D E D I C A T I O N

This thesis is dedicated to two great men

One is Andrew Geddes Bain

The other is Jeremiah Edmund Jennings

Each has left his imprint upon the field
within which this work is rooted

ON THE SIGNIFICANCE OF STRATIGRAPHY IN THE PREDICTION OF THE ENGINEERING BEHAVIOUR OF SOILS AND ROCKS IN SOUTHERN AFRICA

by

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ABSTRACT

ON THE SIGNIFICANCE OF STRATIGRAPHY IN THE PREDICTION OF THE ENGINEERING BEHAVIOUR OF SOILS AND ROCKS IN SOUTHERN AFRICA

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Degree : D.Sc.
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The object of this thesis is to demonstrate that a knowledge
1 of the *stratigraphic unit* occupying a site provides a basis for making broad generalisations regarding likely terrain and foundation conditions which may be used to advantage in designing a relevant site investigation programme. Stated simply: very different exploration programmes would be designed for a house on Orange Grove quartzite and a dam on Transvaal dolomite.

If climate as well as stratigraphy, is taken into account we may recognise a *regional stratigraphic unit* and achieve a higher level of generalisation which will lead to more accurate predictions being made regarding the likely engineering properties of the local rocks and soils. Subdivision of the regional stratigraphic unit into *land patterns*, and of land patterns into their constitutent *land facets*, represent yet further advances from lower to higher levels of generalisation. Apart from specific case histories, however, it is the regional stratigraphic unit that is adopted as the main level of discussion throughout most of the thesis.

The importance of the correct identification of the *origin* of each horizon of the soil profile during site investigation work is emphasised as this, too, may lead to meaningful predictions regarding the engineering behaviour of the soil. The significance of the *pebble marker* as an aid to the correct identification of soil origin is also emphasised.

The introductory chapter concludes with a number of very broad generalisations about the typical forms of the soil profile in Southern Africa and the environmental factors responsible for their development.

Attention is drawn to the engineering significance of the
2 great lithological variability within the ancient metamorphic
bodies of the Basement-complex. The characteristics of residual soils formed by the *in situ* decomposition of these metamorphic rocks are discussed for the four most significant lithological types, viz. 'greenstones', mica schists, phyllites and metagabbros.

Residual greenstones,* where they have been preserved on remnants of the African erosion surface on the margin of the Johannesburg-Pretoria granite inlier, are represented by deep soils which display both potentially expansive and highly compressible characteristics.

The same is true of the residual mica schists of the Zambian Copperbelt. In addition, owing to the preferred orientation of mica particles within these soils, they exhibit anistropic properties.

Residual phyllites in Rhodesia again have somewhat similar characteristics though of a much more variable nature, even within an individual soil profile. Problems of differential movement of structures founded on these soils are thus magnified.

The metagabbros of Salisbury, Rhodesia, produce deep, highly leached residual soils below well-drained slopes, and relatively shallow black plastic clays in situations of impeded drainage. Foundation conditions for medium- and high-rise buildings are nevertheless more favourable on the metagabbros than on the phyllites.

Case histories deal with the successful choice of foundation type for a brewery on residual mica schist in Ndola, Zambia, and with the cracking of buildings built on residual phyllite at a school in Bulawayo, Rhodesia.

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* See footnote on page 8 for definitions and usage of the term 'residual'.

3 Granite-gneiss of the Basement-complex is resistant to weathering in the arid parts of the subcontinent. In the more humid parts, however, and specifically in areas of annual water surplus, it decomposes to produce deep residual soils which exhibit a *collapsible grain structure*. This condition results from the leaching out of kaolinite formed by the hydration of alkali felspars. The local presence of the condition may be predicted from a study of topographic maps and aerial photographs and from simple field tests. Assessment of the degree of severity of the condition ('collapse potential') involves laboratory testing including the double oedometer test. Practical solutions to founding on these soils include different types of piled foundations, suspension of small monolithic structures on three foundation pads, or excavation of the soil and replacement in a compacted condition. A case history describes the tilting of an elevated water tower as a result of inundation of the soil and consequent collapse of the grain structure under two of the column footings, and rectification of the tilt by inundating the soil beneath the other two footings.

In certain types of granite-gneiss, for example in part of the Johannesburg-Pretoria inlier, large blocks of rock remain unweathered though they are surrounded by residual soil which may exhibit a collapsible grain structure. It is argued that the presence of microcline as the main felspar in these *core-stones* may play a more significant role in their preservation from chemical decomposition than has hitherto been recognised. The presence of core-stones within the residual soil may present special engineering problems, particularly in relation to excavation and to the founding of heavy structures.

A residual granite soil which has developed a collapsible grain structure has a grading such that it does not constitute a filter within itself. Fine particles may be washed out from between the coarser particles in situations where there is a sufficient hydraulic gradient, and *pseudokarst* phenomena, such as the development of 'sinkholes', may result. A case history describes how various remedial measures failed to restore a small 'leidam' in the Eastern Transvaal which was leaking as a result of the development of pseudokarst features, and how the dam was eventually abandoned as irreparable.

4 Although exposures of the Witwatersrand System are of limited extent, most of the towns of the Central Witwatersrand, including Johannesburg, are situated largely upon this stratigraphic unit. Excellent examples of stratigraphic constraints on engineering construction and other types of development are also afforded by the nature of the strata comprising this system. Consequently two fairly lengthy chapters are devoted to the Witwatersrand System.

It is shown that the rocks and associated residual soils of the Lower Division of the system are for the most part relatively competent and inert. However, the juxtaposition of hard quartzites and banded ironstones with softer shales and decomposed intrusives, even within the confines of a single building site, poses special foundation problems. Related problems arise from the need to provide lateral support for deep basement-excavations in these strata and consideration is given to the various techniques of support which have been successfully applied in the city of Johannesburg. Two case histories deal with unusual solutions of this problem.

5 Engineering problems associated with the Upper Division of the Witwatersrand System are largely confined to subsidence of the surface as a result of stope closure in undermined areas, and the propogation of earth-tremors emanating from the release of stresses in the hanging-wall strata. It is shown that surface subsidence and consequent damage to structures can take place even in areas where the depth of undermining is of the order of a thousand metres below the surface, though the problems are naturally more severe where the depth of undermining is shallow. Restrictions which are imposed on development in undermined areas are consequently more rigorous in areas of shallow undermining.

Among the factors influencing mining subsidence are the strength and the deformation characteristics of rock constituting the hanging wall, the effect of underground pillars of unmined ore, the development of a fracture-zone around mine openings, the nature and the disposition of dykes and faults and the mining techniques employed. It is shown that four types of mining subsidence are experienced on the Central Rand, viz. the development of 'sinkholes' where excavations of ore at the outcrop have been subsequently filled with soil or rubble, subsidence accompanying the development of underground caverns by the 'frittering' of incompetent strata in the hanging wall, subsidence related to the development of tension-fractures in the hanging wall and subsidence due to normal stope closure. Examples of each of these types of subsidence are recorded in four case histories. Two further case histories deal with differential distortions in the settlement pattern produced by the presence of dykes.

Statistical evidence shows that earth-tremors experienced on the Central Rand are directly related to mining activity. The tremors do not pose any direct constraint on the erection of high-rise buildings on the foot-wall strata. A case history describes the structural damage suffered by a four-storey building located in an unfavourable situation in relation to the disposition of dykes and faults.

Exposures of the Ventersdorp lavas fall into three distinct
6 climatic zones and the engineering problems to be anticipated
with the rocks and the residual soils of each zone may be
predicted from the identification of the regional stratigraphic units so
defined.

In the semi-arid zone which receives less than 500 mm of rainfall per annum there is little development of residual soils and these are usually cemented by calcrete. The presence of soft pockets of pyroclastic materials within the lavas may, however, present foundation problems in heavily loaded structures.

In the sub-humid dry zone which receives an annual precipitation in the range 500 to 750 mm, chemical decomposition of the lava has proceeded as far as the stage of development of clay-minerals of the smectite group and problems of heave may be expected.

In the small sub-humid moist zone confined to the Johannesburg graben, annual precipitation exceeds 750 mm and the residual soils developed on the lavas are deep, highly leached, and characterised by the presence of kaolinite clay in the upper horizons of the soil profile. They are compressible soils with an unusually high coefficient of consolidation. Three case histories relating to this zone deal respectively with the choice of foundation design for a heavily loaded structure, the occurrence of frost-heave in the soils beneath a cold-storage warehouse, and buildings straddling the faulted contact between soft residual lavas and the hard Hospital Hill quartzite.

The most problematical terrain from the engineer's viewpoint
7 is that associated with the Black Reef and the Dolomite Series
of the Transvaal System. It is shown that the occurrence of highly compressible *wad* within these formations is largely related to stratigraphic controls.

An historical record of engineering problems associated with karst phenomena in the Transvaal dolomites culminates with a detailed account of the accelerated development of *sinkholes* and *dolines* in the Far West Rand during the past two decades. It is concluded that the accelerated development of conditions giving rise to various types of surface subsidence in the Far West Rand is directly related to the artificial lowering of the water-table in this area by pumping from mine shafts. A case is made for the writer's contention that the water-table must be regarded as the base level of subsurface erosion in dolomite terrain, and mechanisms are proposed for the development of sinkholes and dolines. It is also contended that *large sinkholes* (more than 45 m in diameter and more than 30 m deep) are not the product of successive collapse of 'multiple arches' as has been proposed by a number of workers.

Attemps at delineation of areas of potential sinkhole and doline development by means of geophysical exploration are discussed and it is concluded that the qravity survey, in spite of its limitations, is the most promising of the available techniques. However three different dolomitic terrain conditions are recognised, each requiring a different interpretation of gravitmetric findings, viz. areas in which the water-table is artificially lowered, areas in which the water-table is static and at shallow depth, and areas in which the water-table is static and deep.

Measures for the protection of engineering structures on dolomite are briefly discussed but it is concluded that prevention of the problem is safer and less costly than protection. On the more positive side, the advantageous uses of dolomite as a construction material are discussed, and the rock is particularly praised for its effectiveness as an aggregate in the manufacture of concrete sewer pipes.

A case history concerning the foundations for the largest complex of silos in Africa, records the timeous discovery of the presence of wad within the stratigraphic sequence of the Black Reef Series.

A further case history deals with the differential settlements observed in kilns at a brickworks situated on a Karoo outlier in the Bank Compartment of the Far West Rand: it is shown that these settlements were due to consolidation accompanying the draining of an irregular mass of wad below the Karoo outlier.

As a general rule it may be stated that the sedimentary rocks
8 of the Pretoria Series provide good founding conditions for
most types of structures. Exceptions to this rule concern the expansive characteristics of a 'stratum' of residual shale within the Magaliesberg Stage and the collapsible grain structure which has been observed in a 'stratum' of residual felspathic quartzite also within the Magaliesberg Stage. There seems to be some evidence of stratigraphic controls in both cases, but the evidence is not regarded as irrefutable.

Residual soils developed on the Ongeluk lavas of the Daspoort Stage often display expansive characteristics. The lavas are characterised by the extreme variability in the depth and the degree of decomposition over relatively short distances. This phenomenon is thought to be associated with variability in joint spacing in the parent-rock.

Expansive soils are also commonly associated with decomposed
9 diabase sills and dykes of the intrusive phase of the Bushveld
Complex. Local hydromorphic effects associated with topographic depressions and natural drainage courses appear to increase the problem. The deleterious effects of differential movement are reflected particularly in structures straddling the contact between residual diabase sills and the Pretoria shales into which they are intrusive. A case history describes the simple remedy adopted in the construction of a church building in such a situation.

The stability of excavations in residual diabase is often problematical, whether in tunnels or in open cut. A case history describes the stability problems encountered in the excavation for a reservoir in a decomposed sill in a particularly unfavourable situation where the residual diabase was being squeezed out under the load of the overlying mountain range of Magaliesberg quartzite.

The most highly expansive soils in South Africa are the black
10 subtropical clays developed as residual soils on the mafic
rocks of the Bushveld Complex. A comparison of indicator properties of these soils with those of similar black clays elsewhere in Africa shows, indeed, that the Bushveld clays rank amongst the worst on the continent.

Experimental work carried out by the CSIR suggests that there may be simple and effective means of overcoming the heave problems in roads that are to be built on these soils. Removal of natural vegetation from the soils well in advance of road construction inhibits the desiccating effects of transpiration. Likewise the desiccating effects of evaporation can be countered by covering the area in advance with an impermeable membrane or, better still, with a thin cover of sand. Pre-treatment of the surface by methods such as these may therefore result in a wetting up of the soil and consequent dissipation of heave in advance of road construction.

Seven characteristic types of soil profile have been found to recur in the terrain underlain by norites and gabbros, at least in the southern part of the Bushveld complex. Generalised soil profiles are given for each of them, together with a discussion of the topographic and lithological controls responsible for their development. Airphoto interpretation has proved to be an effective and rapid means of delineation of these recurrent soil profile types.

The mafic rocks of the Bushveld Complex provide a useful source of natural building stones and concrete aggegates. The locations of existing quarries for these materials suggests the possibility of a 'pseudo-stratigraphic' control.

The use of Karoo sandstones as coarse aggregate in concrete,
11 and of sands derived from the sedimentary rocks of the Karoo
System as fine aggregate, often results in defective construction which manifests itself in four different ways, viz. deflection of reinforced concrete members, development of cracks coincident with reinforcing steel, corrosion of reinforcement and surface crazing. A research programme in which the writer participated during the early 1950's revealed that these phenomena were all produced by excessive shrinkage of the aggregates during the curing period of the concrete.

While the presence of clay-minerals with an expanding lattice was shown to be a contributory factor to the problem in the case of some of the sandstones, it was subsequently found that a more significant factor was the unusually high internal surface-area of the sandstones known to give rise to the problem. In addition the shrinking aggregates possess an unusually low modulus of elasticity.

A case-history describes the deflections which were observed in a bridge which had been made from concrete in which dolerite had been used as the coarse aggregate and local river sand derived from Karoo rocks as the fine aggregate.

In spite of the poor performance of many of the sandstones when incorporated in concrete, Karoo sandstones have been widely and successfully used as natural building stones in many parts of Southern Africa. However, in the more highly porous varieties of Karoo sandstone, the hydration of salts within the pores may generate stresses in excess of the tensile strength of the rock and this can result in disintegration.

The thesis is summarised in the final chapter and it is con-
12 cluded that a knowledge of the local stratigraphy (preferably together with a knowledge of the local climatic environment and the geomorphological history) at any site where an engineering structure is to be built, will act as a means of prediction of the specific nature of the engineering properties of the rocks and the soils which are likely to be encountered at the site. This knowledge may be used to advantage in designing the most effective programme of site exploration for a specific structure at a specific site.

SAMEVATTING

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Die doel van hierdie proefskrif is om aan te toon dat 'n
1 kennis van die *stratigrafiese eenheid* wat onder 'n terrein voorkom, as basis kan dien waarvolgens breë veralgemenings met betrekking tot terrein- en fondamenttoestande gemaak kan word en wat dan voordelig gebruik kan word in die ontwerp van relevante programme van terreinondersoek. Kortweg gestel: ondersoekprogramme wat grootliks verskil sal ontwerp moet word vir 'n huis op Orange Grove-kwartsiet en vir 'n dam op Transvaalse dolomiet.

Indien klimaat sowel as stratigrafie in berekening gebring sou word, kan 'n *regionale stratigrafiese eenheid* herken word, wat lei tot 'n hoër vlak van veralgemening en tot meer akkurate voorspellings met betrekking tot die moontlike ingenieurseienskappe van die plaaslike gesteentes en gronde. Onderverdeling van die regionale stratigrafiese eenheid in *landpatrone* en van landpatrone in hul onderskeie *landfasette*, verteenwoordig verdere voordele wat strek van die laer tot die hoër vlakke van veralgemening. Afgesien van spesifieke gevallestudies, is dit egter die regionale stratigrafiese eenheid wat deurgaans aanvaar word as die hooftema van bespreking in hierdie proefskrif.

Die belangrikheid van korrekte identifikasie van die *oorsprong* van elke horison in die grondprofiel tydens terreinondersoek work beklemtoon, aangesien dit ook kan lei tot betekenisvolle voorspellings met betrekking tot die ingenieursgedrag van die grond. Die belangrikheid van die *rolsteenmerkerlaag* as 'n hulpmiddel by die korrekte identisifisering van die oorsprong van 'n grondsoort word ook beklemtoon.

Die inleidende hoofstuk word afgesluit met 'n aantal breë veralgemenings met betrekking tot tipiese grondprofiele in Suidelike Afrika en die omgewingsfaktore wat aanleiding gegee het tot hulle ontwikkeling.

2 Die aandag word gevestig op die belangrikheid vir die ingenieur van groot litologiese veranderlikheid binne die oermetamorfe liggame van die Fundamentele Kompleks. Die eienskappe van residuele gronde gevorm deur die ontbinding *in situ* van hierdie metamorfe gesteentes word bespreek vir die vier mees belangrike litologiese tipes nl. groensteen, glimmerskis, filliet, en metagabbro.

Residuele groensteen, waar dit bewaar gebly het op oorblyfsels van die Afrikaanse erosie-oppervlak langs die grens van die Johannesburg-Pretoriase granietvenster, word verteenwoordig deur diep gronde wat potensiële uitswelling sowel as hoogs samedrukbare eienskappe toon.

Dieselfde is waar van die residuele glimmerskis van die Zambiese Koperstreek. Daarbenewens, veral na aanleiding van die voorkeurorientasie van die glimmerplaatjies in die gronde, openbaar hulle ook anisotrope eienskappe.

Residuele filliet in Rhodesië het eweneens soortgelyke eienskappe alhoewel van 'n baie meer veranderlike aard, selfs binne 'n enkele grondprofiel. Probleme met betrekking tot differensieële beweging van strukture wat gefundeer is op hierdie gronde word derhalwe vergroot.

Die matagabbro van Salisbury, Rhodesië, lewer diep, hoogs uitgeloogde, residuele gronde onderkant goedgedreineerde skuinstes, en betreklik vlak, swart, plastiese kleigronde onder toestande van swak dreinering. Fondamenttoestande vir middelslag- en hoë geboue is nieteenstaande gunstiger op die metagabbro as op die filliet.

Gevallestudies handel oor die suksesvolle keuse van fondamenttipes vir 'n brouery op residuele glimmerskis by Ndola, Zambië, en met die kraak van geboue wat gefundeer is op residuele filliet by 'n skool in Bulawayo, Rhodesië.

3 Graniet-gneis van die Fundamentele Kompleks bied weerstand teen verwering in die droë dele van die subkontinent. In die vogtige gedeeltes, en spesifiek in gedeeltes waar 'n jaarlikse watersurplus voorkom, ontbind die gesteente om diep residuele gronde te

vorm. Hierdie gronde toon 'n *swigkorrelstruktuur*. Die toestand word veroorsaak deur die uitloging van die kaoliniet wat gevorm is deur die ontbinding van die alkaliveldspaat. Die plaaslike voorkoms van hierdie toestand kan voorspel word deur middel van 'n studie van topografiese kaarte en lugfoto's asook met behulp van eenvoudige veldtoetse. Ten einde te bepaal hoe ver die toestand gevorder het, d.w.s. die swigpotensiaal van die grond, is laboratoriumtoetse nodig, insluitende die dubbele oedometertoets. Praktiese oplossings met betrekking tot fundering op hierdie gronde sluit onder meer in die verskillende tipes heipaalfondamente, suspendering van klein monolitiese strukture op drie fondamentstukke, of die verwydering van die grond en terugplasing in 'n gekompakteerde toestand. In 'n gevallestudie word die kanteling van 'n hoë watertoring as gevolg van die benatting van die grond, en die gevolglike swigting van die korrelstruktuur onderkant twee van die kolomme behandel. Die kanteling is reggestel deur die benatting van die grond onderkant die ander twee voetstukke.

In sekere tipes graniet-gneis, b.v. in 'n sekere gedeelte van die Johannesburg-Pretoria-venster, word gevind dat groot blokke rots onverweerd bly, alhoewel hulle omring is deur residuele grond wat moontlik 'n swigkorrelstruktuur mag besit. Daar word vermoed dat die teenwoordigheid van mikroklien as die vernaamste veldspaat in hierdie *kernstene* 'n groter rol speel in hulle behoud teen chemiese verwering as wat tot hede toe aanvaar is. Die teenwoordigheid van die kernstene binne die residuele gronde mag spesiale ingenieursprobleme tot gevolg hê, veral met betrekking tot diep uitgrawings en die fundering van swaar bouwerke daarop.

'n Residuele granitiese grond wat 'n swigkorrelstruktuur ontwikkel het, het 'n gradering wat sodanig is dat dit nie vanself as 'n filter kan dien nie. Fyn deeltjies mag moontlik uitgewas word tussen die growwer deeltjies onder toestande waar 'n voldoende hidrouliese gradient bestaan, en 'n *pseudokarst* verskynsel, soortgelyk aan die vorming van 'sinkgate', mag ontstaan. 'n Gevallestudie word beskryf waar aangetoon word hoe verskeie herstelmetodes nie geslaag het nie om 'n klein leidam in die Oos Transvaal te red, wat gelek het as gevolg van die ontwikkeling van hierdie pseudokarsttoestande, en hoe die dam uiteindelik as onherstelbaar laat vaar is.

Alhoewel dagsome van die Sisteem Witwatersrand beperk is, is
4 dit waar dat meeste van die dorpe van die Sentrale Witwatersrand, Johannesburg ingesluit, grootliks geleë is op hierdie stratigrafiese eenheid. Uitstekende voorbeelde van stratigrafiese beperkings op ingenieurskonstruksie en ander tipes van ontwikkeling kan toegeskryf word aan die eienskappe van die lae wat hierdie Sisteem opbou. Gevolglik is twee lang hoofstukke gewy aan die Sisteem Witwatersrand.

Daar word aangetoon dat die gesteentes en die verwante residuele gronde van die Onderste Afdeling van die sisteem, vir die grootste gedeelte relatief stabiel is. Die aanwesigheid van harde kwartsiet en gestreepte ystersteen naasaan sagter skalie en ontbinde stollingsgesteenes, selfs binne die perke van 'n enkele bouterrein, veroorsaak egter spesiale fondamentprobleme. Verwante probleme ontstaan wanneer dit nodig word om sydelingse bestutting te voorsien in diep kelderuitgrawings in hierdie lae en oorweging word geskenk aan die verskeidenheid van tegnieke met betrekking tot bestutting wat suksesvol toegepas is in the stad Johannesburg. Twee gevallestudies hanteer ongewone oplossings van hierdie probleem.

Ingeniersprobleme geassosieerd met die Boonste Afdeling van
5 die Sisteem Witwatersrand is grootliks beperk tot die wegsak van die grondoppervlakte as gevolg van die sluiting van afbouplekke in uitgemynde gebiede en die voortplanting van aardskuddings wat ontwikkel as gevolg van die verligting van spanning in die lae aan die dakkant. Daar word aangetoon dat insakking aan die oppervlak en die gevolglike skade aan strukture kan plaasvind in gebiede waar uitgrawing gevorder het tot binne 'n duisend meter van die grondoppervlakte af. Die probleme is egter baie erger waar tot naby die oppervlak uitgemyn is. Beperkings van toepassing op ontwikkeling in uitgewerkte gebiede is gevolglik strenger in gebiede waar tot baie na aan die oppervlak gemyn is.

Onder die faktore wat 'n invloed uitoefen op insinking as gevolg van mynbou is die sterkte en die vervormingseienskappe van die gesteentes aan die dakkant, die effek van ondergrondse pilare wat uit ongemynde erts bestaan, die ontwikkeling van 'n breuksone langs mynopeninge, die geaardheid en die ruimtelike rangskikking van ganggesteentes en verskuiwings en die mynboumetodes wat toegepas is. Daar word aangetoon dat vier tipes myninsinkings ondervind word aan die Sentrale Rand, nl. die ontwikkeling van 'sinkgate' waar die uitgrawing van erts aan die dagsoom opgevul is met grond en verweringspuin, insakking wat saamgaan met die ontwikkeling van ondergrondse grotte as gevolg van afskilfering van swak rotsmateriaal uit die dak, insinking wat gekoppel kan word aan die ontwikkeling van spanningsbreuke in die dakkant en insinking as gevolg van die sluiting van normale afbouplekke. Voorbeelde van elk van hierdie tipes insinking word behandel in vier gevallestudies. Twee verdere gevallestudies handel oor differensiële vervorming in die sakkingspatroon wat veroorsaak is deur die teenwoordigheid van ganggesteentes.

Statistiese gegewens toon aan dat aardskuddings wat ondervind word aan die Sentrale Rand, regstreeks in verband staan met mynbouaktiwiteite. Hierdie aardskuddings plaas geen direkte beperkings op die oprigting van hoë geboue op die vloerkantlae nie. 'n Gevallestudie bespreek die struktuurskade wat aangerig is aan 'n vier-verdiepinggebou, geleë in 'n ongunstige omgewing met betrekking tot die voorkoms aldaar van ganggesteentes en verskuiwings.

6 Dagsome van Ventersdorplawa kom voor in drie duidelike klimaatsones, en die ingenieursprobleme wat verwag kan word met betrekking tot die gesteentes en die residuele gronde van elke sone, kan voorspel word na aanleiding van die identifikasie van die betrokke stratigrafiese eenhede waarbinne hulle val.

In die halfdorre sone waar die reënval minder as 500 mm per jaar is, is die ontwikkeling van residuele gronde gering, en waar hulle voorkom is hulle gewoonlik gesementeer deur kalkreet. Die voorkomste van sagte neste van piroklastiese materiaal binne in the lawa, mag egter fondamentprobleme tot gevolg hê in die geval van swaar belaste strukture.

In die subhumiede droë sone waar die jaarlikse reënval gemiddeld tussen 500 en 750 mm is, het chemiese ontbinding van die lawa reeds so ver gevorder dat die vorming van kleiminerale van die smektietgroep voorkom, en gevolglik kan probleme van uitswelling verwag word.

In die klein subhumiede nat sone, wat beperk is tot die Johannesburgse trog, is die jaarlikse reënval meer as 750 mm. Die residuele gronde wat ontwikkel op die lawa is diep, hoogs uitgeloog en word gekenmerk deur die voorkoms van kaolinitiese kleie in die boonste horisonne van die grondprofiel. Hierdie gronde is samedrukbaar, met 'n ongewoon hoë koefisiënt van konsolidasie. Drie gevallestudies wat betrekking het op hierdie sone handel oor die keuse van fondamentontwerp in 'n swaar belaste struktuur, die voorkoms van bevriesingsuitswelling in die grond onderkant 'n verkoelingspakhuis, en geboue wat bo-oor 'n verskuiwingskontak tussen sagte residuele lawa en harde Hospitaalheuwelkwartsiet geplaas is.

Die mees problematiese terrein, gesien vanuit die ingenieurs-
7 oogpunt, is dié geassosieer met die Serieë Swartrif en Dolomiet van die Sisteem Transvaal. Daar word aangetoon dat die voorkomste van hoogs samedrukbare *mangaanaarde* binne hierdie formasies grootliks gekoppel kan word met stratigrafiese kontrole.

'n Historiese rekord van die ingenieursprobleme wat geassosieer is met die karstverskynsel in die Transvaalse dolomiet word afgesluit met 'n gedetailleerde verslag oor die versnelde ontwikkeling van *sinkgate* en *karstregters* aan die Verre Wes-Rand gedurende die laaste twee dekades. Daar word bevind dat die versnelde ontwikkeling van toestande wat aanleiding gee tot die ontstaan van die verskillende insinkings aan die oppervlak aan die Verre Wes-Rand in regstreekse verband staan met die kunsmatige verlaging van die watervlak in hierdie gebied as gevolg van die uitpomp van water uit die mynskagte. Skrywer stel 'n vir hom aanvaarbare saak daar dat die watervlak as basisvlak van ondergrondse erosie in dolomietterrein beskou moet word, en meganismes word voorgestel vir die ontwikkeling van sinkgate en karstregters. Dit word ook aanvaar dat *groot sinkgate* (meer as 45m indeursnit en meer as 30 m diep) nie die gevolg is van agtereenvolgende instorting van "meningvuldige boogstutte" nie, soos voorgestel deur 'n aantal navorsers.

Pogings om deur middel van geofisiese ondersoek gebiede af te baken waarin sinkgate en karstregters moontlik mag vorm, word bespreek en daar word gevind dat gravimetriese ondersoek, ten spyte van sy beperkings, die mees belowende beskikbare tegniek is. Drie dolomietterreintoestande word egter herken, waarvan elk 'n eie verskillende interpretasie van die gravimetriese ondersoek vereis, nl. gebiede waar die watervlak kunsmatig verlaag is, gebiede waar die watervlak staties en op 'n geringe diepte voorkom, en gebiede waarin die watervlak staties en diep is.

Maatreëls ter beskerming van ingenieurstrukture op dolomiet word kortliks behandel. Daar word egter tot die slotsom gekom dat voorkoming van die probleem baie veiliger en goedkoper is as beskerming.

Aan die positiewe kant word die voordelige gebruik van dolomiet as konstruksiemateriaal bespreek, en die gesteente word aangeprys omrede die doeltreffende aanwending daarvan as aggregaat in die vervaardiging van beton-rioolpype.

'n Gevallestudie word aangebied, wat handel oor die fondamente van die grootste graansuierkompleks in Afrika, sowel as die tydige ontdekking van die voorkoms van mangaanaarde binne die stratigrafiese opeenvolging van die Serie Swartrif.

'n Verdere gevallestudie behandel differensiële sakkings wat waargeneem is by steenoonde wat geplaas is op 'n loslap van Karoogesteentes in die Bankkompartement aan die Verre Wes-Rand. Daar word aangetoon dat hierdie sakkings toegeskryf kan word aan konsolidering as gevolg van die effektiewe dreinering van 'n onreëlmatige massa mangaanaarde onderkant die Karoo-loslap.

As 'n algemene reël mag gekonstateer word dat die sedimentêre
8 gesteentes van die Serie Pretoria goeie fondamenttoestande
verskaf vir die meeste bouwerke. Uitsonderings op dié reël behels die sweleienskappe van 'n laag residuele skalie binne die Etage Magaliesberg en die swigkorrelstruktuur wat waargeneem is in 'n laag van residuele veldspatiese kwartsiet, wat ook binne die Etage Magaliesberg voorkom. Dit wil voorkom asof daar bewys bestaan van stratigrafiese kontrole in beide gevalle, maar hierdie bewyse word nie aanvaar as onbetwisbaar nie.

Residuele gronde wat op die Ongeluklawa van die Etage Daspoort ontwikkel het, openbaar heel dikwels sweleienskappe. Die lawa word gekenmerk deur buitengewone veranderlikheid wat diepte en graad van verwering oor betreklike kort afstande betref. Daar word vermoed dat hierdie verskynsel geassosieer is met die wisselvalligheid van die naatspasiering in die moedergesteente.

9 Swelgronde word in die algemeen aangetref op ontbinde diabaasplate en gange van die intrusiewe fase van die Bosveldkompleks. Plaaslike hidromorfe effekte wat in verband gebring kan word met topografiese laagtepunte en natuurlike dreineringsrigtings skyn die probleem te vererger. Die ongewenste uitwerking van differensiële beweging word in die besonder weerspieël in bouwerke wat opgerig is bo-oor die kontakte tussen residuele diabaasplate en die skalie van die Serie Pretoria waarin hulle intrusief is. 'n Gevallestudie beskryf 'n eenvoudige oplossing wat aangewend is by die konstruksie van 'n kerkgebou in so 'n situasie.

Die stabiliteit van uitgrawings in residuele diabaas is in baie gevalle problematies, hetsy in tonnels of in ope deurgrawings. 'n Gevallestudie beskryf die stabiliteitsprobleme wat ondervind is in die uitgrawings vir 'n stoordam in 'n ontbinde diabaasplaat. Die situasie was besonder ongunstig aangesien die diabaas uitgepers is onder belasting van die oorliggende Magaliesbergkwartsiet wat die bergreeks opbou.

10 Gronde met die allergrootste sweleienskappe word in Suid-Afrika verteenwoordig deur die swart subtropiese klei wat ontwikkel as residuele grond op die mafiese gesteentes van die Bosveldkompleks. 'n Vergelyking van aanwysereienskappe van hierdie gronde met dié van soortgelyke swart kleie elders in Afrika toon dat die Bosveld-kleie as van die slegste op die vasteland beskou kan word.

Eksperimentele werk wat deur die WNNR uitgevoer is, dui daarop dat daar eenvoudige en doeltreffende metodes mag wees om die swelprobleme by paaie wat op hierdie gronde gebou word, op te los. Die verwydering van natuurlike plantegroei vanaf die gronde in 'n vroeë stadium voor padkonstruksie, beperk die uitdrogingseffek van transpirasie. Desgelyks word die uitdrogingseffek van verdamping teëgewerk deur die oppervlakte in 'n vroeë stadium te bedek met 'n ondeurdringbare membraan of, nog beter, met 'n dun lagie sand. Voorbehandeling van die oppervlakte deur middel van hierdie metodes mag dus meebring dat die grond vogtig word, met die gevolglike uitskakeling van opheffing voordat die pad gebou word.

Sewe karakteristieke soorte grondprofiele is waargeneem, wat gereeld aangetref word in die terrein waaronder die noriet en gabbro voorkom, veral in die suidelike gedeeltes van die Bosveldkompleks. Veralgemeende grondprofiele word gegee vir elk van bogenoemde, tesame met 'n bespreking van die topografiese en litologiese kontroles wat verantwoordelik is vir hulle ontwikkeling. Lugfotovertolking het geblyk 'n effektiewe en snelle metode te wees om hierdie gronde, wat so dikwels voorkom, te identifiseer en te omlyn.

Die mafiese gesteentes van die Bosveldkompleks is 'n baie waardevolle bron van natuurlike bouklipmateriaal en betonaggregaat. Die ligging van bestaande klipgroewe wat hierdie materiale lewer, dui op die moontlikheid van 'n 'pseudostratigrafiese' kontrole.

Die gebruik van Karoosandsteen as growwe aggregaat vir beton,
11 en van sande afkomstig van die sedimentêre gesteentes van die
Sisteem Karoo as fyn aggregaat, lei dikwels tot defektiewe konstruksie wat waargeneem kan word in vier hoofverkynsels, nl. die defleksie van wapeningsmateriaal in beton, die ontwikkeling van krake wat saamval met die wapeningstaal, korosie van wapening, en barsies aan die oppervlak. 'n Ondersoekprogram waaraan die skrywer deelgeneem het gedurende die vroeë 1950s, het bewys dat hierdie verskynsels almal veroorsaak is deur buitensporige krimping van die aggregaat gedurende die verhardingstyd van die beton. Alhoewel bevind is dat die teenwoordigheid van kleiminerale met 'n uitsettende traliestruktuur in die geval van sekere soorte sandsteen 'n bydraende faktor was, is daar nogtans later ontdek dat 'n belangriker faktor die buitengewoon hoë interne oppervlakte van die sandstene was wat die probleem veroorsaak het. Bykomend is gevind dat die krimpende aggregaat 'n buitengewoon lae elastisiteitsmodulus het.

'n Gevallestudie beskryf die defleksies wat waargeneem is in 'n brug wat gebou is van beton waarin doleriet gebruik is as growwe aggregaat en plaaslike riviersand, afkomstig van Karoogesteentes, as fyn aggregaat.

Ten spyte van die swak gedrag van baie van die sandstene wat gebruik is in beton, word Karoosandsteen tog wydverspreid en suksesvol gebruik as natuurlike bousteen in groot gedeeltes van Suidelike Afrika. Desnieteenstaande is gevind dat die hidrasie van soute binne die porieë van die besonder poreuse tipes Karoosandsteen, aanleiding mag gee tot spannings wat groter is as die treksterkte van die gesteente en dit mag tot verbrokkeling lei.

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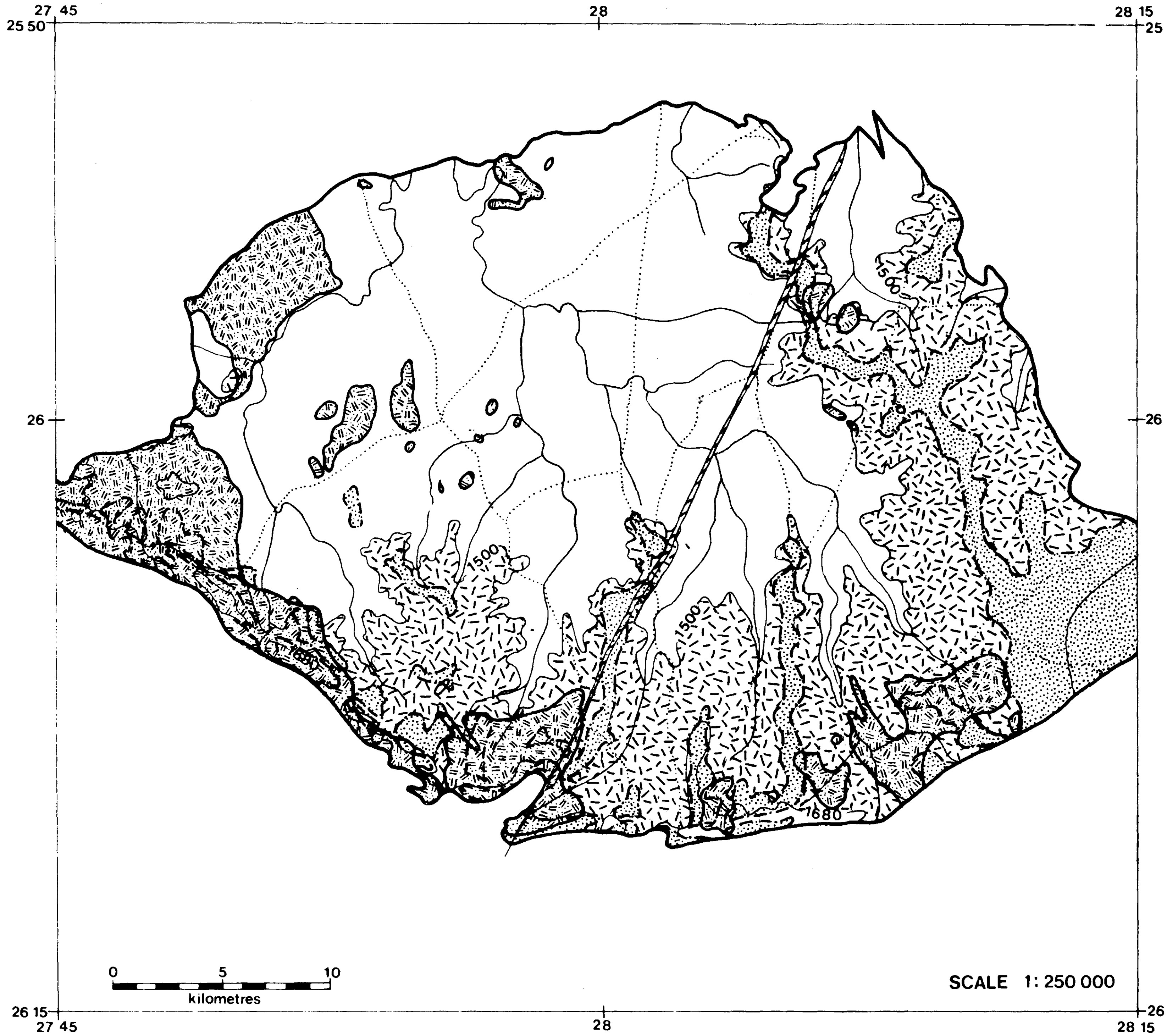

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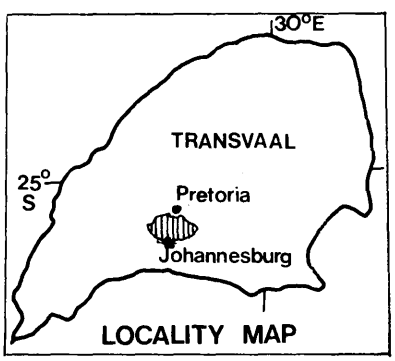

- Outline of Johannesburg - Pretoria granite dome
- Swaziland System
- Mylonite crush-zone
- Remnants of African surface (after Partridge, 1975) with collapsible residual soil
- Additional areas above 1 500m with collapsible residual soil
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FIGURE 3/1 : Distribution of soils with a potentially collapsible grain structure in the Johannesburg–Pretoria granite dome

1 INTRODUCTION

No engineering structure is better than the material of which, and on which, it is built. As practically every structure is built on or in soil or rock, and most are built, at least in part, of soil or rock, the properties of these materials are of fundamental importance in engineering. This fact has led to the development of technologies such as soil mechanics, rock mechanics and materials engineering, all of which draw heavily on the science of geology. To meet the needs of these technologies in particular, and of engineering in general, *engineering geology* has developed as an applied branch of geology in its own right.

In considering the foundation design for any structure, it is the responsibility of the engineering geologist to assess the probable difficulties presented by the site, and that of the engineer to solve them. In considering the use of natural materials within the structure, it is for the geologist to indicate where likely materials may be found, and for the engineer to judge their suitability for the specific purpose.

Engineering geology is thus primarily concerned with making predictions about the nature and the distribution of materials below the surface, and about the behaviour of these materials under load. In order to make such predictions for any specific engineering project, the first step involves the planning of a programme of site exploration which will be aimed at determining -

(i) the nature of the different types of soils and rocks below the surface;
(ii) their distribution on the site both laterally and with depth; and
(iii) their engineering characteristics.

All exploration programmes will thus involve the digging or drilling of trial-holes. Many will involve the testing of soil or rock samples. Some may also involve geophysical surveys. But the programme must naturally be related both to the type of structure to be built and to the type of terrain on which it is to be built: very different programmes would be planned for an air-strip on deep alluvial soil, a high rise building on an outcrop of quartzite, or a dam on dolomite.

Thus the efficacy of a plan of exploration will be in proportion to the amount of information available from the start about both the structure and the terrain. Fairly detailed information will usually be available about the proposed structure even at this early stage. Often, however, no more will be known about the terrain than can be read from a small-scale geological map, i.e. what *stratigraphic unit* occupies the site. But is is hoped to demonstrate in this thesis that even this broad level of access to information can provide a basis for making generalisations about terrain conditions that can be used to advantage in designing the exploration programme.

In order to know with what degree of confidence a statement about soil or rock behaviour may be regarded, it is necessary to be quite clear about the level of generalisation under which the statement has been made. It is therefore necessary, at this stage, to define more clearly what is meant by the terms that we shall use to represent different levels of generalisation, starting with the stratigraphic unit as the broadest level, and the land facet as the most precise level.

The broadest level of generalisation about terrain is, then, the *stratigraphic unit*, and it is mainly with this level that the ensuing chapters deal. Where possible, the stratigraphic units will be subdivided regionally, usually on the basis of different climatic zones, and each such *regional stratigraphic unit* treated at a somewhat narrower level of generalisation. In this regard the work of H.H. Weinert is of the utmost importance. Weinert (1964) has demonstrated that mechanical disintegration is the predominant mode of rock weathering in areas where his climatic N-value is greater than 5, whereas chemical decomposition predominates where the N-value is less than 5. The N-value is calculated from climatic data as follows:

$$N = \frac{12.\ E_j}{P_a}$$

where: E_j = evaporation during January

P_a = annual precipitation

The climatic N-value of 5 is plotted for Southern Africa in Figure 1/1. Apart from the significance of the N-value regarding the engineering behaviour of weathered dolerite and other basic igneous rocks in road construction, which was the consideration that led to Weinert's original

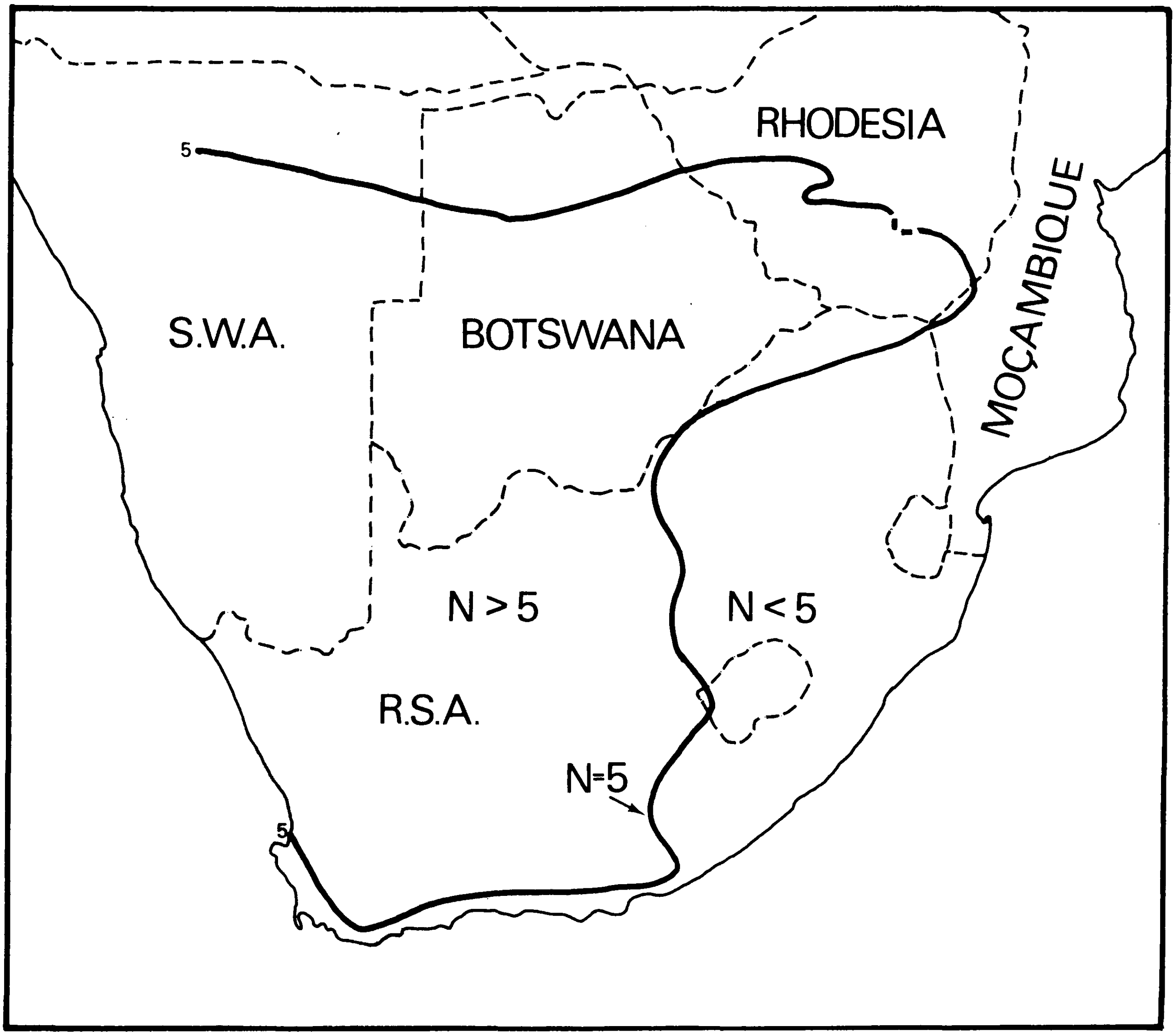

FIGURE 1/1 : Climatic N–value = 5 plotted for Southern Africa

(After Weinert, 1967)

conclusions, a number of other very useful, if admittedly very broad, generalisations can also be made. For example, in the more arid western part of the subcontinent where N is greater than 5, residual soils are generally shallow, transported soils vary greatly in thickness, and pedocretes, where present, are likely to be calcretes or, less commonly, silcretes. In the more humid eastern part where N is less than 5, residual soils are generally deep, transported soils shallow, and pedocretes, where present, are likely to be in the form of ferricretes.

Arising from the development of techniques for soil engineering mapping in South Africa (Brink and Williams, 1964), much emphasis has been placed on higher levels of generalisation concerning terrain units, and the engineering significance of *land patterns* (formely called land systems) and *land facets* has been fully appreciated (Brink *et al*, 1968), in spite of the history of controversy surrounding these concepts. The *land facet* is, from a practical point of view, the smallest unit of terrain which can be mapped and about which high levels of generalisation may be made (Brink *et al*, 1965). The soil profile is reasonably uniform within any specific land facet and the engineering properties of the materials within any specific horizon of that soil profile are also reasonably uniform. It is a terrain unit of the order of size that can be delineated on aerial photographs within a range of scale of 1:10 000 to 1:80 000.

Lower levels of generalisation may be made about the *land pattern*. This is the terrain unit containing a recurrent pattern of genetically linked land facets. It is of an order of size that can conveniently be mapped at scales between 1:250 000 and 1:1 000 000.

Yet lower levels of generalisation may be made about *land regions*, which are contiguous land patterns grouped together on a basis of common lithology or close lithological association and that fall within the same climatic zone. Broadly speaking, therefore, the land region is the unit of terrain which has already been referred to as a *regional stratigraphic unit*.

To summarise: we may advance from low to high levels of generalisation regarding the engineering characteristics of terrain in terms of the following sequence of units:

stratigraphic unit
regional stratigraphic unit (or land region)
land pattern
land facet

While reference will occasionally be made in the text that follows to the engineering characteristics of certain land facets and land patterns, the level of generalisation that will generally be adopted will be that concerning the regional stratigraphic unit. The purpose of adopting this level will be to try and demonstrate that a site exploration programme can, and indeed should, be designed on the basis of the predictions that can be made concerning the local engineering properties of the soils and rocks by simply identifying the stratigraphic unit on a small-scale geological map and appreciating the significance of the local climatic environment.

In attempting to defend the above statement, a limited number of stratigraphic units within Southern Africa will be examined. Those selected for this purpose will be dealt with in strato-chronological order from the oldest to the youngest, namely:

Primitive Systems of the Basement-complex
Archaean granite
Lower Division of the Witwatersrand System
Upper Division of the Witwatersrand System
Ventersdorp System
Black Reef and Dolomite Series of the Transvaal System
Pretoria Series of the Transvaal System
Diabase associated with the Bushveld Complex
Mafic Phase of the Bushveld Complex
Sedimentary rocks of the Karoo System

It will be seen from the above list that the old basis of stratigraphic terminology in terms of systems and series will be employed rather than the currently proposed terminology involving subdivision into supergroups, groups and formations. The reason for this decision is simply that the findings of the various committees currently engaged in erecting the new hierarchy of stratigraphic units have not been published at the time of writing. It will also be seen that the above list is not a comprehensive one for the whole of Southern Africa. The reason for this is simply that the writer has selected for his stated purpose those stratigraphic units with which he is most familiar. Thus the exclusion of such widespread units as the Waterberg and the Cape System, the Karoo dolerites and the Stormberg basalts, must not be taken as an indication that these units are not as significant from the engineering viewpoint as are those which have been selected for treatment here: indeed the Karoo dolerites, to mention but one of those excluded here, have received the full attention they deserve as important construction materials in the geotechnical literature of South Africa.

Furthermore, no chapter will be devoted to Pleistocene deposits, or, in engineering terminology, 'transported soils'. Again, this must not be taken to mean that these deposits and their engineering characteristics are of little consequence to the engineer. The contrary is true. Transported soils, and pedocretes developed within them, are not considered in this thesis simply because, as a general rule, they are not shown on the 1:1 000 000 geological map of South Africa. And since it is our specified object to demonstrate what a useful tool that map is in designing a site investigation programme, Pleistocene deposits must not be considered, except insofar as occasional reference can be made to them as having been locally derived from those older strata under consideration, or insofar as they may form an integral part of a specific case history. It is only in the case histories, then, that there will be a departure from the broad level of generalisation and that attention will be focussed on engineering behaviour of rocks and soils at a specific site.

Finally, reference must be made in this introduction to the engineering significance of the *soil profile*. Below any point on the earth's surface there exists a certain succession of materials. Before an engineering structure is placed on or below the surface, be it a building, bridge, dam, road, tunnel, or any other structure, it is necessary to identify and describe this succession of subsurface materials, and to assess the likely behaviour of each layer in the succession to the degree to which it will interact with the proposed structure. The degree of precision with which such assessments must be made will naturally be dependent on the nature of the engineering structure. At dam sites, for example, many deep diamond-drill holes may be required to determine the load-bearing and water-retaining capacities of the underlying *rocks*, whereas at sites for relatively light buildings exploration need seldom extend below the *soil* cover and predictions regarding the behaviour of the underlying rocks may usually be based solely on a knowledge of the local stratigraphy.

Thus for a wide range of engineering structures the emphasis in site exploration is placed on the succession of *soil horizons* and the engineering characteristics of the materials within each such layer or horizon. Often no more will be required of a site investigation for a relatively light or small structure than the accurate description of each horizon of the soil profile. Accurate description of the soil profile is also a prerequisite in site investigation for heavier structures, though here it may be necessary to extend the description into the underlying bedrock or the succession of bedrock horizons. The number of samples that need be taken and tested in any site investigation will decrease in proportion to the accuracy of description of the various horizons encountered. Consequently a standard terminology for the relevant descriptors in soil profiling has emerged in South African practice, and a standard terminology in core-logging is in the process of being established by the Association of Engineering Geologists (South Africa Section) in collaboration with the Geotechnical Division of the South African Institution of Civil Engineers.

The emphasis on accurate and standarised soil profiling and core-logging which has developed in South Africa is a logical consequence of the close association that has been established here as a tradition in geotechnical circles between geologists and engineers. The geologist can only be of service to the engineer if he is able to communicate his geological knowledge effectively, and the starting point for effective communication is the use of standard descriptive terms which carry with them a specific engineering meaning.

In all of the soil profiles which are reproduced in this thesis the descriptors relating to each soil horizon have specific and accepted meanings. Each soil horizon is described in terms of the six descriptors which are defined in the paper entitled 'Revised guide to soil profiling for civil engineering purposes in Southern Africa' (Jennings *et al*, 1973). These descriptors are moisture condition, colour, consistency, structure, soil type and origin (or 'MCCSSO' for short). It will be seen from the paper that the *origin*, especially of residual soil horizons,* has a specific stratigraphic connotation. To state the intention of this thesis in a different way, it will attempt to demonstrate that a correct identification of the *origin* of a soil or rock horizon may lead to meaningful predictions concerning the engineering behaviour of that horizon.

It would be mistaken to believe that the above contention is a truism or is universally accepted. Very little emphasis is given to the significance of origin in the geotechnical literature of England, Europe and America and the inclusion of origin as a descriptor in soil profiling practice in these countries is the exception rather than the rule. One reason for this may be the presence of vast accumulations of unsorted glacial till in the soil profiles of many parts of these countries, and the relatively shallow occurrence of residual soils. By contrast there are no glacial tills in Southern Africa and the residual soil horizons often extend to great depth. This introduction will conclude presently with some further generalisations about the typical forms of Southern African soil profiles. But reference must first be made to the fact that, even within South Africa, some engineers, and even some geologists, have repeatedly questioned the value of recording the origin of soil and rock horizons in soil profiling, and have suggested that there is no benefit to be derived from including the stratigraphic identification of mapping units in soil-engineering maps. Thus it has been suggested that the *lithological* identification of the bedrock which forms the base of a succession of soil horizons in a soil profile or in a mapping unit will suffice for engineering purposes: as long as the bedrock under such a unit is listed in the legend to a map as 'sandstone', it is of no consequence to the engineer whether it is identified as 'Table Mountain sandstone', 'Daspoort sandstone', 'Ecca sandstone' or Cave sandstone'. The writer's strongly held conviction is that there is a world of difference between the engineering behaviour of sandstones (and other lithological types) of different age and different mineralogical compositions, by virtue of their different modes of origin. The residual soils to which they

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* Jennings *et al* (1973) advocate the use of the word 'residual' followed by the appropriate stratigraphic and lithological terms for denoting the origin of a residual soil. Thus a soil formed by *in situ* decomposition of Magaliesberg shale is, for the sake of brevity, recorded simply as 'residual Magaliesberg shale'. This convention is adopted throughout this thesis.

will give rise on decomposition will likewise possess distinctive engineering properties which are predictable, even if only at a broad level of generalisation, if the stratigraphic origin of the parent material has been correctly identified and recorded.

Finally, then, some further broad generalisations about the different typical forms of the soil profile in Southern Africa and the environmental factors responsible for their development:

(1) The climatic factor and its influence on the mode of rock weathering has already been mentioned. In very broad terms it may be stated that the soil profile contains deeper residual soils and a concentration of iron and aluminium oxides in the more humid zone east of the N = 5 line shown in Figure 1/1; and shallower residual soils with a concentration of soil carbonates in the more arid zone to the west of this line.

(2) Owing to the absence of Pleistocene glaciation the soils, both transported and residual, are preserved over large parts of the subcontinent.

(3) The erosive effect of episodic rainfall results in the local transportation of large quantities of soil and its deposition downslope, particularly in the more arid parts of the subcontinent.

(4) The absence of natural lakes and the consequent discharge of rivers straight into the ocean results in the generally shallow depth of water-transported soils, particularly in the more humid parts. Notable exceptions to this are the deep estuarine deposits near the mouths of major rivers where gorges as much as 60 m deep which were carved during Pleistocene eustatic periods of low sea level has subsequently become choked with substantial accumulations of sediment as the sea level rose. A further notable exception is the great depth of transported soil within the vast Kalahari Basin.

(5) The most characteristic feature of the soil profile in Southern Africa is the presence, in both humid and arid zones, of a *pebble marker* sandwiched between the transported soils and the residual soils. Attention is drawn to the description of this important feature, in the revised guide to soil profiling referred to above (Jennings *et al*, 1973).

2 THE BASEMENT-COMPLEX

METAMORPHIC BODIES OF THE "PRIMITIVE" AND LATER SYSTEMS

Treatment of the engineering geology of the metamorphic bodies within the Basement-complex will be largely confined to broad generalisations about occurrences of these materials near Johannesburg and in Rhodesia and Zambia.

Little is known about the engineering characteristics of rocks of the Kheis System in Namaqualand and South West Africa, beyond the fact that the high degree of foliation, coupled with advanced mechanical disintegration, often cause them to be vulnerable to slope stability problems in open cuts. On the whole, the arid climatic environment is not conducive to the development of residual soils, so that foundation problems would be confined to the transported soil zone. That is, however, not invariably the case. Phyllites of the Gariep System in the Namib Desert in the south-western part of South West Africa locally show kaolinisation to depths of up to 50 m beneath remnants of silcrete cappings where these are preserved on the well-planed African erosion surface of the west coast monocline*. These deep residual clays may well be associated with problems of low bearing capacity.

Within the eastern half of the Republic, the only extensive occurrences of ancient rocks of the Primitive Systems are in the Barberton Mountain Land, the Murchison Range and the Messina area.

Relatively little engineering development has taken place in these areas, and consequently little is known of the engineering problems that they may present. Being elevated masses for the main part, it is unlikely that residual soils would be deep, but it may be predicted with confidence that clayey soils developed on the mafic and ultramafic rocks, even if they are shallow, would exhibit expansive characteristics. This is found to be the case in the two occurrences of the Swaziland System on the southern margin of the Halfway House granite, one to the

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* Personal communication from Dr T.C. Partridge, 1973

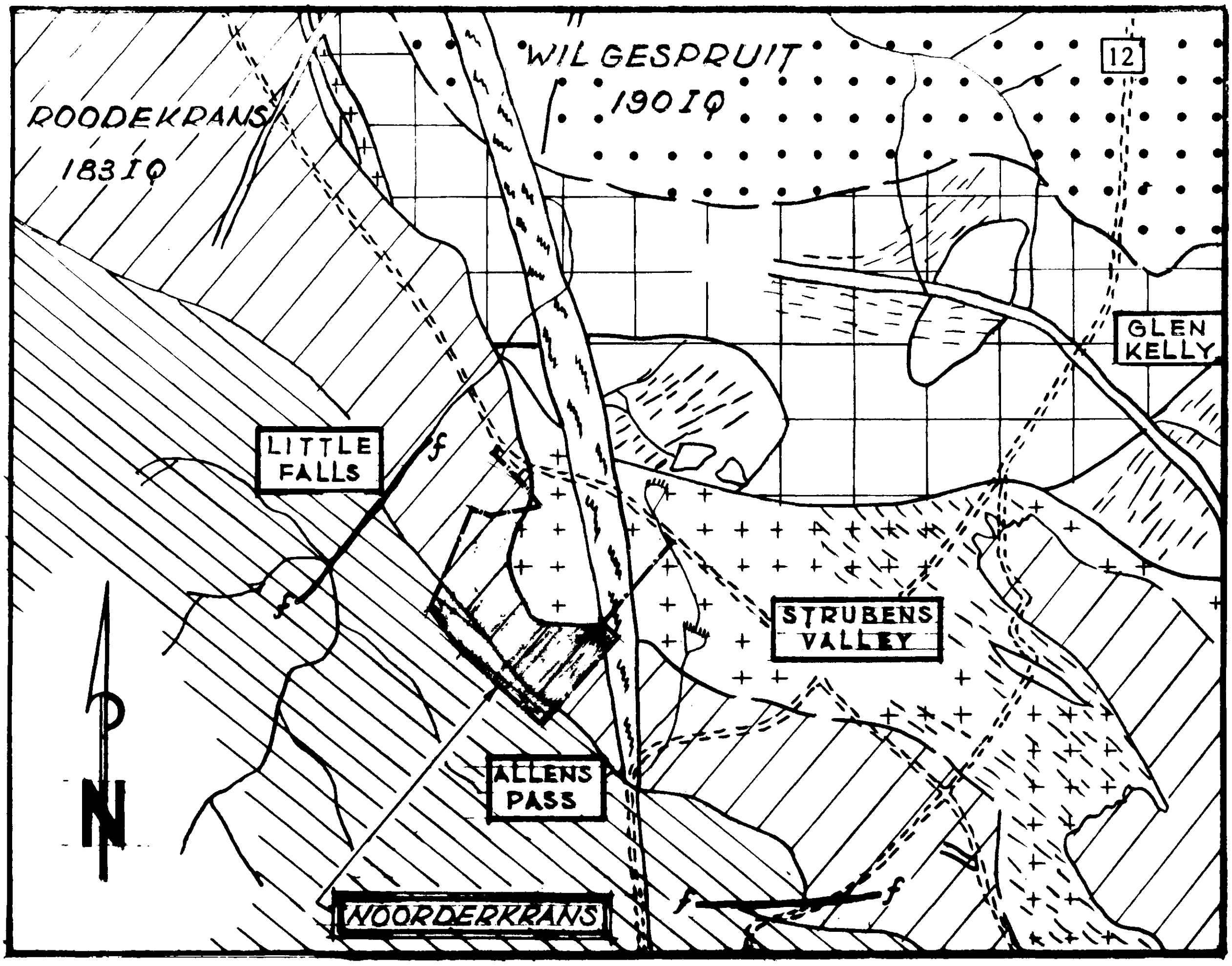
west of Muldersdrift and the other south of Modderfontein (which are erroneously shown on the 1970 geological map of the Republic as "Jamestown Igneous Complex"). Heavy expansive clays, which are also notably fertile, are developed on the 'greenstones', i.e. the amphibolites, serpentinites and phyllitic rocks. These residual soils are highly variable in thickness, even over short distances; they are seldom more than 3 m thick except where they have been protected from erosion on remnants of the African erosion surface or on exhumed remnants of the pre-Karoo erosion surface, where they may extend to depths of more than 20 m.

In Zambia and Rhodesia the ancient metamorphic rocks are of particular engineering importance. It is largely within these rocks that gold and copper mineralisation has taken place, and consequently most of the towns are located upon them. In the subtropical environment of Rhodesia these rocks are often found to be decomposed into a clayey or sandy, micaceous silt to variable depths, in places up to 40 m or more, while, under tropical conditions in the Zambian Copperbelt, decomposition often extends to well below the water-table, to depths of 75 m or more.

The engineering characteristics of soils formed by the advanced chemical decomposition of these ancient metamorphic rocks will be discussed under the headings of the four most significant lithological types, viz. "greenstones" (near Johannesburg), mica schists (Zambia), phyllites (Rhodesia) and metagabbros (Salisbury, Rhodesia).

RESIDUAL GREENSTONES

The occurrence of deep residual greenstone soils described here is situated at Noorderkrans, about 15 km west of the central part of Johannesburg and 4 km north-east of the Roodepoort town centre. The geological setting of the area, which is within the Swaziland System on the south-western margin of the Johannesburg-Pretoria granite inlier, is shown in Figure 2/1. A mafic and ultramafic suite of rocks consisting of serpentinites, amphibolites and basic schists has been recognised here by Anhaeusser (1971), on whose mapping Figure 2/1 is based. For the purpose of the present study this suite of parent-rocks from which the local residual soils have been formed has been given the general sack-term title of "greenstones". The greenstones represent a primitive



LEGEND

MAFIC DYKES (DIABASE AND DOLERITE).

PILANESBERG DYKES:(COMPOSITE DIABASE AND COARSE PORHYRITIC QUARTZ MONZONITES AND SYENITES) DYKES PRODUCE PINKISH-RED STAINING IN IMMEDIATELY ADJACENT GRANITES.

CRUSH-OR SHEAR-ZONES: QUARTZ VEINS, QUARTZ SERICITE SCHISTS AND SHEARED QUARTZ AUGEN-SCHISTS.

WITWATERSRAND SYSTEM UNDIFFERENTIATED QUARTZITES, AURIFEROUS CONGLOMERATES, SHALES AND BANDED IRON FORMATIONS.

HOMOGENEOUS MEDIUM-GRAINED GREY PORPHYRITIC GRANODIORITE SUITE; PEGMATITES COMMON.

MEDIUM TO COARSE-GRAINED PINKISH-GREY HOMOGENEOUS GRANODIORITES; PEGMATITES RARE.

HORNBLENDE AND/OR BIOTITE GNEISSES, HYBRID MAFIC, ULTRAMAFIC AND GRANITIC ROCKS, DIORITIC GNEISSES.

SWAZILAND SYSTEM ;UNDIFFERENTIATED MAFIC AND ULTRAMAFIC PLUTONIC AND VOLCANIC ROCKS NOW MAINLY ALTERED TO SERPENTINITES, A VARIETY OF AMPHIBOLITES (HORNBLENDE TREMOLITE-ACTINOLITE) AND CHLORITE TALC, TALC CARBONATE AND TALC CHLORITE SCHIST. HYBRID GRANITIC ROCKS DEVELOPED IN PLACES.

FOLIATION TRENDS

FIGURE 2/1 : Geology of the south-western margin of the Johannesburg-Pretoria granite inlier showing the gological setting of Noorderkrans (After Anhaeusser, 1971) crustal remnant, intruded by Archaean granite and overlain unconformably by the Orange Grove quartzite of the Witwatersrand System.

The study area is situated on the northern slope of the watershed formed by the linear range of koppies capped by quartzites of the Lower Division of the Witwatersrand System. The crests of the koppies represent part of the pre-Karoo erosion surface which was exhumed during the African cycle of erosion. The resistant Orange Grove quartzite has locally protected much of the residual greenstone soil from erosion during the currently operating post-African cycle of erosion, which accounts for the preservation of an exceptionally thick (> 20 m) mantle of residual soil below the north-facing talus slope and even below the upper portions of the pediment. The same phenomenon has been observed elsewhere along the same escarpment, even where the soils are residual from granite (see Hospital Hill Series in Chapter 4).

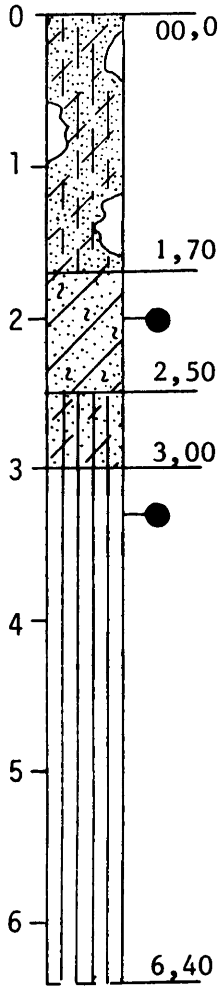
Calculated surface gradients of the local terrain units, from south to north, are given in Table 2.1:

Altitude	Terrain Unit	Surface Gradient
Above 1 730 m	crest on quartzite	10% - 20%
	free face on quartzite	80% - 100%
1 730 - 1 610 m	talus slope on quartzite	15% - 30%
Below 1 610 m	pediment on greenstone	5% - 10%
1 556 m	hydraulic gradient on stream channel (Wilgespruit)	2%

TABLE 2.1 : Surface gradients at Noorderkrans

From a total of 37 trial-holes augered in the 135 hectare study area, three representative soil profiles have been selected for reproduction: profile NK1 is typical of the talus slope (Figure 2/2), NK2 is typical of the junction between talus slope and pediment (Figure 2/3), and NK3 is typical of the pediment (Figure 2/4). (It is of interest to note, in passing, the increase in thickness of the overlying hillwash, and the decrease in thickness of the pebble marker, as one progresses downslope from an elevation of 1 620 m to an elevation of 1 570 m).

NK 1



Coarse, medium and fine, sub-angular to angular, quartz, quartzite and phyllite gravels and quartzite boulders, in a matrix of moist, light brown, in profile dark reddish brown, loose to firm, clayey silty medium and fine sand with roots; talus pebble marker.

Moist, orange brown, in profile dark reddish brown indistinctly blotched reddish orange, firm, micro-fissured, sandy clay with roots; reworked residual greenstone.

As above, but predominant colour reddish orange and a clayey silt.

Moist, dark yellow, in profile, yellow blotched and streaked black and reddish orange, firm, jointed (stained black), silt; residual greenstone.

NOTES

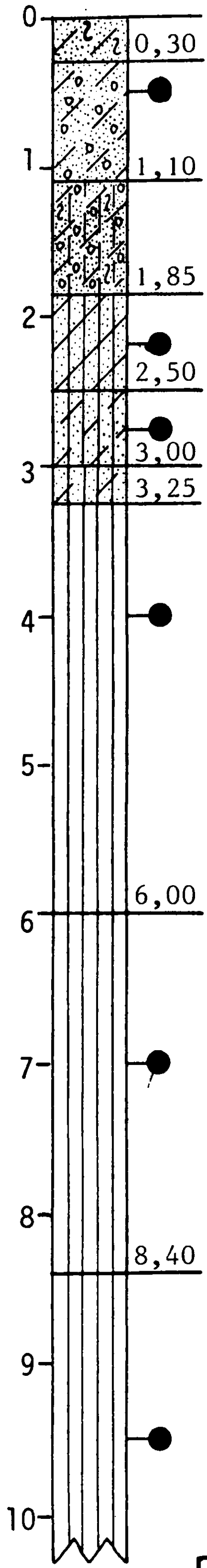
1. Hole augered to 6,40 m with Hughes LDH 120 Digger but not to refusal.
2. Water table not encountered.
3. Disturbed soil samples taken at 2,00 m and 3,30 m.

GBM/HAW
21/3/75

FIGURE 2/2 : Typical soil profile on talus slope at Noorderkrans

Surface elevation of trial-hole 1 620 m

NK 2



Slightly moist, dark brown, in profile dark brown, stiff, intact, clayey medium and fine sand with roots; hillwash.

Moist, brown, in profile reddish brown, firm, intact, clayey medium and fine sand with occassional medium quartz gravels; hillwash.

Coarse, medium and fine, angular, quartz and quartzite gravels scattered in a matrix of moist, brown, in profile brown, firm, intact, silty, clayey, coarse, medium and fine sand with few fine roots; pebble marker.

Moist, orange brown, in profile reddish orange blotched yellowish orange and brown, soft, intact, slightly sandy clay-silt; reworked residual greenstone.

As above but buff blotched reddish orange and brown, clayey silt; residual greenstone.

As above but in profile pale green streaked reddish brown; residual greenstone.

Moist, dark yellow, in profile dark yellow blotched pale red, firm, jointed (stained pale red and near vertical disposition), silt; residual greenstone.

Moist, buff, in profile buff, soft, jointed (stained black and pale red with near vertical disposition), silt; residual greenstone.

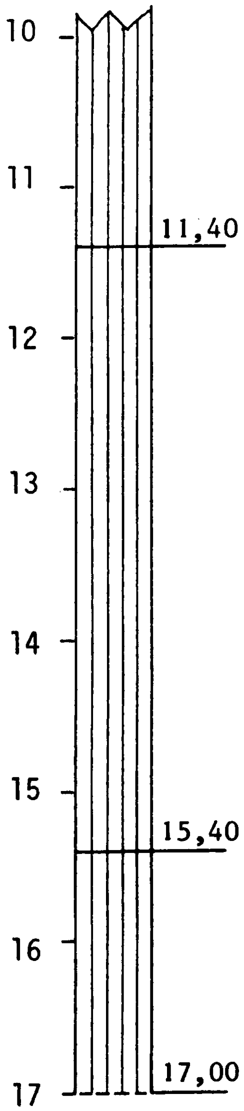
Moist, buff, in profile buff to dark yellow blotched and streaked pale red, occasionally thinly banded light yellow, stiff becoming stiffer with depth, jointed (stained pale red and black), silt; residual greenstone.

FIGURE 2/3 : Typical soil profile at junction of talus slope and pediment at Noorderkrans

Surface elevation of trial-hole : 1 610 m

(Continued on next sheet)

NK 2 (Continued from previous sheet)



Slightly moist, light olive, in profile pale green blotched light reddish brown, stiff with occasional pockets of firm to soft material, jointed (stained black), silt; residual greenstone.

Slightly moist, green, in profile dark green indistinctly banded pale green, stiff to very stiff, jointed (stained black), silt; residual greenstone.

NOTES

1. Hole augered to 17,00 m with Hughes LDH 120 Digger but not to refusal.

2. Water table not encountered.

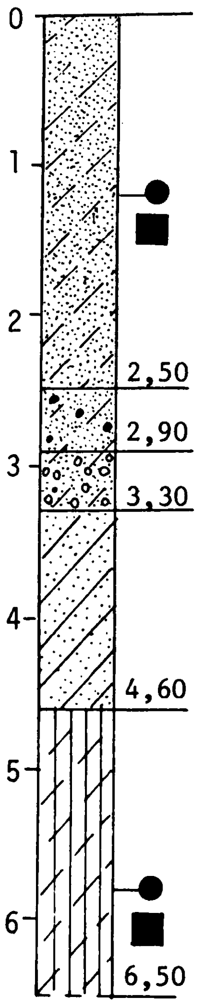
3. Horizon between 6,00 m and 8,40 m breaks readily along near vertical poorly defined planes and is seen to form a V-shaped body about ½ m thick when exposed on side wall. Arms of V as observed about 1,5 m wide.

4. Disturbed soil samples taken at 0,50 m, 2,20 m 2,75 m 4,00 m, 7,00 m, 9,50 m and 12,50 m.

GBM/OMW
26/3/75

FIGURE 2/3 (contd) : Typical soil profile at junction of talus slope and pediment at Noorderkrans

NK 3



Moist, brown, in profile dark reddish brown, firm, intact, clayey, medium and fine sand with roots; hillwash.

As above but slightly moist and firm to stiff with ferruginised soil nodules, lightly ferruginised hillwash.
Coarse, medium and fine, subangular to subrounded, quartz and quartzite gravels in a matrix of stiff, densely ferruginised hillwash; densely ferruginised pebble marker.

Moist, brown, in profile dark red blotched yellowish and greyish brown and black, firm, micro-fissured, sandy clay; reworked residual greenstone.

Moist, orangish brown, in profile reddish orange speckled and blotched buff, yellow and brown, very soft to soft, jointed, clayey silt; residual greenstone.

NOTES

1. Hole augered to 6,50 m with Hughes LDH 120 Digger but not to refusal.
2. Water table not encountered.
3. Undisturbed and disturbed soil samples taken at 1,20 m and 5,80 m.

GBM/HAW
20/3/75

FIGURE 2/4 : *Typical soil profile on pediment at Noorderkrans*

'Surface elevation of trial-hole : 1 570 m

At the top of the residual zone, i.e. immediately below the pebble marker, the decomposed greenstone has been reworked by biotic action. This is thought to be due largely to the activity of termites. This reworking is reflected by a dark red or reddish brown colour (the colour of the overlying hillwash and pebble marker matrix) and the presence of a sand fraction which is absent from the underlying undisturbed residual soil.

The upper horizons of the residual greenstone which has not been biotically reworked have the texture of a clayey silt owing to the advanced degree of chemical decomposition. At greater depth the texture is that of a pure silt. It will be seen from Figure 2/3 that hole NK2 was drilled to a depth of 17 m without encountering refusal of the Hughes LDH 100 Digger. It is on this evidence that the depth of residual soil is assumed possibly to exceed about 20 m.

The range of indicator properties of the residual greenstone and the reworked residual greenstone are given in Table 2.2:

	Reworked Soil	Residual Soil
No. of Samples Tested	19	34
Liquid Limit	40 - 64%	45 - 59%
Plasticity Index	15 - 29%	10 - 29%
Linear Shrinkage	5 - 19%	5 - 14%
Clay Fraction	20 - 44%	10 - 29%
Unified Soil Class	CH	CH and MH-ML
Mean Expansiveness	High	Medium

TABLE 2.2 : Indicator properties of residual greenstone from Noorderkrans

Applying the method of Van der Merwe (1964b) to the generalised soil profile developed on these greenstones (in which the transported soils are non-expansive) a total potential heave of 40 mm is calculated. Owing to large variations in both the thickness of individual horizons and their degrees of expansiveness, however, it was calculated that total heave would probably vary from about 25 mm to 50 mm.

The undisturbed sample of residual greenstone taken at a depth of 5,8 m in trial-hole NK3 had a bulk density of 1 592 kg/m^3. The sample was subjected to consolidometer testing under saturated conditions and showed a normal consolidation settlement of 3,6 per cent when loaded to 200 kPa. Under loadings for single-storey structures (i.e. about 50 kPa) founded directly on the residual greenstone, a normal consolidation settlement of the order of 5 mm is calculated.

RESIDUAL MICA SCHISTS

Owing to the high degree of leaching, residual mica schists on the Zambian Copperbelt have very low *in situ* densities, commonly less than 1 500 kg/m^3 and, in places, as low as 1 000 kg/m^3.

They are fairly highly compressible in spite of having been subjected to high preconsolidation pressures, and they are characterised by an unusually high coefficient of consolidation, i.e. their *rate* of consolidation is abnormally rapid. Coefficient of consolidation (C_v) values determined on a large number of these soils from Ndola and Kitwe ranged from 1 to 5 mm^2/sec, for applied foundation pressures in the range 375 to 575 kPa. At the same time they often exhibit expansive characteristics of a moderate order, as illustrated by the plots of Atterberg Limits in Figure 2/5 (Kantey and Brink, 1952).

The mineralogy and the fabric analysis of these soils throw light on their engineering behaviour. The fresh schist consists essentially of muscovite, plagioclase felspar and quartz, the muscovite flakes being oriented parallel to the planes of schistosity. By virtue of the stress conditions obtaining at the time of metamorphism, the schistosity is usually very steeply inclined or even vertical. Decomposition of the plagioclase felspar gives rise initially to expanding 2:1-lattice clays which predominate in the lower horizons of the profile and account for the heaving potential of the residual soils. In the more highly leached upper horizons these clays have been converted into the non-expanding 1:1-lattice clay of the kaolinite group. Consequently one finds that the Atterberg Limit values and the Activity (Plasticity Index divided by clay content i.e. percentage less than 2 micrometres) increase with depth in spite of a progressive decrease in clay content (Van der Merwe, 1955).

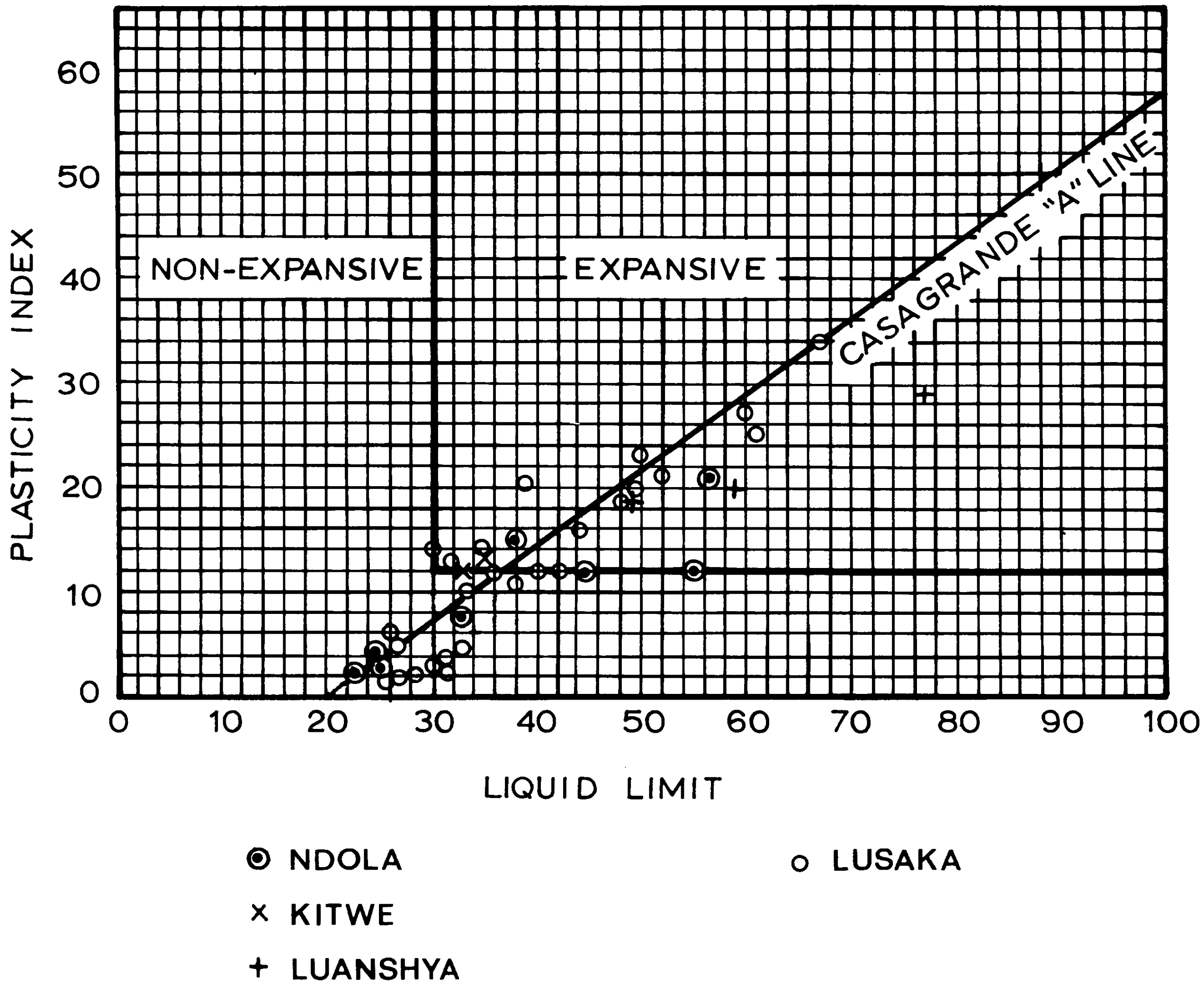


FIGURE 2/5 *Atterberg Limit plots for samples of residual mica schist in Zambia from depths between 3 metres and 10 metres*

(After Kantey and Brink, 1952)

Under intense tropical weathering some of the muscovite breaks down to form clay, but much of it remains practically unaltered and retains its steeply inclined orientation. The unusual pattern of consolidation under load, with the major part of the consolidation taking place very rapidly, may be attributable to the reorientation of mica flakes rather than to consolidation of the soil as a whole.

In a study on residual mica schists from Luanshya, Mackecnhie (1967) concluded that "the amount of consolidation is greatest at the lowest angle of dip of the schistosity and least at the highest angle of dip. The difference, however, is not large and is least in the most highly leached state". In this study the high rate of consolidation was attributed to the relatively high permeability considered to be associated with the schistose structure, and to possible planes of preferential drainage existing in the soil structure.

Figure 2/6 shows strength lines obtained by Blight (1963) from both triaxial compression and triaxial extension tests performed on saturated and partly saturated samples of decomposed mica schist. The samples were taken from a depth of about 75 m below surface at the Nchanga opencast copper mine. Samples tested in extension, in which the radial stress is the major principal stress and the axial or minor principal stress is decreased until failure occurs, failed *along* foliation planes, whereas those tested in compression failed *across* the foliation planes. The variability in properties from one sample to the next has accordingly affected the results of the extension tests, and has caused a considerably greater scatter than in the case of the compression tests.

These test results give an idea of typical values of shear strength for this type of material, particularly the $\emptyset$ values. Cohesion values have been found to decrease with a progressively increasing degree of decomposition, but the degree of decomposition has little effect on the $\emptyset$ values.

Trial-holes put down in parts of Tzaneen, northern Transvaal, in isolated patches of decomposed mica schist within the Basement-granite, showed a minimum depth of 7 m of residual soil which exhibited properties similar to those of the soils described above.

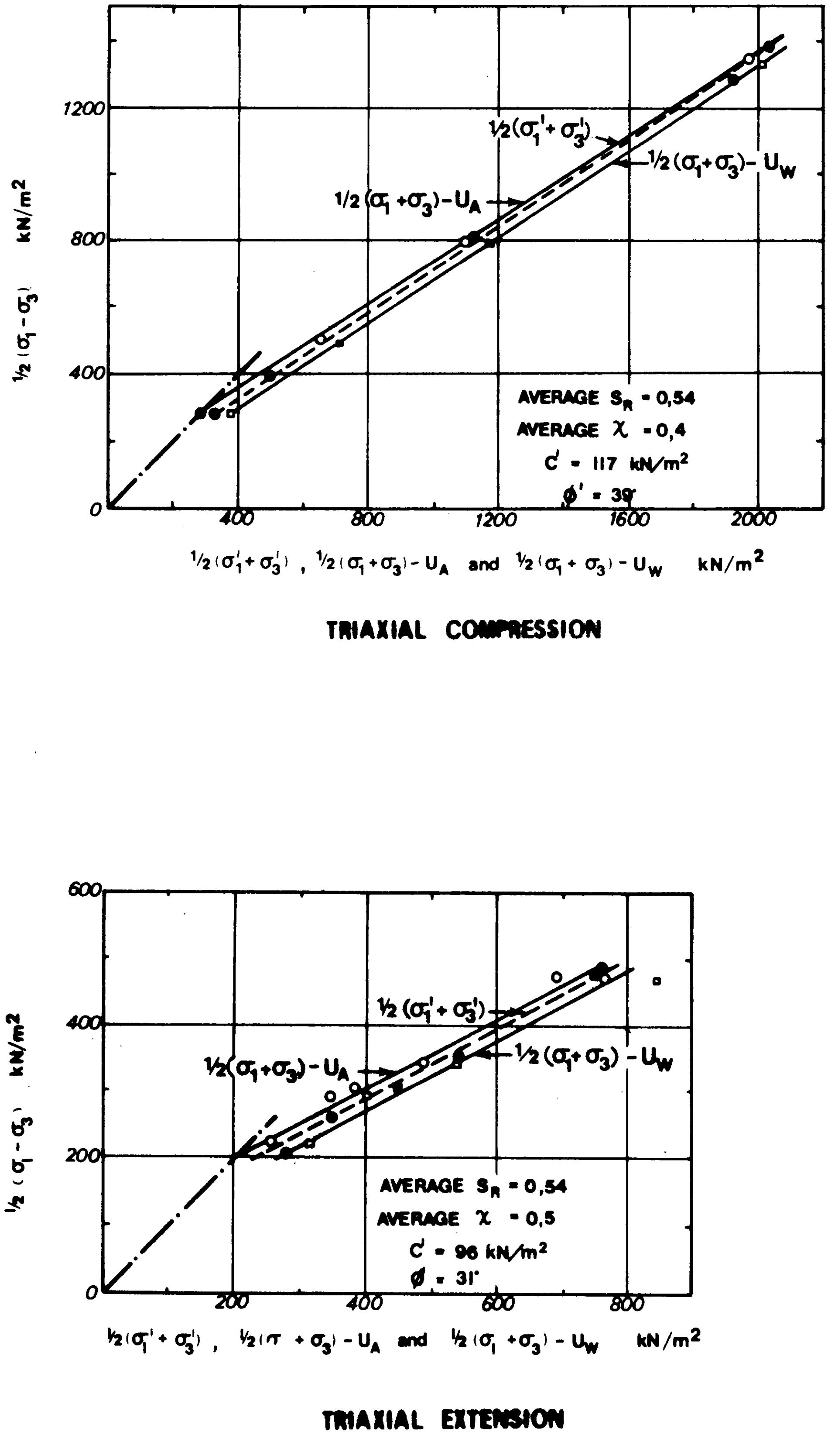


FIGURE 2/6 : Strength lines for triaxial compression and extension tests on saturated and partly saturated samples of residual mica schist (After Blight, 1983)

The most satisfactory solution to the founding of heavy structures in these soils has been the use of piled foundations. Where the soils are highly expansive, and the profile deep, it is necessary to found in the zone of stable moisture content below the water-table, due consideration being given to the seasonal fluctuation in the level of the water-table. The saturated soils below the water-table are invariably more highly compressible than the partially saturated soils above this level, and it is the consolidation, or settlement, characteristics rather than the shear strength of the material that determines the bearing pressures which can safely be used for end-bearing piles. However, using an average value of C_v of 3 mm^2/sec, it can be deduced that 90% of the total consolidation, under an end-bearing pile pressure of 475 kPa, will take place within four months of the foundation loads being applied. In practice, therefore the major part of the settlement takes place during construction, and only minor differential settlement occurs after completion of the structure and application of finishes.

RESIDUAL PHYLITE

The foundation problems associated with residual soils formed by the decomposition of phyllitic rocks are in many ways similar to those described above. In general these soils are somewhat less compressible and, while still possessing an unusually high coefficient of consolidation, the rate at which consolidation takes place is slower than in residual mica schists. The upper horizons of the residual soils are commonly highly expansive but, unlike the residual schists, the degree of potential expansiveness decreases rapidly with depth. Hard bands of quartzite and banded ironstones are frequently intercalated with the phyllites and give rise to extreme contrasts in founding conditions, even within the confines of a small construction site. Whereas the quartzites are seldom found to be decomposed, the phyllites may be decomposed to great depth. Salisbury is largely underlain by phyllites of the Iron Mask Series, which have been known to produce residual silts and clayey silts to depths of as much as 40 m.

The phyllites, and the soils formed on them, are highly variable. This applies even within a single soil profile, as illustrated by the consolidation characteristics of soil samples taken from one trial-hole shown in Figure 2/7.

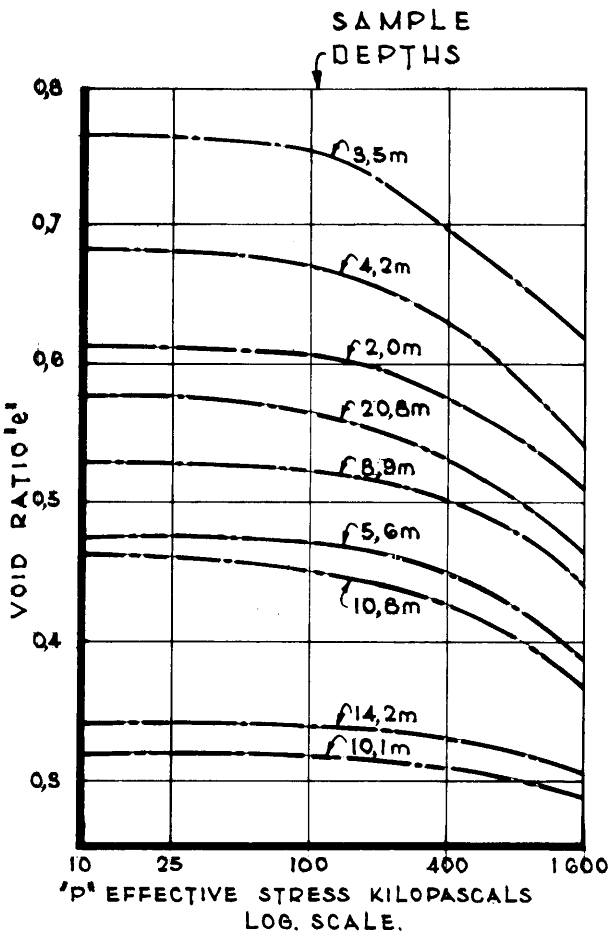


FIGURE 2/7 Consolidation characteristics of samples from a single soil profile in leached phyllite

(After Macknechnie, 1975)

The two main factors responsible for this are lithological variations in the parent-rock and the topographic situation upon which the soil profile has developed. In Salisbury, Machechnie (1975) has recognised two distinct lithological forms of phyllite: (i) a dark grey or blue schistose rock which is often present in conformable association with banded ironstones and quartzites and which provides generally sound foundation conditions, and (ii) a deeply weathered and highly leached material, ranging in colour from white and red through to dark blue, and containing bands of basic igneous material usually in the form of epidiorite or amphibolite. The basic igneous material in the latter type is present in a random and often lenticular form, the lenses pinching out in both vertical and horizontal directions. Sulphide mineralisation, in the form of pyrite and pyrrhotite, is common in the latter type. In places these minerals are found in substantial quantities. Whereas a diamond-drill core of unmineralised phyllite may have a typical density of 2 800 kg/m^3, mineralised cores may have densities exceeding 3 200 kg/m^3. Being themselves highly vulnerable to oxidation and hydration, the sulphides are responsible for the generation of salts and acids which promote weathering, particularly along the planes of schistosity on which the mineralisation is concentrated, to produce residual soils as deep as 30 m to 40 m with densities as low as 1 600 kg/m^3.

The problem of differential settlement can be particularly severe where a structure straddles the contacts between the sound blue phyllite and the weathered paler variety. Professor Machechnie (1975) comments that, whereas structural engineers often show a marked reluctance to mix foundation types, he would not hesitate, with his current experience of the Salisbury phyllites, to mix spread and piled foundations in the same structure, provided, of course, that he had assured himself that the sounder areas where spread footings were proposed were confirmed as such by adequate proof-drilling.

The other major factor controlling the type of soil profile developed on the phyllites is the topographic control. A more advanced degree of weathering and a greater depth of residual soil are observed in topographic situations favouring good internal drainage and decomposition by mildly acidic leaching waters. Non-expanding 1:1-lattice clays predominate (though, in some localities, the upper horizons of the redisual zone may be expansive) and the highly leached soils are compressible. The converse is true for areas of impeded drainage which produce waterlogging in a reducing and alkaline environment. The soils developed under such conditions are highly expansive but of limited thickness. The presence of highly plastic vlei-soils at the surface inhibits the rate of weathering at depth. Smectite clays predominate. Typical soil profiles corresponding to these two extremes of internal drainage conditions are illustrated in Figure 2/8: these examples are based on published data by Mackechnie (1975), modified by the writer's own observations.

RESIDUAL METAGABBRO

Though of relatively limited extent within the greenstone belts of Rhodesia and elsewhere, the metagabbro is of crucial significance in Salisbury as it outcrops over an area of about 25 sq km in the heart of the city, immediately to the north-west of the central business district, as shown in Figure 2/9.

The metagabbro is intrusive into the phyllites, or metamorphosed mudrocks, of the Basement-complex. The intrusion of the gabbro probably took place prior to the granitisation of the host rock (Tyndale-Biscoe, 1957).

The outcrop area of the metagabbro in Salisbury is manifested by the presence of weathering spheroids at the surface. The topographic control on soil profile development is again as marked as it is in the case of the phyllites. On well-drained slopes where acidic decomposition is operative, deep, highly leached, red silt profiles develop to a depth of 10 or 15 metres. In situations of impeded drainage where alkaline decomposition operates, heavy black plastic soils rich in 2:1-expanding lattice clay-minerals predominate, and decomposition seldom extends to depths greater than about 6 metres below surface. Typical soil profiles, based largely on the work of Mackechnie, are shown in Figure 2/10.

From his widespread experience in site investigation work in Salisbury, Professor Mackechnie (1975) concludes that "development of the central business district should be aimed, for foundation engineering rather than purely planning reasons, at the remaining open space in which the gabbro occurs and thereafter that redevelopment for multi-storey purpose should take place in the north-central area which is underlain by the gabbro rather than on the phyllites".

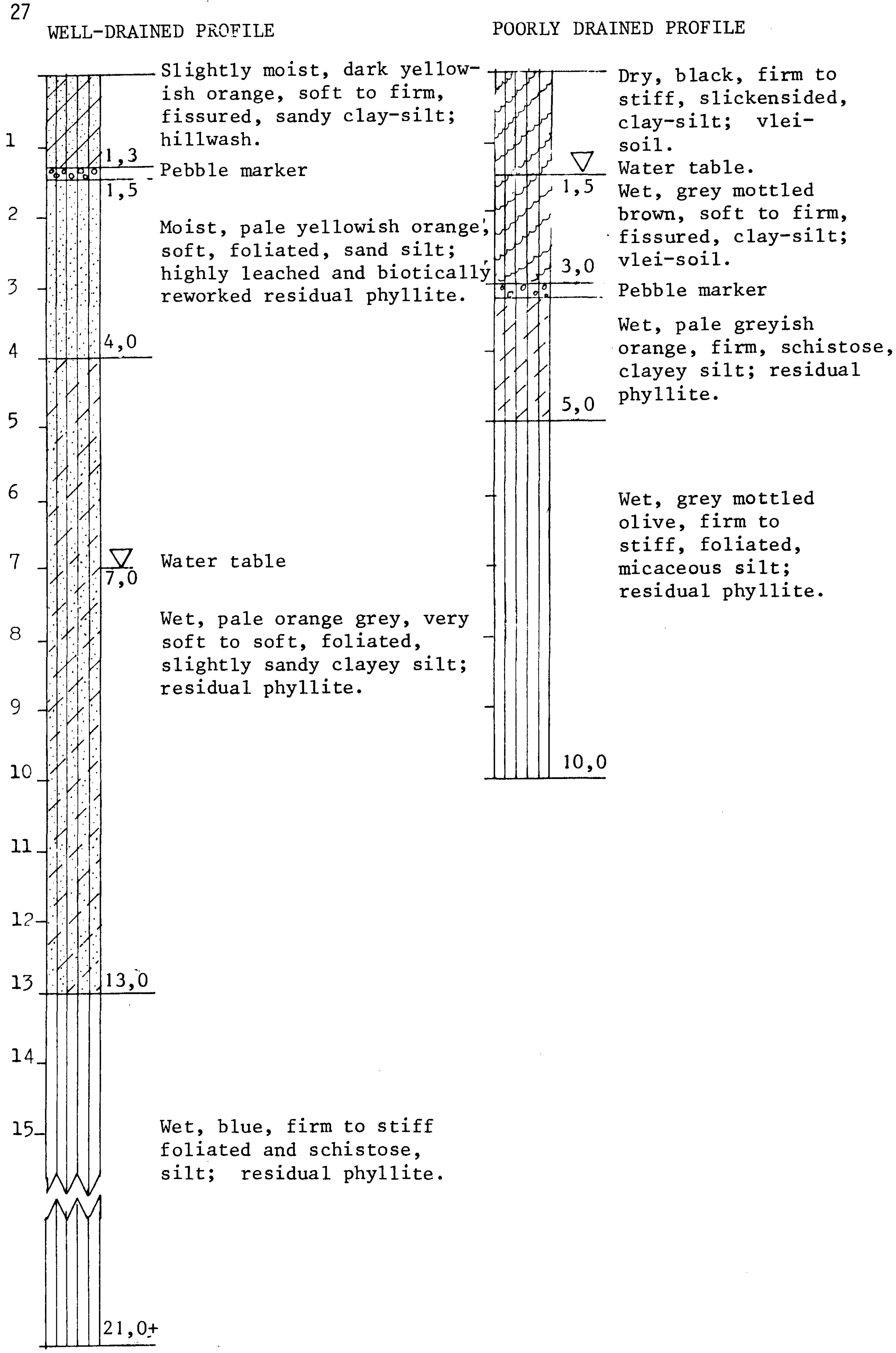


FIGURE 2/8 : Typical soil profiles developed on phyllite of the Iron Mask Series in well drained and poorly drained topographic situations in Salisburg, Rhodesia

(Largely after Mackechnie, 1975)

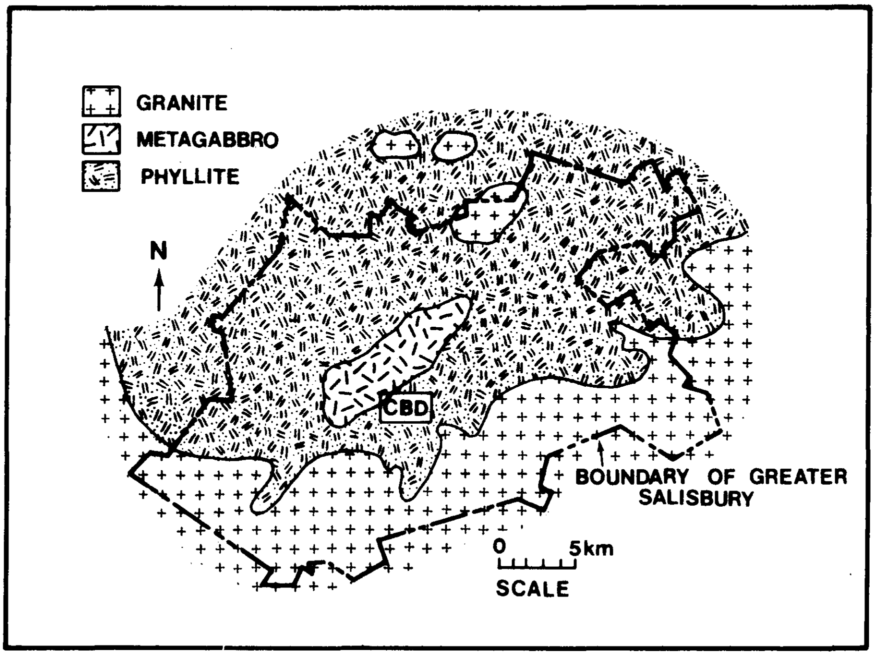


FIGURE 2/9 : Simplified geological map of Salisbury

(After Mackechnie, 1975)

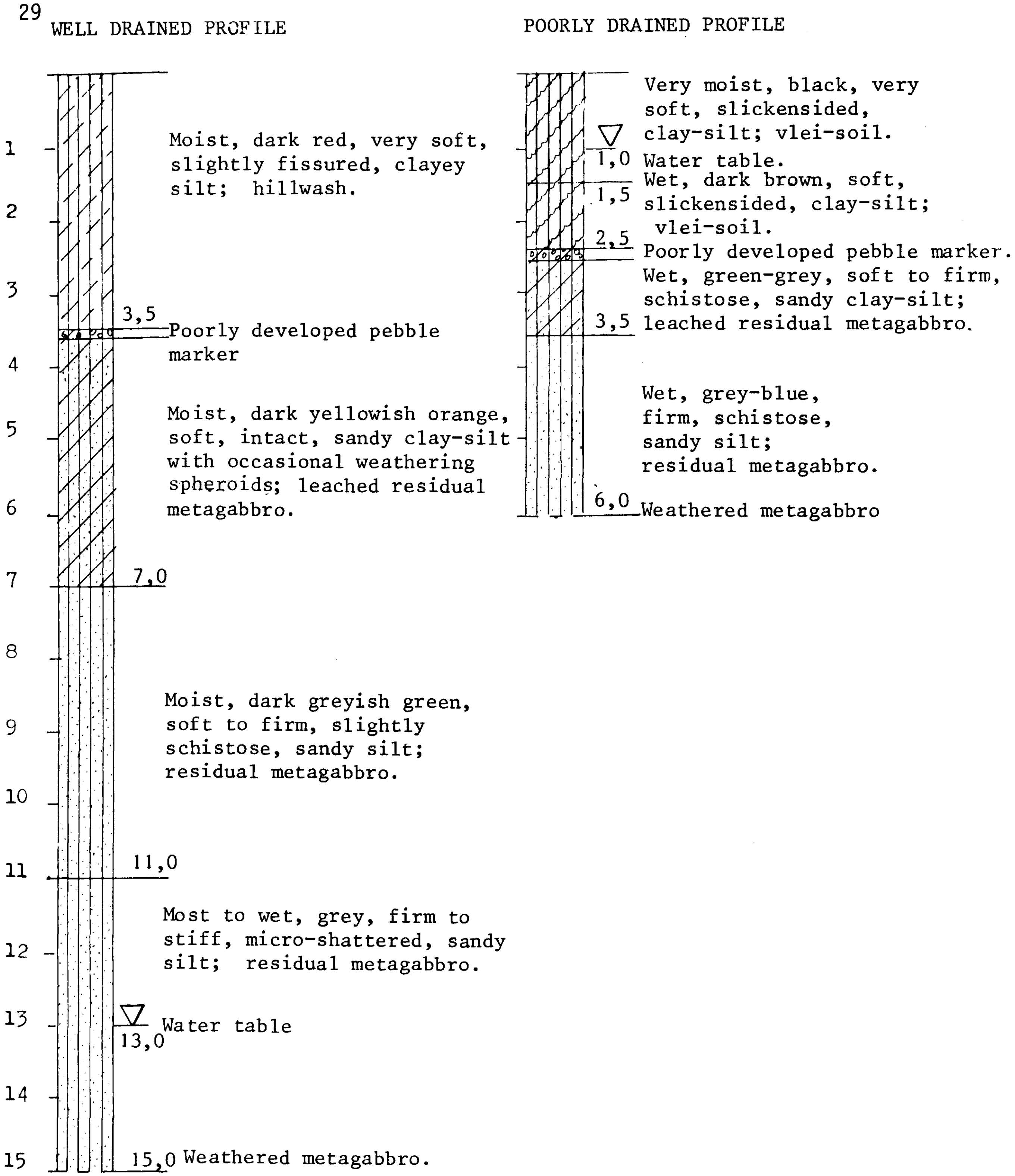


FIGURE 2.10 Typical soil profiles developed on metagabbro in well drained and poorly drained topographic situations in central Salisbury, Rhodesia

(Largely after Mackechnie, 1975)

CASE HISTORY I

RESIDUAL MICA SCHIST

SITE FOR COMPLEX OF BREWERY BUILDINGS AND INSTALLATIONS: NDOLA, ZAMBIA

A geotechnical investigation carried out at the above site during 1957 revealed soil profiles which have been generalized in Figure 2/11.

Atterberg Limit determinations indicated slightly expansive characteristics within the lateritic zone above the pebble marker. Tests on seven samples of the residual schist from different depths gave a range in Liquid Limit from 23 to 57, in Plasticity Index from 2 to 21, in Linear Shrinkage from 2 to 7 and in clay content from 4 to 19 per cent, indicating a very low degree of potential expansiveness.

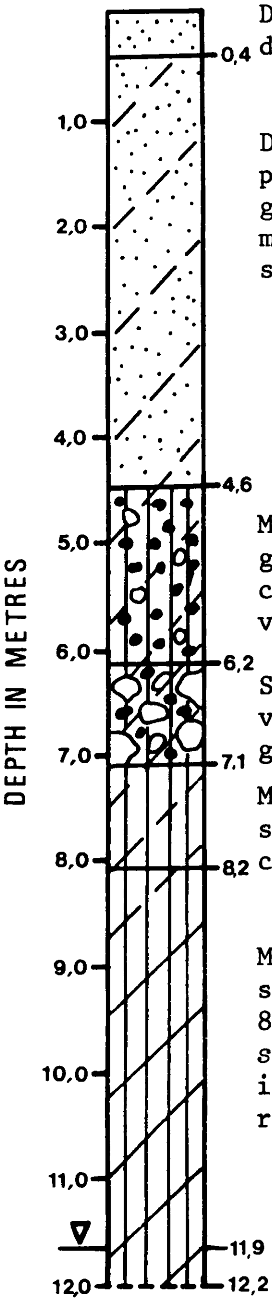
Triaxial shear and consolidation tests were carried out on undisturbed samples of residual soil from depths 8,5 m and 10,5 m, typical results of which are reproduced in Figures 2/12 and 2/13 respectively.

Based on these findings, three alternative forms of foundation treatment were considered:

* raft foundation at depth 5 m in lateritic zone, at bearing pressure 275 kPa and with predicted settlement of the order of 50 mm;

* under-reamed piles founded at depth 8,5 m: triaxial shear tests indicated bearing capacity of 525 kPa, but consolidation characteristics indicated settlement of 40 mm (most of which would take place during construction) under a loading of 275 kPa;

* under-reamed piles founded at depth 10,5 m: settlement of 30 mm calculated for bearing pressure of 430 kPa, most of which would take place during the course of construction, with negligible differential settlement after application of finishes.



Dry, light brown, in profile yellowish brown, medium dense, intact, silty sand; leached aeolian sand.

Dry becoming progressively moister with depth; orange, in profile brownish orange, medium dense at top becoming progressively looser with depth, intact, clayey, fine and medium sand; altered aeolian sand (with collapsible grain structure).

Moist, reddish brown, in profile red mottled yellow, grey and brown, firm to stiff, slightly shattered, clayey silt with ferruginous concretions and fine vein-quartz gravel; lateritic zone.

Subangular, medium and fine gravel-sized fragments of vein-quartz and quartzite, in matrix of moist, ferruginised, red and yellow clayey silt; pebble marker.

Moist, buff, in profile grey mottled red and yellow, soft to firm. slightly shattered, weakly ferruginised clayey silt; transition zone.

Moist, buff, in profile grey streaked yellow and red, soft to firm, foliated, with schistosity dipping about 85^{o}, highly micaceous clay-silt, formed by advanced *in-situ* decompostion of mica schist (Muva System), with intercalations of friable, decomposed quartzitic seams; residual Muva schist.

NOTES:

1. Hole augered with 750 mm diameter bucket of calweld rig, but not to refusal.
2. Water-table encountered at 11,90 m.

FIGURE 2/11 : Generalised soil profile for brewery site, Ndola

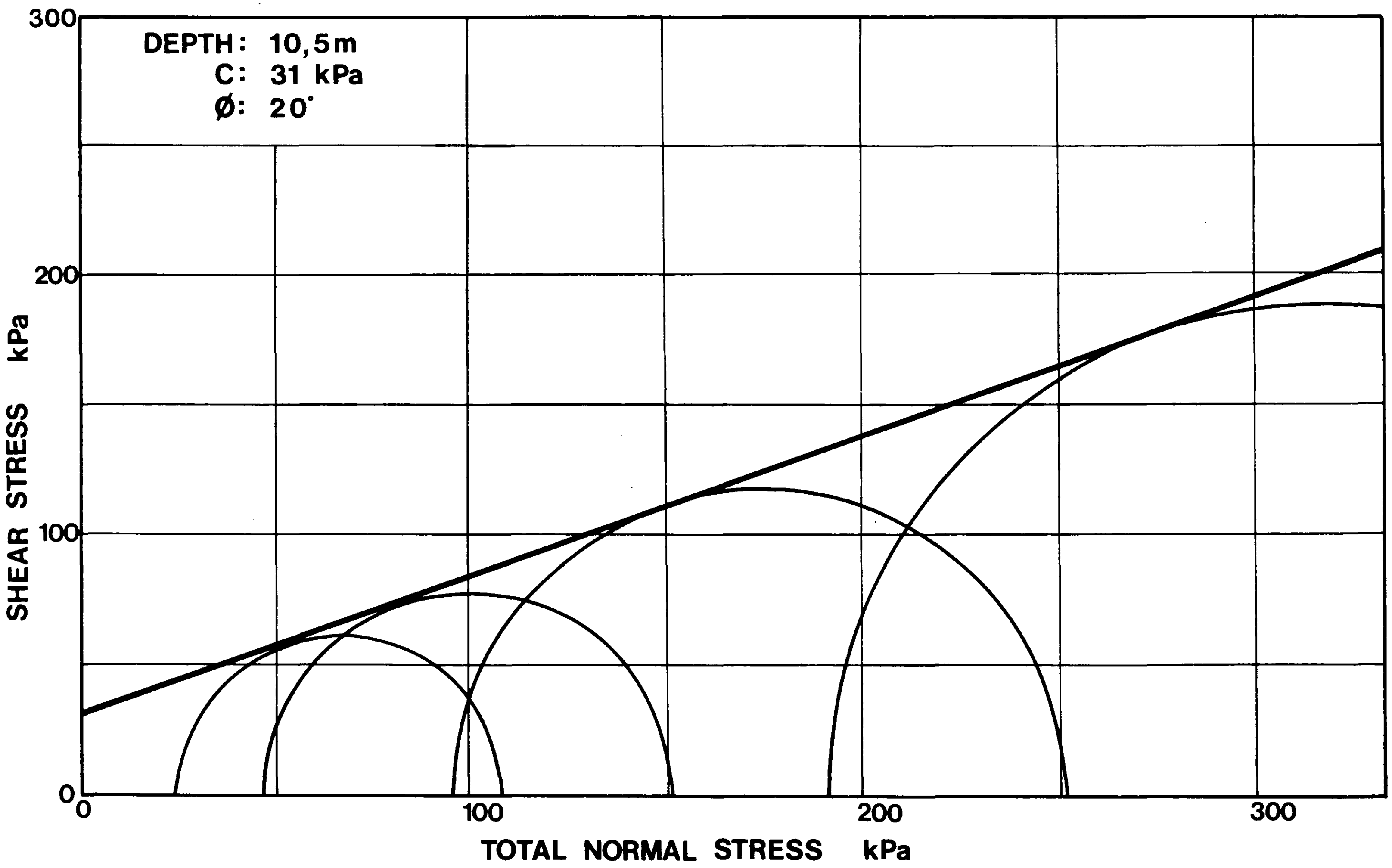


FIGURE 2/12 : Saturated, consolidated, undrained triaxial shear tests on sample of residual mica schist from brewery site, Ndola

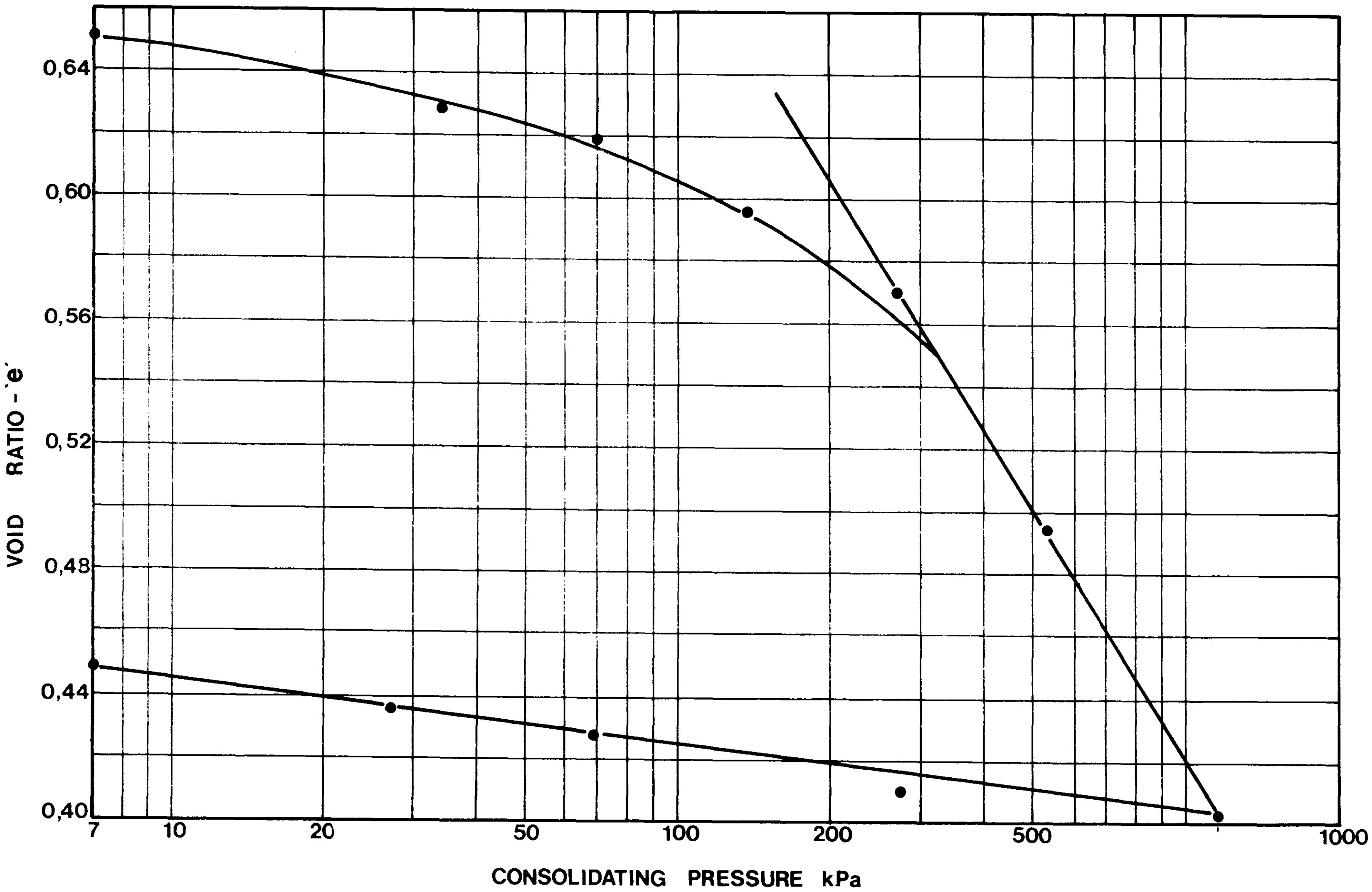


FIGURE 2/13 : Consolidation test results for sample of residual mica schist from brewery site, Ndola

The following forms of foundation treatment were finally adopted as the most economical:

* heavy structures (including a three-storey block) founded at average depth 10,5 m, some on under-reamed *cast-in-situ* augered piles, some on driven *cast-in-situ* piles, and others on tubed percussion-bored piles;

* light structures (single-storey blocks and items of brewing plant) founded on lateritic zone, at average depth 5 m, on miniature, 200 mm diameter, driven piles;

* ground floor slabs suspended and supported on strapped beams over piles.

Construction took place during 1958 and there has to date been no evidence of foundation movements of a sufficient order to produce distortion or cracking in any of the structures.

CASE HISTORY 2

RESIDUAL PHYLLITE OF GREENSTONE SERIES

SCHOOL BUILDINGS: BULAWAYO, RHODESIA

Figure 2/14 shows the layout of some of the buildings of a school in Bulawayo. Soil profiles of about 100 augered pile holes, foundation excavations and trial-holes were recorded in January 1960 before the foundations for the administration block were cast. By plotting these on the site plan it was possible to draw up a detailed geological map of the site, a simplified version of which is shown in Figure 2/14. A cross-section from east to west, coinciding with a row of column bases for the administration block, is shown in Figure 2/15. At the same time a survey was made of the cracking patterns in the existing buildings on the site, some of which has been built as early as 1917.

It will be seen from the legend in Figure 2/15 that five different materials were encountered on the site:

(1) dark brown, shattered, sandy clay; hillwash
(2) angular quartzite and vein-quartz gravel and boulders in matrix of red sandy clay; pebble marker
(3) brown, shattered, residual clay formed by advanced decomposition of phyllite
(4) stiff to very soft rock weathered phyllite, varying in colour and with marked foliations dipping eastward
(5) very dense to very soft rock weathered quartzite with quartz veins.

Severe cracking had taken place in the brickwork of the double-storey dormitory block, particularly above the door and window openings, and a crack about 100 mm wide ran approximately north-south in the first floor concrete slab. Many cracks in the brickwork were seen to intersect one another and the cracking pattern clearly indicated a reversal of movement suggesting cyclic heaving and settlement. It was also learned from the Principal that the cracks had actually been observed to open up during

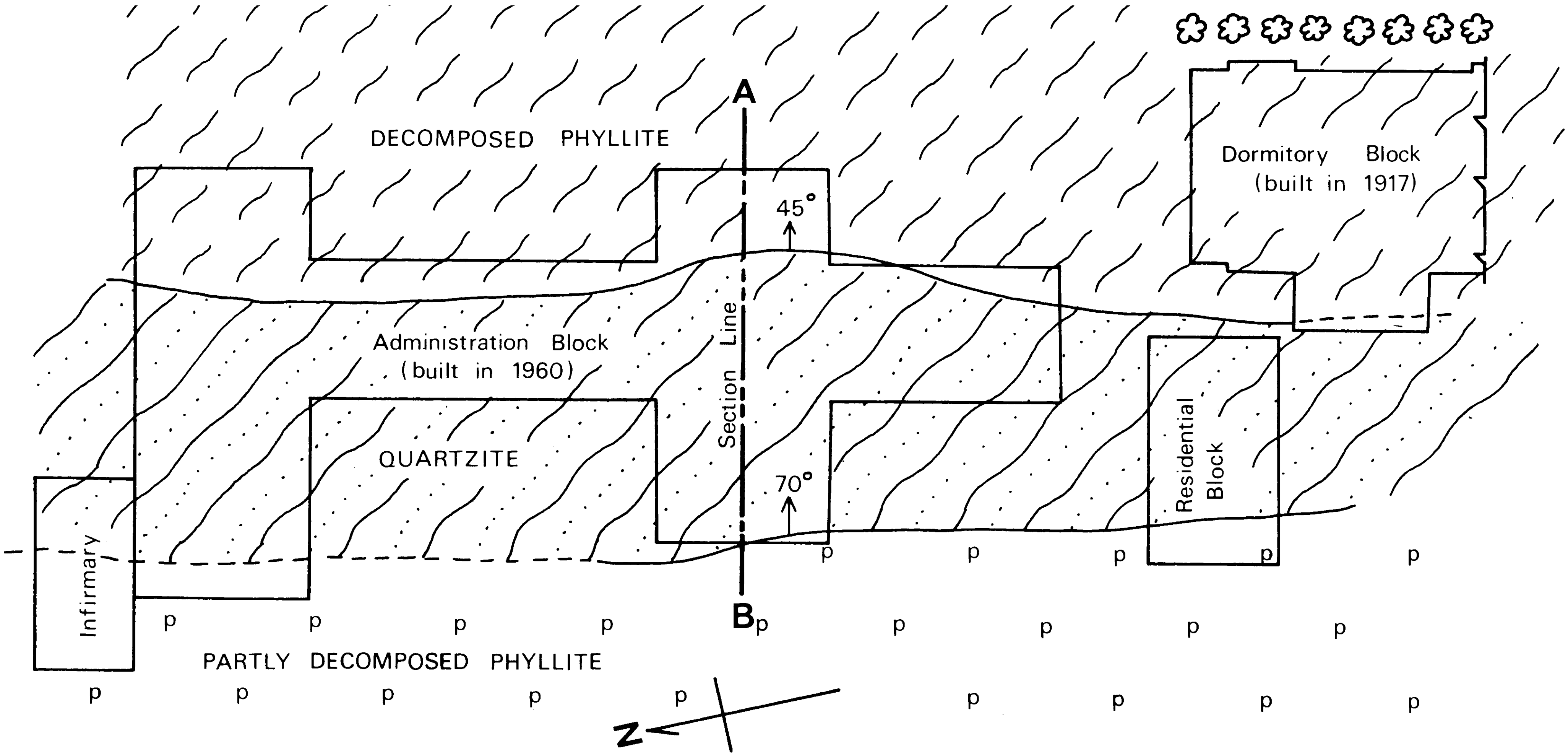


FIGURE 2/14 : Layout of some of the buildings of a school in Bulawayo

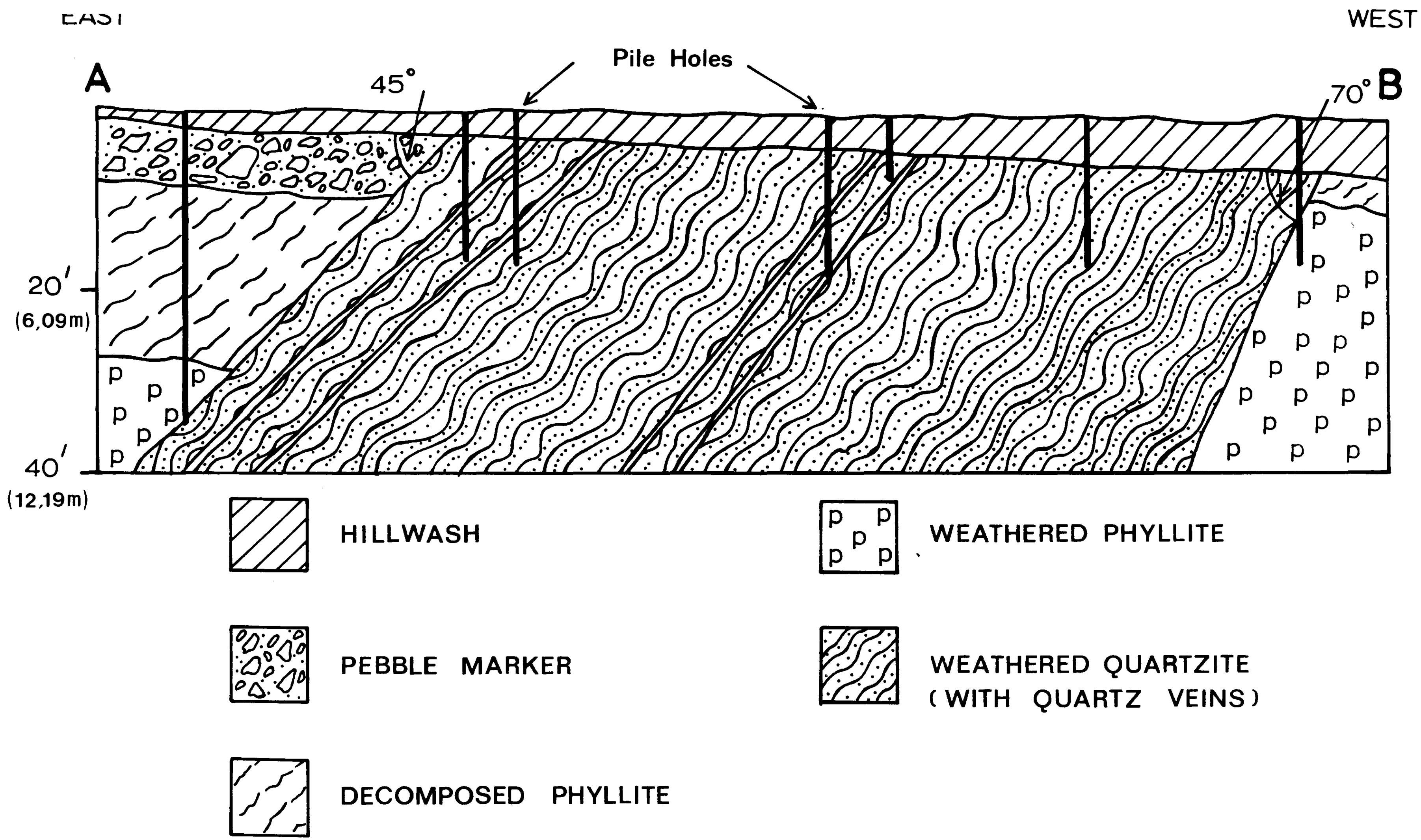


FIGURE 2/15 : Geological cross-section A–B from east to west coinciding with a line of column bases for the Administration Block of a school in Bulawayo the dry season and to close during the wet season. Maintenance costs for continual repairs of the building had been very considerable.

It will be seen from Figure 2/14 that the dormitory block was sited almost entirely on decomposed phyllite. The residential block, founded mainly on quartzite, showed slight cracking only on the western side where the foundations were on partly decomposed phyllite. As will be seen from Figure 2/16, the infirmary building was also fairly badly cracked. The eastern half of this building was founded on quartzite and the western side on partly decomposed phyllite. The cracking pattern indicated heaving on the western side and clear indications of seasonal reversal of movement.

Cracks in the dormitory block had clearly been caused by seasonal expansion and contraction of smectite clays in the deeply decomposed phyllite, aggravated by the desiccating effects of a row of Jacaranda trees on the eastern side of the building. Although heaving movements caused by the shallow thickness of expansive residual phyllite under the western half of the infirmary were of a very much lower order, cracking of this building was accentuated by the fact that the eastern half of the building was founded on quartzite. Because of the severity of the cracking and the high maintenance costs both of these buildings have now been demolished.

Ground floor beams for the administration block were designed to span between the pile caps of under-reamed piles, and were cast on a bed of ash about 100 mm thick. An inspection of this building in 1969 revealed the presence of fairly small cracks in the brickwork on the eastern side of the building where ground beams were cast over the decomposed phyllite.

FIGURE 2/16 : Cracking in walls of infirmary at a school in Bulawayo

No movement at eastern side founded on quartzite, but heaving at western side founded on decomposed phyllite

3 THE BASEMENT-COMPLEX (CONTINUED)

GRANITE-GNEISS

The Basement-granites, or granite-gneisses, and their associated residual soils are exposed over large areas of the northern and eastern Transvaal, Swaziland, Natal and Namaqualand. In the arid and semi-arid regions mechanical disintegration has been the dominant weathering process and residual soils are shallow and coarse-textured, resulting in the sound foundation conditions that can be expected to obtain throughout the large areas of relatively young Basement-granites of Namaqualand (see Figure 3/2). Any foundation problems that may arise in these areas would certainly be associated with deep transported soils and not with the granites as such.

COLLAPSIBLE GRAIN STRUCTURE

In humid regions, however, granite may often be decomposed to great depths into residual soil. Quartz remains unaltered in the form of sand grains, often slightly rounded by partial solution. Mica particles, particularly muscovite, remain partially unaltered except in the upper zones of the soil profile where they have decomposed. But the felspars become thoroughly kaolinised by chemical reaction with water charged with carbon dioxide:

$$\underset{\text{ORTHOCLASE}}{4\,KAlSi_3O_8} + 4\,H_2O + 2CO_2 = \underset{\text{KAOLINITE}}{(OH)_8Al_4Si_4O_{10}} + 8\,SiO_2 + 2\,K_2CO_3$$

So fine-grained are the particles of colloidal kaolinite that, in areas of relatively high rainfall, and in situations conducive to leaching or suffosion*, they are largely removed in suspension by circulating ground-waters, leaving behind a spongy residuum of micaceous silty

* The term *suffosion* is defined later in this chapter.

sand*. Colloidal coatings which adhere to individual quartz grains impart an apparently high strength to the soil when dry, and an unwary engineer would not hesitate to apply moderately high foundation pressures after examining the soil profile (Brink and Kantey, 1961). If the soil should become saturated under load, however, the colloidal bridges between quartz particles become lubricated and lose strength instantaneously; the grains fall into a denser state of packing which may lead to sudden foundation settlements of some magnitude.

This condition has become known as *collapsible grain structure* (Jennings and Knight, 1956 and 1957). It has resulted in severe differential settlement of buildings and other engineering structures where precautions have not been taken in foundation design.

Deep residual soils formed on Basement-granite and which exhibit collapsible grain structure appear to be confined to old erosion surfaces, or pediplain remnants, which have been preserved from denudation in subsequent erosion cycles. This is particularly well illustrated in the Johannesburg-Pretoria granite inlier, where soils with a collapsible grain structure have been shown to be confined to crests and marginal slopes of the African and pre-Karoo erosion surfaces which have been protected from dissection in the currently operating Post-African cycle (Brink and Partridge, 1967). As a general guide it may be stated that residual granite soils above the 1 500 m contour (Figure 3/1) are likely to possess a collapsible grain structure. These soils have been the cause of unsightly cracking in many buildings in the northern suburbs of Johannesburg and in Randburg and Sandton, although they are relatively shallow, seldom more than 15 metres deep, in contrast with the residual granites in more humid environments such as the Swaziland and Transvaal Highveld and the Zambian Copperbelt where decomposition often extends to more than twice this depth.

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* "Waters, seeping from the granite formations after heavy rains, contain a marked milky tubidity consisting of a fine colloidal suspension, which is so fine that a filter paper used for fine precipitates does not separate it from the water .... The suspended material in the filtrate after having been passed through a very fine filter paper, amounts to 0,892 gm per litre at 105^0 ... The molecular silica: alumina ratio if 6:8 indicating ... kaolinite with abundant silica of colloidal sizes" (C.R. van der Merwe, 1962).

The phenomenon of collapsible grain structure in these soils has been definitely established in Tzaneen, White River, Nelspruit, Mbabane the Johannesburg-Randburg-Sandton area, Salisbury, Chingola and Mufulira (Knight, 1963). The known occurrences within the Republic are all found to fall within, or close to the borders of, the area of annual water surplus (Schulze, 1958), as shown in Figure 3/2, which emphasises the rôle played by thorough leaching in the development of these soils.

It is of interest to note that a general correlation exists between the distribution of Basement-granites within the area of annual water surplus as shown in Figure 3/2, and the distribution of *ferralitic* soils as shown in Figure 3/3. As the mnemonic title suggests, ferralitic soils contain relatively high percentages of iron and aluminium hydroxides. They are highly leached old soils developed on the African erosion surface (initiated during the early Tertiary), under a tropical or semi-tropical climate. The climate has been sufficiently moist and the duration of weathering time sufficiently long to cause thorough depletion of bases from the soil. This has led to the formation of kaolinite and gibbsite from the weatherable minerals. The soil thus has a very low base status, and is characterised by a friable and porous structure and the presence of 1:1-lattice kaolinitic clays.

PREDICTION OF THE PRESENCE OF COLLAPSIBLE GRAIN STRUCTURE IN RESIDUAL GRANITIC SOILS

1. Prediction from maps and aerial photographs

Using the annual water surplus map (Figure 3/2) as a first guide to suspect areas, an examination of 1:50 000 topographic maps also assists in delineating erosion surface remnants. Further, sterescopic study of aerial photographs has proved invaluable in this regard, particularly where small-scale photography is available.

2. Prediction from field evidence

Though the consistency of the soil will depend on its moisture content, varying from stiff (or even very stiff) in a dry soil to very soft or loose in a moist soil, the high void ratio and the

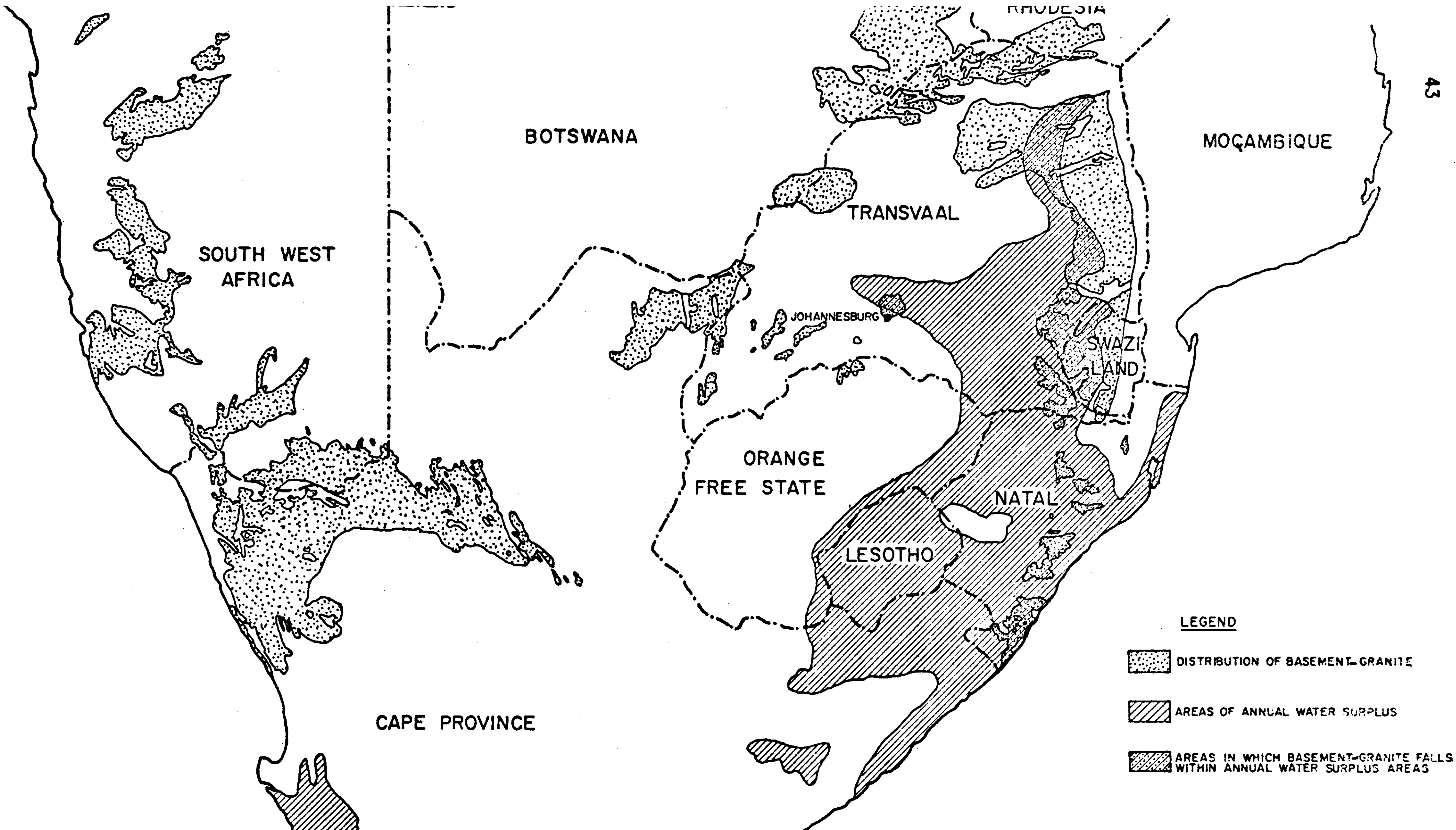


FIGURE 3/2 DISTRIBUTION OF GRANITE-GNEISS OF THE BASEMENT-COMPLEX IN RELATION TO AREAS OF ANNUAL WATER SURPLUS

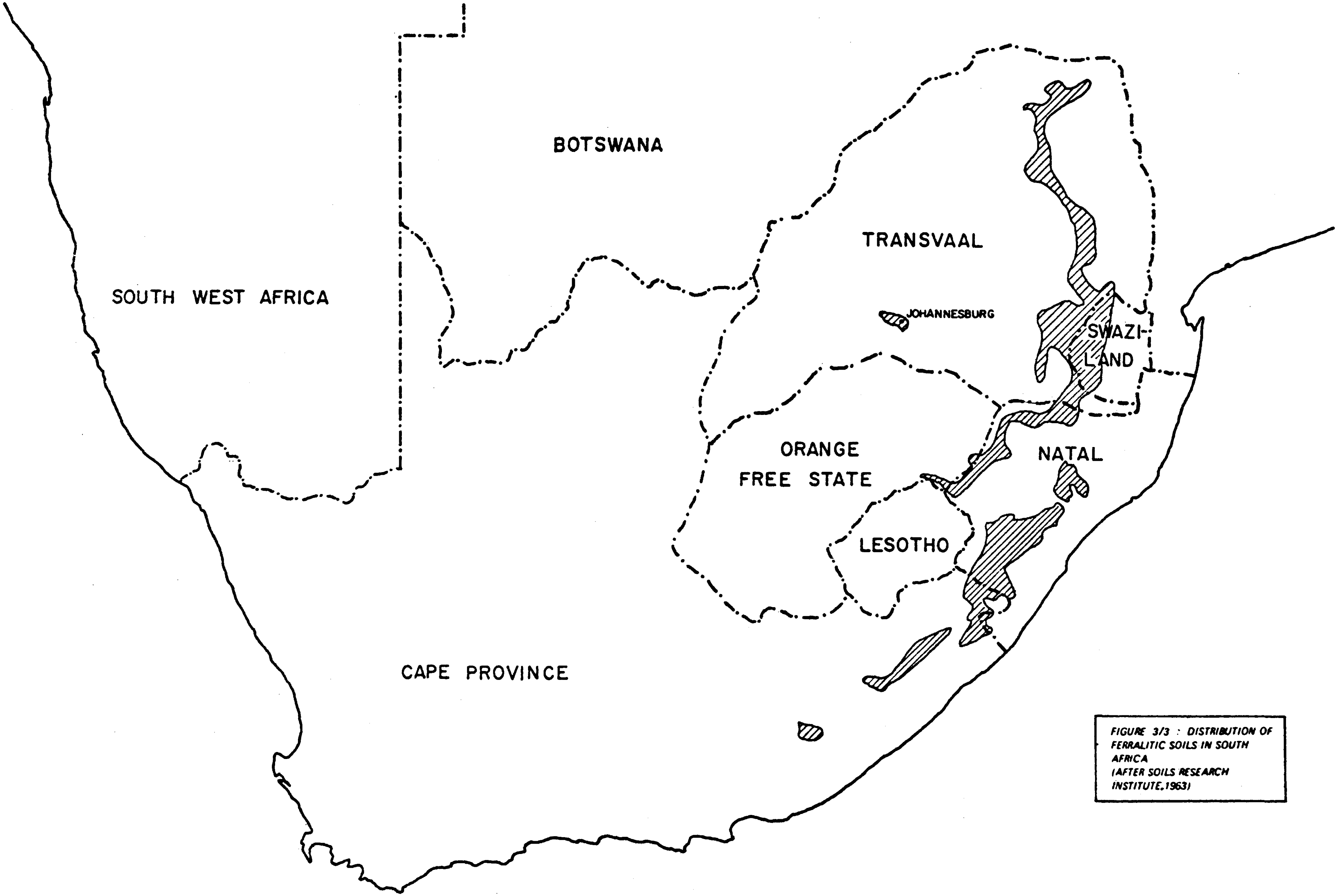


FIGURE 3/3 : DISTRIBUTION OF FERRALITIC SOILS IN SOUTH AFRICA
(AFTER SOILS RESEARCH INSTITUTE, 1963)

porous structure which characterise the collapsible condition will usually be clearly evident while recording the soil profile. The colloidal coatings around quartz grains, and the clay bridges between them, are easily recognised under a hand lens.

A simple field test may be carried out by carving two small cylindrical samples of undisturbed soil as nearly as possible to the same volume, wetting and kneading one of the samples and remoulding it into a cylindrical shape of the original diameter. An obvious decrease in length when compared with the disturbed twin sample will confirm a collapsible grain structure. This may be checked by repeating the procedure on the second sample and comparing the two remoulded volumes.

A similar reduction in volume on even partial "remoulding" may be observed by back-filling a pit with the soil originally excavated from it. If the soil possesses a collapsible grain structure, the back-filled soil will fail to fill the pit completely, the unfilled portion of the pit accounting for as much as 10 per cent of its original volume. This effect is particularly striking when the pit being back-filled contains water.

The most significant field evidence, however, concerns the observation of failure in existing buildings in the vicinity of a suspect site. Rigid concrete structures will tilt towards the area of maximum collapse; flexible steel buildings will show distortion of the less rigid members; unreinforced masonry will produce a cracking pattern related to points of local entry of water into the soil, e.g. broken sewer-pipes or points of discharge of rain-water.

3. Prediction from laboratory evidence

Examination of thin sections of residual granite soil under a microscope will indicate whether or not a collapsible grain structure has developed. In order to prepare thin sections the soil must first be impregnated with a hard-setting liquid, for which purpose the use of a liquid epoxy resin has been found the most satisfactory (Knight, 1959). The loose and porous grain structure of a collapsible soil, and the colloidal coatings around quartz grains and clay bridges between grains may be readily identified under the microscope.

Owing to their high void ratio, the dry density of soils with a collapsible grain structure is unusually low, varying from about 900 to 1 500 kg/m^3. The determination of dry density on a series of samples from different depths will indicate the level below which the degree of decomposition and/or the degree of leaching have been insufficient to produce a collapsible grain structure: safe depths of founding for piles or piers can be readily determined by this means.

Double oedometer tests may be used to predict the amount of settlement which will take place as a result of collapse of grain structure if the soil becomes inundated under any particular applied load (Jennings and Knight, 1957; Knight, 1958). The method involves running two consolidometer tests on twin undisturbed samples simultaneously, one under saturated conditions and the other at natural moisture content. The *collapse potential* may also be determined from a single oedometer test in which the soil sample is saturated under an applied pressure of 200 kPa (Jennings and Knight, 1975).

FOUNDATION TREATMENT FOR STRUCTURES ON RESIDUAL GRANITE WITH A COLLAPSIBLE GRAIN STRUCTURE

Practical solutions to the problem are based mainly on founding at a depth where the collapsible phenomenon is either absent or of neglible proportions. Where this depth is not excessive (less than about 12 metres), end-bearing augered piles usually offer the most economical solution. If the bearing capacity of the residual granite at the founding depth is insufficient to support a normal pile, the base of the pile may be underreamed to spread the load.

Where the collapsible condition extends to excessive depth the material may be pre-collapsed *in situ* by driving displacement piles to a limited depth. During a site investigation for the erection of additional buildings at Northview School, situated on a remnant of the African erosion surface in the Johannesburg-Pretoria granite inlier, the opportunity presented itself for a detailed investigation of this method of approach (Brink and Kantey, 1961). *Cast-in-situ* displacement piles with an expanded bulb were driven on the site and undisturbed samples extracted before and after driving to attempt to observe the effect of driving such piles on this type of soil: In the actual event, the particular site proved to be extremely variable and therefore somewhat unsatisfactory for test purposes, but Figure 3/4 provides some indication of the percentage increase in density that was obtained around the piles. As total collapse requires an increase in density of the order of 7 to 10 per cent it would appear that pre-collapse can be achieved by this method. Alternatively, foundations may be designed on the basis of permissible settlement in addition to permissible bearing capacity. In a very few cases involving light structures the problem has also been met by elaborate precautions for keeping water away from the foundations. In one case, a school building in White River, Eastern Transvaal, classrooms were individually supported on three short augered piles, and provision made for jacking up of any point of support which might settle.

Another method of foundation treatment which has been suggested, for sites where collapsible soil is less than about 10 metres thick, involves augering to the stable underlying material and backfilling the holes with the auger spoils, moistened to achieve an approximate Optimum Mositure Content, and compacting the backfill in layers by means of a drop-weight. "Soil piles" produced in this fashion should be capable of supporting surface bearing pressures of the order of 300 to 400 kPa, which would be adequate for relatively light structures (De Beer, 1965).

Yet another solution for certain structures lies in excavation of the soil to a depth of 1,5B (where B is the width of the foundation), and backfilling to founding level with the same material compacted at Optimum Moisture Content to a specified density in relation to the final load that is to be applied.

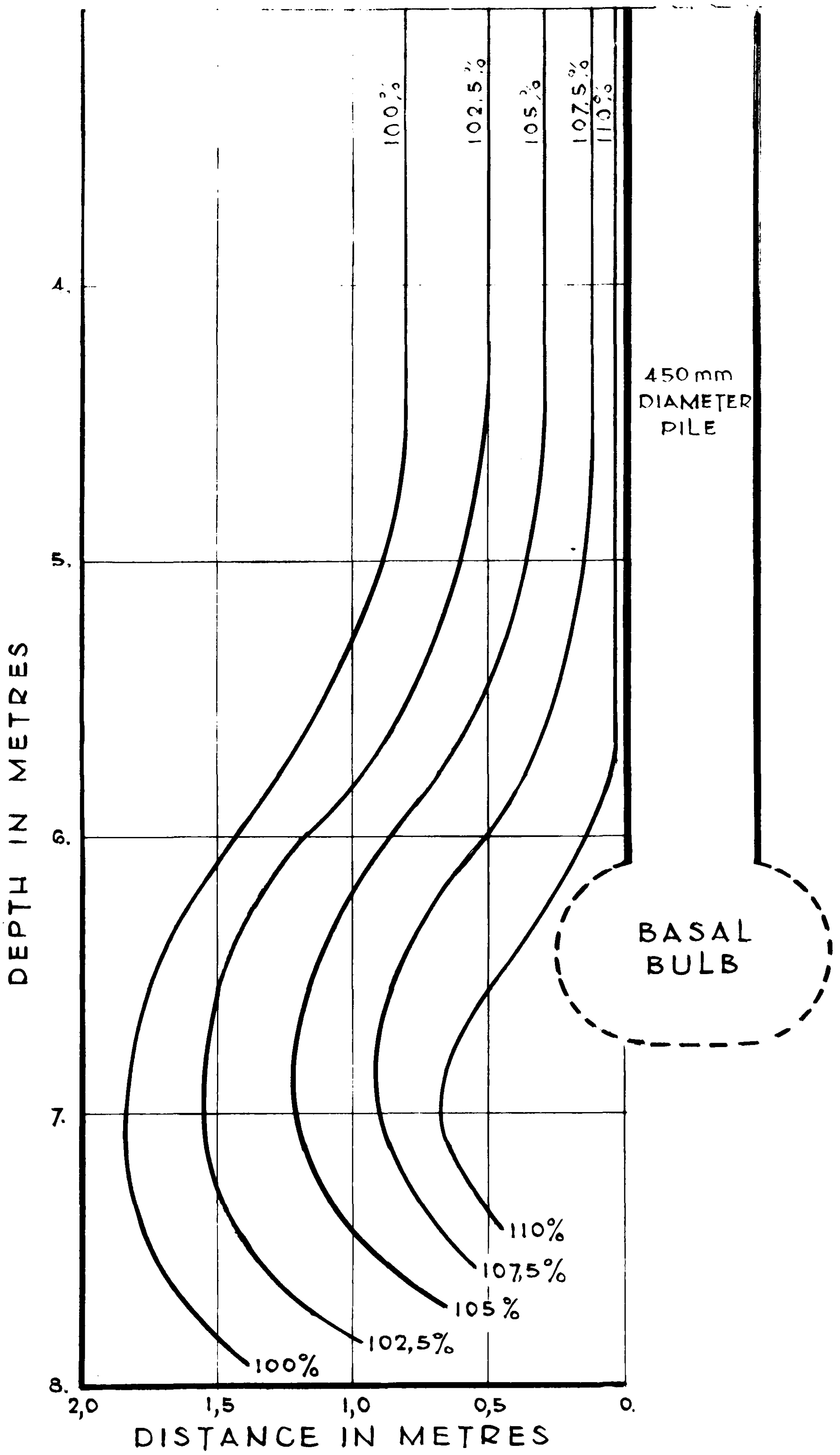


FIGURE 3/4 : Compactive effect of cast–in–situ displacement pile expressed as percentages of the original density of the residual granite : Northview School, Johannesburg

(After Brink and Kantey, 1961)

CASE HISTORY 3

RESIDUAL BASEMENT-GRANITE

DIFFERENTIAL SETTLEMENT OF WATER TOWER: WHITE RIVER, EASTERN TRANSVAAL

The erection of a 125 000 litre reinforced concrete water tower, consisting of a circular tank supported on four columns 15,25 m high, was completed in July 1953. Foundation bases for each of the columns were founded at a depth of about 1,5 m below ground level on a residual granite soil which was dry and stiff and which appeared to be perfectly sound material. As shown in Figure 3/5, the bases were 2,5 metres square, resulting in a bearing pressure of 145 kPa from dead load and water load, excluding wind loads.

Four years later, in August 1957, the occupant of a nearby house noticed a marked tilt of the tower in an easterly direction. Investigation by the writer and the engineer who had designed the structure showed that the top of the tower was more than 300 mm out of plumb, and that the NE base had settled 152 mm, the SE base 160 mm, the NW base 57 mm and the SW base 75 mm, since they had been cast in January 1953. It transpired that the tank had overflowed for a period of several days during April 1957, inundating the soil around the NE base. Initial settlement of this base caused a feed-pipe 100 mm in diameter to break at the elbow resting on top of the base, resulting in a flow of water towards the SE base along the backfilled feed-pipe trench. This flow of water continued unnoticed for several months, inundating the soil under the NE and SE bases, while the two western bases were protected from inundation by the natural slope of the ground.

It appeared, therefore, that the settlement of the western bases of 57 mm and 75 mm had probably taken place as normal consolidation settlement during construction, while the additional settlement of 75 to 100 mm of the eastern bases had resulted from the collapse of grain structure of the residual granite soil on becoming inundated.

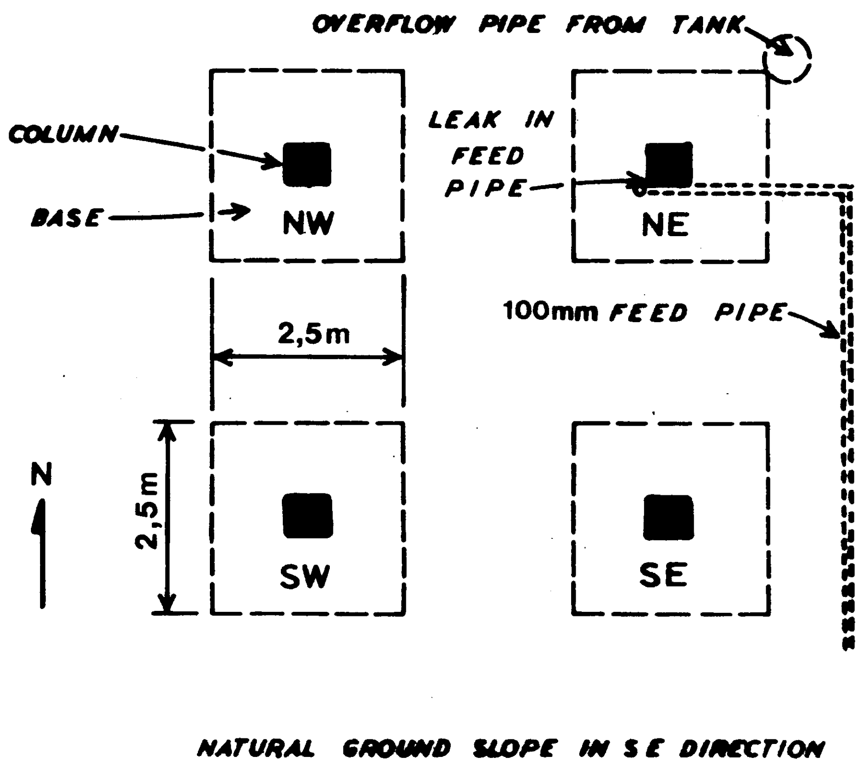


FIGURE 3/5 : Foundation plan of White River water tower

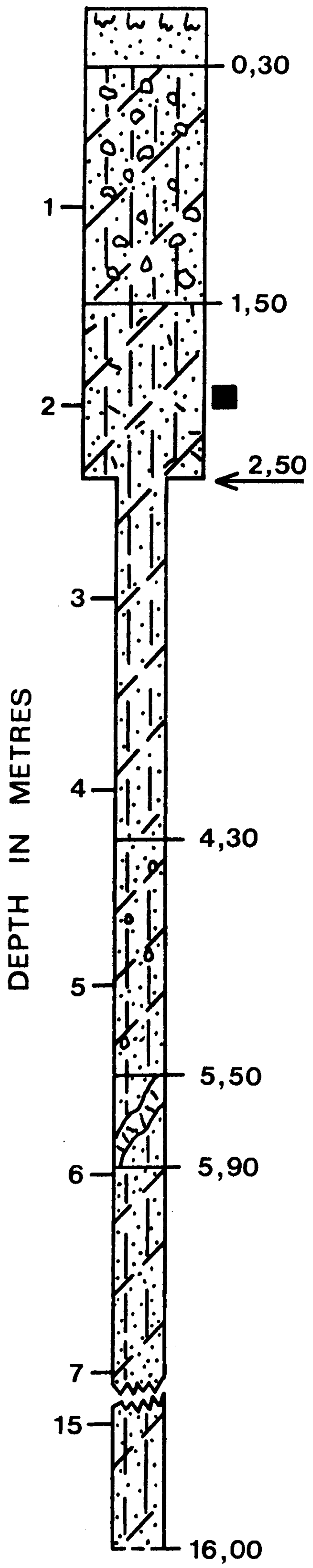
A trial-hole near the NE base which was dug by hand to a depth of 2,5 m and continued by hand-augering to a depth of 16 m, revealed a typical soil profile for thoroughly leached residual granite (Figures 3/6 and 3/7).

Undisturbed samples which were taken from a depth of 2 m were subjected to double oedometer testing and a further sample, loaded in the oedeometer at natural moisture content to the foundation pressure of 145 kPa, exhibited an instantaneous collapse of grain structure on being saturated (Figure 3/8). This was the first case on record when a residual granite soil was recognised to have a collapsible grain structure. Prior to this the phenomenon had only been recognised in altered aeolian sands (Jennings and Knight, 1956 and 1957).

A settlement analysis carried out to obtain the stress distribution to a depth of 6 m below the bases indicated that the theoretical settlement of a base founded on this soil at a natural moisture content of about 20 per cent (i.e. 70 per cent saturation) whould be 90 mm and that a further settlement of 187 mm could be anticipated under conditions of full saturation (Low, 1959). The calculated consolidation settlement of 90 mm was in close agreement with the measured settlements of 57 mm and 75 mm in the western bases. The fact that the calculated *total* settlement of 277 mm was approximately 45 per cent greater than the observed settlement of 152 mm for the eastern bases is probably attributable to a faulty assumption that the samples from 2 m were representative of the whole depth of soil affected by the loading. It is now recognised that the degree of development of collapsible grain structure, and hence the collapse potential, generally decrease with depth.

In considering remedial measures to right the tower the obvious decision was taken to attempt to induce additional settlement of the western bases by inundating the soil around them. This was carried out, after first taking the precautionary measure of guying the tower across the east-west direction with winch-controlled cables, by allowing water to flow into trenches around the western bases for a period of 110 hours at an average rate of about 2 000 litres per hour. The acceptance of water at this rate of flow indicated a soil permeability equivalent to 0,04 mm/sec.

For the first 15 hours no movement was observed on the bases. Then, as will be seen from Figure 3/9, the two western bases started to settle at



Moist, dark grey, in profile grey, firm, intact, organic, clayey coarse sand; biotic topsoil.

Subangular medium and fine gravel with coarse, medium and fine sand, in a matrix of orange, firm, clayey-silt; biotic stone line.

Undisturbed soil samples taken from depth 2,00 m	Clay content	= 14%
	Silt content	= 13%
	Dry Density	= 1 385 kg/m^3
	Liquid Limit	= 33
	Plasticity Index	= 8
	Linear Shrinkage	= 2,7

(bottom of test-pit: start of hand-auger hole)

Moist, orange, in profile orange with white and grey pockets and mottles, firm, intact, coarse, medium and fine sand in clay-silt matrix; with occasional milky quartz veins; abundant tree roots throughout; residual granite.

Residual granite as above but lighter in colour and containing more coarse material.

Residual granite as above, but predominantly white in colour and very coarse-grained, probably owing to presence of thick quartz vein.

Residual granite as above but yellowish in colour, and considerably moister; material apparently becoming progressively moister with increasing depth, but water-table not encountered.

Bottom of hand-auger hole; but not at refusal.

FIGURE 3/6 : Soil profile of test-pit and hand-auger hole at site of White River water tower

FIGURE 3/7 : Donga in residual granite near White River water tower, showing biotic stone line

(Photo by A.A.B. Williams)

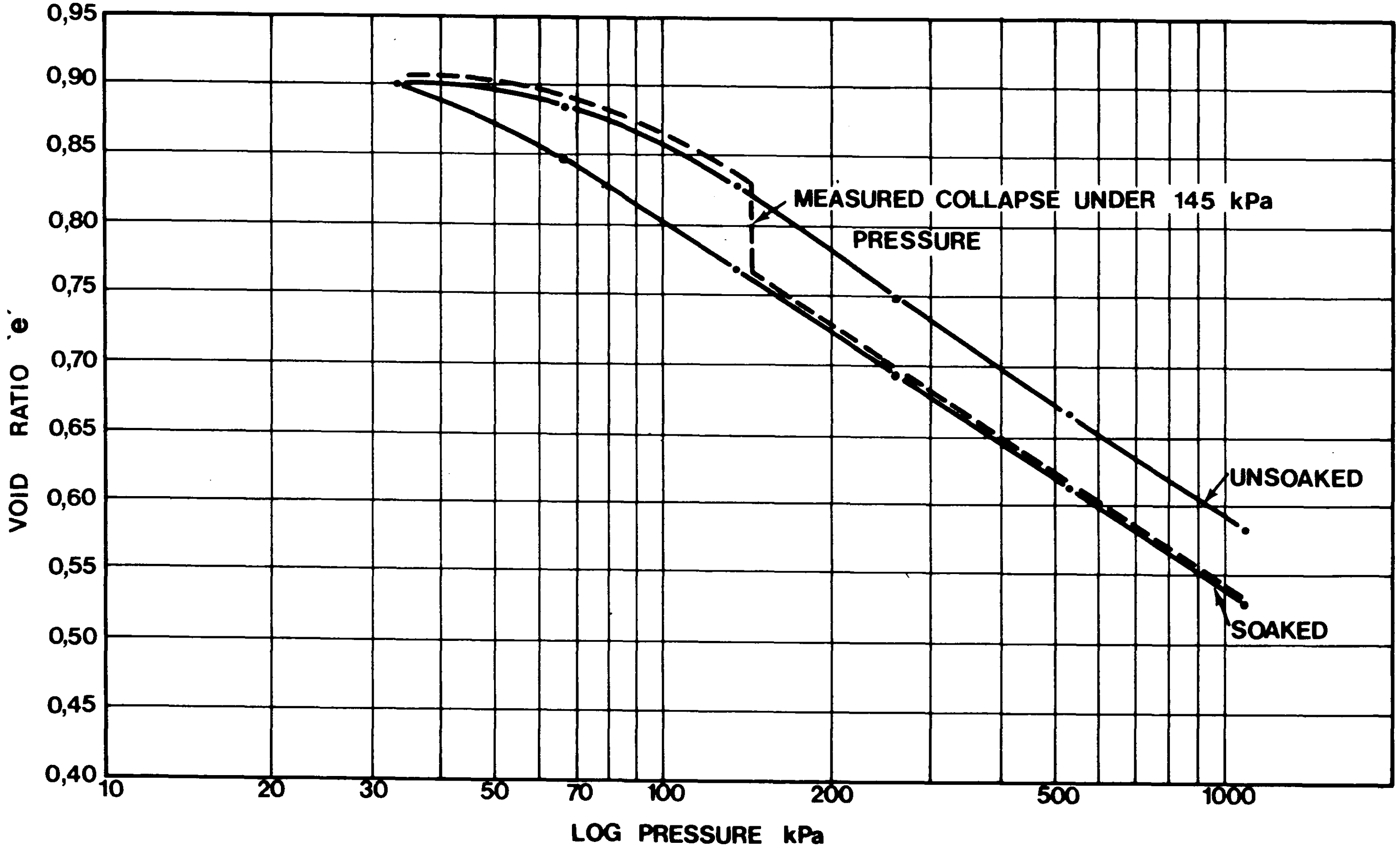


FIGURE 3/8 : Consolidation curves for residual granite from 2 m depth at White River water tower

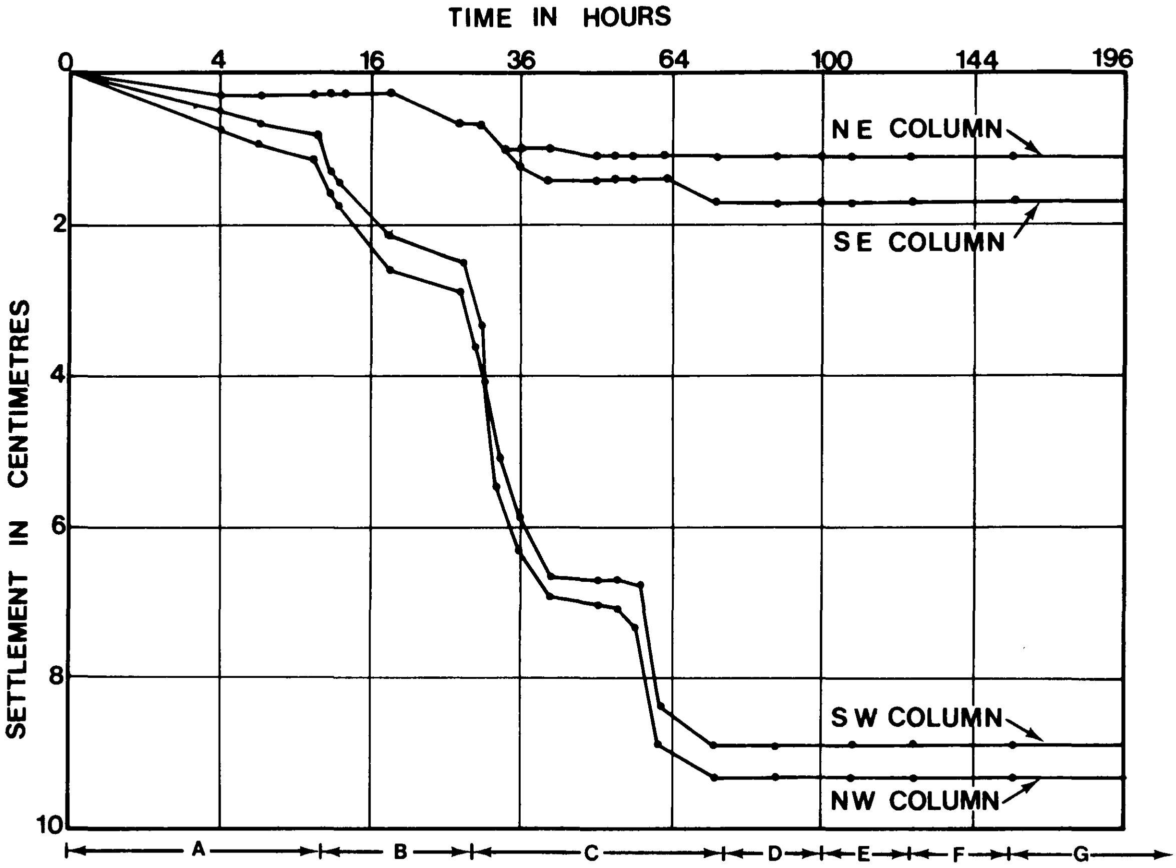


FIGURE 3/9 : Time/settlement curves for the four column footings of the White River water tower during correction of tilt a steady rate. The rate of settlement was accelerated by filling the tank with a water load of 136 tonnes, while slackening the eastern guy and tightening the western one. Settlement of the western bases continued for a further 65 hours; a minor additional settlement on the two eastern bases served to correct a slight tilt towards the south. Final overall settlement, as from January 1953, amounted to 167 mm for the NE base, 170 mm SE base, 165 mm NW base and 150 mm SW base. In this way the final tilt on the tower was reduced from more than 300 mm to about 50 mm which is hardly perceptible to the eye (Figure 3/10).

Calculation of ultimate bearing capacity, based on the results of triaxial shear testing which had been carried out on the undisturbed samples from the 2 m depth, had indicated that the factor of safety under dead loading alone, and for fully saturated conditions, was less than 2.

It was therefore decided to increase the size of the bases from 2,5 metres square to 3 metres square, thereby reducing the bearing pressure from 145 kPa to 95 kPa. This work was completed in May 1958. Three months later the tank again overflowed for a period of about 36 hours causing complete inundation of the soil around the bases, but no further movement took place, and indeed no further movement has been recorded since then.

Interesting comparison of the engineering properties of the soil was afforded by a more recent investigation carried out by the writer for a new reservoir situated 30 metres north of the water tower.

Undisturbed samples were taken from a test-pit on the new site at depths 5,3 and 6,0 metres. Comparative test data are given in Table 3.1.

From the test data it can be concluded that:

(i) Atterberg Limits of both samples, though having a 4,0 m difference in depth, are very similar;

(ii) dry densities as obtained from the triaxial tests are greater for the sample at 6,0 m (1973) than at 2,0 m (1957) possibly due to higher compaction by the greater thickness of overburden;

(iii) as would be expected, the collapse potential as derived from the consolidation curves is greater at the shallower depth of 2,0 m than at 6,0 m.

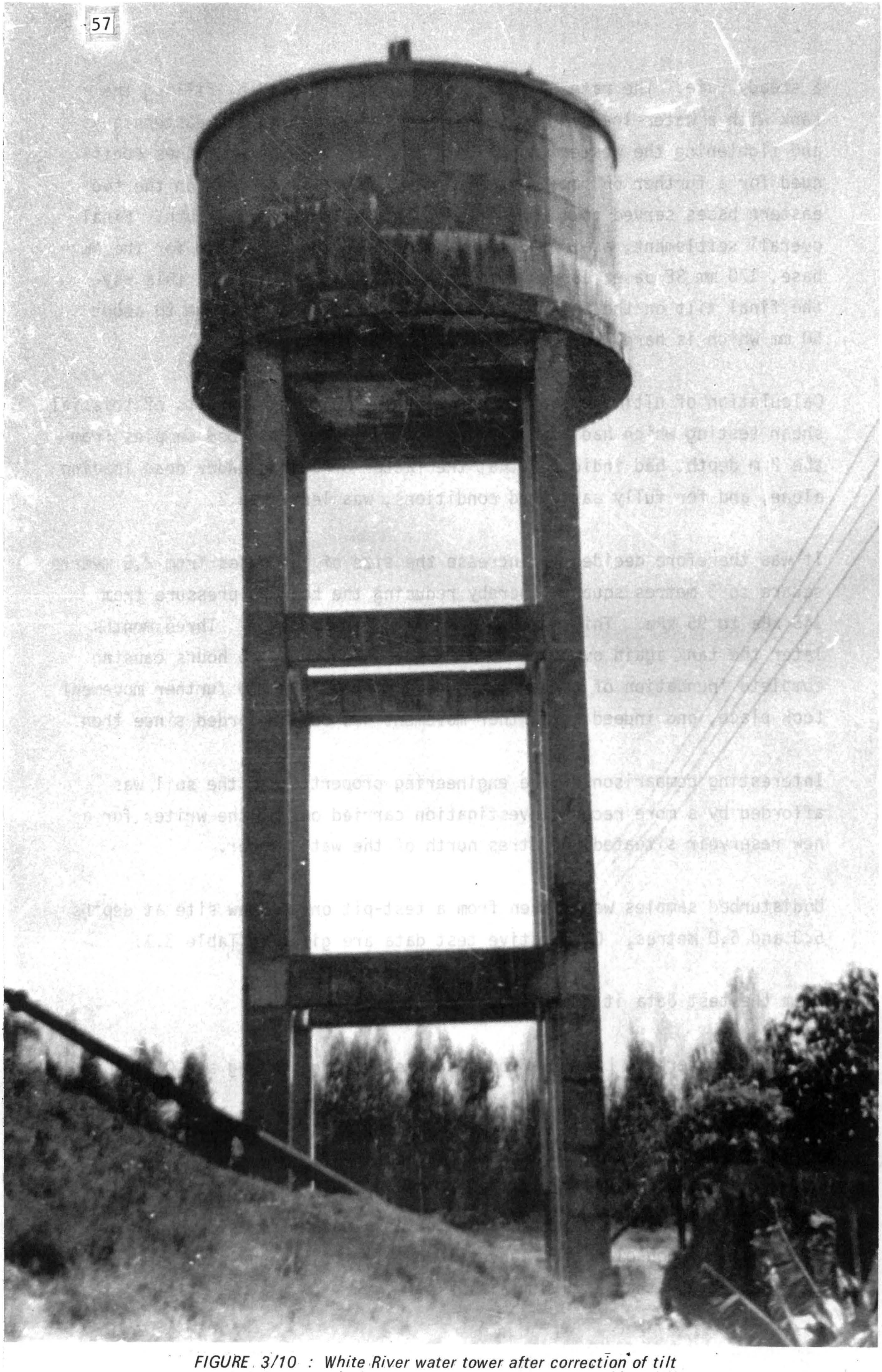


FIGURE 3/10 : White River water tower after correction of tilt

Atterberg Limits	Water Tower Site sampled in 1957 Depth 2,0 m	New Reservoir Site sampled in 1973 Depth 6,0 m
Liquid Limit	33	34
Plasticity Index	8	8
Linear Shrinkage	2	2
Collapse Potential at 200 kPa	3,2%	2,9%

Water Tower Site sampled in 1957				New Reservoir Site sampled in 1973		
Triaxial Shear Tests	Sample depth in metres	Bulk density kg/m^3 γT	Dry density kg/m^3 γd	Dry density kg/m^3 γd	Sample depth in metres	Triaxial Shear Tests
Scu	2,0	1 554	1 330	1 368	6,0	Suu
	2,0	1 564	1 306	1 411	6,0	
	2,0	1 564	1 330	1 424	6,0	
	2,0	1 552	1 333	1 333	6,0	
Ucu	2,0	1 538	1 330	1 382	5,3	
	2,0	1 589	1 347	1 416	5,3	
	2,0	1 576	1 360	1 401	5,3	
	2,0	1 592	1 355	1 431	5,3	
	2,0	1 698	1 378	1 355	5,3	
		Ccu = 10,3 kPa Øcu = 20^o		Cuu = 35 kPa Øuu = 8^o		

Table 3.1: Comparative test data for residual granite samples from elevated water tower site and adjacent new reservoir site, White River

THE PROBLEM OF "CORE-STONES" WITHIN RESIDUAL GRANITE

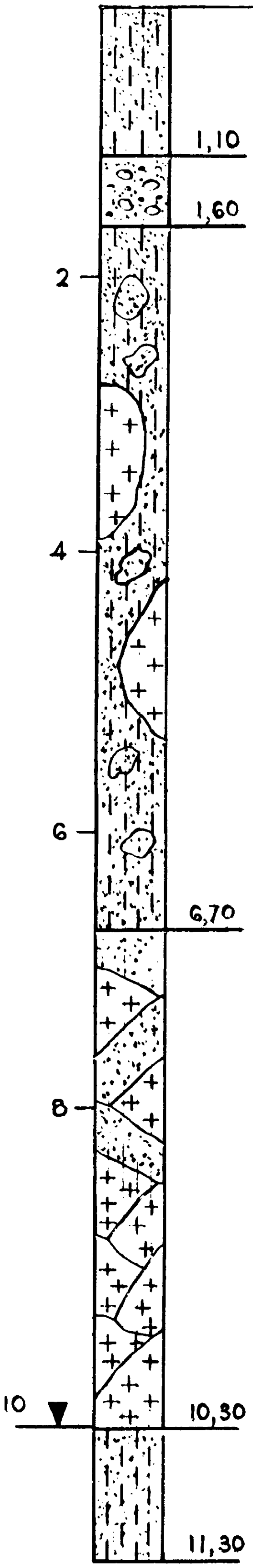
The presence of core-stones of hard granite within the residual granite soil presents special problems in foundation engineering. The problem is particularly acute where the residual soil possesses a collapsible grain structure, and the core-stones are too large too be removed by hand from an augered pile hole: piling in such a situation becomes virtually impossible.

An excellent opportunity for examining the nature and occurrence of core-stones was provided in a site investigation at a cement factory in Salisbury, Rhodesia. The investigation was conducted by Professor J.E. Jennings in 1974. A test-pit of plan dimensions 1,80 by 1,20 m revealed the soil profile shown in Figure 3/11, which led Jennings to the conception of core-stone development depicted in Figure 3/12. This conception was subsequently verified by examination of a large excavation opened up for the foundations of several heavy structures built on the site (Figure 3/13).

"Sheeting" in the granite is mainly in the horizontal plane, and jointing mainly in vertical planes. Weathering of the granite as a result of hydration of the felspars in carbonated rain-water (the carbon dioxide having been taken into solution partly from the atmosphere but mainly from the soil air in the plant-growing topsoil) has been largely confined to the zones immediately bordering on the joints and sheeting planes. Decomposition is thus wholly dependent on the concentration of carbonic acid and the directional pattern of water flow. Consequently, as depicted in Figure 3/12, the occurrence and size of core-stones will increase with depth in the vadose zone, and the typical shape of a core-stone will be more rounded at the top than at the base of a joint-block. At the level where most of the carbon dioxide in the water has been dissipated by chemical reaction with the felspars, the joints in the parent-rock will be discernible and little rounding off of joint-blocks will be seen. This will become progressively more apparent until the water-table is reached.

In the phreatic zone below the water-table the flow pattern changes from vertical to more nearly horizontal (along the phreatic flow-net) and the granite here, though still weathered, will conform with a pattern which is more "corrosive", with a lessening of the tendency to core-stone development.

Natural Ground Surface



Moist, brownish red, in profile red, very soft, intact, silty fine sand; altered aeolean.

Coarse, medium and fine, rounded schist gravels, lateritised in matrix of soil as above; pebble marker.

Moist, reddish orange, very soft, intact, silty coarse, medium and fine sand; few very hard rock rounded granite boulders up to 2 m diam; many soft decomposed granite boulders 500 mm pink and yellow speckled black; silty sand between boulders probably collapsible; residual granite.

Hard rock granite boulders tending to be rounded at top and angular on joints towards bottom; closely packed; matrix where present pinkish orange silty coarse sand.

Wet pinkish orange, firm to stiff, intact, silty coarse sand; residual granite

NOTES:

1. Sides of hole from 1,60 - 6,70 m show few tension-cracks.
2. Note change in weathering pattern at water-table: 10,00 m.
3. One core-stone on bottom of test-pit about 30% of area at bottom.

FIGURE 3/11 : Soil profile of hand-excavated test-pit (1,8 x 1,2 m) at Salisbury Portland Cement Company

(After J.E. Jennings)

HW/JEJ
28.1.74

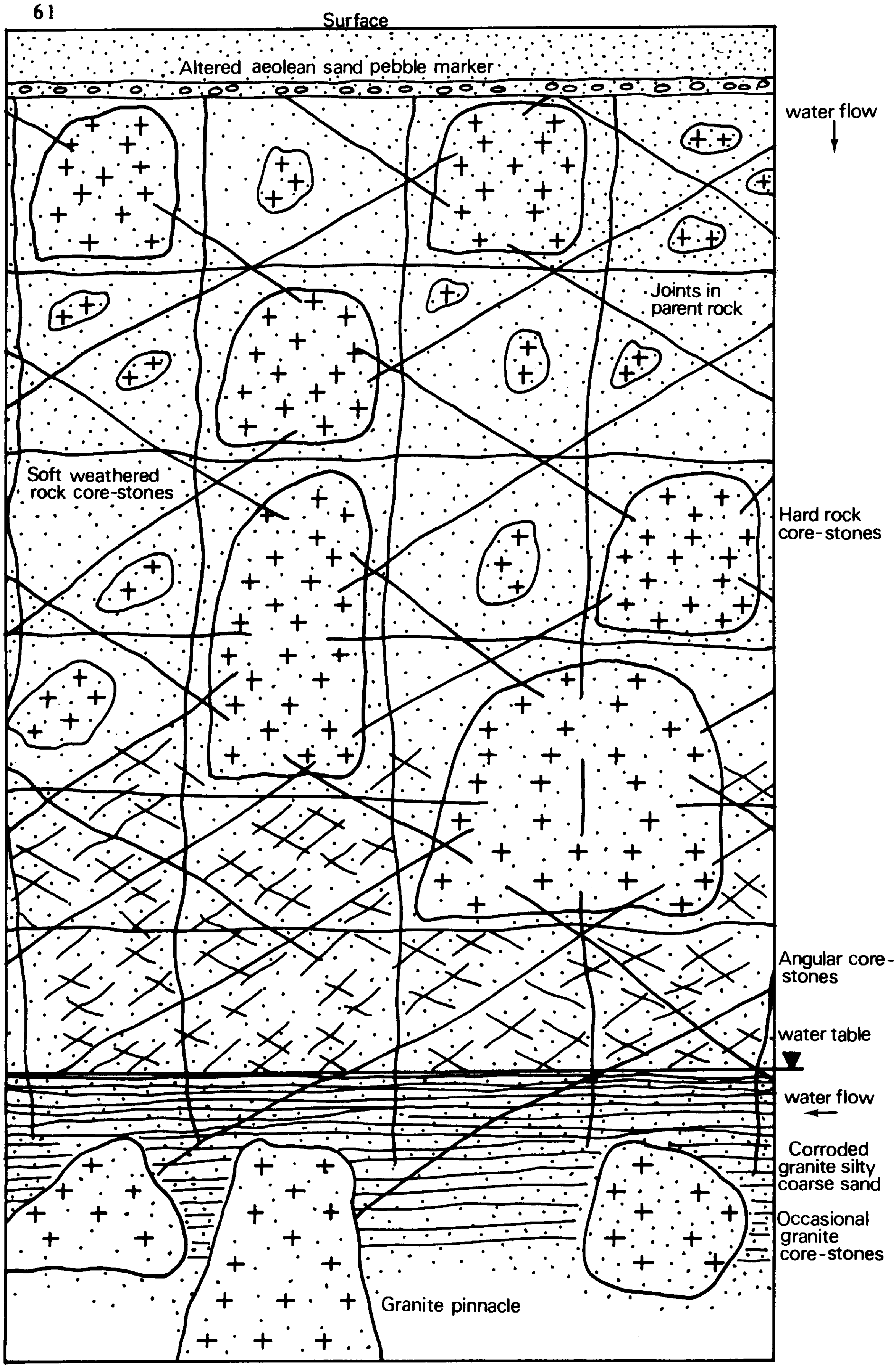


FIGURE 3/12 : A conception of the development of core-stones in the weathering of granite (After J.E. Jennings)

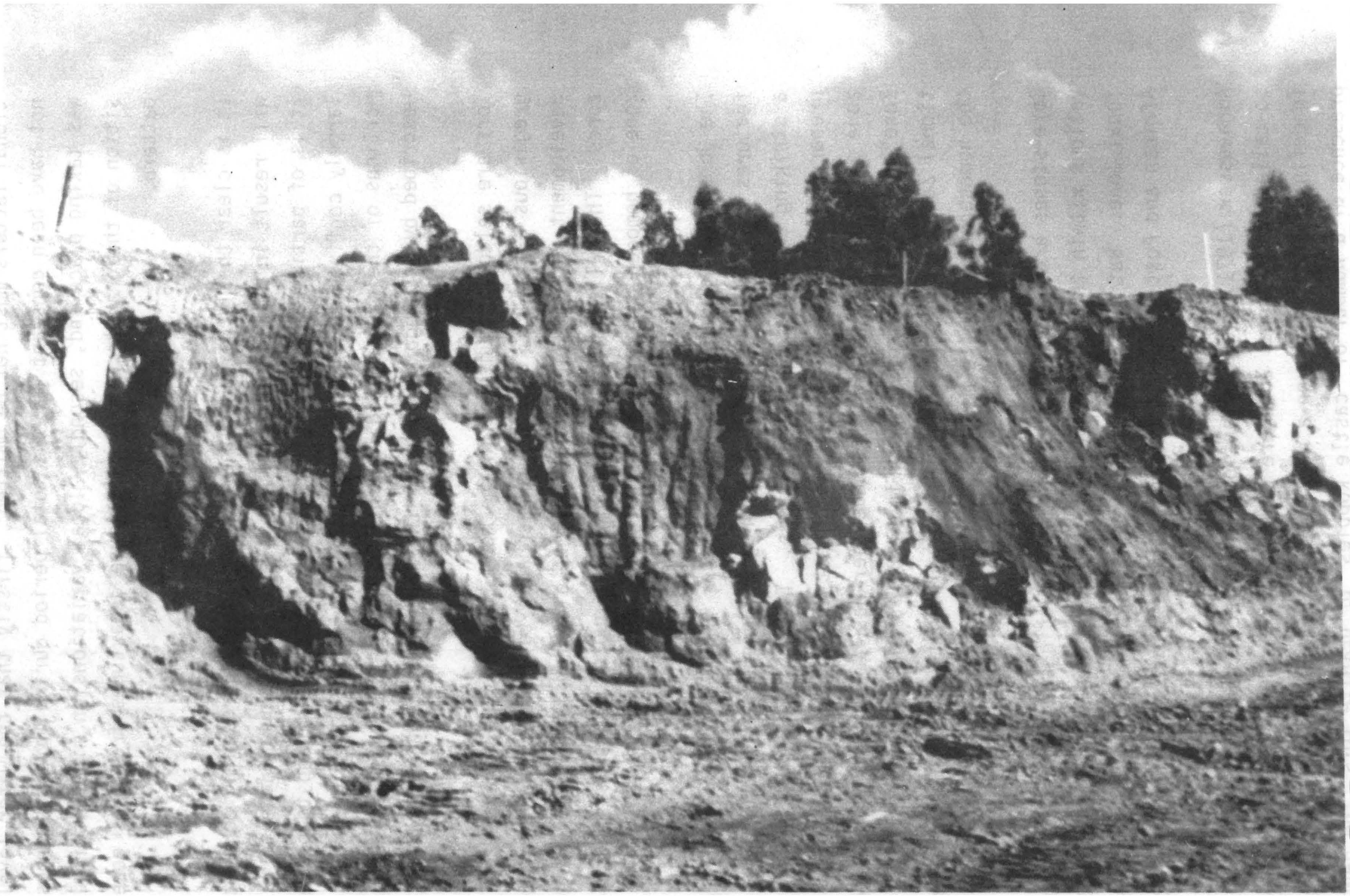


FIGURE 3/13 : Core-stones in residual granite at cement factory, Salisbury

(Photo by J.E. Jennings)

These comments must obviously be viewed only as guidelines. Two additional factors must not be omitted: firstly that the water level may not have been constant for the total period during which decomposition was taking place and, secondly, local variations in mineralogical composition of the granite which could cause departures from this general pattern.

It is clear that erosion of the residual soil surrounding core-stones will result in the formation of "castle koppies" or granite tors. And it is of particular interest to note that tors are found to be very largely confined to zones of homogeneous granite, often near the central portions of the oval-shaped "gregarious batholiths" of granite, as described by Macgregor (1952) for the Rhodesian Archaean Complex.

Tors are seldom encountered in the more tonalitic peripheral zone of a gregarious batholith, and it is reasonable to assume that core-stone development is similarly unlikely to be a common feature of the peripheral zones. The Salisbury cement factory site quoted above is situated in a zone of homogeneous granite in the Chindamora batholith.

The Johannesburg-Pretoria granite inlier, if it may be regarded as having features in common with one of Macgregor's gregarious batholiths, provides a striking example of this hypothesis. The geological map of the inlier prepared by Anhaeusser (1971) shows that tors are confined to the central zone of granitic rocks (which Anhaeusser has named the "Transitional Zone"). It will be noted from Figure 3/14, furthermore, that the Transitional Zone is confined within an area of Post-African II incision into the homogeneous granite mass.

Core-stones are common features in road cuttings and foundation excavations within the homogeneous granites immediately south of the Transitional Zone. But as this area is largely occupied by remnants of the African and Post-African I erosion surfaces, tors, as such, are absent.

Anhaeusser (1971) draws attention to the fact that "the limits of the so-called Transitional Zone are of a purely arbitrary nature although, in defining this region on a map (he) has made use of the restricted presence of a number of 'castle koppies' or granite tors, the development of which may be linked genetically to the behaviour of the various granites in their vicinity".

Anhaeusser notes, further, that in a study of the occurrences of tors at Witkoppen, Brook (1970) concluded that the mineralogical variation in the granites of the widely spaced sites in the largest group of tors in the area was of sufficient magnitude to suggest that the tors owed their resistance to other than compositional characteristics. A total of 24 samples had been examined by Brook, and he had found that the felspar content ranged from 54 to 76 per cent of the total rock, with the potash-rich variety, microcline, accounting for between 42 and 53 per cent of the felspars, and the rest consisting of plagioclase felspar.

Brook had proposed that the tors were formed by strong jointing, coupled with differential deep weathering beneath the Post-African I surface of King (1962): Post-African II incision had exhumed the core-stones so formed.

However, while Brook has maintained that, because of the apparent lack of correlation between mineralogical composition and topography the tors could not be explained in terms of rock composition, Anhaeusser's contention is that the reasons for tors being confined to the central part of the granite inlier are indeed connected with geological as well as geochemical variations within the so-called Transitional Zone. He feels that the high felspar content noted by Brook has not been given the attention that it deserves. He considers that the K-rich felspar in the Transitional Zone was introduced by metasomatic replacement of pre-existing, more sodic felspar in gneisses and migmatites, and that "the homogenisation process, although incomplete, nevertheless produced material having a tendency to joint in a manner suitable for the production of subangular joint blocks and weathered core-stones ..... these core-stones ultimately gave rise to the precariously perched boulders so typical of the castle koppies or granite tors found in the area".

The present writer would go even further in contending that the mineralogical composition of the tor blocks and core-stones, with their exceptionally high content of microline felspar, is of paramount significance in the mode of weathering - or rather resistance to weathering - of the homogeneous granite occupying the central and southern parts of the granite inlier.

Microcline, being an ordered polymorph, is usually formed at lower temperatures than is the disordered polymorph orthoclase or any other felspar. Consequently microcline is far more resistant to decomposition in the zone of weathering than is any other felspar, and a granite containing a high percentage of microcline will therefore be more susceptible to development of core-stones than granites containing a high percentage of sodic felspars, such as those constituting the peripheral zone of the inlier or of a typical gregarious batholith.

A good example of core-stone development is to be seen in the excavation for part of the Johannesburg Western Bypass (Road N 1-20). Where this road passes beneath the Witkoppen Road (Road P79/1) at Fourways township, excavation was severely hampered by the necessity to blast a number of very large core-stones which were embedded in the residual soil as shown in Figure 3/15.

The resistance of microcline to decomposition in the zone of weathering is well illustrated by the fact that pegmatite veins, in which microcline is the principal felspar, are more resistant to weathering in both arid and humid environments than is the granite host-rock within which they are present. Figure 3/16 illustrates this fact.

PSEUDOKARST PHENOMENA

Closely associated with the development of collapsible grain structure are the *pseudokarst* phenomena resulting from *suffosion* in residual granite soils. Before proceeding to a description of these phenomena, it may be as well to define the terms pseudokarst and suffosion, as they are words which have hitherto seldom, if ever, appeared in South African scientific literature.

A terrian in granular materials (either transported or residual soils) may be classified as pseudokarstic if it has landforms similar to those of a *karst** terrain, but which are produced by water flow (and attendant

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* *Karst* is the aggregate of the landforms and subsurface features produced in carbonate or evaporite rock terrain by solution, the downward flushing of sediment or residuum into subjacent solution cavities (i.e. solution-induced suffosion) and the consequent subsidence and collapse of such sediments or residuum (Quinlan, 1966).

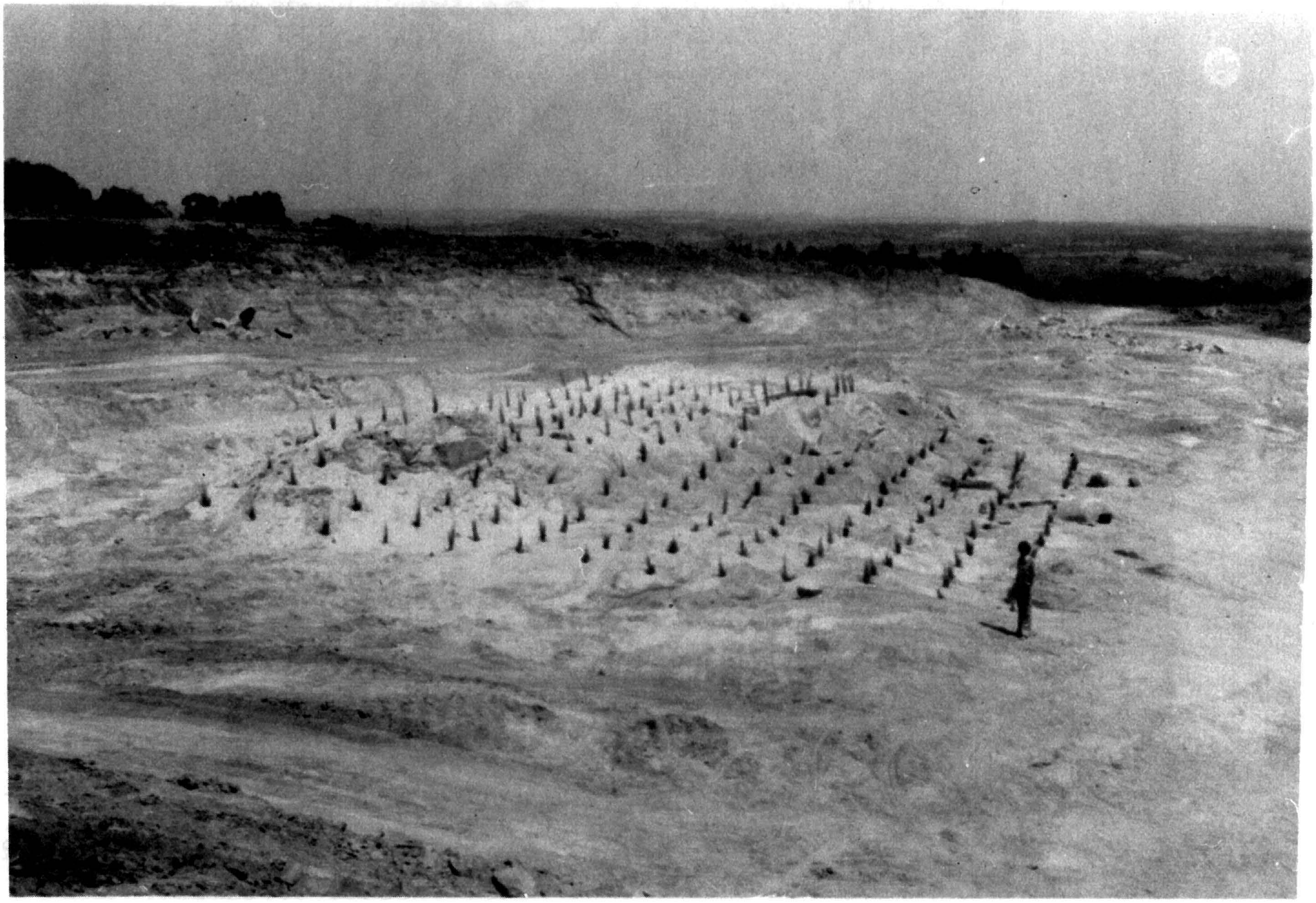


FIGURE 3/15 : Excavation for part of Johannesburg Western Bypass (Road N 1–20) showing core–stone within residual granite

The core-stone has been pre-drilled ready for blasting

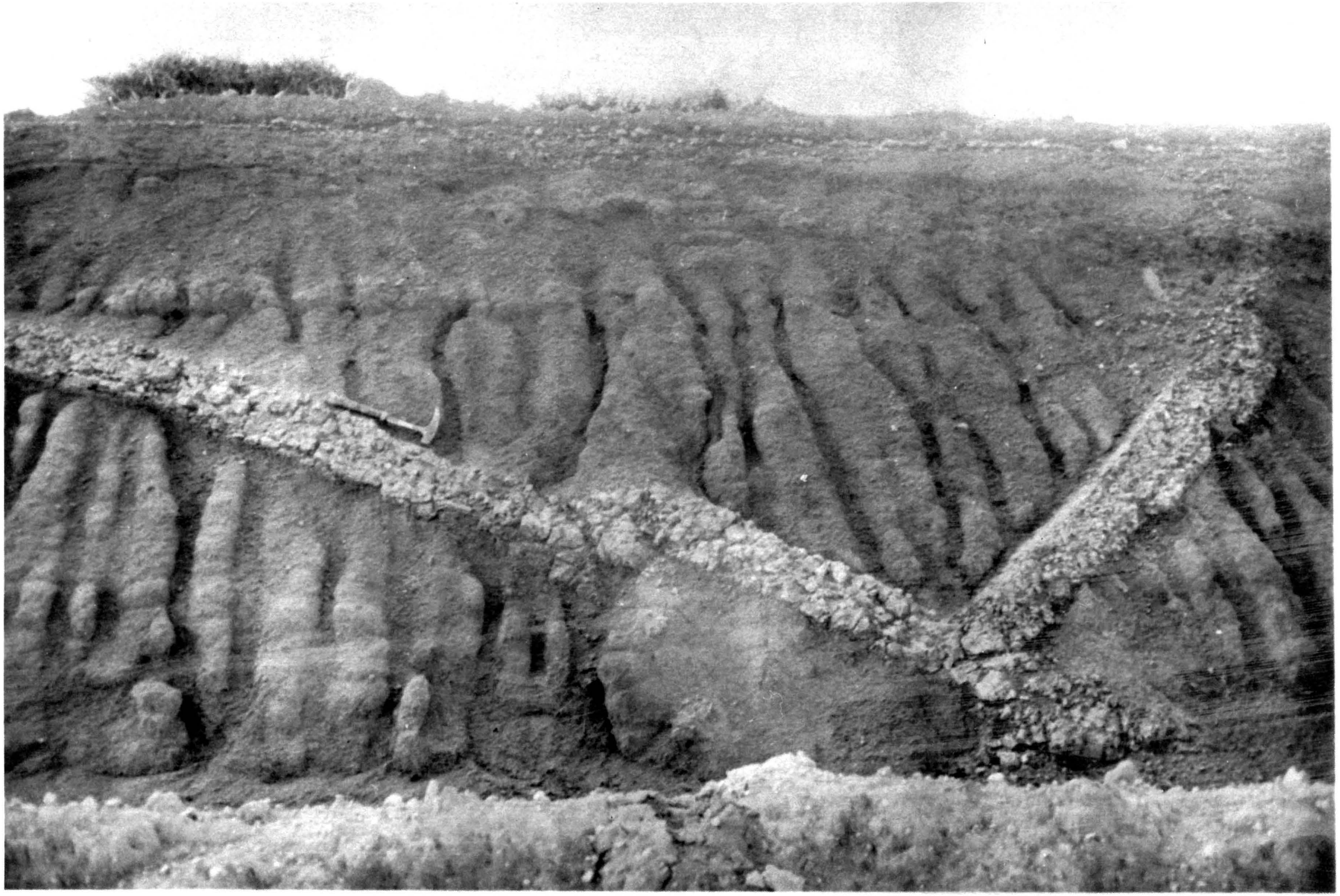


FIGURE 3/16 : Fresh pegmatite veins rich in microcline felspar within residual granite : road cutting in Witkoppen, Sandton, Transvaal processes such as deflocculation of clays) taking place within the material itself.

Suffosion, a term first introduced by Pavlov in 1898, refers to the undermining of transported or residual soils by the mechanical and chemical action of underground waters. *Selective mechanical suffosion*, the process that concerns us here, involves the selective outwashing of fine particles from between coarser particles, and is one of several processes often referred to as *piping* (Quinlan, 1966). That residual granite soils are particularly susceptible to this process is illustrated by the typical particle size distribution curves shown in Figure 3/17 (Donaldson, 1963). These curves show that the soil is not a filter within itself (Blight, 1959) and that, under a sufficient hydraulic gradient, the finer particles will be washed out from between the coarser particles.

Suffosion is thus the process responsible for the development of collapsible grain structure in residual granite soils, by the selective washing out of deflocculated kaolinite colloids. When this process is accelerated under the influence of an increased hydraulic gradient, either natural or induced by man, subsidence of surface materials into the voids so created results in the formation of "sinkholes", "dolines" and other pseudokarstic features. The granite terrain in parts of the Northern Transvaal and Swaziland in situations where rapid incision by streams and dongas has locally created increased hydraulic gradients, affords spectacular examples of pseudokarst (Figure 3/18).

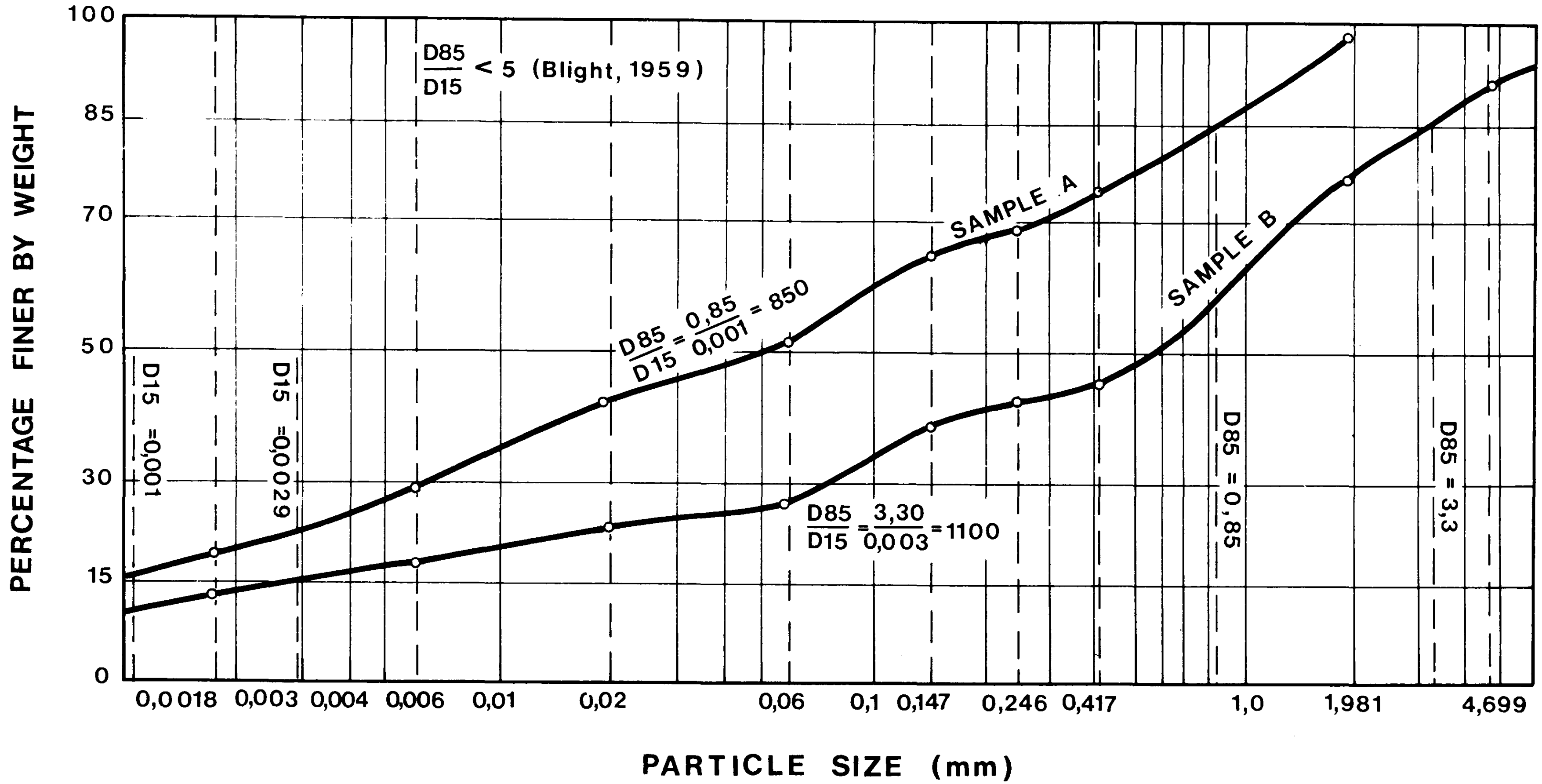
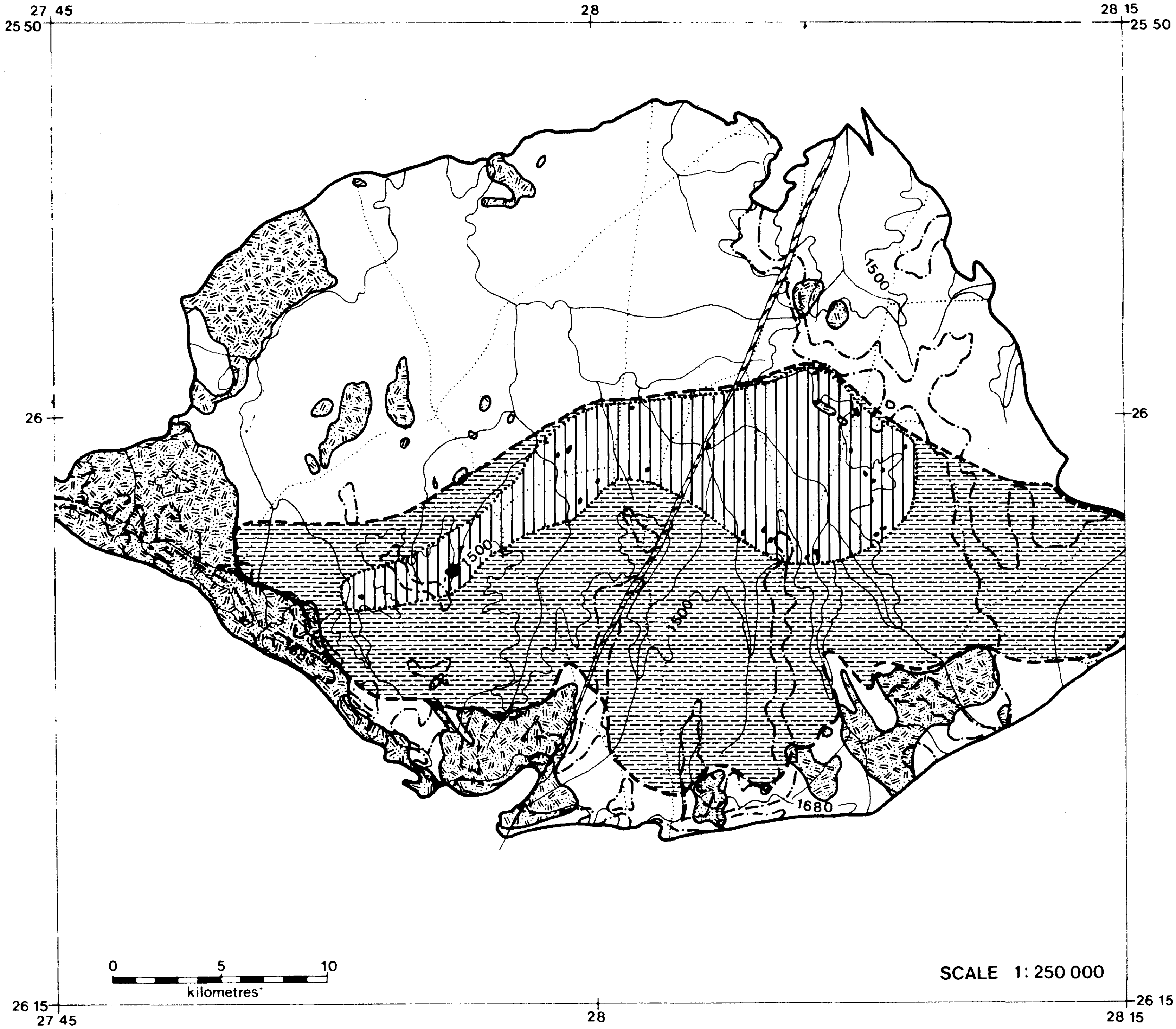
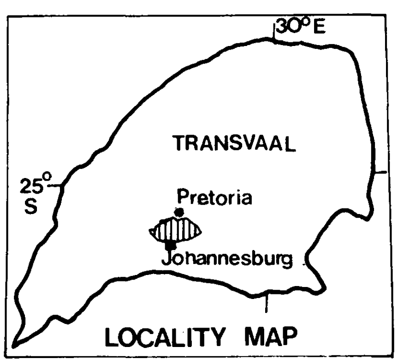


FIGURE 3/17 : Particle size distribution curve for residual granite soil from the site of Westphalia Leidam (see Case History 4)

D_{85} is a particle size such that 85% by weight of particles are smaller than D_{85}. A similar definition applies to D_{15}. The ratio D_{85}/D_{15} is a measure of the shape of the particle size distribution curve. The larger this ratio becomes, the flatter the grading curve becomes. If the ratio exceeds 5, as in this case, the soil will be susceptible to suffosion.



LEGEND



- Outline of Johannesburg - Pretoria granite dome
- Swaziland System
- Mylonite crush-zone
- Transitional zone with tors and core stones (after Anhaeusser, 1971)
- Inferred additional extent of core stones (homogeneous granite)
- Remnants of African surface(after Partridge 1975)
- Contour interval in metres
- Major drainage pattern
- Principal roads

FIGURE 3/14 : Distribution of residual soils containing core-stones within the Johannesburg-Pretoria granite dome

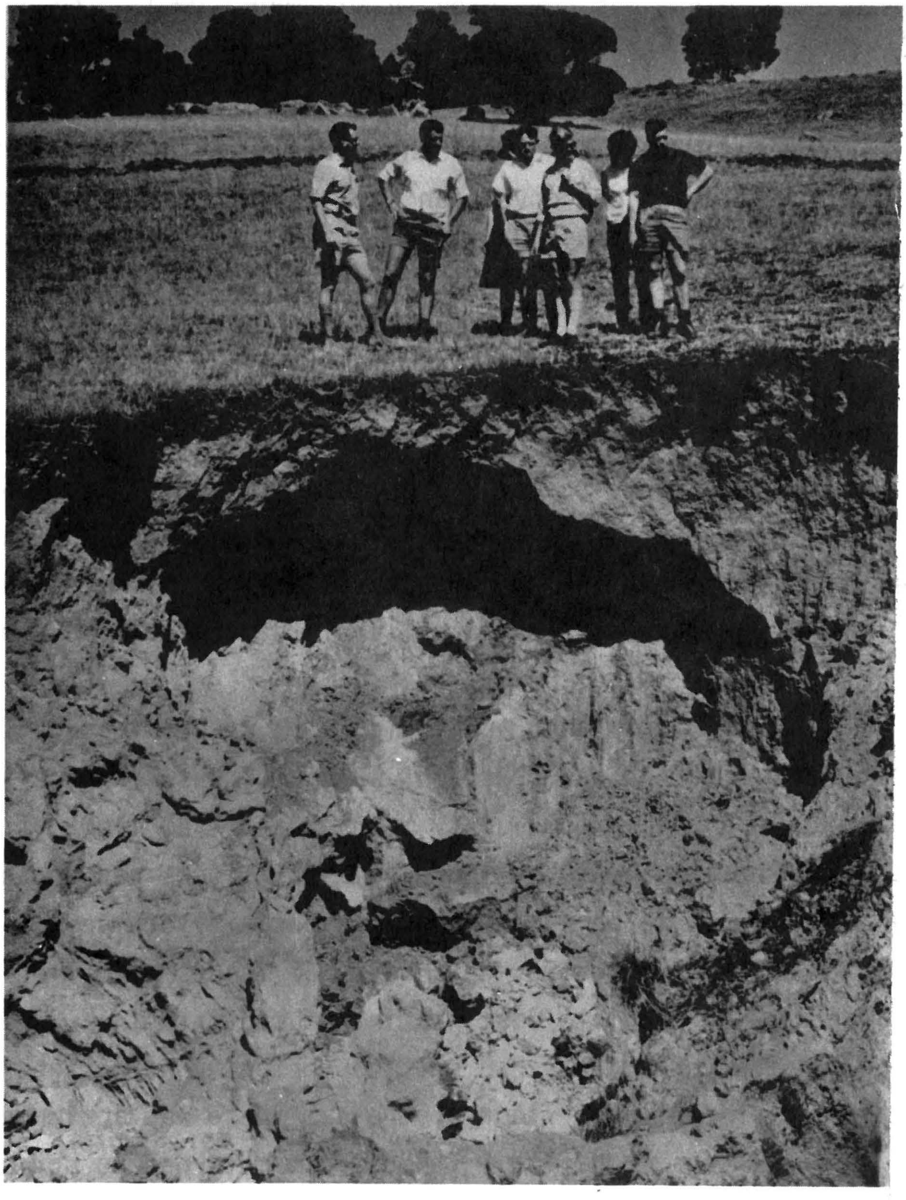


FIGURE 3/18 : Pseudokarst sinkhole in residual granite near Hlatikulu, Swaziland

CASE HISTORY 4

RESIDUAL BASEMENT GRANITE

FAILURE OF WESTPHALIA LEIDAM, EASTERN TRANSVAAL

The Westphalia Leidam was completed in November, 1960. The design was simple: it consisted of a hole dug into residual granite on the saddle of a ridge (Figure 3/19), with the excavated material compacted on three sides to form an earth embankment, and with the inlet from a canal on the highest side. The dam covers an area of nearly one hectare and was designed for a capacity of 36 million litres. At the time of construction it was not realised that the residual granite had a collapsible grain structure, and the engineers were therefore puzzled by the fact that their calculation of quantities proved to be about 40 per cent out: the hole had to be dug much deeper than originally specified, in order to provide sufficient earth to bring the embankment up to the required height. In retrospect it is clear that this miscalculation was a result of due consideration not being given to excessive loss of volume which would take place as the excavated collapsible material was compacted.

Some weeks after the dam had been filled with water, leakage at the rate of 85 litres per second was observed. A gauge measuring the flow of a small stream in a valley some sixty metres below the floor of the dam on the southern side showed an increase of 29 litres per second. The leakage was thought to be due to the high porosity of the soil below the dam and, in an attempt to provide a seal, 45 tonnes of milled exfoliated vermiculite and vermiculite dust was introduced into the water at the inlet. When the vermiculite had settled to form a watertight layer 60 mm thick over the whole floor of the dam there was an immediate reduction in leakage, and flow in the stream returned to normal for the next few months.

Then the dam started to leak badly again. The water was drained out, and it was found that large cracks, up to 500 mm wide and two metres long, and small sinkholes about 150 mm in diameter (Figure 3/20) had developed in the floor. It was also found that the embankment on the southern side had subsided about half a metre.

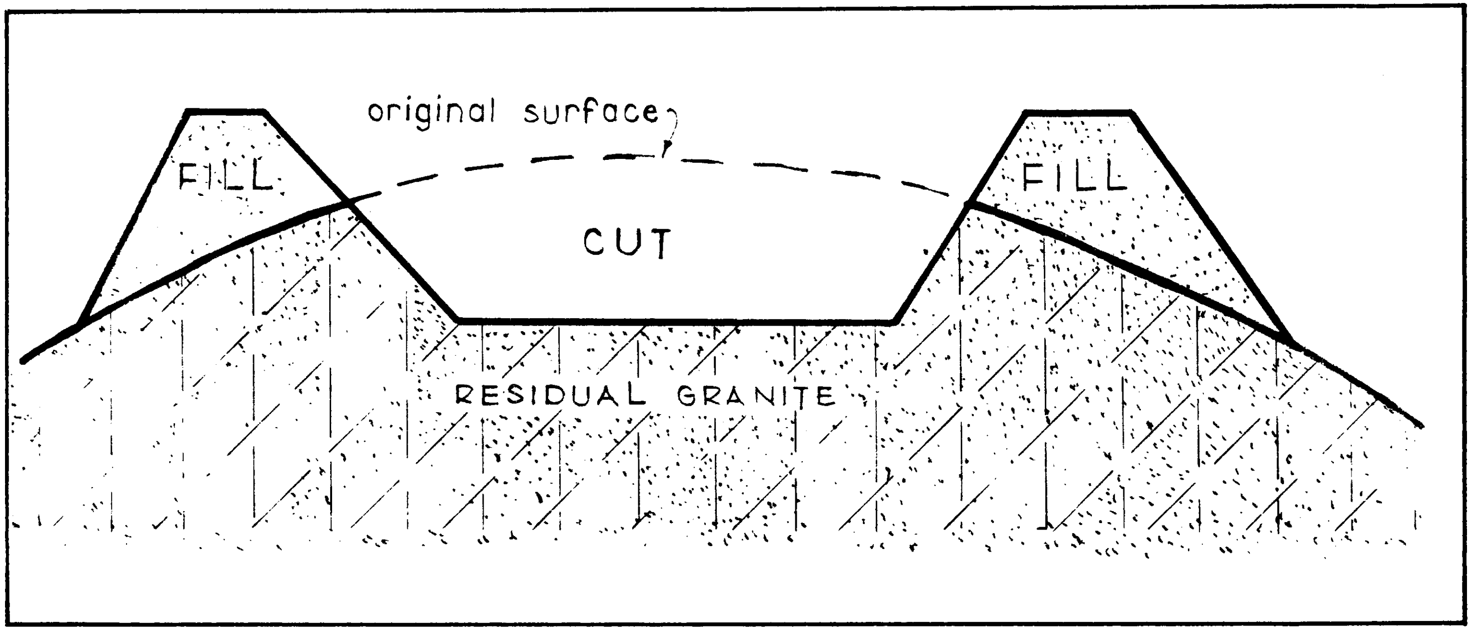


FIGURE 3/19 : Diagrammatic cross-section through Wesphalia Leidam

FIGURE 3/20 : Small 'sinkhole' in floor of Westphalia Leidam

The cracks were repaired by trenching them open to a width of 1,25 m and to a depth of 1,25 m and backfilling with the local soil plus a two per cent admixture of bentonite. In addition, the entire floor of the dam was lined with PVC sheeting, factory welded into large sheets which were then glued together and the joints covered over with adhesive strips (Figure 3/21). This work was completed in September 1963 and the dam was filled once more.

These remedial measures proved totally ineffective, however, and the dam started leaking badly again within a few months. A further inspection revealed more cracks in the floor and holes in the PVC sheeting above the larger cracks. An additional settlement of 300 mm was also observed in the embankment on the southern side. Further attempts at remedial measures were not considered economically justifiable and the dam had to be abandoned.

The failure of the dam may be ascribed to two related processes: collapse of grain structure in the saturated residual soil below the embankment, coupled with accelerated suffosion resulting from the introduction of an artificial hydraulic gradient.

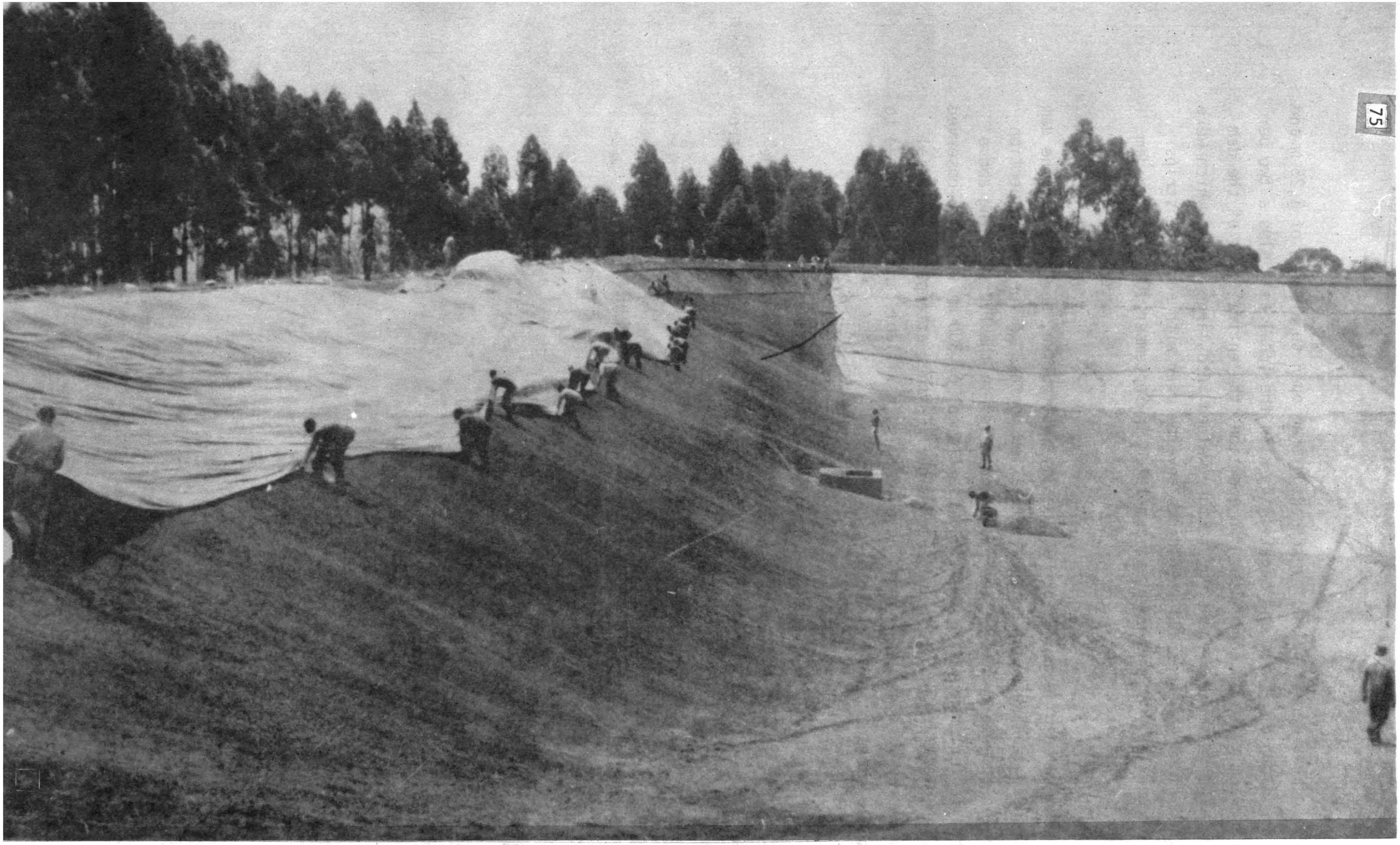


FIGURE. 3/21 : Lining the Westphalia Leidam with PVC sheeting : August 1963

4 LOWER DIVISION OF THE WITWATERSRAND SYSTEM

Outcrops of the Witwatersrand System are confined to relatively small areas. Nevertheless, foundation problems associated with the rocks of this system, and with the soils derived from them, are of particular significance as the city of Johannesburg and most of the towns of the Central Rand are situated largely upon them.

All of the exposures are situated within the area of relatively advanced chemical decomposition with climatic N-value (Weinert, 1964) less than 5. Consequently the shales, and in some cases even the quartzites, are weathered to considerable depths. In general the residual soils possess relatively high strength and low compressibility and contain clay minerals with stable lattice structure. They therefore present no major problems in foundation design for normal buildings. However, much of the central city area of Johannesburg is underlain by sediments of the Lower Division of the Witwatersrand System which strike east-west and in which relatively fresh quartzites are interbedded with residual shales and deeply decomposed diabase sills. Where a heavy building straddles the contacts between such diverse materials it is often necessary to provide deep piled foundations to support a row of columns, whereas an adjacent row might be adequately supported on normal footings at shallow depth.

This juxtaposition of soft residual soils and hard rocks also poses problems in any works requiring extensive excavation. In contrast with the relative ease of tunnelling by shield in the homogeneous stiff clays underlying the city of London, any planning for an underground railway for Johannesburg would have to take adequate account of the local geotechnical difficulties associated with the complexity of its geological setting. Because of this complexity the detailed geological map of Johannesburg compiled by J.H. de Beer (1965), and the Geotechnical Urban Data Bank which has recently been established in the City Engineer's Department, are both proving of inestimable value to engineers and planners concerned with development of the city.

Special problems of local significance are associated with the growing demand for deep basement excavations in the densely built-up city area.

HOSPITAL HILL SERIES

Undoubtedly the best founding conditions for heavy structures on the Central Rand are provided by the three bands of hard rock within the Hospital Hill Series, all of which form prominent topographic features, viz. the Orange Grove quartzite, the Contorted Bed and the Hospital Hill quartzite.

In the case of the Orange Grove quartzite, however, this statement must be qualified to a certain extent, as this band rests unconformably on rocks of the Basement-complex with a southward dipping contact. Protected from erosion by the hard capping of quartzite, deep residual soils on the Basement-granite have developed a collapsible grain structure, so that the wedge-shaped quartzite outcrop is underlain by a highly unstable soil. One of the dangers resulting from such an anomalous situation was illustrated some years ago in the suburb of Killarney when the side of a basement excavation straddling the contact collapsed after a heavy shower of rain, causing a short section of roadway in close proximity to slump into the excavation. Costly remedial measures included the installation of a retaining wall on the contact, which involved strapping the wall back by means of stressed cables drilled into the quartzite, before construction could be continued and the roadway reinstated.

Current development in Johannesburg includes the erection of several tower blocks of unprecedented height situated on the Contorted Bed, at bearing pressures of the order of 1 000 to 2 000 kPa or more. This intricately folded and faulted stratum is locally about 30 m thick and consists of finely banded siliceous and jaspery rocks interbedded with iron ores including magnetite, hematite and some pyrite. Within a metre or two below the surface outcrop the Contorted Bed is a hard rock, but engineers were in the past wary about the possible behaviour of the rock as a founding material in view of the commonly held notion that the contorted structure owed its origin to volumetric changes which accompanied chemical reactions after deposition (Hatch and Corstorphine, 1904). Examination of exposures in road cuts indicates, however, that the contortions and small-scale faulting have been caused by overthrusting movements from the south. The red shales both above and below the Contorted Bed have suffered the development of steeply inclined cleavage planes as a result of these stresses, indicating high confining pressures during the period of deformation. As cleavage is known to develop only in consolidated rocks, by brittle fracture, it may be concluded that the period of deformation occurred long after deposition of the sediments and was probably associated with the large-scale basining of the Witwatersrand System. Banded ironstones, wherever they are found, exhibit an 'incompetent' response to stress, owing to the intercalation of hard cherty bands with relatively softer iron-ores, and it is for this reason that the Contorted Bed has suffered more severe strain than the relatively 'competent' - though now much softer - shales above and below it. There is no evidence of residual stresses still operating within the rock, and it is therefore concluded that the Contorted Bed should provide adequate founding for heavy structures.

Brixton Tower is situated on the Hospital Hill quartzite. The tower has a mass of 6 350 tonnes and is founded on a ring beam 26 m in outside diameter and 6 m wide at a depth of only 2,5 m below the surface (Zunz *et al* 1965).

The shales of the Hospital Hill Series are seldom decomposed to any great depth, though refusal to a power-driven auger, where the shales are fissile, is sometimes as deep as 20 m. The shales are usually hard at a depth of about 6 m below the water-table. Except where the residual soils contain excessive termite workings, they are generally stiff in consistency and provide adequate founding for normal structures at relatively shallow depth. However, piled foundations have had to be used on sites where the decomposed shales have been thoroughly reworked by termite activity. Termite nests and channels sometimes extend to depths of as much as 12 m below the surface and occupy up to 75 per cent of the total soil volume. Such disturbance by termites has the effect of drastically decreasing the shear strength and increasing the compressibility, even to the degree of causing substantial consolidation collapse of the material when it becomes inundated under load.

In contrast with the 2,5 m depth of founding of the Brixton Tower, the Hillbrow Tower is founded at a depth of 40 m. The top of the 268,5 m high Hillbrow Tower is within an altitude of 1,5 m of the top of the Brixton Tower. But whereas the latter has a mass of only 6 350 tonnes and is founded on Hospital Hill quartzite, the former has a mass of 18 144 tonnes and is founded on shales of the same series. The shales dip south at an angle of 45^{0}. Down to a depth of 12 m the weathered shale could be excavated by hand. Below this depth the sequence consists of alternating bands with consistency varying from very stiff (soil) to soft rock, and with gouge material of soft residual soil up to 200 mm thick in the bedding planes. The water-table was present at a depth of about 22 m. The tower rests on a 4,2 m thick ring beam 13,72 m in diameter, which is supported on eight reinforced concrete piers 3,2 m in diameter. The piers were cast in shafts which were excavated by conventional mining methods. The material below a depth of 12 m had to be blasted, and the shafts were lined with unreinforced concrete as excavation proceeded. The shafts were underreamed at an angle of 60^0 at the bottom, to a diameter of 5,5 m. Below the founding depth of 40 m the gouge seams were considered to be of negligible thickness, and the consistency of the shale was soft to hard rock.

GOVERNMENT REEF AND JEPPESTOWN SERIES

In contrast to the rocks of the Hospital Hill Series, the shales and often even the quartzites of the Government Reef Series and particularly of the Jeppestown Series, are decomposed to very considerable depths, producing residual soils of limited bearing capacity. The depth of refusal to a power-driven auger may be as much as 25 m in residual argillaceous quartzite and even deeper in the arenaceous shales. The trough-like depression along the total strike of the Jeppestown Series, a depression formerly occupied by undrained pans and marshes of which Florida Lake is a dammed-up remnant, is attributed to the low resistance of these sediments to weathering and erosion. The water-table today, confined within ground-water compartments of various sizes bounded by diabase dykes and sills, may be encountered at depths varying from 6 m to over 30 m. However, there is a predominance of yellow residual soils in this sequence which indicates a hydrated state of the ferric oxides. This is further evidence of the recent lowering of the water-table after a prolonged condition of complete saturation. Decomposition of the Jeppestown sediments has thus taken place under saturated conditions, and the soils are amongst the few deep residual soils of the Highveld which have not suffered desiccation.

One isolated case is on record, however, where local desiccation of residual Jeppestown shale produced a heaving potential in the soil. The municipal swimming bath at Primrose, on the northern side of Germiston, was built during 1953 on a site which had formerly been occupied by a plantation of blue-gums (Rigby *et al*, 1954). Before the bath was opened to the public, leakage losses of about 115 litres per hour were observed and cracking started to develop in the building occupied by offices and change rooms. Enquiry revealed that the soil had been excessively dry during excavation. Although no cracks had developed in the bath, which had been constructed in 7,5 m squares separated from one another by adequate expansion joints, investigation showed that heaving movements had been responsible for causing leakage through the joints. The cracking pattern in the building confirmed that heaving movements were taking place, and the severest cracking was observed at points situated above broken drains. Tests on samples of the residual soil gave a maximum Liquid Limit of 45, Plasticity Index of 16, Linear Shrinkage of 10 and clay content of 18 per cent, indicating potential expansiveness of a relatively mild order from which, under normal conditions, little heaving movement would have been expected. However, owing to local desiccation caused by the blue-gums which had been felled just prior to construction of the bath, coupled with the presence of large quantities of water leaking from the bath and broken drains, conditions favourable to substantial heaving movements had been created. Applying D.H. van der Merwe's (1964b) method for the prediction of heave, it is calculated that a total heave of 52 mm would be expected for the whole soil profile wetting up from a desiccated condition, and the observed cracking pattern was indeed commensurate with movements of this order. It is of interest to note that all but very minor movements ceased within a few months, and that after the expansion joints, drains and cracks had been repaired, maintenance costs were only a little higher than normal. Recent enquiries indicate that this continues to be the case to the present time.

DEEP BASEMENTS

It was originally with the residual soils and rocks of the Jeppestown Series, together with associated deep residual diabase soils, that the problem of lateral support for deep basement excavations was concerned in Johannesburg. During the past decade, however, the techniques for providing such support, which were developed in the Jeppestown Series, have been successfully applied in other series of the Witwatersrand System and also in the residual soils of the Ventersdorp System in the city, as shown in Table 4.1.

Associated with the problem of lateral support in an excavation which may be as much as 30 m deep, is the problem of the effect of such excavation on the foundations of adjacent structures. The development of techniques for the stabilisation of such deep holes in a built-up area is a relative innovation, and one in which Johannesburg engineers and geologists have made pioneering contributions. In the words of George Rhodes-Harrison (1967), the distinguished Johannesburg architect and planner, in introducing the 1967 Symposium on Deep Basements:

> "At the outset it should be said that the demand for multi-purpose deep basements is not something occurring anywhere, but is peculiar to the Central Business Districts (or CBDs) of large cities. This demand manifested itself first in Johannesburg about 1962 or 1963. Until then, the basements commonly constructed in the CBD of Johannesburg consisted of one or two levels with, on level sites, an excavation depth of approximately 9 metres. These excavations seldom encountered any serious subterranean water problems ....
>
> "Suddenly a new demand arose for multi-level basements with depths of 14 m and 18 m and later 30 m. This demand for an increased number of levels and increased depth occurred shortly after a significant increase in site area to 60 m square, or whole city blocks, was adopted for property development. In fact it is doubtful whether, without this increased site area, multilevel basements can be planned with high enough usable areas to be economic ....
>
> "We in Johannesburg stand therefore at the threshold of evolving or developing new techniques in exploration, design, documentation and construction. We face the problems of recognising new risks and legal implications and we must attempt to describe them contracturally correctly. In short, we are founding a new tradition or practice in regard to which we must still retain flexibility. Whilst our knowledge of the intimate geology has expanded tremendously and our consultants can talk with authority and confidence regarding their structural solutions, it is too early to claim that all implications, particularly town planning aspects, have in fact been fully identified ....."

From a geological point of view, the most important part of George Rhodes-Harrison's statement is that "these excavations seldom encountered any serious subterranean water problems". Indeed it is for this reason that the techniques evolved in Johannesburg differ radically from the traditional methods for lateral support in saturated ground developed in countries in the Northern Hemisphere.

One of the most ingenious solutions to the problem of stabilising the side-walls of a deep basement excavation in "partially saturated" soils in a built-up area followed from the development of power-driven bored piling machines capable of augering to 36 m in decomposed rock, coupled with the development of techniques for stressing cables which have been anchored into small diameter holes drilled with rock-drilling equipment. The augered piles had been developed originally for foundations in expansive soils, and the stressed cables as an extension of the rock-bolt principle in deep mining operations: a combination of the two techniques, developed and refined during the past decade, has provided a novel solution for supporting the sides of a deep basement excavation. This was first applied in Johannesburg on the Rand Daily Mail site in 1963 (Parry-Davies, 1967).

Cast-in-situ augered piles are first installed at closely spaced intervals around the periphery of the site to be excavated. When excavation commences, the piles provide lateral support based on the principle of the cantilevered bulkhead. As excavation proceeds to greater depth, the piles are tied back by means of the stressed anchor-cables and further support is provided, based on the principle of the anchored bulkhead. This is illustrated diagrammatically in Figure 4/1.

In practice this procedure for provision of lateral support is complicated by a number of factors, not the least of which is the juxtaposition of soft soils and hard rock within the confines of the site, and the presence of geological discontinuities in the rock. Account has to be taken of the nature and the orientation of such discontinuities and of the hydrostatic pressures involved when water accumulates in defective planes which have an unfavourable orientation relative to the free excavation face. Problems of side support are increased where the water-table is shallow and the excavation acts as a sump. Experience continues to be gained in dealing with these complications, however, and the method shows promise of providing one of the most economical solutions for stabilising deep excavations in densely built-up areas.

This, and other methods which have been pioneered in Johannesburg for stabilising deep city excavations, have led to the preparation by the South African Institution of Civil Engineers of a Code of Practice for "Lateral Support in Surface Excavations" (1972) which is the first manual of its kind to have been published anywhere in the world. In

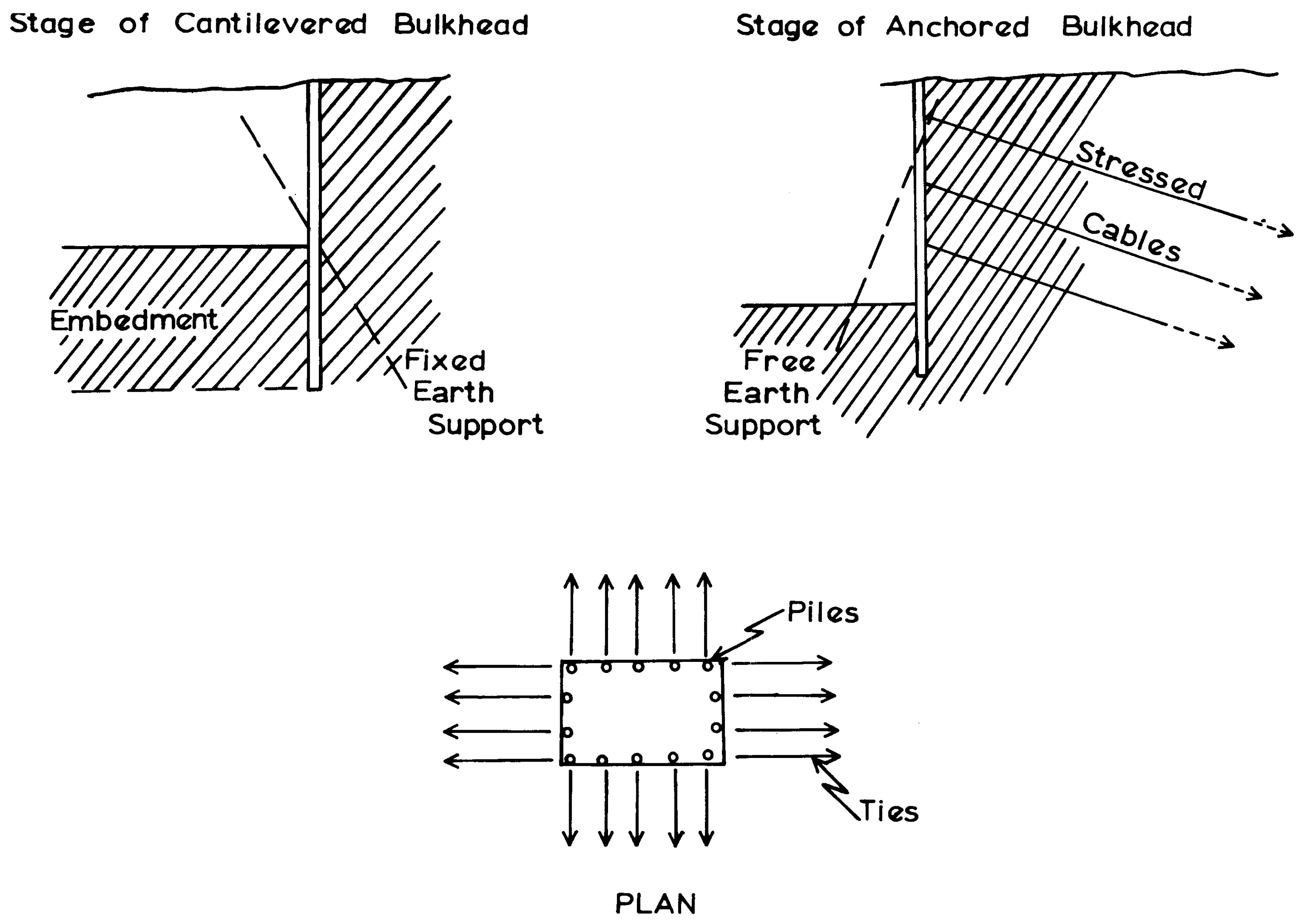


FIGURE 4/1 : Diagrammatic presentation of the principles of the cantilevered bulkhead and the anchored bulkhead in deep excavations in a built-up area addition to the method described above, this manual describes a variety of systems used for lateral support, including the use of batters, inclined shores, flying shores (i.e. bracing right across the excavation), cantilever piling without the application of tie-back anchors, *in situ* walling cast in trenches where mud slurry support is required, and a number of unconventional support systems.

The choice of the most suitable method of lateral support for any particular project is naturally dependent on a number of factors, the most important of which, in the view of the writer, is the nature of the geological formation into which the excavation is to be made. Table 4.1 lists some of the deep basements which have been excavated in Johannesburg during the past decade, together with an indication of the geological formation at each site and the method of lateral support used.

Street Location	Name of Building	Area m^3	Depth m	Number of Levels	Geology	Lateral Support Method Used	Remarks
Saratoga Harrow	Ponte Flats				Hospital Hill quartzite	This was not a basement. Twenty 100 tonne permanent vertical cables were installed to anchor the building down	
Hancock Quartz	Joubert Park Nursing Home	47 x 32 = 1 404	7 - 10	3 to 4	Ventersdorp lavas	Ground anchors	This was one of the early applications of ground anchors in soft ground: great difficulties were experienced.
Height Nind	Perskor	60 x 33 = 1 980	13	3	Ventersdorp lavas	Ground anchors installed through perimeter piles	Anchors in soft ground proved very successful.
Plein Hoek	Damelin College	27 x 61 = 1 647	13,2 to 8,6	3	Ventersdorp lavas	Ground anchors, combined on east face with piles for underpinning	
Bree Von Brandis	Wembly Building	30 x 30 = 900	9	3	Government Reef quartzite (Q3)	Ground anchors	
Bree End	Sandbury	28 x 41,5 = 1 032	9,0 to 6,4	4 half levels	Government Reef quartzite (Q2)	Ground anchors	

TABLE 4.1 : Deep basements excavated in Johannesburg

Street Location	Name of Building	Area m^3	Depth m	Number of Levels	Geology	Lateral Support Method Used	Remarks
Kerk Pritchard Fraser	Sage Building	46 x 30 = 1 380	11	3	Government Reef quartzite (Q5) shale (S5) and quartzite (Q6)	Ground anchors	
Jeppe Pritchard Von Wielligh Delvers	Sanlam Centre	137 x 77 = 10 550	11	3	Government Reef quartzite (Q3) diabase and Government Reef quartzite (Q4)	Ground anchors	
Pretoria Quartz Claim	Elkam	63 x 31 = 1 953	11	3	Contorted Bed and Hospital Hill shale	Ground anchors	
Pretoria Cavell	Harmo	46 x 30 = 1 380	7,5	2	Hospital Hill shale	Ground anchors	Anchors inserted only in one face under adjacent building.
Kotze Claim	Medical Hill	31 x 30 = 930	7 - 7,5	3	Hospital Hill shale	Ground anchors	
Jorissen Melle	Black Rose	26 x 17 = 442	7 - 8	2	Ventersdorp lavas & Hospital Hill shale	Ground anchors	
Jorissen Loveday	Traduna	67 x 27 = 1 809	6 - 7	2	Hospital Hill shale and quartzite	Ground anchors	
De Korte Simmonds	Telephone Exchange	48 x 30 = 1 440	5	1	Hospital Hill quartzite	Ground anchors	

TABLE 4.1 (cont'd)

Street Location	Name of Building	Area m^3	Depth m	Number of Levels	Geology	Lateral Support Method Used	Remarks
Smit Rissik	Wanderers View	78 x 66 = 5 148	13 - 16	3 + 1 over half site	Ventersdorp lavas (and quartzite "floater")	Ground anchors	Founded on or near quartzite "floater".
Smit Wolmarans Twist	Athlone Hotel	45 x 34 = 1 530	6	2	Ventersdorp lavas	Ground anchors	
Market Rissik Joubert	Belmark	78 x 32 = 2 496	7 - 8	2	Jeppestown lower arenaceous shales (jS1)	Ground anchors	
Market Eloff	Katz & Lurie	31 x 30 = 930	10	3	Jeppestown lower arenaceous shales (jS1)	Ground anchors	
Fox New Street North Rissik	Homes Trust	47 x 35 = 1 645	10 - 12	3	Diabase and Jeppestown quartzite (jQ)	Ground anchors	
Main Marshall Troye Polly	Sandglen Towers	79 x 63 = 4 977	9,7 - 13	3 plus plant-room	Jeppestown lower arenaceous shales (JS1) and quartzite (jQ) with diabase sill and diabase dyke	Ground anchors. Soldiers installed before excavation in boreholes during Phase I. Phase II normal method	
Albert Loveday Rissik	M.I.P.F.	38 x 64 = 2 432	6,5 - 13	3	Main-Bird quartzite and diabase dyke	Ground anchors	

TABLE 4.1 (cont'd)

Street Location	Name of Building	Area m^2	Depth m	Number of Levels	Geology	Lateral Support Method Used	Remarks
Market Kort Commis-ioner West Diagonal	Markort	120 x 63 = 7 560	8 ±	2 plus one for plant	Jeppestown quartzite: southern half of site on diabase dyke	Ground anchors	
Hoek Wanderers Plein	Cathedral Towers (Darragh House)	63 x 23 = 1 450	10,3 - 12,4 (9,5 - 6,6 below Cathe-dral)	4	Ventersdorp lavas	Ground anchors on S, E and W faces; anchors through piles on N face under Cathedral	
Marshall Kruis Main	I.B.M. Building	47 x 63 = 2 976	15,9 - 17	5	Jeppestown quartzite (jQ)	Ground anchors including some permanent anchors	
Pretoria Claim Kotze Twist	Highpoint	6 600	9,1 - 14,9	4 (max)	Hospital Hill shales and Contorted Bed	Ground anchors	
Fox Von Brandis Main Eloff	Trust Bank Building	63 x 63 = 3 969	29	8	Jeppestown quartzite (jQ) lower arenaceous shales (jS1) and diabase dyke	Continuously curved peripheral wall with application of pressure between corner arches and face of excavation, by means of grouting, to produce ring compression.	

TABLE 4.1 (cont'd)

CASE HISTORY 5

JEPPESTOWN SERIES

EXCAVATION OF BASEMENT FOR THE STANDARD BANK CENTRE: FOX, HOLLARD, MAIN & SIMMONDS STREETS: JOHANNESBURG

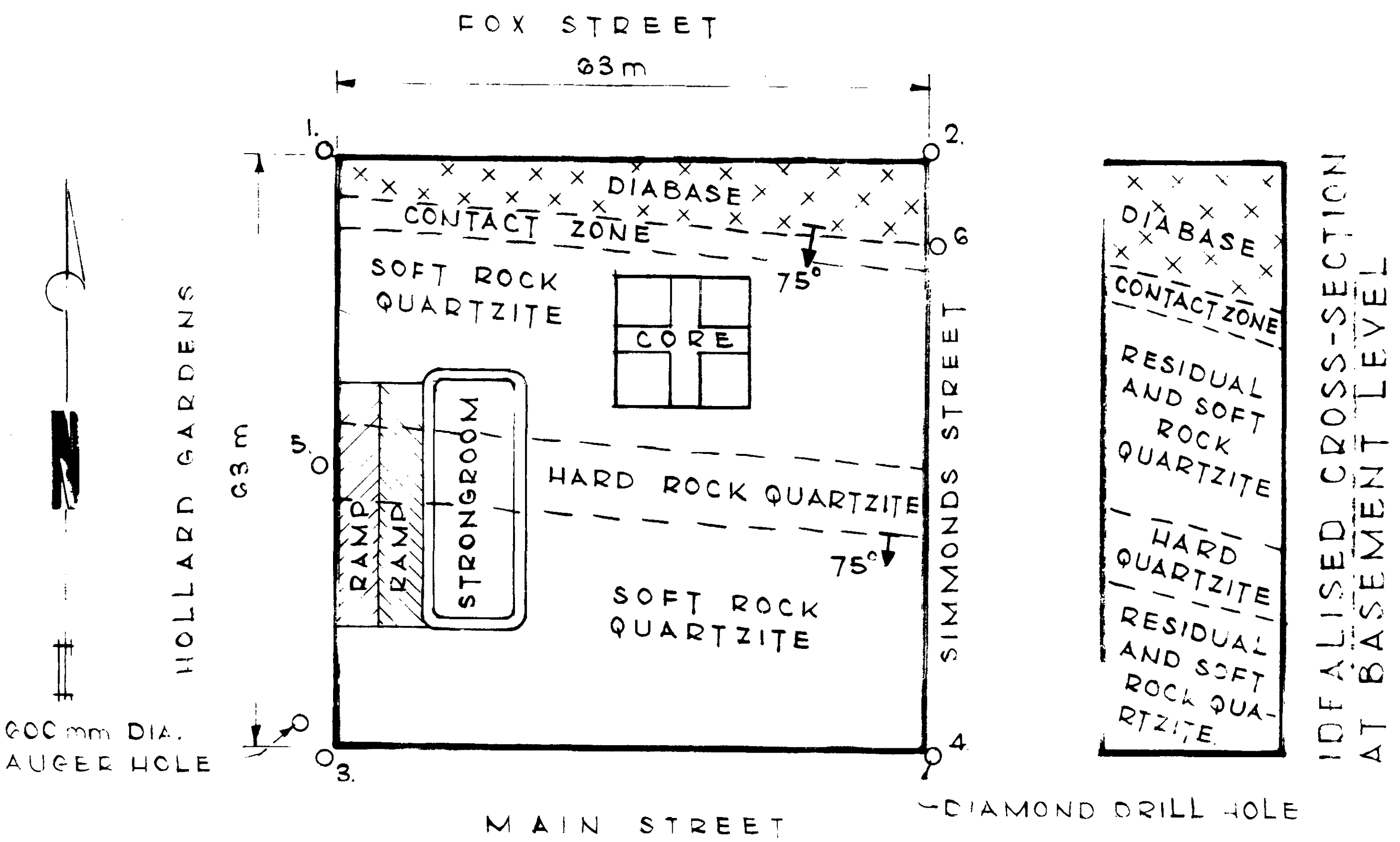
The site is a typical 63 m square central city block. The building is a 140 m high office tower formed by three groups of frames cantilevered off a central core which rises from an 18 m deep basement.

Site investigation carried out during 1966 included:

1. Six preliminary diamond-drill holes to a depth of up to 45 m around the perimeter of the site, with a piezometer installed in one of the holes to determine the depth of the water-table. These holes indicated that the site was occupied by residual quartzite of the Jeppestown Series (locally overtilted and dipping steeply to tbe north) with a transgressive sill of decomposed diabase striking east-west along the northern perimeter and dipping south, as shown in Figure 4/2. Hole 2 encountered highly jointed diabase, holes 3 and 4 residual quartzite grading into soft rock quartzite and hole 5 hard rock quartzite. Holes 1 and 6 passed through residual quartzite into diabase, with a highly weathered contact-zone about 6 m thick between the two consisting of "dark reddish orange, soft to firm, highly slickensided and jointed with soft clay gouge in fractures, clayey silt residual from diabase".

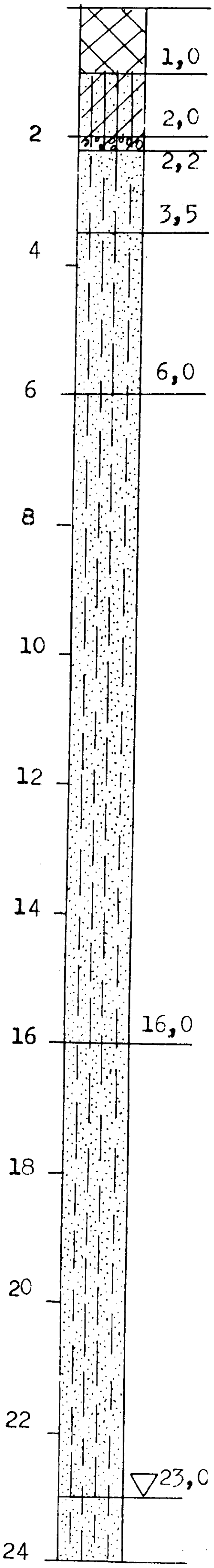
2. Seven 900 mm diameter trial-holes augered to refusal of a Hughes LDH 100 Digger, to establish the consisitency of the material and the extent of fissuring of the weathered rock. Refusal depth varied greatly but was on average 17 m in the diabase and 6 m in the quartzite.

3. A special 600 mm diameter hole augered at the south-west corner of the site to a depth of 24 m in the quartzite to allow detailed visual examination of the material and to enable contractors to inspect the material before tendering. The soil profile of this hole as recorded by the writer is given in Figure 4/3.



BOTH PLAN AND CROSS-SECTION TAKEN AT BASEMENT LEVEL

FIGURE 4/2 : Idealised geological map and cross-section of Standard Bank Centre site as inferred from the site investigation



Poorly compacted soil and rubble fill.

Moist, dark red, in profile brick red, soft, intact, sandy clay-silt; hillwash.

Vein quartz gravel in matrix as above, pebble marker.

Slightly moist, brown, in profile red and yellow, medium dense or firm, intact, medium and fine silty sand; residual quartzite with admixture of hillwash from above.

Slightly moist, brown, in profile predominantly yellow but with occasional broad purple speckled white bands, dense, apparently intact but with wide-spaced black-stained joints, silty medium and fine sand; residual quartzite with random veins.

Slightly moist, dark yellow, in profile yellow but occasional broad purple speckled white bands, very dense (but purple speckled white bands dense), randomly jointed at fairly wide spacing, with slickensided joints black-stained or colloid-coated, at random orientation but mainly steeply dipping, slightly silty medium and fine sand; residual quartzite with quartz veins. At about 12 m seam of soft red colloidal clay up to 75 mm thick dipping steeply (75° - 80°) south.

Moist, yellowish brown, in profile predominantly yellow but with occasional purple speckled white bands, very dense to very soft rock (but purple speckled white bands dense to very dense), randomly jointed at irregular spacings, with slickensided joints black-stained or with thin colloidal coatings, randomly oriented but mainly vertical or steeply dipping, slightly silty fine and medium sand; weathered quartzite of Jeppestown Series.

Water-table

ABAB
24/3/1966

FIGURE 4/3 : Soil profile recorded in 600 mm diameter Hughes LDH 100 Digger trial-hole on south-west corner of Standard Bank site, Johannesburg

4. After demolition of pre-existing buildings on the site, further diamond-drill holes to confirm the foundation conditions for the tower block core.

The site investigation showed the degree of decomposition of both the quartzite and diabase to decrease with depth. The residual diabase, a silty clay, had a shear strength at depth 9 m of 70 to 90 kPa which increased to 345 kPa immediately above the depth of auger refusal. Below the depth of auger refusal the weathered rock graded rapidly into fresh diabase of very hard rock to extremely hard rock consistency, with crushing strengths ranging from 40 to 100 MPa. The residual Jeppestown quartzite had a shear strength of 700 kPa at auger refusal depth, and then graded into weathered quartzite with a consistency of soft rock at about 32 m below surface.

The choice of excavation technique was largely dictated by two factors: the nature of the geological formation, and a critical path analysis which indicated that an early start on the office tower was fundamental to completing the contract in three years (Heydenrych and Yawich, 1967). It was thus decided to leave batters from the streets as far as possible before commencing a vertical excavation supported by anchors to accommodate the office tower and strongroom. Batters were the predominant form of lateral support on the eastern and southern perimeters where they could extend well into the site, while anchors provided the bulk of the support on the northern and western sides where there was no room for wide batters. The earth face was then strutted on removal of the batters from the central portion of the final basement structure.

The forces from the retained earth face on a basement of this depth are large. These forces can normally only be carried by balancing the loads across the site. This involves a complex construction programme with earth batters being removed uniformly on opposite sides of the site. In this case, however, the two major structures of the office block core and the strongroom construction were able to resist unbalanced loading conditions and allowed a simplification of the construction procedure.

The construction sequence was complicated by many factors but was essentially as follows (Zunz *et al*, 1971):

Stage 1: Excavation with 45^0 batters to the - 13,7 m level (Figure 4/4 A).

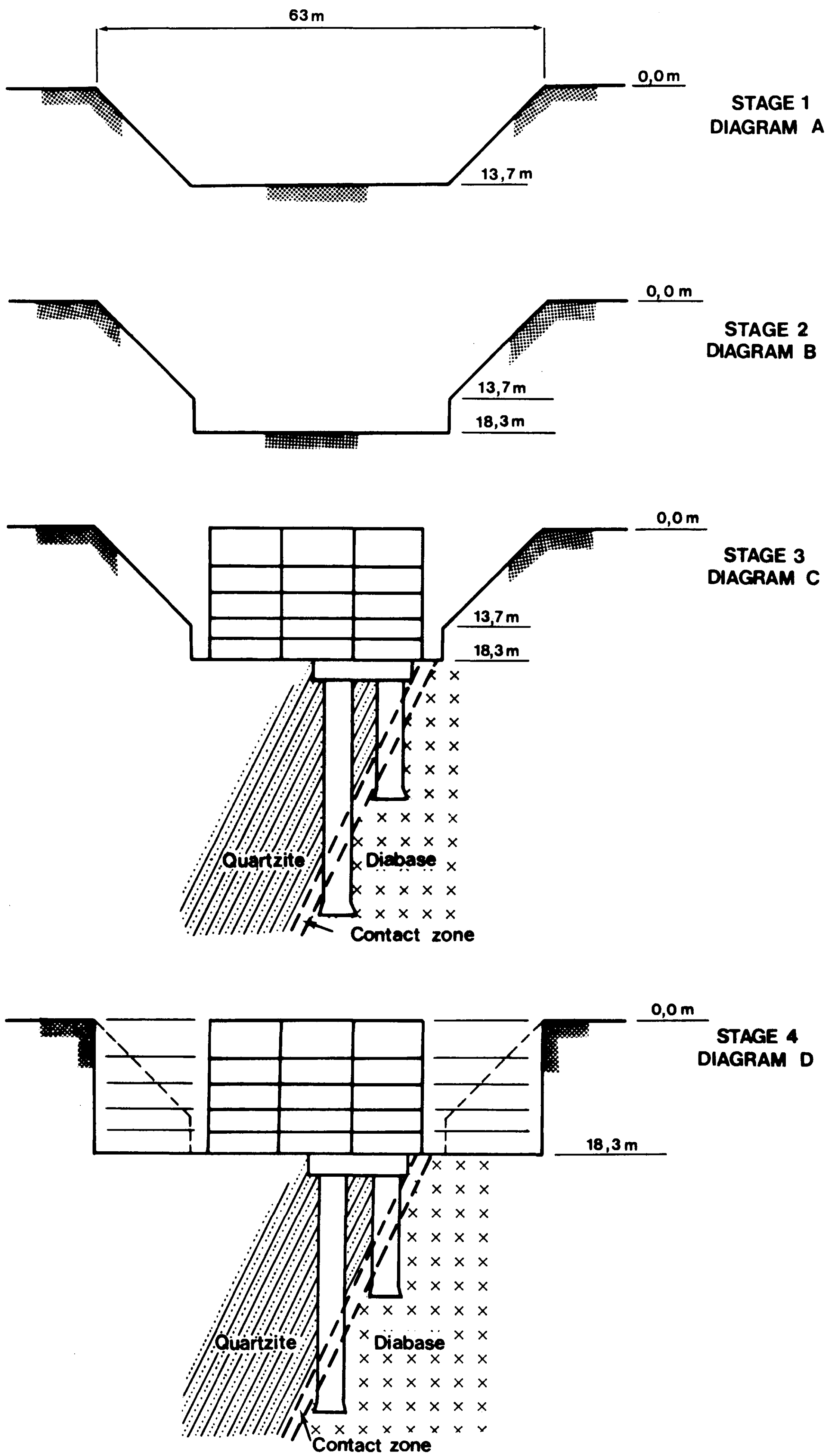


FIGURE 4/4 : Four stages in the excavation and construction on the Standard Bank site, Johannesburg (Largely after Zunz et al, 1971)

Stage 2: Excavation of the central portion of the site to the -18,3 m level, bracing the vertical face with ground anchors (Figure 4/4 B). The beginning of this stage coincided with a period of unusually heavy rains which continued throughout this stage of the excavation. The soft, deeply decomposed contact-zone flanking the diabase sill was exposed during this stage and a local slip occurred in this zone. Stability was re-established by installing additional ground anchors and by sealing the eroded residual soil behind the anchors with mass concrete. It is of interest to note that excavation of the east and west faces, disposed at right angles to the strike of the strata, produced no problems. Local wedge failures were confined to the north face. Bedding planes here dipped towards the open excavation and instability was further exacerbated by the unfavourable disposition of the soft diabase contact-zone. The most stable face of the excavation was the south face, where bedding planes in the quartzite dipped away from the open excavation.

Stage 3: Construction of foundations and basement structure in this central pit (Figure 4/4 C). It will be noted from the idealised geological map and from the section in Figure 4/2 that the diabase sill, together with its flanking contact-zone, dips towards the south where it passes beneath the core block. Even at the great depth involved, the contact-zone was found to be decomposed into a soft residual soil with the texture of clayey silt, and this necessitated the foundations for the office tower being taken down to solid rock. Solid diabase was present at the varying depths dictated by the dip of the sill, the deepest founding being on the southern side at a depth of 45 m (see Figure 4/5).

Stage 4: Removal of the earth bank while bracing the face from the completed structure, with construction of retaining walls from the top down as excavation proceeded (Figure 4/4 D).

Design considerations for the stability of the excavation were based on a computer analysis of a variety of wedge failure and slip circle failure patterns, and the factors of safety against failure were found to be in excess of 2 for the design adopted. Nevertheless, stabilising a deep

FIGURE 4/5 : *Excavation for Standard Bank, Jonannesburg* Taken from south-western corner between Stages 2 and 3 (See diagram C of Figure 4/4) basement with batters is in many respects less predictable than the application of other strutting techniques. Monitoring of ground movements was, therefore, carried out by making forthnightly checks for both vertical and horizontal movements on two concentric rings of control survey pegs. The inner ring of pegs was placed on the site boundary and the outer ring on the pavement opposite the perimeter of the site. Two further control pegs to act as bench-marks were located some distance from the site.

It was clear from the levelling observations on the survey pegs that the ground movements on the streets were directly related to the nature and dispostion of the geological formations. A portion of Fox Street in proximity of the northern face of the excavation subsided about 50 mm. The nature of the subsidence can best be described as a slumping condition caused by plastic flow in the contact-zone of the diabase. This was the only major subsidence: very small movements were observed on the other three streets, and these were more in a horizontal direction, towards the excavation, than in a vertical direction.

The piezometer readings and subsequent observations as the excavation proceeded indicated a natural water-table at 23 m below the original ground level. The presence of water was thus only a problem in the foundations of the tower core which penetrated to a maximum depth of 45 m below ground level. Water-proofing of the completed structure was provided by a cavity construction in the retaining walls: the 75 mm drainage cavity is enclosed between the concrete wall and a brickwork skin, the concrete wall being perforated with rows of no-fines concrete filter blocks which prevent the build up of external water pressure. A drainage layer is incorporated below the surface bed.

CASE HISTORY 6

JEPPESTOWN SERIES

EXCAVATION OF BASEMENT FOR CARLTON CENTRE: MAIN, KRUIS, COMMISSIONER AND VON WIELLIGH STREETS, JOHANNESBURG

The Carlton Centre is a R50 000 000 development comprising two major towers - a 50 storey office block 200 m high and a 37 storey hotel 122 m high - rising from a seven-level basement nearly 30 m deep. More than half-a-million cubic metres of soil and rock was excavated from the 122 m x 158 m site occupying four central city blocks. A shallower basement, below the parking garage on a adjacent block to the south, is connected to the main basement by a tunnel below Main Street.

The site is characterised by extremely variable geological conditions as shown in Figure 4/6. Shales, quartzites and argillaceous quartzites of the Jeppestown Series have been intruded by a conformable dolerite sill and an irregualr transgressive diabase body. The Jeppestown rocks have been locally overtilted and dip to the north at an angle of about 75^0. The dolerite, argillaceous quartzites and shales are deeply decomposed, the average depths to bedrock being about 35 m, 40 m and 45 m respectively. Typical soil profiles and test data for these materials are shown in Figure 4/7. The diabase is less deeply decomposed, bedrock generally being encountered at depths less than 15 m, while the arenaceous quartzite in the south is for the most part only moderately weathered.

The site slopes diagonally in a south-easterly direction, resulting in an excavation that varied in depth from 29,25 m in the north-western corner to 23,15 m in the south-eastern corner. The enormous size and varied depth of the excavation required a perimeter retaining wall that had to withstand tremendous earth pressure at extreme depths. Bracing had to be kept to a minimum, however, to permit up to two thousand men with heavy equipment to move around freely on site. The bracing structure eventually adopted comprised an octagonal concrete grid which carried the horizontal loads in compresson and tied closely spaced perimeter bulkhead piles into a monolithic retaining wall, as shown in Figures 4/8, 4/9 and 4/10.

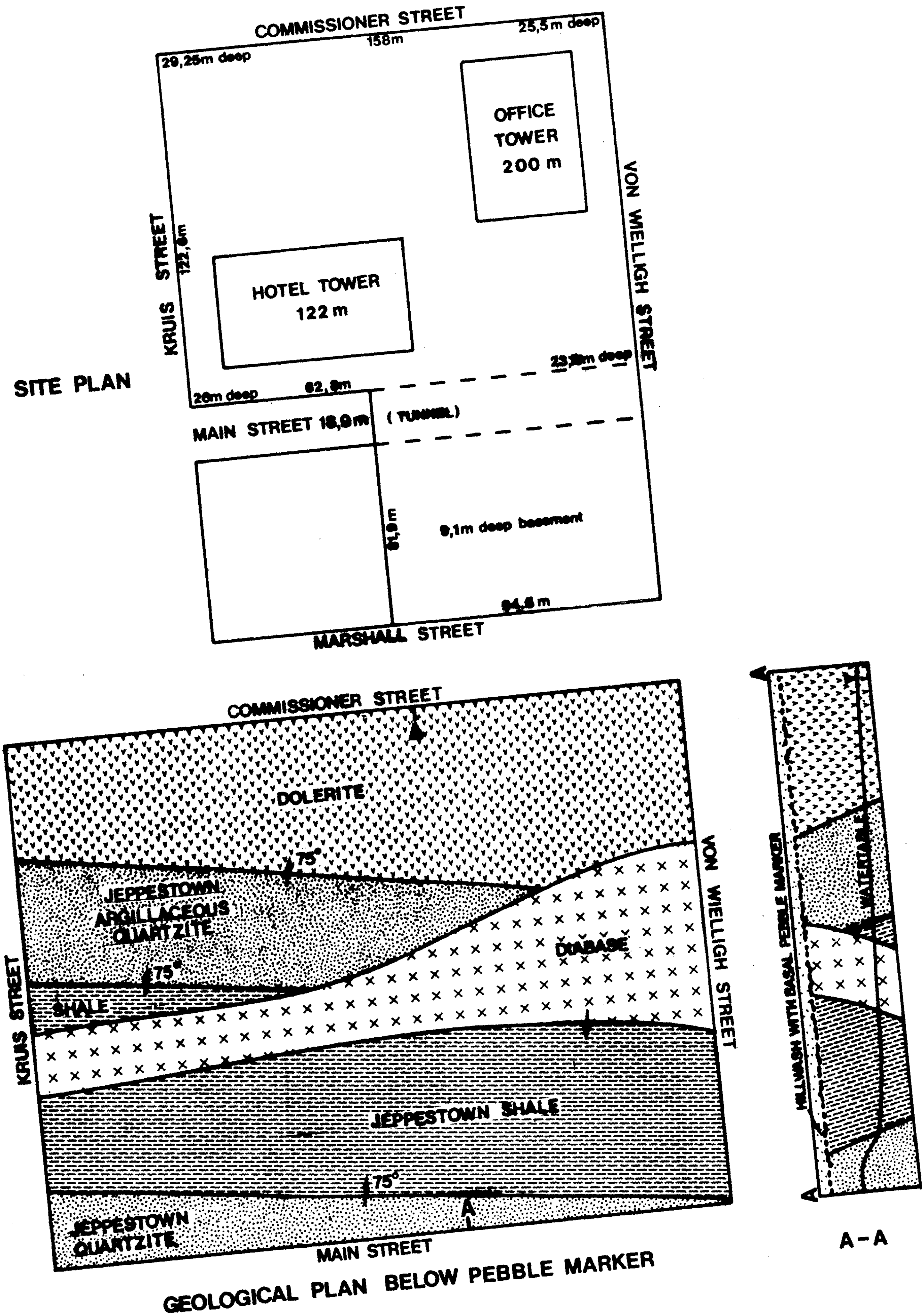


FIGURE 4/6 : Site plan of Carlton Centre site, Johannesburg, with geological plan and cross-section (After Isaacs and Heydenrych, 1987)

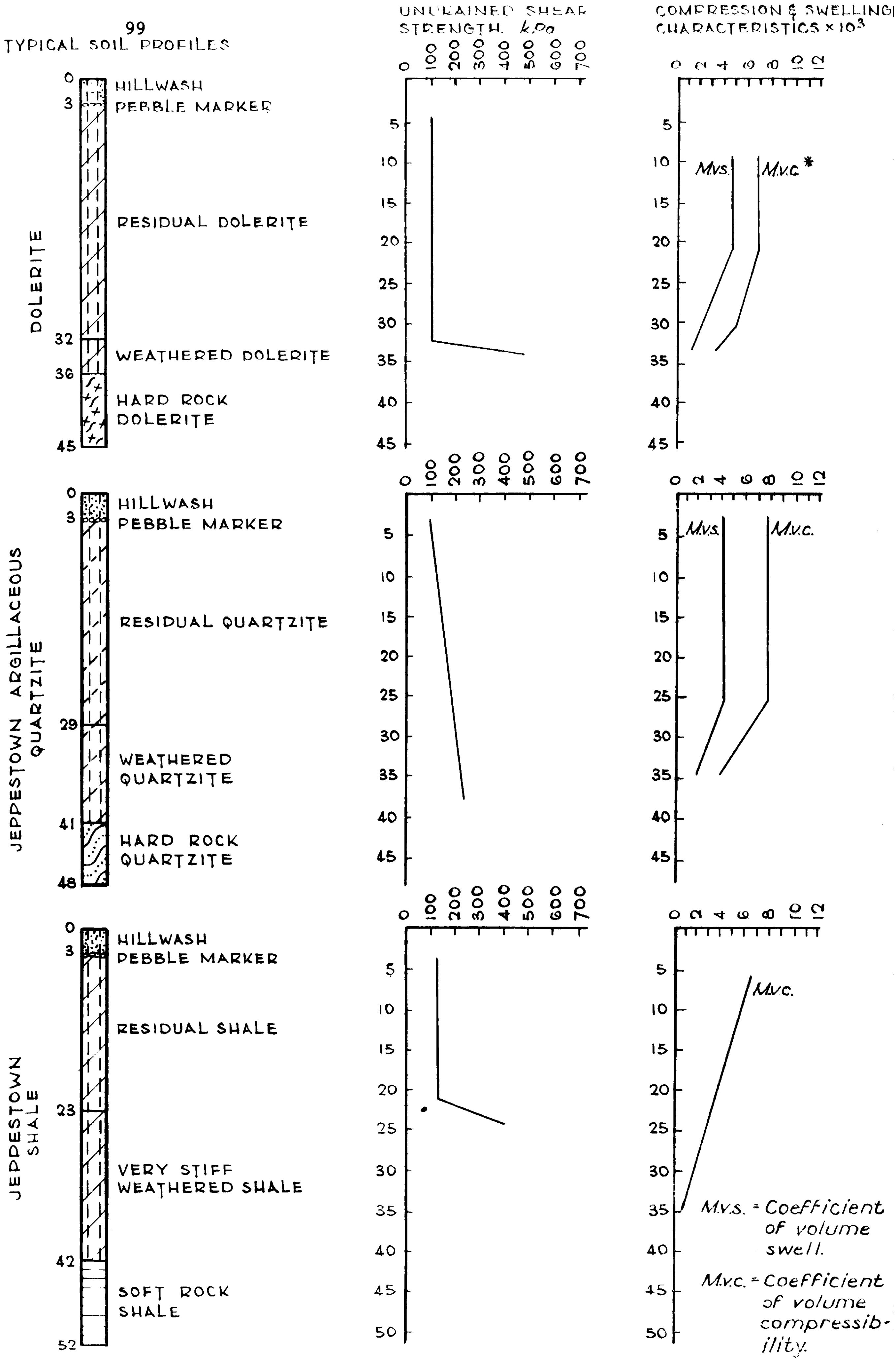
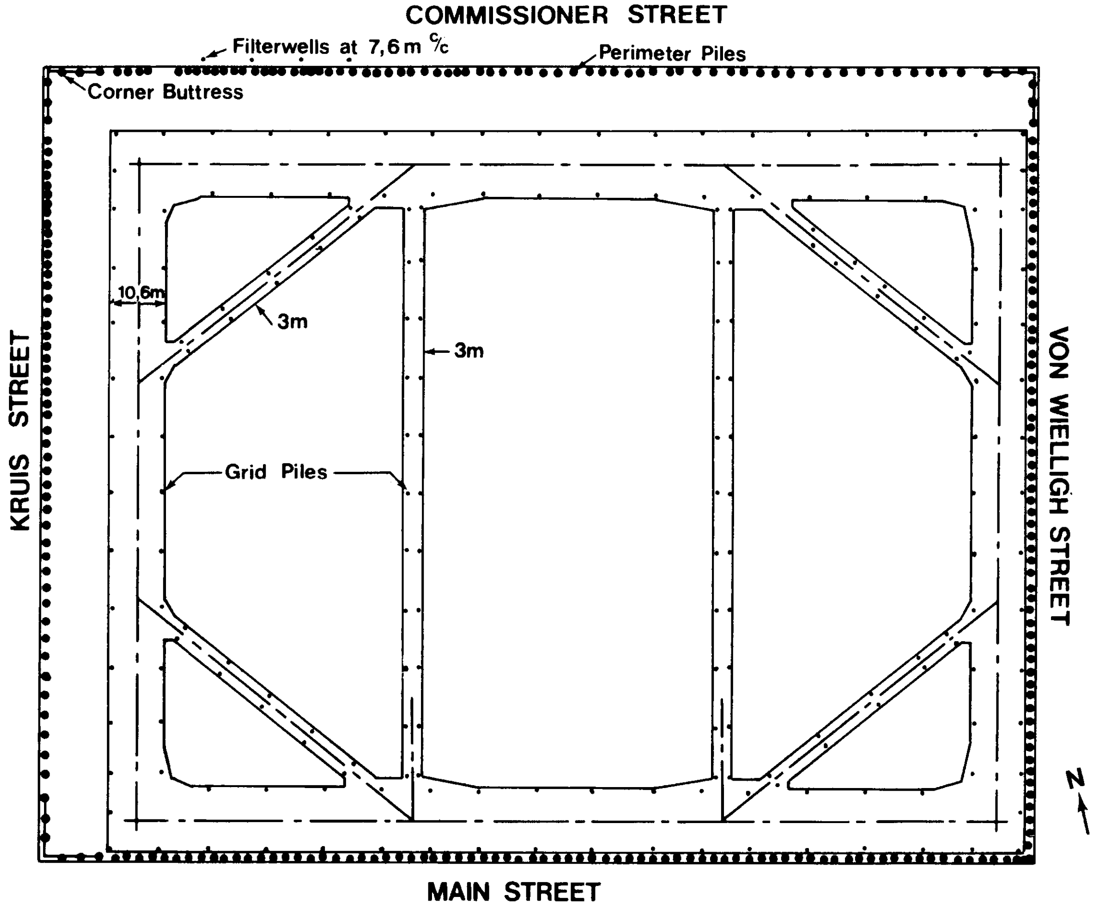
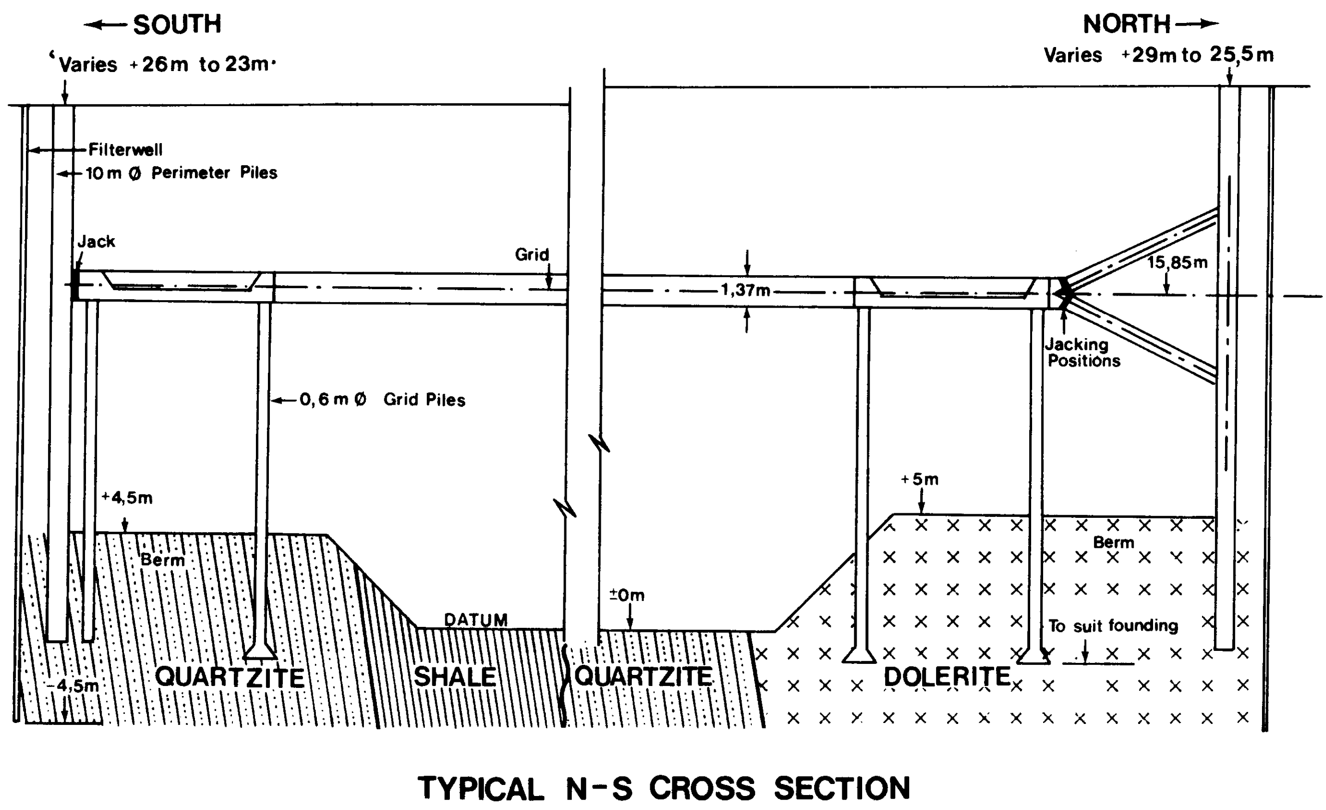


FIGURE 4/7 : Typical soil profiles and laboratory test data for materials on Carlton Centre site, Johannesburg

(After Isaacs and Heydenrych, 1967)



PLAN OF BRACING STRUCTURE



TYPICAL N-S CROSS SECTION

FIGURE 4/8 : Plan of bracing structure and typical cross-section through the excavation for Carlton Centre, Johannesburg

(Largely after Heydenrych & Isaacs, 1967)

FIGURE 4/9 : Excavation for Carlton Centre showing perimeter bulkhead piles and polythene sheeting protecting potentially unstable batter in residual diabase

FIGURE 4/10 : *Excavation for Carlton Centre showing octagonal grid bracing structure* (Photo by A.A. Gordon)

The construction sequence was as follows (Isaacs and Heydenrych, 1967; Heydenrych and De Beer, 1975):

1. Installation of filter wells around the perimeter to lower the water-table to below basement level. Design of the filter wells was based on experience gained on the Rand Daily Mail site and on a trial filter well installed in Commissioner Street. The filter wells were installed in 500 mm diameter auger and jumper-drill holes sunk to 4,5 m below basement level. Immediately after drilling, casing (which was later withdrawn) was installed to prevent collapse of the sides. Slotted metal pipes 100 mm in diameter were then installed in the holes. Filter sand was poured between the slotted pipes and the earth, and this was compacted by vibrating the inner tube. The base of the pipe was sealed with a welded plate. Slots, 300 mm long by 1,5 mm wide, were cut in the pipe to 6 m above the water-table. Finding a suitable filter sand proved difficult; grading analyses of the residual dolerite and of numerous pit and river sands in the Transvaal showed that naturally occurring sands were unsuitable. Sand from the Jukskei River near Hartebeestpoort Dam seemed best, and this was used initially on site. However, this sand did not meet the specification for a filter medium in which 60% of the material must be of larger grain size than the diameter of the slots in the casing; an excessive amount passed through the slots during consolidation by vibrating. The problem was eventually solved by mixing two sands in suitable proportions: the Jukskei River sand, and a washed and graded quartzite crusher-run from Brits, in the proportions 3:1 respectively. The quantity of water pumped during the first six months was approximately 180 000 litres per day.

2. Augering from surface of 280 pile holes, one metre in diameter and an average of 30 m deep, along the perimeter of the site at centres varying from 1,5 to 3,0 m. Piles cast in these holes were reinforced to act as vertical columns to carry the earth loads to the bracing grid.

3. Excavation to the level of the bracing grid, leaving berms to limit the maximum cantilever of the bulkhead piles to about 10 m.

4. Installation of 200 piles to support the bacing grid. These later became free-standing columns after the surrounding soil had been excavated.

5. Construction of prestressed concrete grid, and application of axial loading by means of Freyssinet-type flat jacks.

6. Excavation of the site to 4 m below the grid and installation of inclined props against perimeter piles where necessary.

7. Completion of the remainder of the excavation leaving in position a 5 m high berm against the perimeter piles at the bottom of the excavation (Figure 4/8).

8. Construction of basement. After the earth pressures were transferred to the basement floors the bracing grid was demolished. To facilitate demolition of the grid, duct tubing had been incorporated into the concrete sections, and these were later used for the placing of demolition explosives.

On account of the extremely variable nature of the geological formations, and the variable depth and degree of decomposition suffered by the different rock types, the design criteria had to be varied around the site. The pressure distribution in the diabase and the arenaceous quartzite is dependent on the effect of the gradual strengthening of the material with depth as a result of the progressive decrease in weathering. The design of the grid and bracing system was tested for a series of possible overload conditions: this including the possibility of a burst water-main causing temporary increased water pressures. Caution had also to be exercised in assuming that the rock forms an unyielding base to the overlying residual soils. In addition, the problem of overall stability in the highly jointed rock mass required careful consideration.

Because of the transition from the residual soil, through weathered rock to fresh rock, the top of the "rock" was arbitrarily defined as material which caused refusal of a one metre diameter auger bucket using a Hughes LDH 100 Digger machine. As the rock below the depth of refusal was jointed, it was necessary to provide some lateral restraint to the rock in the form of smaller diameter piles drilled from the bottom of the large pile holes, and designed for a nominal horizontal pressure, in order to prevent spalling of blocks of material from the face of the excavation. It was possible to do this as it was found that an auger-bucket of smaller diameter could penetrate jointed rock to depths well below the refusal depth of the auger-bucket of one metre diameter.

The two tower blocks were founded on 3 m diameter unreinforced concrete shafts. The shafts were excavated from the bottom of the basement excavation by means of conventional mining techniques using hand excavation and *in situ* concrete lining. It was necessary to sink the shafts through alternate bands of hard and soft rock before they were eventually underreamed to the required diameters at depths varying from 43 to 63 m below street level, and subsequently filled with concrete. The remainder of the complex is founded on spread footings, except those parts underlain by argillaceous quartzite which required piled foundations.

The presence of hard quartzite under the whole of the planned parking site between Main and Marshall Streets forced the architects to move part of the parking into the basements under the towers, and to limit excavation on the southern section to a depth of 9,1 m.

Because of the nature of the local geological formation at the site, 54 per cent of the total development is below ground level. The office tower at Carlton Centre is the tallest building on the continent of Africa. Even though situated in the valley occupied by the Jeppestown Series, it contrubutes significantly to the imposing skyline of the city. Were it to have been built in a more commanding position, on one of the three ridges in the northern part of the city, the hard quartzite or banded ironstone of the ridges would have made excavation of a deep basement totally uneconomical.

5 UPPER DIVISION OF THE WITWATERSRAND SYSTEM MAIN-BIRD AND KIMBERLEY-ELSBURG SERIES

GEOLOGY OF THE GOLD-BEARING STRATA ON THE CENTRAL RAND

The Upper Division of the Witwatersand System comprises some 2 800 m of quartzites and gold-bearing conglomerates with only one shale horizon of any consequence. On the Central Witwatersrand the beds strike roughly east-west and dip to the south at steep angles which flatten with depth. Dykes and sills of basic igneous rock, mainly diabase, are intruded into the sedimentary rocks, the dykes frequently being intrusive along fault-planes.

The most extensively developed conglomerate zones are the Main Reef, Bird Reef, Kimberley Reef and Elsburg Reef groups, and it is the first of these which is of the greatest significance. In this group, which is up to 75 m thick, the following conglomerate reefs are encountered interbedded with quartzites: North Reef (lowermost), Main Reef, Main Reef Leader, Middle Reef, South Reef and South South Reef (uppermost). Of these reefs it is the Main Reef, the Main Reef Leader and the South Reef which have the highest gold contents and which have been most extensively mined on the Central Witwatersrand and specifically within the confines of the Johannesburg municipal area with which the following account is mainly concerned.

The Main Reef is 1 to 6 metres thick. In places it has been denuded and replaced by gold-bearing Pyritic Quartzites in the erosion channels up to 25 metres thick. Pay-streaks within the Pyritic Quartzites have been mined to stope widths of as much as 8 metres in places. The Main Reef Leader varies in thickness from 0,5 to 2 metres and lies directly upon the Main Reef or in close proximity to it. The South Reef usually lies some 30 metres above the Main Reef and has a thickness of about 0,5 to 1 metre.

Where all three of these reefs have been mined the aggregate stoping width is normally 4 to 6 metres. Where Pyritic Quartzites have been mined, the aggregate stoping width may be more than double this.

While the reefs dip southward at high angles on outcrop, sometimes more then 80^0, these steep dips generally persist to depths of no more than 120 to 150 m below surface before they flatten to an angle of about 25^0 or 30^0.

During the first decade or two of mining, from 1886 onwards, the Main Reef and the Main Reef Leader received practically all the attention, as the outcrop values in these two reefs were higher than those in the South Reef. When the relatively narrow 'middling' between the two stopes collapsed, as it often did, the surface excavations became more enlarged. Support in the stopes mined at shallow depth consisted mainly of untreated timber props and waste-filled 'pigsties'.

When the workings were later kept open to serve as upcast air-ways the humid return air accelerated the decomposition of the untreated timber (Pyne-Mercier, 1970). Individual props became rotten and local falls of the hanging wall were common. The same fate was suffered by timbers forming the cradles of pigsties, and the waste packs then slid down the steeply inclined stopes dislodging further props deeper down and leaving large areas of hanging wall unsupported. This resulted in stope closure on a more extensive scale and the outcrop areas became enormous excavations which had to be fenced off in the interest of safety. With the passage of time it was thus the areas immediately south of the outcrops which posed the greatest danger of surface subsidence.

MINING SUBSIDENCE

Thus, although good founding rocks in themselves, the quartzites of the Main-Bird Series present special problems of a largely unpredictable and unresolved nature, viz. subsidence associated with the undermined areas where gold-bearing banket reefs have been stoped out.

The amount and the extent of surface movement is variable and often difficult to predict, being dependent on the local dip, depth and thickness of the mined-out reefs, the number of reefs one below the other, the presence and disposition of dykes and faults, the type and extent of underground packing, the age of the stopes and the frequency and magnitude of earth-tremors.

The solution to the problem is rendered more difficult by the fact that, while the outcrop areas are in many places still accessible and amenable to treatment to ensure safety, the stopes down dip cannot be re-entered to introduce fresh support except at prohibitive cost (Pyne-Mercier, 1970).

Furthermore, as stated above, the problem is largely of an unpredictable nature. Indeed the words of Professor H. Briggs (1929), a pioneer worker in this field, expressed nearly half a century ago, still seem largely to apply to the Witwatersrand today:

> "Mining subsidence is not amenable to mathematical analysis and still less to mathematical synthesis. Empirical formulae are sometimes useful as guides and will probably become more so as experience extends and is made more fully available but, as things are at present, only the simplest rules are acceptable, and they should be received with ample reservations and applied with judgement."

Empirical formulae have indeed proved themselves useful as guides, and *are* becoming more so as experience extends. In addition, several fundamental aspects of ground movements have become apparent from observations south of the reef outcrops on the Central Witwatersrand, for example that:

(i) substantial subsidence and surface damage may occur above undermined ground even where the depth of undermining is more than 160 m;

(ii) subsidence may take place in areas which are not currently being mined;

(iii) collapse of a stope can result in horizontal movements at the surface in addition to vertical subsidence;

(iv) the removal of a pillar from a mined-out stope, for example in reclamation-mining, may cause a volume of subsidence in excess of the volume of the pillar removed.

The following examples illustrate the above observations:

Shallow depth of mining : less than 160 m

1. Fergusson's Halt at Randfontein Estates (1950):
Mining had taken place at a depth of 100 m and a maximum subsidence of 3 m occurred on surface. Surface cracks of up to 300 mm were observed (Else, 1957).

2. New Pioneer Central Rand Gold Mine (1926):
The removal of pillars from a worked out area resulted in a saucer-shaped depression at the surface, with vertical subsidence of 350 mm, horizontal movements of up to 152 mm and cracks up to 230 mm wide (Else, 1957). The depth of the stope was 90 m and the stope width was 1,8 m.

3. Simmer and Jack (1912):
Subsidence due to the mining out of two reefs to a stoping width of 3,35 m and to depths between 50 m and 160 m, resulted in the corner of a trading store building settling about 150 mm vertically and the wall tilting 406 mm out of plumb. Numerous cracks developed on surface (Else, 1957).

4. Main Reef Road near Longdale Township (1960):
Subsidence of one metre in tarred road surface. (See Case History 7).

5. House in Cleveland Road (before 1969):
Subsidence of house by about two metres without structural damage. (See Case History 12).

6. Nourse Mines Limited (1942):
Vertical differential movements of up to two metres occurred suddenly over an outcrop area with cracks of up to 500 mm wide opening up. (See Case History 11).

Intermediate depths of mining : 160 - 500 m

7. City and Suburban (1926):
About 780 m of a Johannesburg sewer had to be relaid after suffering differential settlement of up to 810 mm. The sewer was underlain by a stope 2,3 m wide, at depths between 61 m and 348 m.

8. Randfontein Estates Gold Mines (1926):
Ground movements resulting in surface cracks 100 mm wide and with vertical displacement of 75 mm continued for a period of two weeks. The underlying stope was between 56 m and 332 m in depth.

9. Geldenhuis Deep (1920):
Crushing of underground pillars resulted in surface cracks up to 27 mm wide, even over the deeper portions of a stope which lay at depths between 150 m and 457 m.

10. Johannesburg Motorway (1969):
A large crack in the hard rock was exposed in the excavation for a bridge for City Deep Access Road. It extended to a depth of 6 m and over a distance of 18 m. The width of the crack varied but was up to 12,5 mm, and it extended downwards into the rock. Undermining was at a depth of 396 m, but the crack appeared above an unmined dyke. It was undoubtedly associated with tensile stresses acting at the surface.

11. Rand Leases (1946):
In 1946 it was decided to extract the Main Reef in an area where the Main Reef Leader and the South Reef had already been mined out. A substantial subsidence of the ground was observed in an area where the depth to the Main Reef varied from 195 m to 238 m. The subsidence was adjacent to a dyke dipping at an angle of about 80^0. A vertical step appeared at the surface, with a displacement of about 300 to 450 mm. A crack at the foot of the step measured about 150 to 230 mm wide. Level pegs were inserted in the ground, five on each side of the crack and some 8 metres away from it, and periodic observations were made to determine horizontal movements. Between 1947 and 1953 relative horizontal movements of up to 150 mm were recorded between opposing pairs of pegs on either side of the crack. It is understood that movements have increased since that date, the maximum relative horizontal movement being of the order of 250 mm.

12. Sunlight House (before 1954):
Differential settlement of more than 330 mm in a building situated above mine workings 175 m deep. (See Case History 10).

Great depths of mining : 500 - 1 500 m

13. Village Main Reef and Village Deep:
A small strip-pillar was left at the boundary of the two mines. The stope was at a depth of 558 m below surface. Bursting of the strip-pillar, accompanied by an earth-tremor, resulted in cracks, 6,5 m wide, opening up in a tennis court on surface.

14. Turf Shaft, Robinson Deep (1957 - 1959):
Subsidence of the ground surface as a result of the extraction of the Turf Shaft pillar at 1 060 to 1 457 m below surface was measured by A.N. Brown (1960). The actual subsidence over a period of two years is shown in Figure 5/1. It will be seen that the surface subsided 76 mm over this period, and that subsidence was accelerating with time. This was caused by the increased percentage extracion. The removal of the shaft pillar resulted in rejuvenation of the rate of surface subsidence in the areas overlying previously mined-out stopes, some of which had been mined as early as 1912. The amount of surface subsidence measured during the period covered by the investigation was greatest in areas direcly overlying the shaft pillar, as may be seen from Figure 5/2.

Subsidence of the order of two metres has thus not been uncommon on the Central Rand. 'Sinkholes' developed at outcrop level in waste-filled stopes can be tens of metres deep. As these hazards can present a serious threat to life and to buildings, certain restrictions have been laid down from time to time by the Government Mining Engineer with regard to building on proclaimed ground.

RESTRICTIONS ON DEVELOPMENT

Current restrictions apply not only to areas which are already undermined but also take account of the future demands of mining in the event of further increases in the price of gold, when previously unpayable reefs under a proposed building or township could be mined out. The restrictions listed in Table 5.1 act as a general guide to prospective township developers and others at the present time.

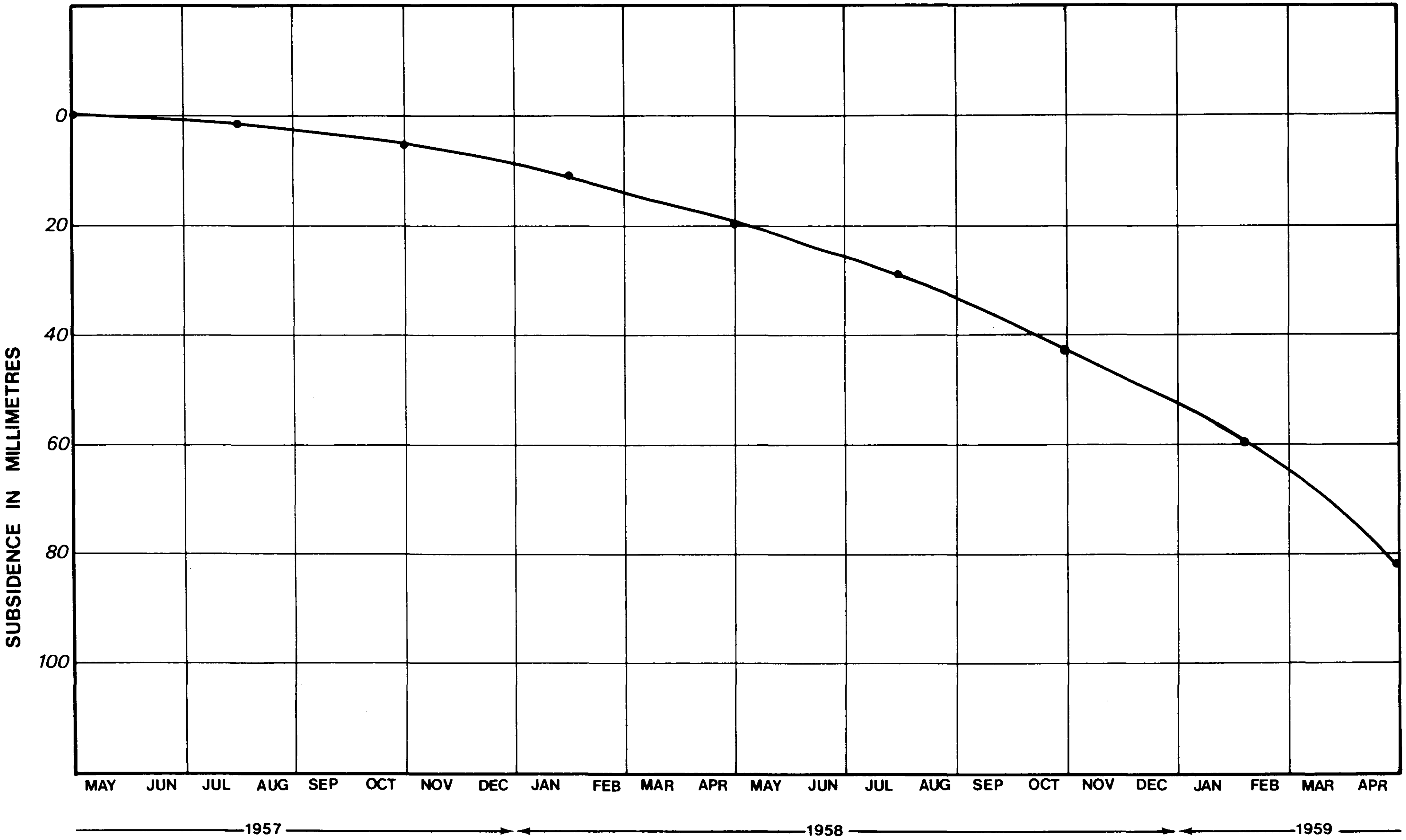


FIGURE 5/1 : Plot of time against surface settlement above Turf Shaft Pillar

(After Black and Brown, 1960)

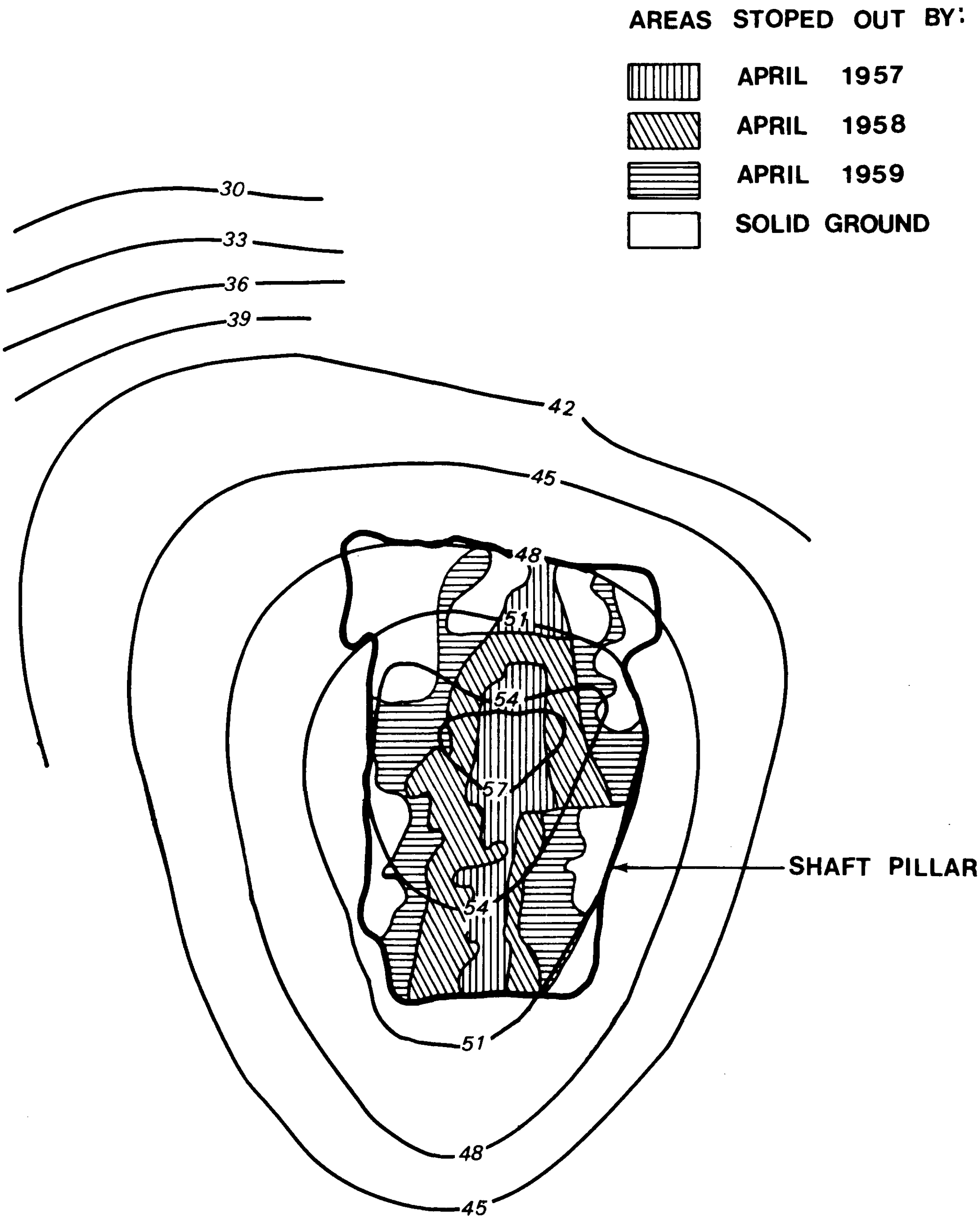


FIGURE 5/2 : Surface subsidence associated with the extraction of the Turf Shaft Pillar, Robinson Deep : 1957 to 1959

Figures on contour lines represent surface subsidence in millimetres

(After Black and Brown, 1960)

DEPTH OF SHALLOWEST WORKINGS OR POTENTIALLY ECONOMIC REEF BELOW SURFACE		NUMBER OF STOREYS ALLOWED IN PROPOSED BUILDING	MAXIMUM ALLOWABLE HEIGHT OF WALLS OF BUILDINGS	
METRES	FEET		METRES	FEET
0 - 91,4	0 - 300	None	-	-
91,4 - 122	300 - 400	1 with 1 basement	4,88	16
122 - 152,4	400 - 500	2 with 1 basement	8,23	27
152,4 - 183	500 - 600	3 with 1 basement	11,58	38
183 - 213,4	600 - 700	4 with 1 basement	14,94	49
213,4 - 244	700 - 800	5 with 1 basement	18,29	60
> 244	> 800	No building restrictions except where excessive stoping widths exist		

TABLE 5.1 : Current building restrictions on undermined areas or areas underlain by potentially economic reefs on the Central Witwatersrand

As shown on De Beer's (1965) geological map, the zone within which the above restrictions are applied seldom exceeds half a kilometre in width within the Johannesburg municipal area.

The following restrictions also apply:

* No building may be erected within a zone commencing 3 m on the foot-wall side of the outcrop and extending to where the reef or stope is 91,4 m below surface.

* No buildings where people sleep or congregate may be erected where the hanging wall of the shallowest stope or economic reef is from 91,4 m to 244 m below surface.

* Such buildings as are allowed in terms of Table 5.1 must be constructed of reinforced concrete, timber or steel frameworks, with panels of corrugated iron, asbestos or other flexible material.

These rules apply generally to cases where one, two or possibly three reefs have been mined, or may be mined, and the combined stoping width does not exceed 1,25 per cent of the depth to the shallowest stope or reef. However, they are not hard and fast rules. In some cases the Government Mining Engineer finds it necessary either to relax the rules or to apply greater restrictions. Two examples of extreme cases illustrate this:

(i) permission to erect a multi-storey building over the outcrop area where appropriate techniques of ground stabilisation involving grouting had been applied; and

(ii) restriction of building height to 18,29 m (five storeys) where the shallowest economic reef was situated between 274,3 and 304,8 m below surface but where there was evidence of excessive stoping widths and poor support of the stopes.

Before relaxing restrictive measures or applying stricter ones consideration is taken of such factors as local geological conditions, angle of dip of the reefs, degree and nature of support in the workings, date of extraction of the reefs and amount of stope closure that has already taken place.

Where existing structures are situated above reefs which are to be mined, restrictions are also imposed on the mining company concerned. These restrictions make provision for adequate support of workings under the existing structure. However, restrictions on mining operations under a structure which is to be protected would naturally not be economically warranted if too generous a building height had been permitted in the first instance without consideration of future mining interests.

The effect of all these provisions is still evident today in the presence of low buildings which either straddle the outcrops or are built on undermined ground to the south of them, as may be seen from a stereoscopic examination of Figures 5/3 to 5/5. A dramatic jump in the height of buildings is to be seen immediately to the north of the Main Reef outcrop, whereas this is succeeded southwards by buildings which gradually increase in height until tall buildings are again present where the depth to the uppermost reef (the South Reef) is about 213 metres. Isolated tall buildings immediately south of the outcrop area are invariably situated on intact dykes.

FACTORS INFLUENCING MINING SUBSIDENCE

The effect of strength and deformation characteristics of the hanging-wall rock

The hanging walls of all mined-out stopes on the Central Witwatersrand consist without exception of quartzite. Strength and deformation characteristics for samples of fresh quartzite from the Main-Bird and Kimberley-Elsburg Series are given in Tables 5.2 and 5.3. While these figures may be taken as a guide to the strength and deformation properties of hanging-wall quartzites at deep levels of mining, they in no way reflect the properties of quartzite in the weathered zone which, on the Central Rand, generally extends to a depth of about 45 metres. Below this depth the compressive strength of the grey quartzite is generally in the range 175 MPa to 375 MPa. From a depth of 30 m to 45 m the compressive strength usually drops to within the range 25 MPa to 55 MPa, and above 30 m the strength is generally lower and highly variable.

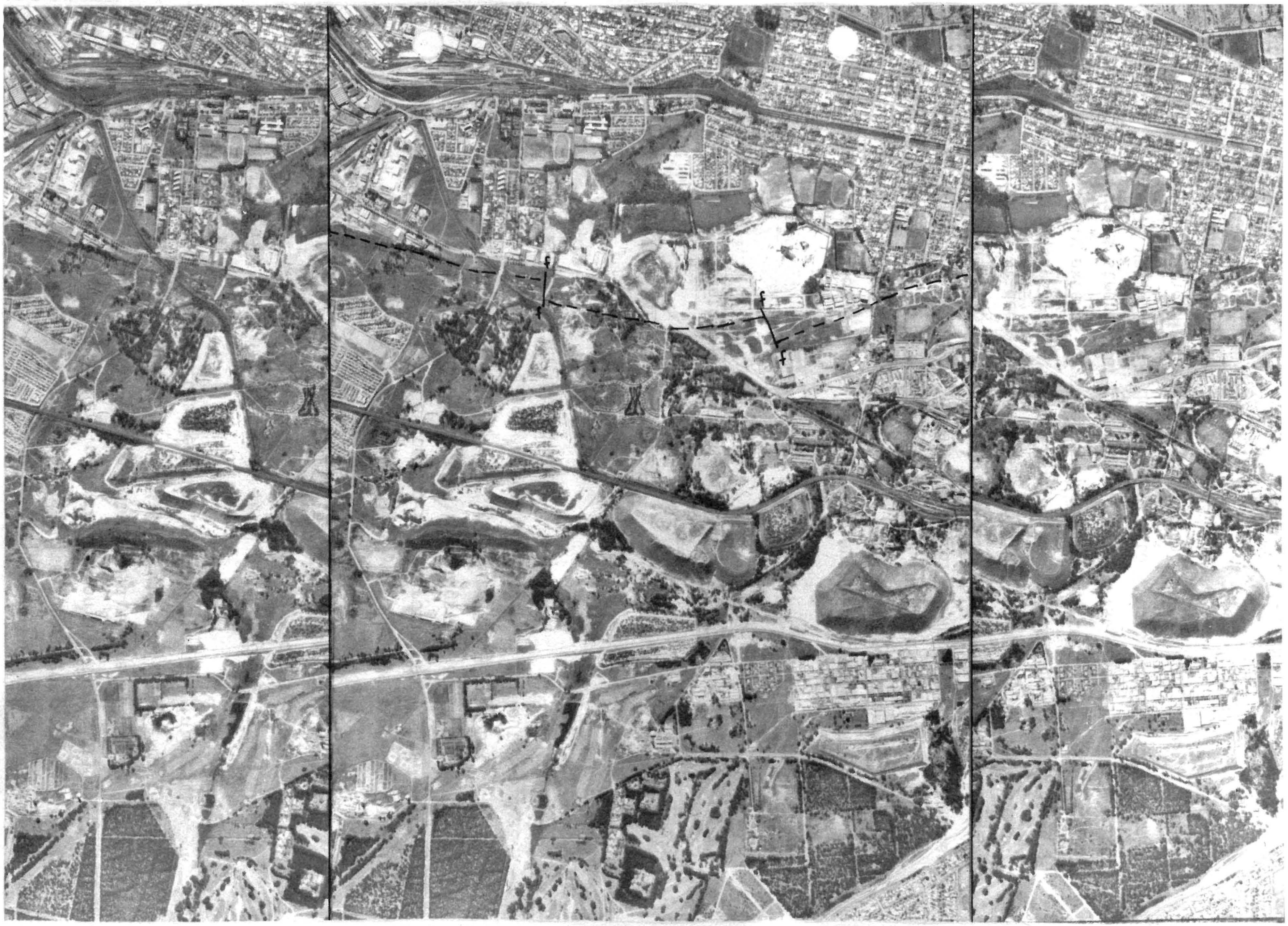


FIGURE 5/3 : Stereo–triplet of aerial photographs from Baragwanath Road in the west to Treu Road in the east showing Main Reef outcrops

FIGURE 5/4 : Stereo–triplet of aerial photographs from Treu Road in the west to Mooi Street in the east showing Main Reef outcrop

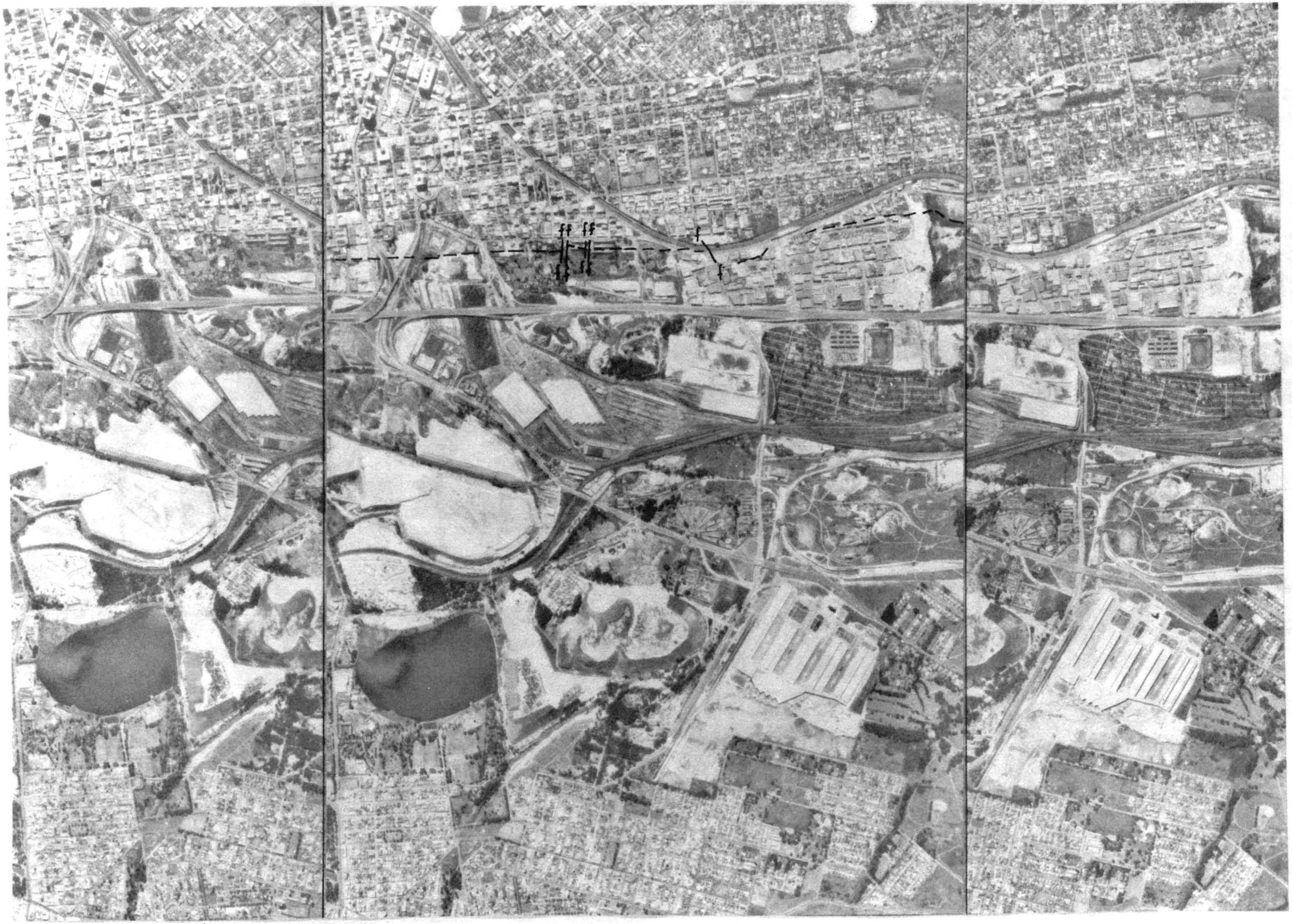


FIGURE 5/5 : Stereo–triplet of aerial photographs from Mooi Street in the west to Pentz Street in the east showing Main Reef outcrop

(Photo by AOC)

LOCALITY: Western Deep Levels Mine (Sample tested 1971)	σ_1 / σ_3 Ratio	Strength MPa	Number of specimens tested	Standard deviation %
TRIAXIAL COMPRESSIVE STRENGTH	20,8	296	10	6,0
	13,3	369	10	5,3
	9,1	488	10	9,1
UNIAXIAL COMPRESSIVE STRENGTH	∞	237	10	4,1
UNIAXIAL TENSILE STRENGTH	$-\infty$	10,8	10	11,0

DENSITY : kg/m^3	2 710	MODULUS OF ELASTICITY GPa	79
POROSITY	1%	POISSON'S RATIO	0,13
MOISTURE CONTENT	0,02%		

PETROGRAPHIC ANALYSIS		
QUALITATIVE	COMPOSITION (VOL %)	GRAIN SIZE DISTRIBUTION
Strongly bonded quartzite, with grains surrounded by thin film of amorphous silica	Quartz : 67% Muscovite) Chlorite) Amorphous) silica) : 32% Pyrite : 1%	0,1 - 2,6 mm Quartz : 1,2-2,6 : 60%; 0,6-1,2 : 30%; <0,6 : 10% Pyrite : mostly angular grains 0,06 mm (few 1,2 mm)

TABLE 5.2 : Strength and deformation characteristics of Main-Bird quartzite

(Data provided by NMERI-CSIR, 1975)

LOCALITY Stilfontein Gold Mine : East Subvertical Shaft Pillar Depth : 1 304,5 m	σ_1 / σ_3 Ratio	Strength MPa	Number of specimens tested	Standard deviation %
TRIAXIAL COMPRESSIVE STRENGTH	38,10	388	10	2,91
	20,95	501	8	4,30
	13,35	938	9	3,92
UNIAXIAL COMPRESSIVE STRENGTH	∞	284	12	1,32
UNIAXIAL TENSILE STRENGTH	- ∞	22,3	3	4,20

DENSITY : kg/m^3	2 650	MODULUS OF ELASTICITY GPa	83,4
POROSITY	1%		
MOISTURE CONTENT	2%	POISSON'S RATIO	0,106

PETROGRAPHIC ANALYSIS		
QUALITATIVE	COMPOSITION (VOL %)	GRAIN SIZE DISTRIBUTION
Recrystallised sandstone, angular to subrounded grains. Grains partly welded by recrystallisation, partly separated by interstitial matrix.	Grains (mainly quartz and chert) : 69,4 Matrix (sericite-chloritoid) : 26,6 Accessories (mainly pyrite) : 4,0	< 10 µm 22,3% < 100 µm 4,9% < 200 µm 7,8% < 400 µm 11,6% < 800 µm 22,8% < 1,2 mm 9,1% < 2,0 mm 8,3% > 2,0 mm 13,2%

TABLE 5.3 : Strength and deformation characteristics of Kimberley-Elsburg quartzite

(Data provided by : NMERI-CSIR, 1975)

The strength of even the fresh quartzite is not an absolute property as it is dependent on the state of stress, the moisture content and the period of loading. The state of stress varies with the degree of fracturing which will be considered later. It has been shown by Wiid (1968) that the strength of most hard rocks is about 50 per cent less in the saturated state than in the dried out state, and by Bieniawski (1967) that the strength of norite is 26 per cent less than that determined according to standard procedures if the load is applied for a period of more than 17 days. As a result of these effects of moisture content and time the hanging wall may eventually collapse, even if it has been stable for many decades (see Case History 11).

According to the deformation properties of rock and soil it may be described as:

(i) bulk solid (or granular) material, e.g. waste mine dumps;
(ii) microplastic material, e.g. wet clay residual from diabase dykes;
(iii) elastic material, e.g. unfractured and unweathered quartzite and diabase;
(iv) macroplastic material, e.g. fracture-zones in quartzite around mine openings.

All four types of these materials are encountered in the Central Rand between the surface and the underlying mine workings. As bulk solid material is confined to waste dumps and microplastic materials to the upper zone of weathering in the diabase intrusions, further consideration will be given only to elastic and macroplastic rock.

The deformation of anisotropic rock is characterised by a number of elastic constants which decrease in number as the symmetry of deformation increases. Five elastic constants characterise transversely isotropic rock* and two characterise homogeneous elastic rock. If the influence of bedding planes or interbedded weak partings in the strata are not pronounced, the Witwatersrand quartzites may be regarded as

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* *Transversely isotropic* is a term used for material which is horizontally laminated and in which the stress properties do not vary horizontally but only vertically.

homogeneous rock and described by two elastic constants. Grobbelaar (1957) has demonstrated from extensive tests on quartzites from the ERPM mine that these rocks respond elastically in uniaxial compression testing.

Using the elastic theory Salamon (1963) has shown that the volume of underground closure in mine workings is equal to the volume of surface subsidence and to the volume of subsidence at any elevation between the surface and the workings. This implies that elastic closure of one cubic metre in a stope would produce a volumetric displacement between the original and the subsided ground surface of one cubic metre. Thus as long as there are voids in the mine workings there will be potential energy for the generation of further surface subsidence. This potential is only fully consumed when the ground surface has subsided to such an extent that the void between the original and final ground surfaces is equal in volume to the volume of rock excavated by mining.

The effect of underground pillars

Further work by Salamon (1968) has demonstrated that, in the case of a hypothetical two-dimensional pillar in unfailed elastic rock, stope closure will take place at distances exceeding Lc, where:

$$Lc = \frac{Sm\ E}{4(1 - d^2)\ q}$$ = half the critical span within which stope closure takes place at mid-span.

where:

Sm = stoping width

E = Young's Modulus

d = Poisson's Ratio

q = overburden stress

Using this equation it can be shown that at a depth of 1 500 m below surface the value of Lc is approximately 120 m per 300 mm of stoping width. An unmined pillar may therefore prevent a substantial amount of underground closure and hence surface subsidence from taking place (Grobbelaar, 1970). However, the subsequent extraction of an isolated underground pillar may cause a volume of surface subsidence which may be of the order of a thousand times more than the volume of the pillar itself (cf. extraction of Turf Shaft pillar at Robinson Deep Mine mentioned earlier). If the stoping width in the vicinity of the pillar is not uniform owing for example to local unpay areas in one of several reefs, differential rock movement will take place over the non-uniform stoping width which may cause differential settlement and damage at surface (Grobbelaar, 1970).

The above observations apply only in the case of underground pillars of substantial size and at depths where the rock will behave elastically. Small pillars have finite strengths. The load on a small pillar may be increased to beyond its strength by mining at distances less than 2 Lc from the pillar. The failure of such a pillar may cause excessive surface subsidence or may generate high-amplitude seismic shocks.

Also based on the elastic theory, the finite element method has been used for the prediction of stress and displacements around mine workings in the Witwatersrand. Two highly idealised analyses were conducted by the Chamber of Mines Research Organisation (Oravecz, 1972) by modelling stopes dipping at 70^0 and extending from the surface to a depth of 1 524 m (5 000 ft). In one model a strike pillar 30,5 m (100 ft) in width was placed at a depth of 304,8 m (1 000 ft) and in the second the stope was left unsupported. The difference in displacements (anywhere within the displacement field) calculated for the second and the first models represents the effect of excavating the strike pillar. This difference may also be regarded as providing a very rough estimate of the effect of the collapse of some regional underground support.

The results of the analyses are summarised in Figure 5/6. The differential vertical and horizontal displacements on suface are presented here in terms of the distance from the outcrop. It may be seen that both the maximum vertical and horizontal displacements took place in the hanging wall at the outcrop with magnitudes of 760 mm and 860 mm respectively. Horizontal differential displacement remained constant for a horizontal distance of about 305 m (1 000 ft) from the outcrop corresponding, on the hanging wall side, to a depth of cover of 762 m (2 500 ft). Horizontal displacement on the hanging wall side then decreased at a nearly uniform rate of 25 mm per 100 m (3 in per 1 000 ft) with increasing distance from the outcrop. On the foot-wall side the maximum horizontal differential displacement amounted to 96 mm.

Vertical differential displacement was greatest at the outcrop and then decreased at a nearly uniform rate of 67 mm per 100 m on the hanging

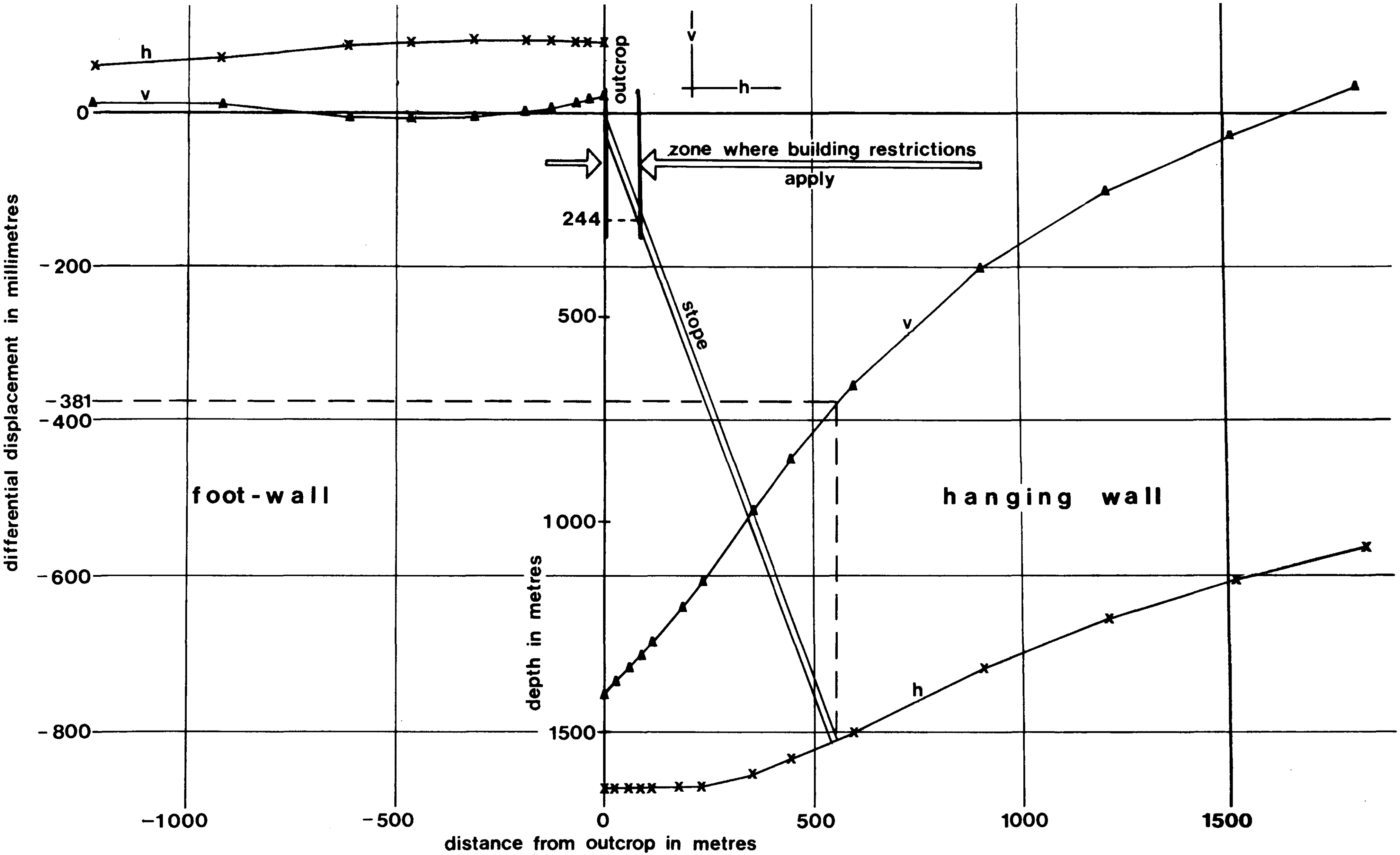


FIGURE 5/6 : Differential displacement at surface as derived from finite element solutions of open stope and stope with a pillar

(After Oravecz, 1972)

wall side. Therefore, immediately above the limit of stoping on the surface the differential vertical displacement still amounted to 381 mm (15 in). The maximum differential vertical displacement on the footwall side of the outcrop amounted to 25 mm.

In reporting the findings of this work Oravecz emphasised the following points:

(i) that the hypothetical case is a gross exaggeration of the worst possible condition that could exist in practice;

(ii) that the example is used mainly to show that the extraction of such a pillar (or the regional collapse of such a pillar if collapse could occur) would result in the dangerously large displacements extending beyond the limit of stoping;

(iii) that it was assumed that the stopes were completely open in both instances, i.e. that support provided by ore remnants, packs and sand-filling was completely ignored; and

(iv) that the reponse of the strata throughout was assumed to be perfectly elastic.

Before concluding this discussion on the effect of underground pillars, attention must be drawn to the phenomenon of tension-cracks which develop above shallow pillars in fractured rock. This phenomenon has a wide-spread occurrence on the Central Rand. It is illustrated in Figure 5/7, taken from an unpublished report by Dr F.G. Hill.

Hill explains the mechanism of such tension-failure as follows:

> "Assume that all the reef is mined out from surface to the fulcrum point 'B', that is, to the northern edge of the large pillar which for reasons of unpayability has been left intact; the rock mass 'A' moves towards the excavation at right angles to the plane of the reef; because movement at point 'B' is arrested, the surface comes under tension and a crack may develop."

The same phenomenon may be encountered where a rigid dyke rather than an unmined pillar provides the fulcrum. One of the most dramatic examples of this type is described in Case History 11.

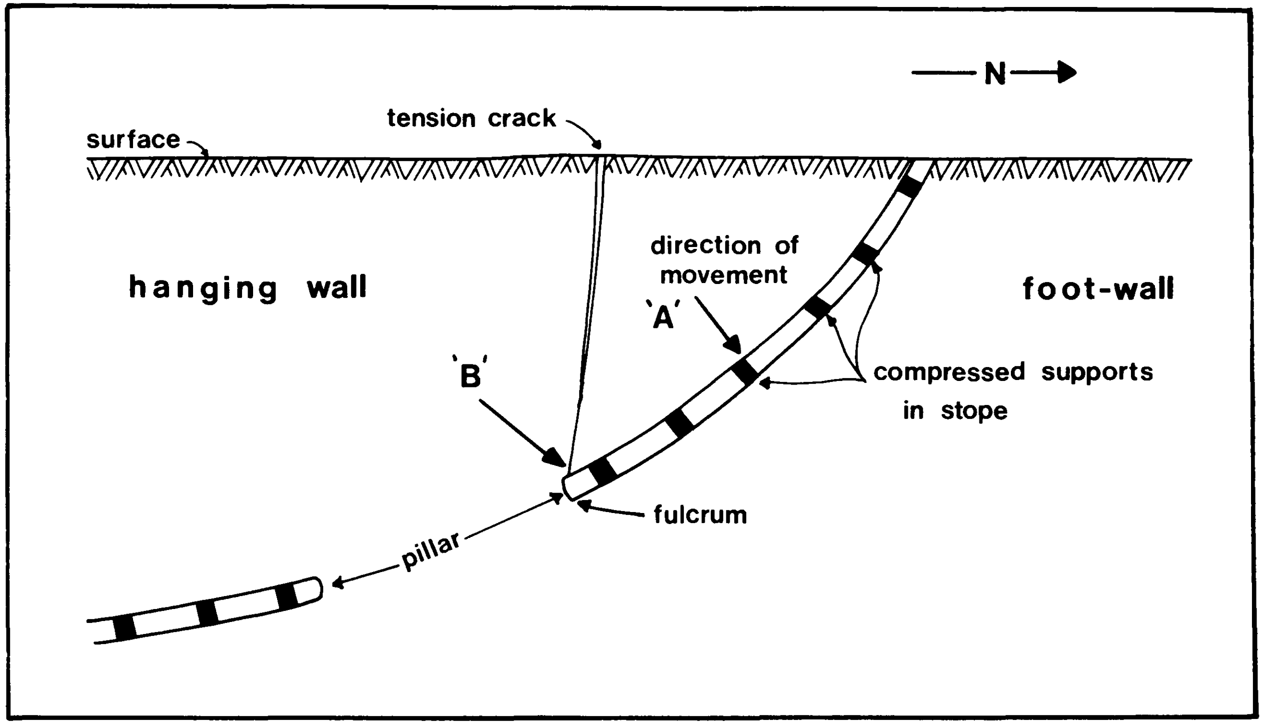


FIGURE 5/7 : Diagram illustrating the development of a tension-crack due to the presence of an unmined pillar at shallow depth (After F.G. Hill)

The effect of the fracture-zone around mine openings

If underground workings exceed a critical size fracture-zones develop around the openings (Leeman, 1958). The rock then no longer exhibits elastic behaviour, but rather a frictional or 'slip-stick' type of behaviour if the fracture-zone is of limited extent, or a macroplastic behaviour if the fracture-zone is extensive. Macroplastic behaviour obtains if the fracture-zone extends to the surface. While the average strain is to some extent predictably within the zone of macroplastic movement, strain may vary locally at random and local relative displacements are thus common occurrences (Grobbelaar, 1970).

In brittle quartzites exhibiting a 'slip-stick' type of behaviour deformation is not gradual and uniform, but tends to be zero during periods of 'stick' and may manifest itself in the form of rock-busts during 'slip' episodes. Thus a potential or exisitng fracture-plane may show no relative movement over a certain period of time. The gravitational energy released during a 'slip' episode or a rock-burst is often in excess of the energy required to deform the rock. The excess energy manifests itself in the form of seismic shocks which are propagated in the form of shock waves.

The extent of the fracture-zone around a mine opening in virgin ground in which no mining had previously taken place was measured at Harmony Mine, OFS, during the period 1955 to 1957 (Denkhaus *et al*, 1958). Mining commenced here in the shaft pillar at a depth of 1 235 m and level observations were taken at various depth points in the shaft. Growth of the fracture-zone as the tabular ore body was mined out is shown in Figure 5/8. It will be seen that the fracture-zone grew in size as the size of the excavation increased. Furthermore, since the strength of rock is time dependent as demonstrated by Bieniawski (1967), it follows that the fracture-zone will also increase in size with time even if mining operations are suspended. Figure 5/9, from the work of Denkhaus and Hill (1960), is of interest concerning the nature of the fracture-pattern around a mine opening at great depth.

Where shallower depths of mining are concerned, however, as in the case of the zone immediately south of the outcrop on the Witwatersrand, the nature of the fracture-zone is somewhat different. This is best illustrated by the sketches of Dr F.G. Hill which are reproduced in Figures 5/10 and 5/11.

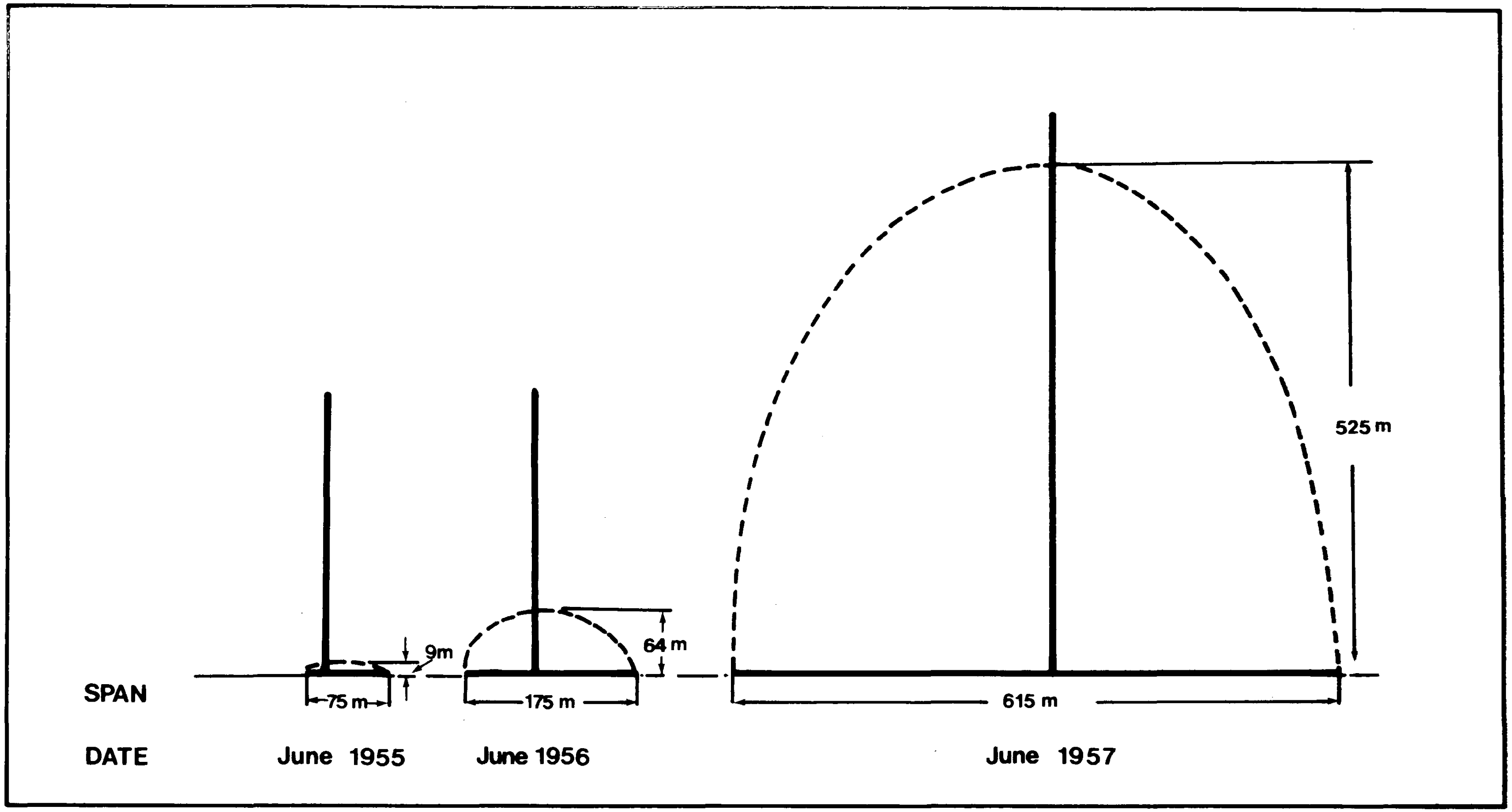


FIGURE 5/8 : Development of the zone of fracture above the shaft pillar excavation at Harmony Gold Mine, O.F.S.

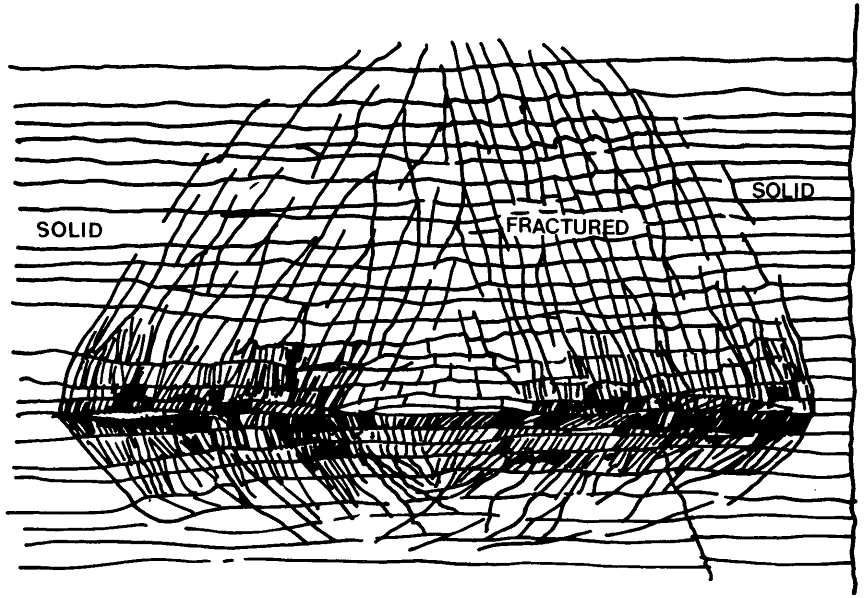


FIGURE 5/9 : Diagrammatic sketch of the probable fracture-zone around an excavation in hard sedimentary rock at great depth (After Denkhaus and Hill, 1960)

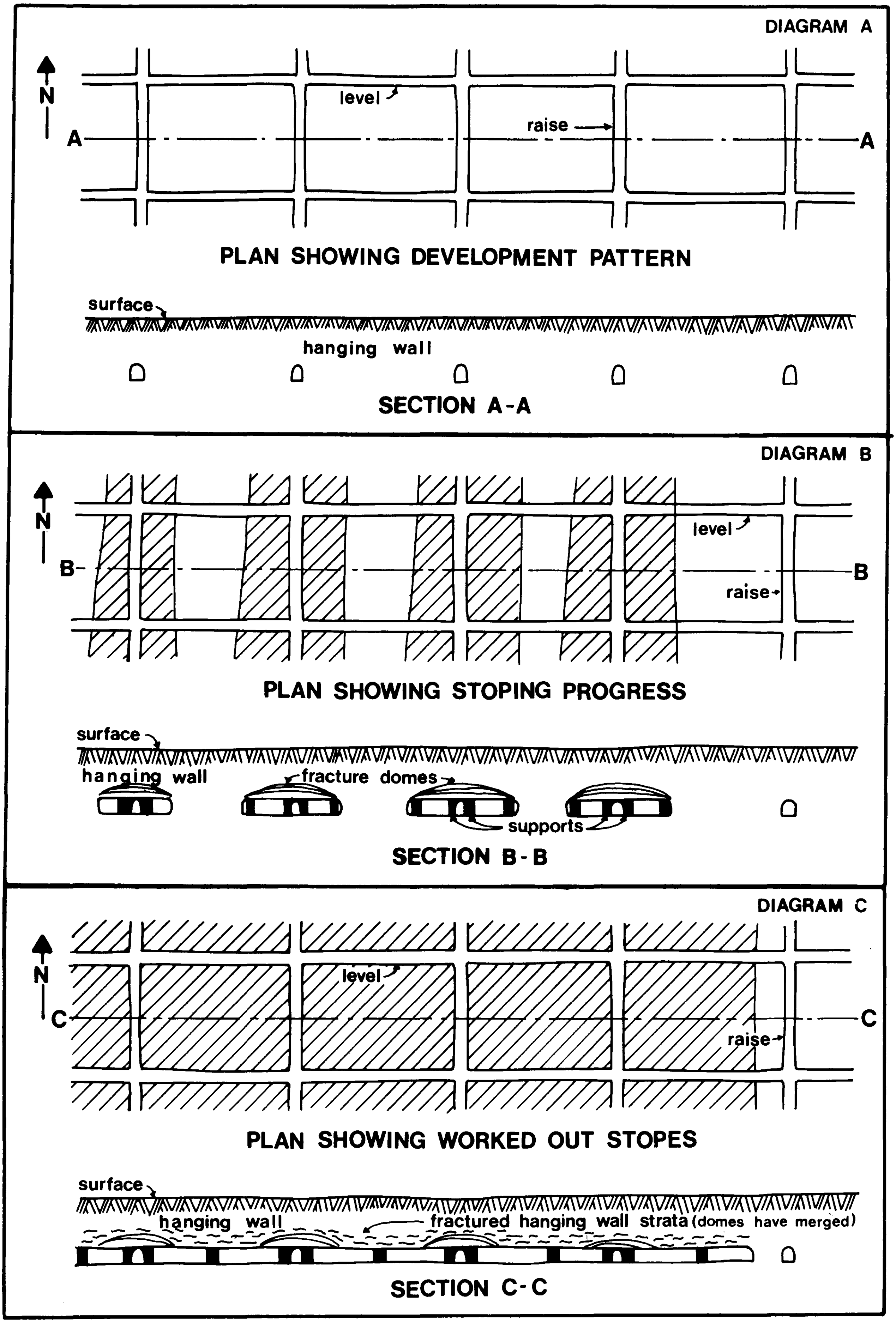


FIGURE 5/10 : Sketches showing development of fracture-zone around stoped-out areas (After F.G. Hill)

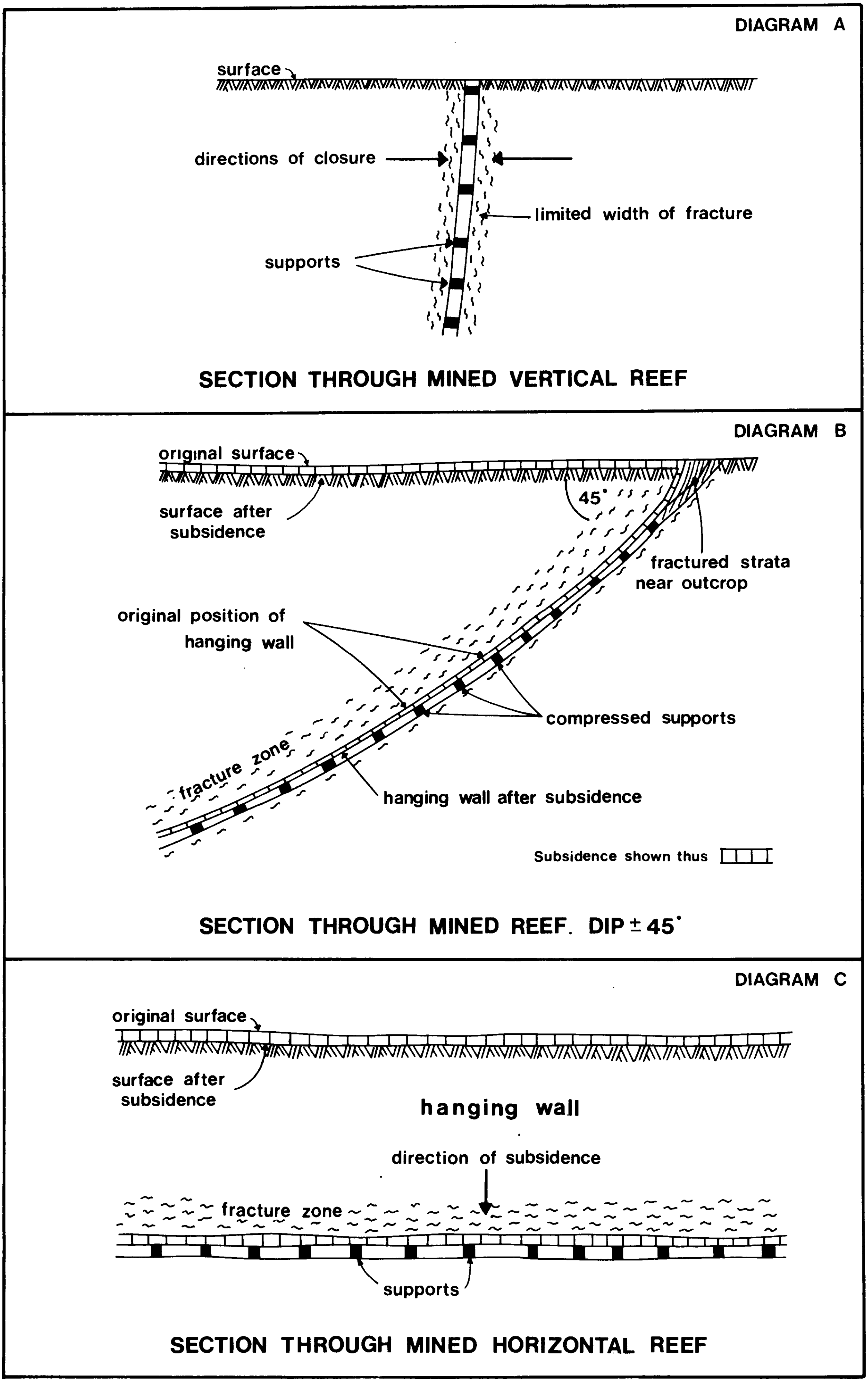


FIGURE 5/11 : Influence of dip of reef on development of fracture-zone
(After F.G. Hill)

Dr Hill's commentary on the diagrams in Figure 5/10 is broadly as follows. Diagram A shows a typical pattern of development as practiced on the Central Rand, especially at shallow depth. Diagram B shows a plan and section of the reef a year or more after the commencement of stoping. As mining progressed east and west of each raise supports were installed and, as the hanging wall settled, fracture-domes formed above the worked-out stopes. The plan and section after the reef has been completely stoped out is portrayed in Diagram C. The most important feature illustrated here is that the fracture-domes between raises have merged and that a zone of fractured rock overlies the entire stoped out area. After the strata above the fracture-zone have settled the latter becomes so compact and the supports so compressed that further settlement ceases. The hanging-wall strata have, in other words, come to rest, and stope closure is complete. However, the time to reach this state may be as much as 10 to 15 years. In this regard much depends on the dip and the depth of the reef or reefs.

Concerning the influence of dip and depth on the size of the fracture-zone, Dr Hill comments as follows on the sketches in Figure 5/11. The three diagrams illustrate the pattern of fracture to be expected for reefs of different inclination. In the case of the vertical or near vertical reef (Diagram A) the extent of fracture into the side-walls is very limited - only a few metres - because gravity plays very little part in inducing fracture. Fracture development will depend on the competence of the rock. Thus in the quartzites forming the side-walls of the South Reef in the Johannesburg area there will be virtually no fracturing, whereas in the weak strata confining the Main Reef Leader fracturing may extend for several metres. As the angle of dip decreases the effect of gravity becomes more pronounced and the extent of the fracture-zone above the stope increases, as illustrated in Diagrams B and C. However, another factor here comes into play, namely depth below surface. Whereas, as in the case of Harmony Mine where the reef was stoped at a depth of 1 235 m, the fracture-zone at different stages of mining extended well into the hanging-wall strata (Figure 5/8), in the shallow outcrop areas of worked out mines on the Central Witwatersrand (e.g. Ferreira, Mayfair, Croesus and Rietfontein Consolidated Mines Limited), exploratory holes drilled for Dr Hill showed fracturing to be confined to a zone no more than 23 m above the stopes. The important fact to be borne in mind, however, is that the rock mass above the fracture-zone remains intact and, as it settles, it does so *en bloc*.

FIGURE 5/12 : Plan of Johannesburg showing Main Reef outcrop, section lines for Figures 5/13 to 5/26, locations of Case Histories 7 to 13, positions of surface level pegs and all mine dumps and slimes dams

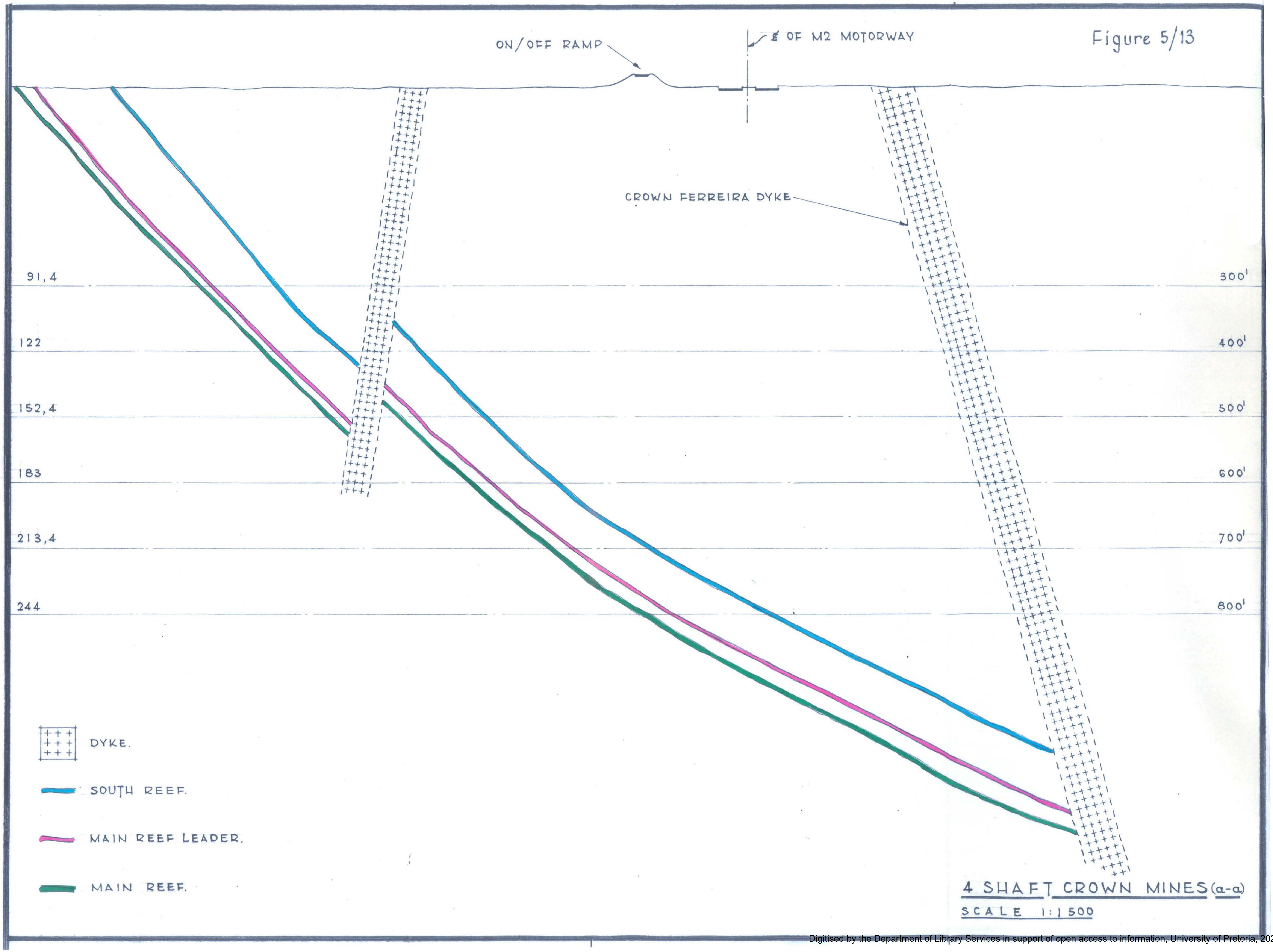
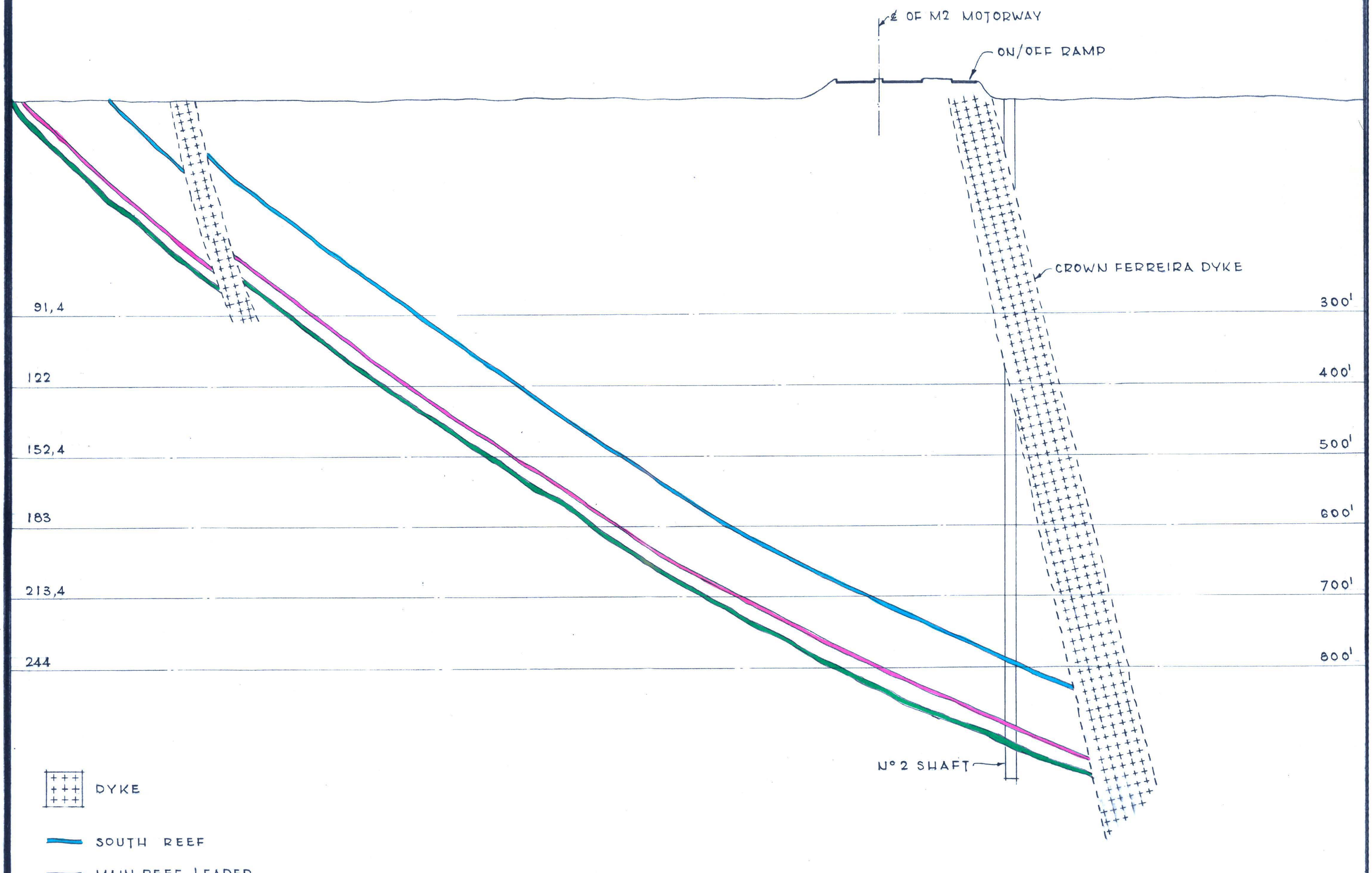


Figure 5/13
ON/OFF RAMP
℄ OF M2 MOTORWAY
CROWN FERREIRA DYKE
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
DYKE.
SOUTH REEF.
MAIN REEF LEADER.
MAIN REEF.
4 SHAFT CROWN MINES (a-a)
SCALE 1:1 500

Figure 5/14



2 SHAFT CROWN MINES.

SCALE 1 : 1 500 (b-b)

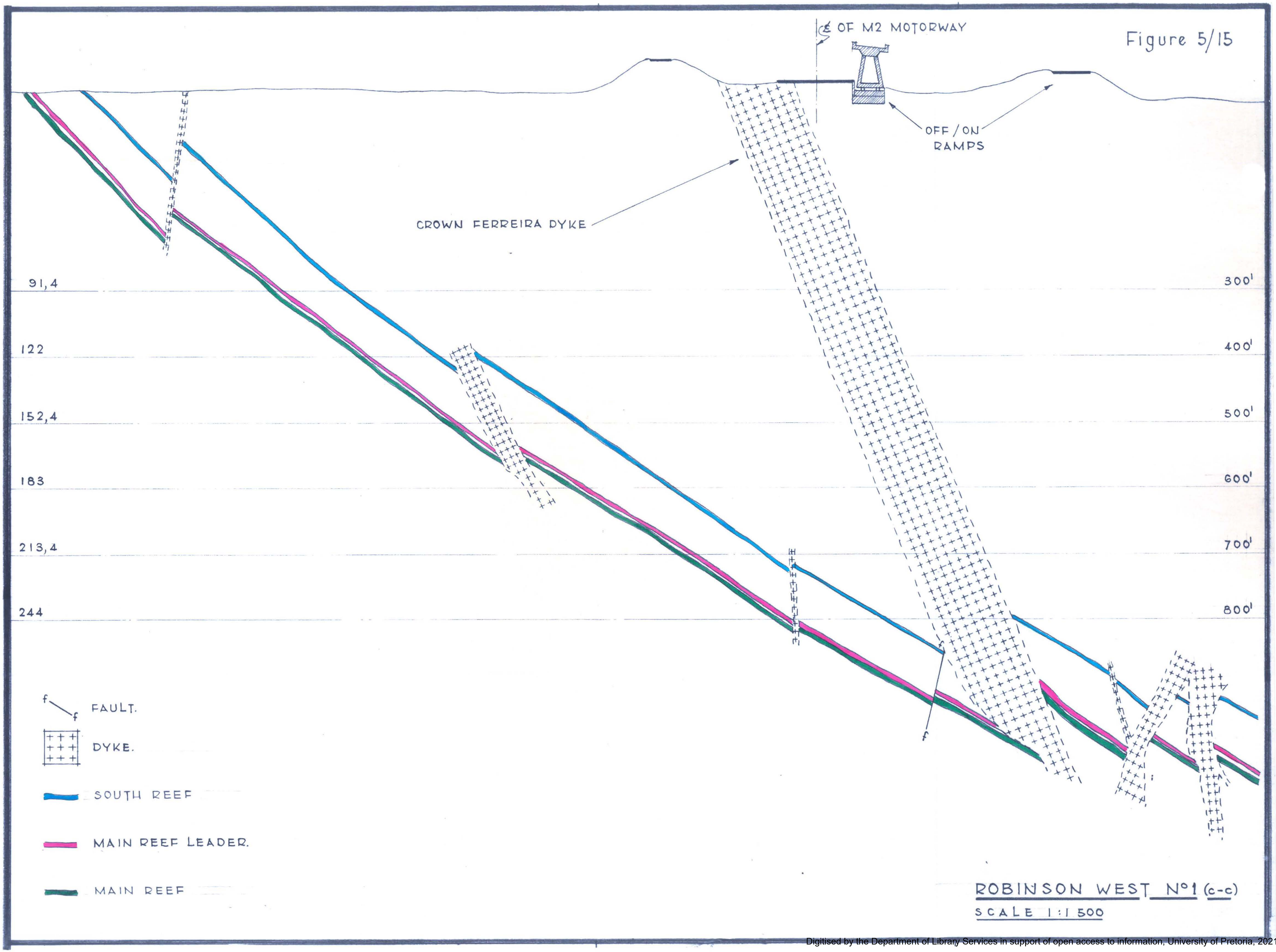


Figure 5/15
℄ OF M2 MOTORWAY
OFF / ON
RAMPS
CROWN FERREIRA DYKE
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
FAULT.
DYKE.
SOUTH REEF
MAIN REEF LEADER.
MAIN REEF
ROBINSON WEST Nº1 (c-c)
SCALE 1:1 500

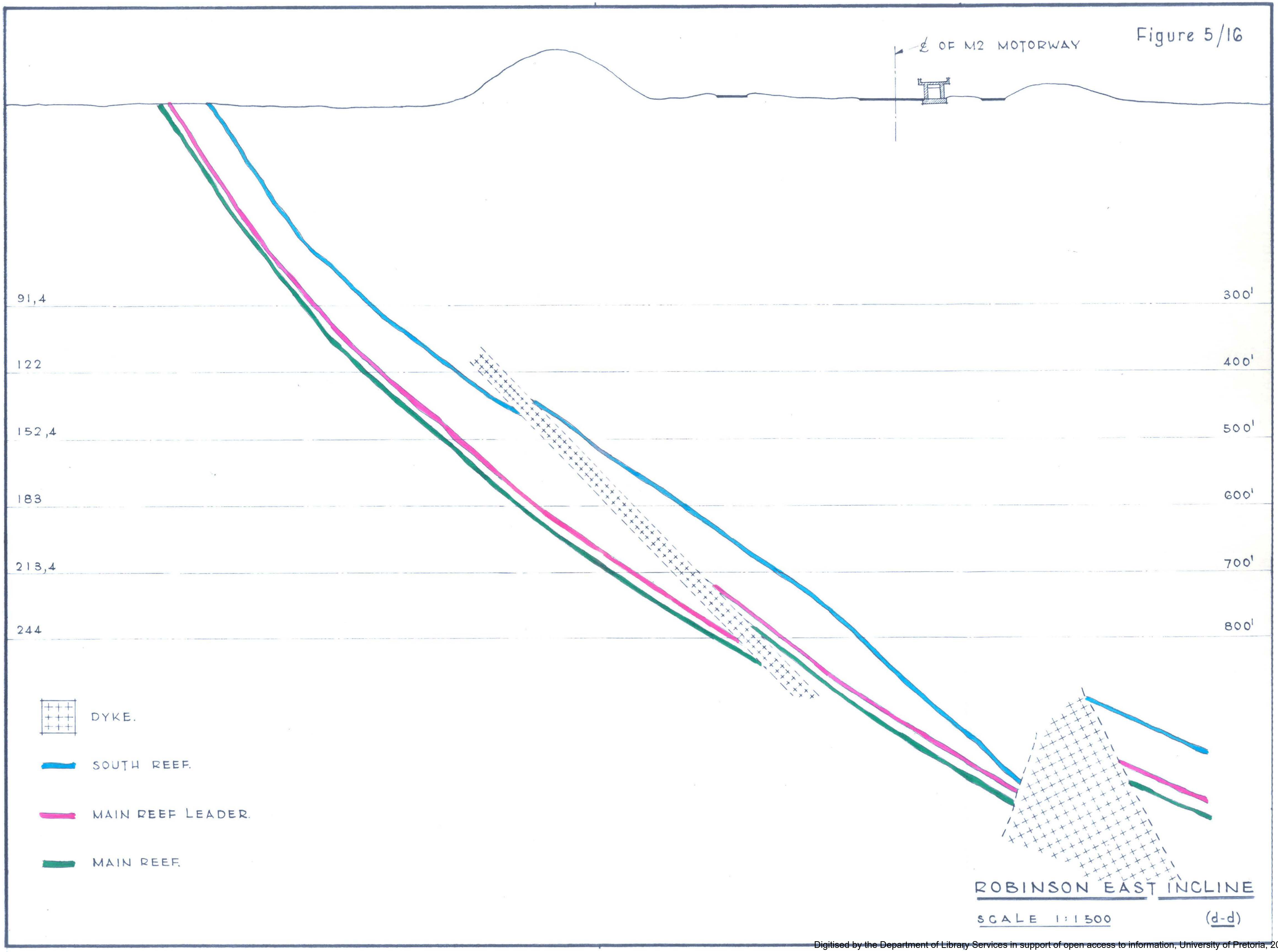


Figure 5/16
℄ OF M2 MOTORWAY
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
DYKE.
SOUTH REEF.
MAIN REEF LEADER.
MAIN REEF.
ROBINSON EAST INCLINE
SCALE 1:1 500
(d-d)

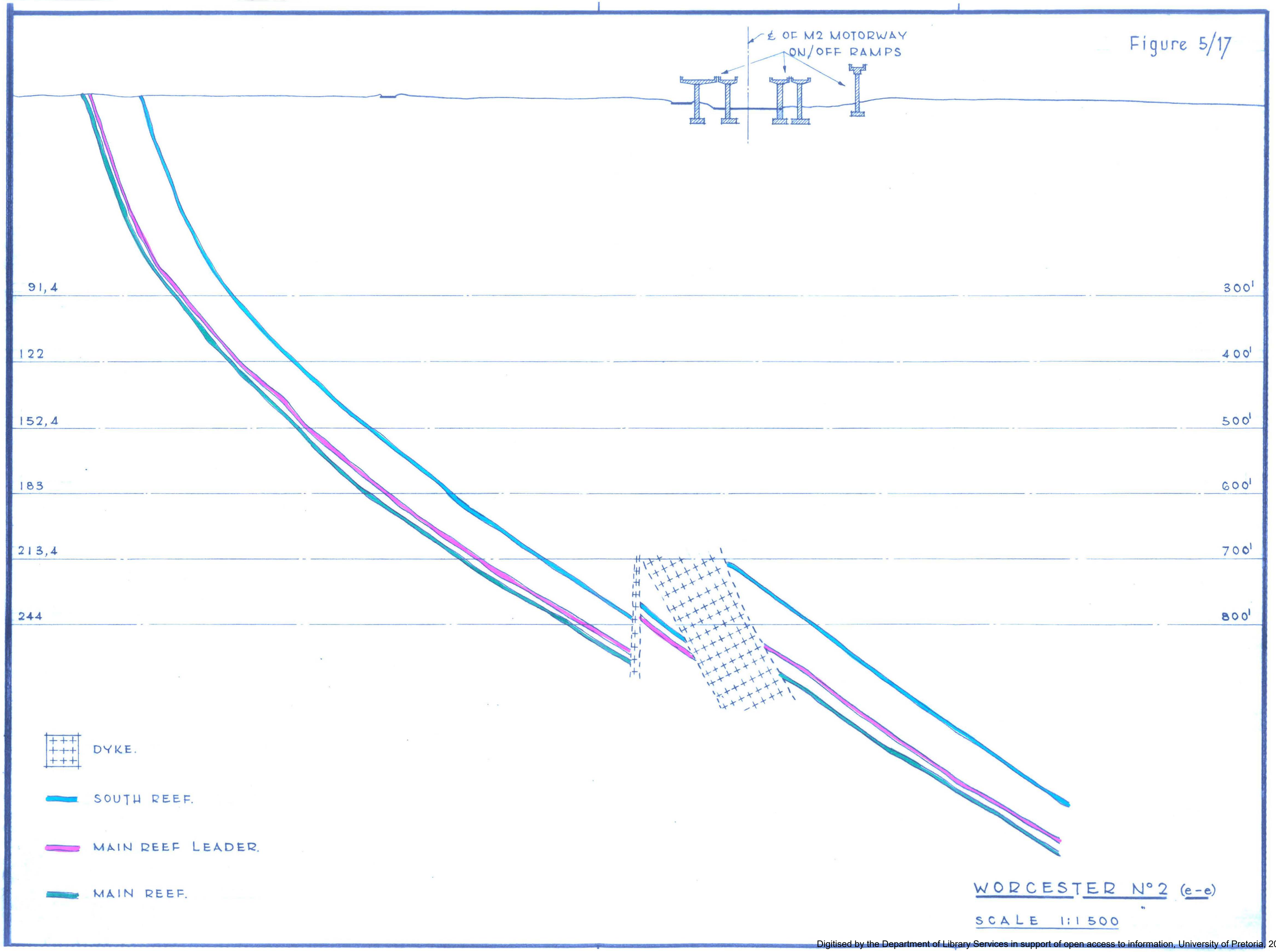
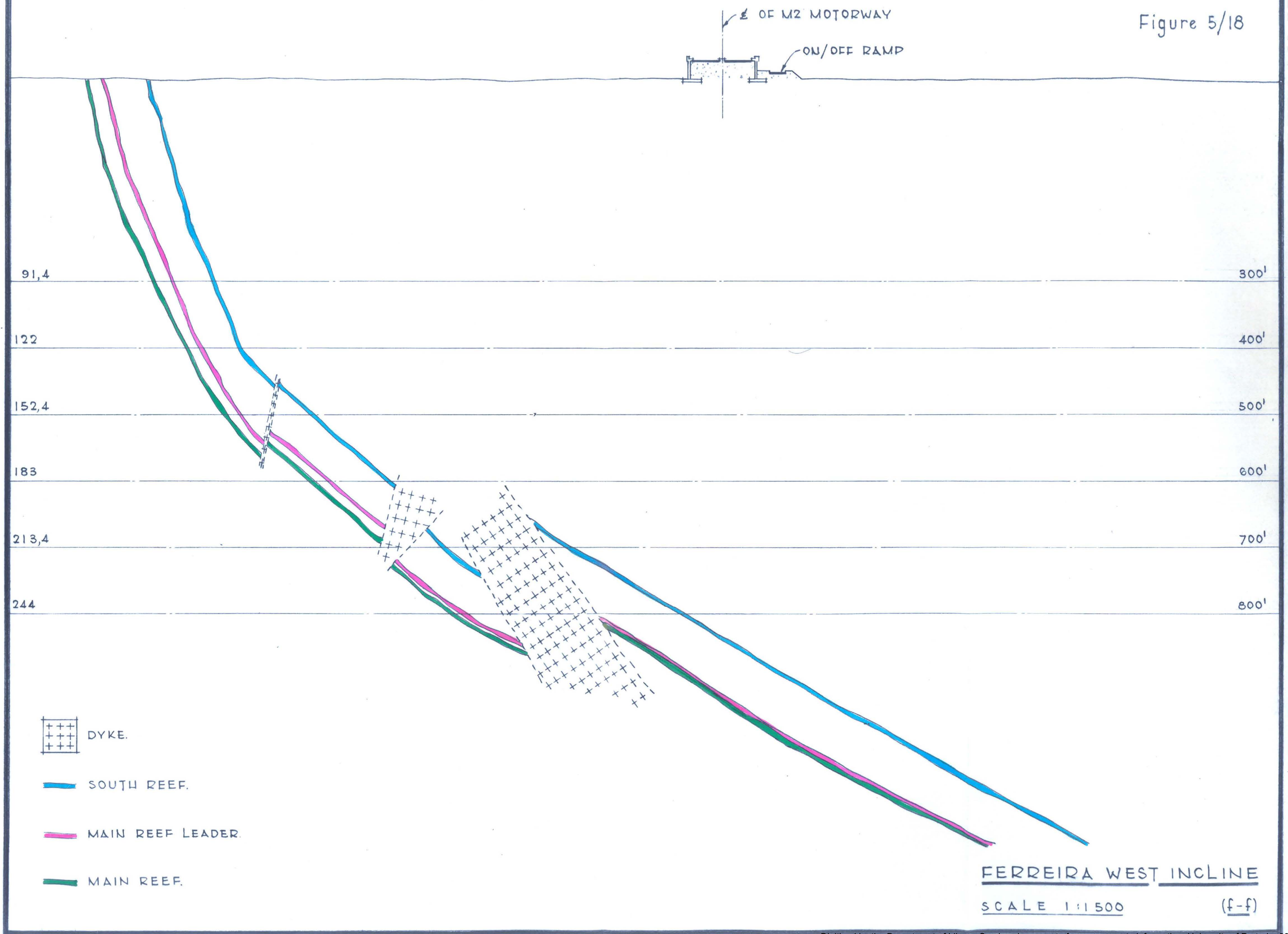


Figure 5/17
℄ OF M2 MOTORWAY
ON/OFF RAMPS
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
DYKE.
SOUTH REEF.
MAIN REEF LEADER.
MAIN REEF.
WORCESTER N°2 (e-e)
SCALE 1:1 500

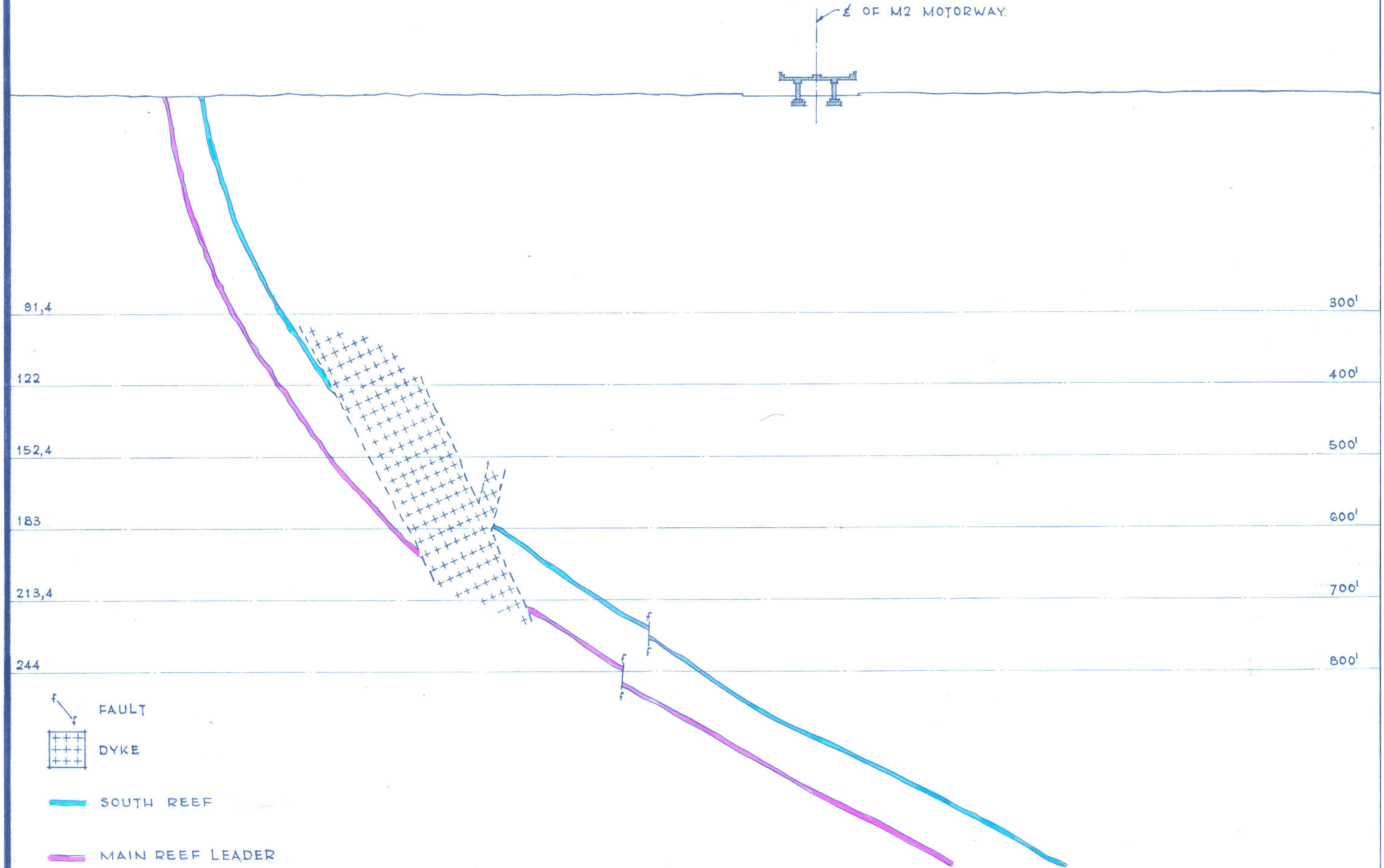
Figure 5/18



FERREIRA WEST INCLINE

SCALE 1:1 500 (f-f)

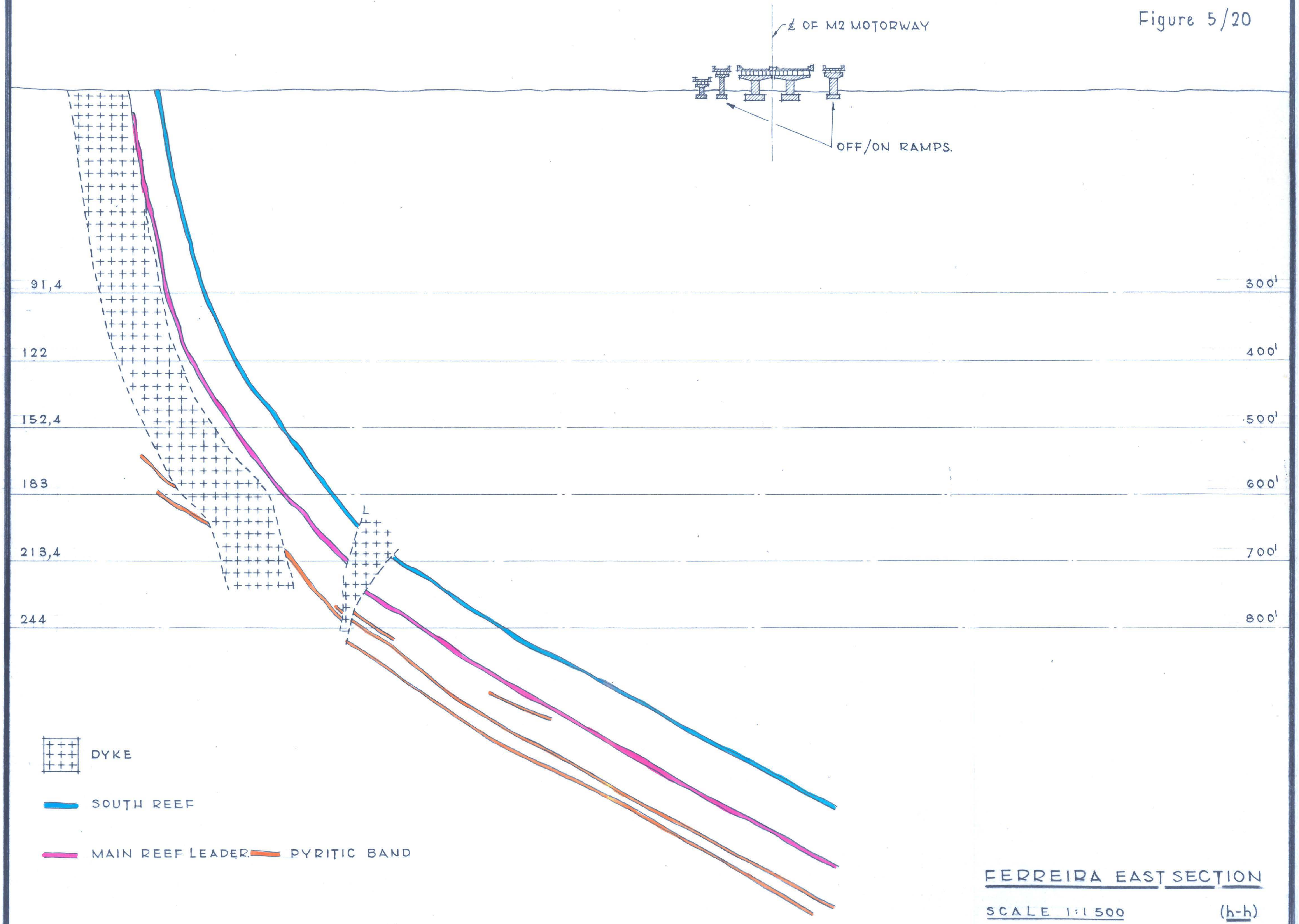
Figure 5/19



FERREIRA EAST INCLINE

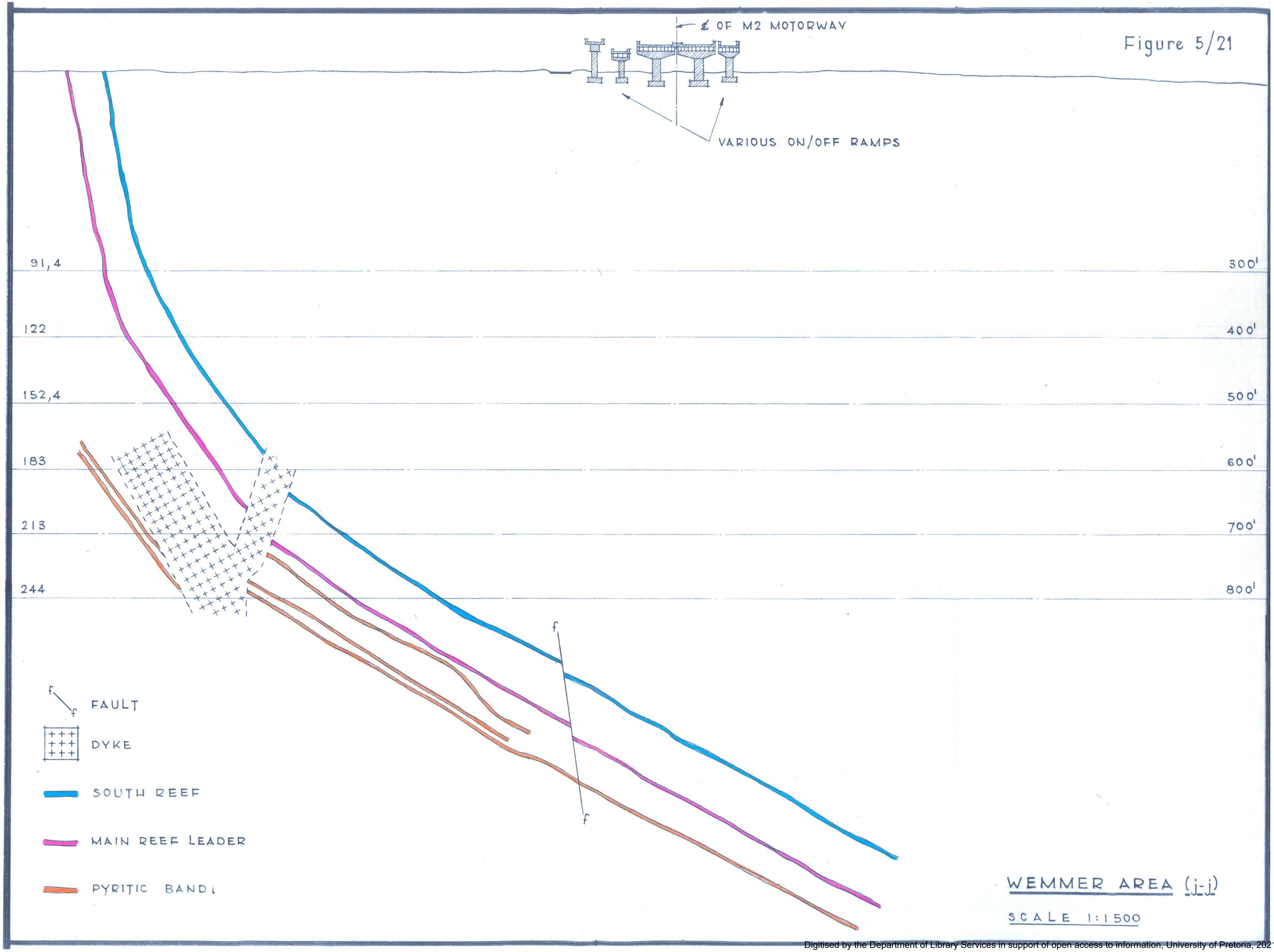
SCALE 1:1500 (g-g)

Figure 5/20



FERREIRA EAST SECTION

SCALE 1:1 500 (h-h)



℄ OF M2 MOTORWAY
Figure 5/21
VARIOUS ON/OFF RAMPS
91,4
122
152,4
183
213
244
300'
400'
500'
600'
700'
800'
f
f
FAULT
DYKE
SOUTH REEF
MAIN REEF LEADER
PYRITIC BAND
WEMMER AREA (j-j)
SCALE 1:1500

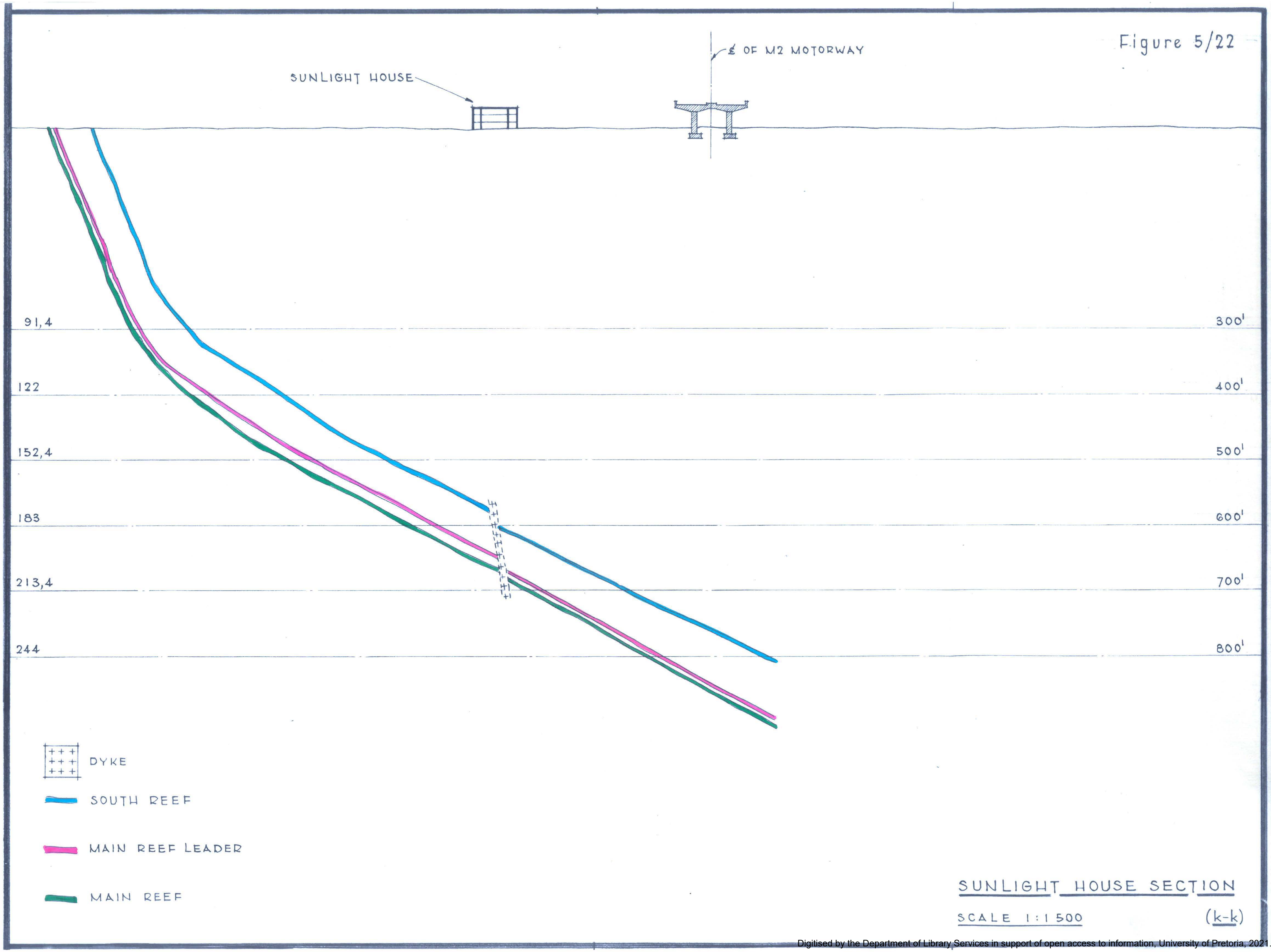


Figure 5/22
℄ OF M2 MOTORWAY
SUNLIGHT HOUSE
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
DYKE
SOUTH REEF
MAIN REEF LEADER
MAIN REEF
SUNLIGHT HOUSE SECTION
SCALE 1:1 500
(k-k)

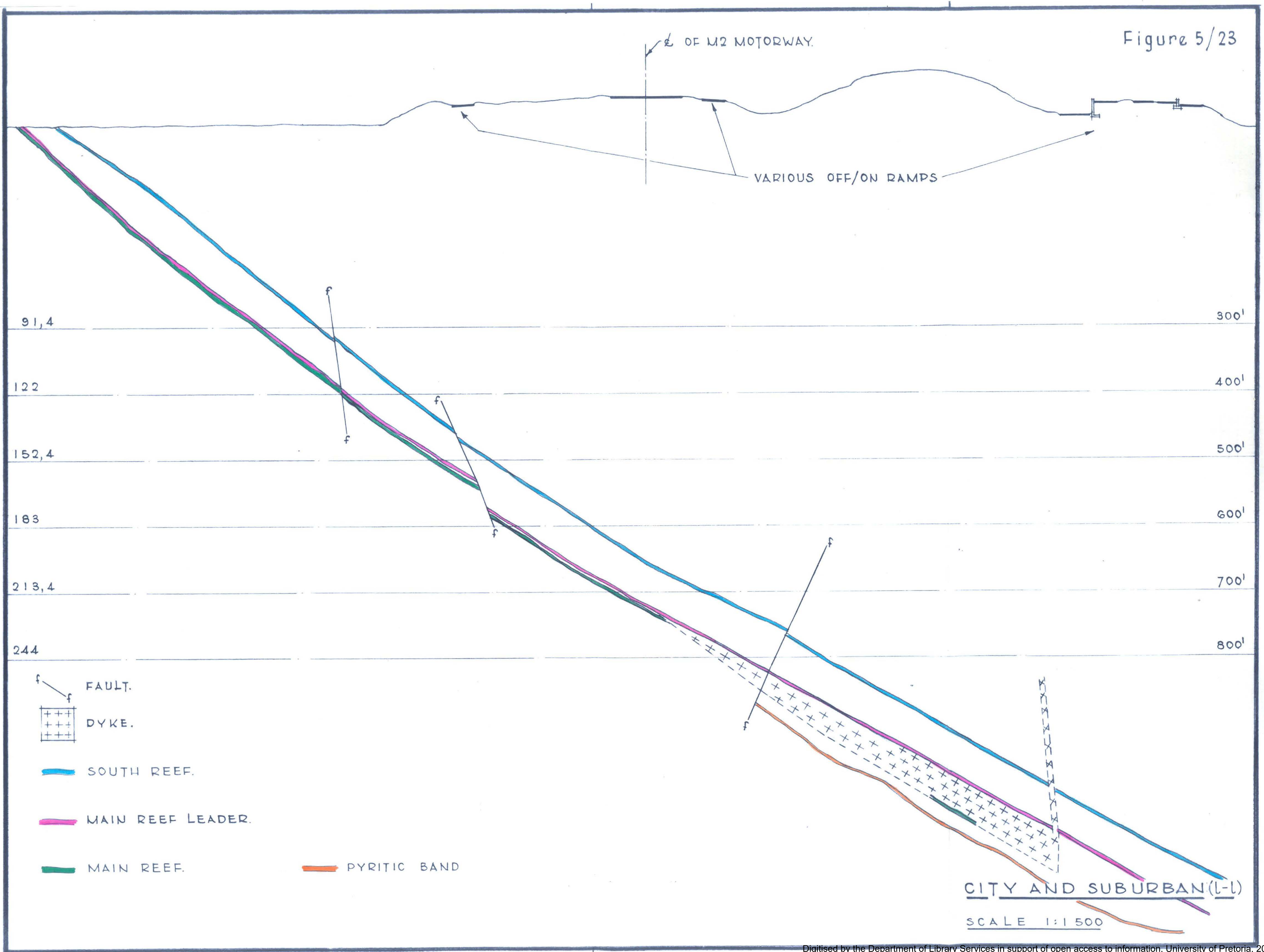


Figure 5/23
℄ OF M2 MOTORWAY.
VARIOUS OFF/ON RAMPS
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
f
FAULT.
DYKE.
SOUTH REEF.
MAIN REEF LEADER.
MAIN REEF.
PYRITIC BAND
CITY AND SUBURBAN (L-L)
SCALE 1:1 500

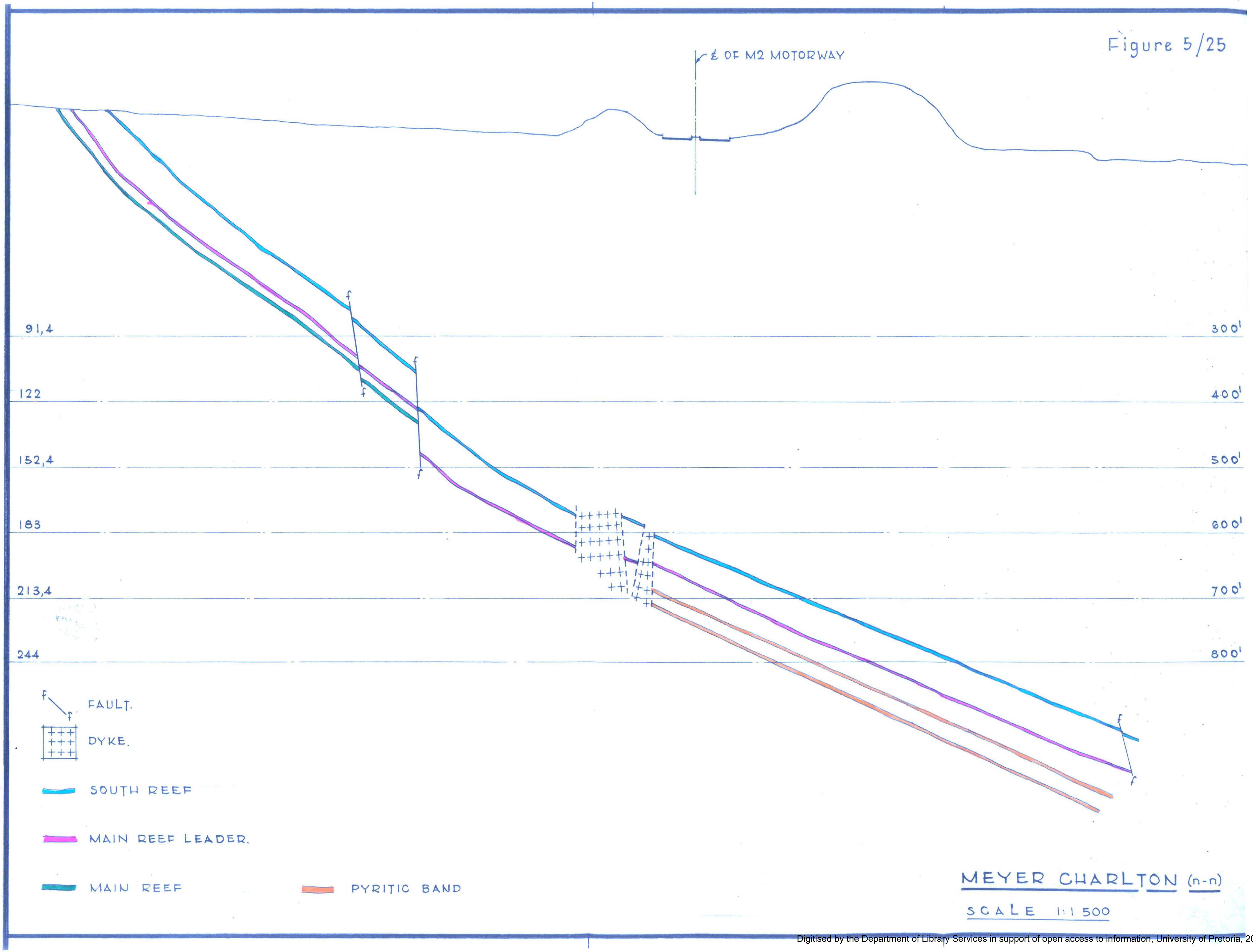
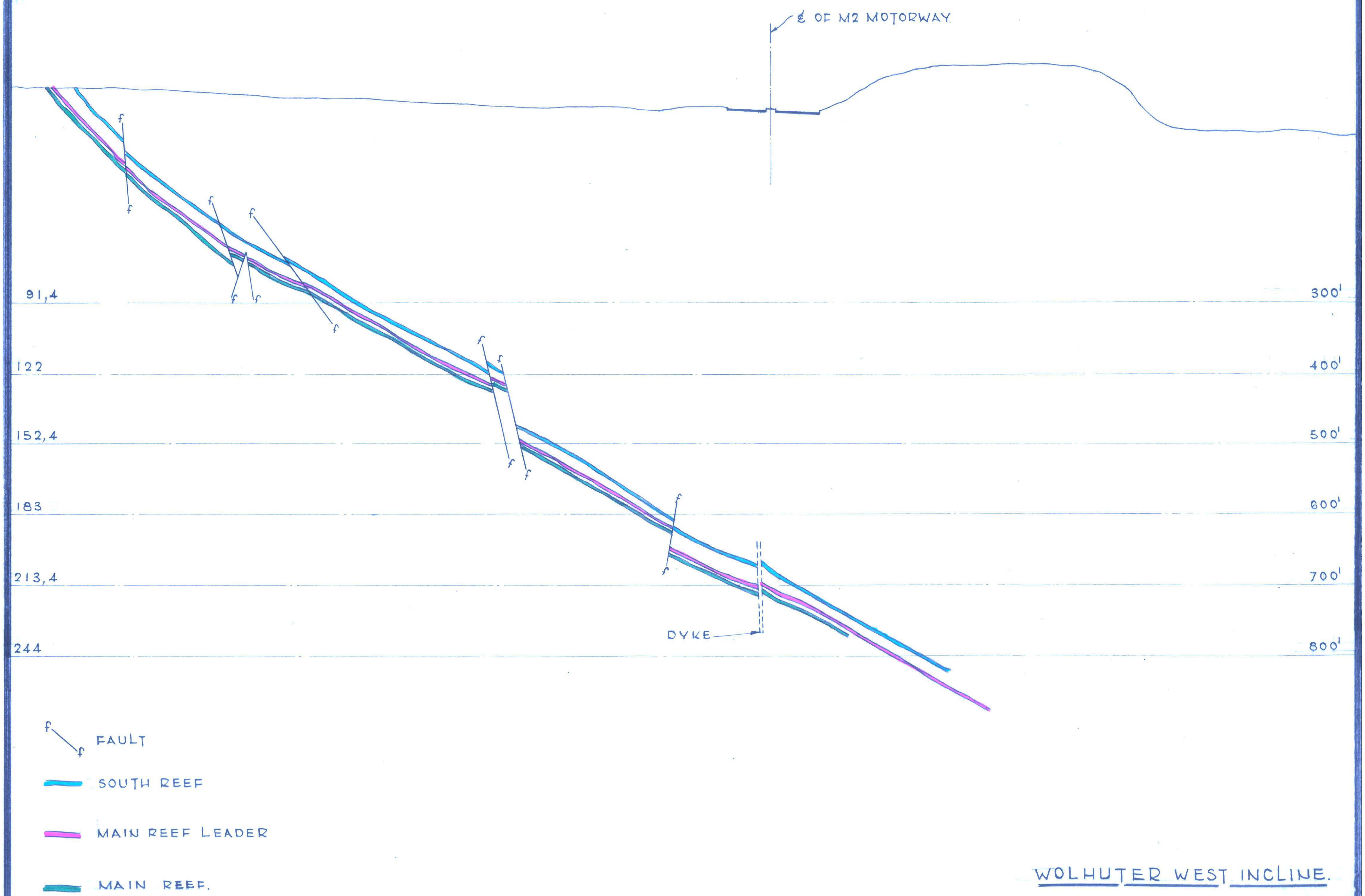


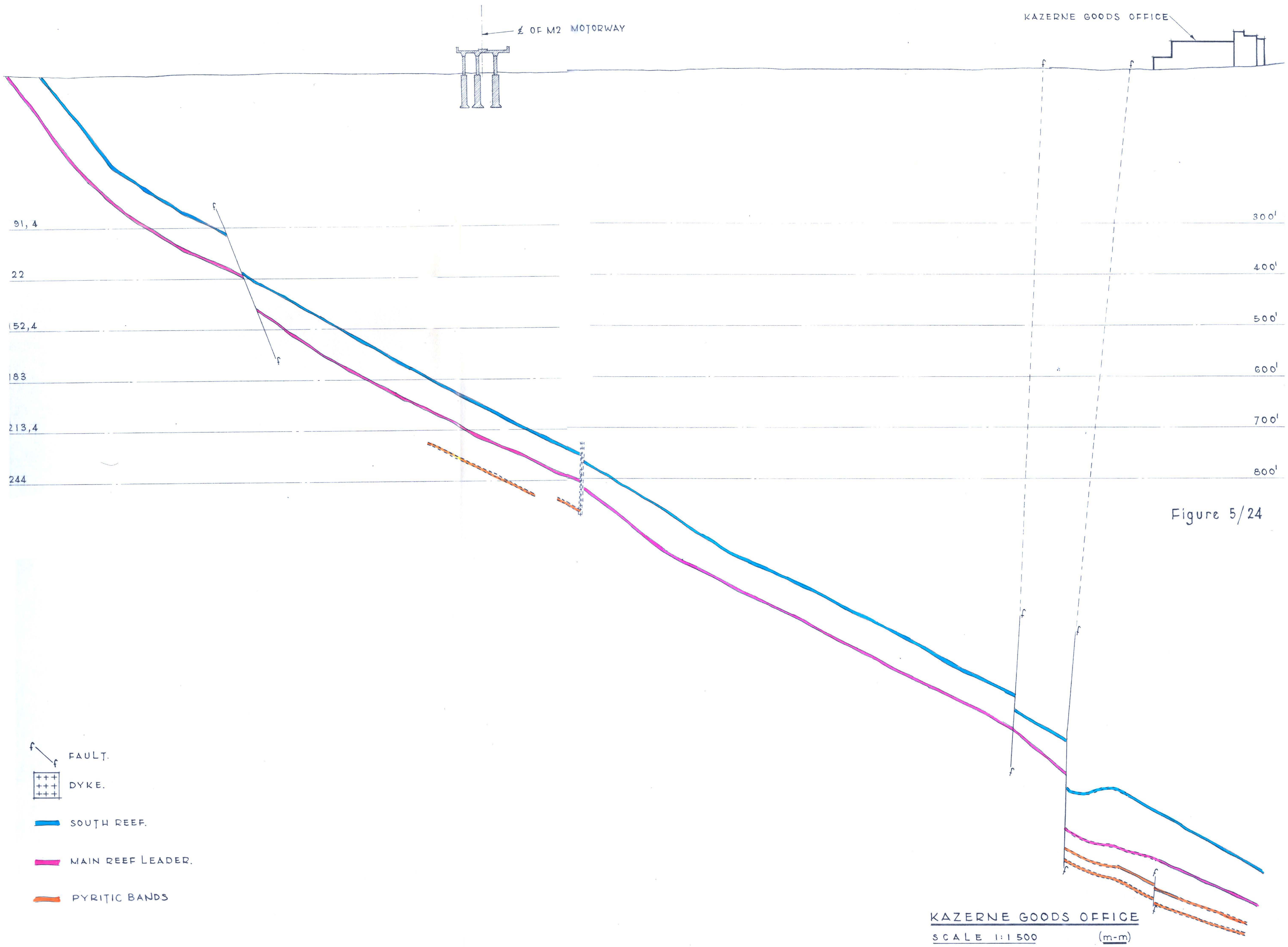
Figure 5/25
℄ OF M2 MOTORWAY
91,4
122
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
f
FAULT.
DYKE.
SOUTH REEF
MAIN REEF LEADER.
MAIN REEF
PYRITIC BAND
MEYER CHARLTON (n-n)
SCALE 1:1 500

Figure 5/26



WOLHUTER WEST INCLINE.

SCALE 1 : 1 500 (o-o)



℄ OF M2 MOTORWAY
KAZERNE GOODS OFFICE
91, 4
22
152,4
183
213,4
244
300'
400'
500'
600'
700'
800'
FAULT.
DYKE.
SOUTH REEF.
MAIN REEF LEADER.
PYRITIC BANDS
KAZERNE GOODS OFFICE
SCALE 1:1 500 (m-m)

Figure 5/24

Furthermore, subsidence takes place at right angles to the plane of the reef - a fact established over the years by measurements taken during the removal of reefs as well as sagmeter stope-closure readings on mines such as Durban Roodepoort Deep, Crown Mines, City Deep and ERPM. Thus in the case of Diagram A closure would be horizontal, in Diagram B it would be at right angles to the dip of 45^0, and in Diagram C it would be vertical.

The effect of dykes and faults

A glance at the underground mine plans of any of the mines on the Central Rand suffices to show the extent to which dykes and faults criss-cross the whole of this gold-field. The same impression will be gained by an examination of the fourteen typical cross-sections through the undermined area immediately south of the Johannesburg city area: these are included here as Figures 5/13 to 5/26 and their locations are shown on the master plan in Figure 5/12. These cross-sections illustrate not only the frequency of occurrence of faults and dykes but also the fact that the former are invariably occupied by the latter or, conversely, that the dykes are invariably present as intrusions along fault-planes. While only strike- and oblique faults appear in these cross-sections which are drawn at right angles to the strike, it may also be stated that dip-faults are abundant and that these, too, are very frequently occupied by dykes.

The largest dyke in this area is the Grahamstown Dyke with a width of over 200 metres in places: the outcrop area of this dyke is shown on De Beer's map (1965).

The presence of these features poses great problems in the extraction of the ore-body. Dislocation of the reefs by faulting greatly impedes the rate of mining progress, while the larger dykes create the need for special treatment of haulages traversing them in order to avoid closure due to rock-bursts to which they are particularly susceptible. Reference will be made later to the propagation to the surface of shock waves via dykes and faults.

Were it not for the presence of dykes and faults the prediction of surface subsidence would be a relatively straightforward matter. Irregularities in surface subsidence caused by faults and dykes have already been referred to in many of the examples cited and further reference to their effects will be found in several of the case histories to follow. Mention need only be included here of subsidence problems related to the failure of dykes in the underground workings. Most dykes less than 3 m wide almost certainly failed at the time of mining. Dykes more than about 100 m wide will probably not fail within the next five centuries (Grobbelaar, 1970). Dykes between 5 and 15 m wide which have not yet failed are liable to do so violently at a time when the instantaneous stress exceeds their instantaneous strength*. In an extreme case the failure of, for example, a dyke 10 m wide, at a depth of 300 m below surface, could cause a sudden subsidence of about 30 per cent of the total unclosed stoping width in the area, which could result in differential surface subsidence of a metre or more (Grobbelaar, 1970).

The effect of mining technique and of renewed mining activity in dormant areas

Developments in mining techniques have had their repercussions on the surface. When the jackhammer rock drill replaced the old type bar-rigged slogger machines a considerable reduction in stoping width followed. This in turn made it possible to work previously unpayable areas, particularly on the South Reef (Pyne-Mercier, 1970). When South Africa went off the gold standard in 1933, and again when the price of gold increased in 1949 and 1974, mining was resumed in areas which had been regarded as unpayable before. These three dates are of the utmost importance because, while it is a firmly established fact that surface subsidence diminishes with time after the cessation of local mining activity, and the surface usually reaches a state of equilibrium some ten to fifteen years after mining has ceased, the resumption of mining in previously dormant areas brings with it the possibility of the resumption of surface subsidence. The Government Mining Engineer is well aware of this fact and has to try and reconcile the interests of the mining companies with those of the owners of existing buildings or potential developers in areas affected in this way. During 1974 there was a spate of reclamation-mining in progress on the Central Rand or planned for the immediate future, and the attendant dangers in terms of

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* As the strength of a pillar is dependant on moisture content, the effects of cyclic loading and the progressive growth of fractures, the *instantaneous strength* refers to its strength at a given moment.

surface subsidence were once more of dire concern. The more recent drop in the price of gold at the moment of writing must be regarded as a temporary respite from the inevitable surface consequences of reclamation mining or, from another point of view, possibly a temporary setback!

THE DIFFERENT TYPES OF MINING SUBSIDENCE EXPERIENCED ON THE CENTRAL RAND

From the above discussion, and from the examples of subsidence already cited and those to follow in the case Histories, it is concluded that four distinct types of mining subsidence are known on the Central Witwatersrand.

1. 'Sinkholes'

Where the outcrop excavations have in the past been filled with soil or rubble, as is the case over extensive areas, and where water from a surface or near-surface source seeps in sufficient quantities through the fill, loose material is washed down the stope to greater depths. By a process of 'headward erosion' from the bottom of the fill upwards, the stage is eventually reached where the plug of remaining fill at the top of the outcrop excavation is no longer capable of spanning across the void and it collapses to form a sinkhole. The mechanism is in many ways similar to that occurring in the dolomites which is fully discussed in Chapter 7.

By the same process sinkholes have been known to develop not only in fill within the outcrop excavations but also within the soil wedge immediately south of the outcrop. Dr F.G. Hill has made an intensive study of the outcrop area over a strike distance of 16 km from Durban Roodepoort Deep in the west to the Village Main Reef mine in the east and has found no single instance of a sinkhole having developed more than 15 m to the south of the South Reef outcrop. The reason that he advances for this is the strength of the hanging-wall rock at a depth of some 30 m below surface: tests have shown the compressive strengths at this depth to be within the range 25 MPa to 55 MPa. He has also observed that sinkholes are more prone to develop where the dip of the reefs on outcrop is steep.

It is important to stress that, whereas most forms of mining subsidence continue only for a limited period of time after cessation of mining activity, the sinkhole type of subsidence may be activated at any time, even centuries after mining ceases, if no preventive measures have been taken. As stated above it is usually caused by erosion of fill by water flowing down from the old stope and may thus be initiated at any time by the development of a leak in a nearby pipe. Three possible methods of providing protection against sinkhole development have been suggested and applied by Dr F.G. Hill. These are illustrated in Figure 5/27 together with a diagram showing the circumstances under which sinkholes develop. However, remedial treatment after the development of a sinkhole type of subsidence invariably involves more sophisticated methods, as illustrated by the example given in Case History 9.

2. <u>Subsidence accompanying cavern development</u>

Closely related to the sinkhole type of subsidence but of less common occurrence is surface subsidence associated with the development of a cavern by 'frittering' of the decomposed hanging-wall rock immediately south of the outcrop area. A well-documented example of this type of cavern development with accompanying surface subsidence, and the remedial treatment which was applied is presented in Case History 7. This example concerns the development of a vault-shaped cavern in decomposed quartzite under part of the Main Reef Road near Longdale Township. It would seem that a shallow dip on outcrop favours the development of this phenomenon because it is in these circumstances that the stope passes through a substantial area of decomposed quartzite. Consequently the potentially affected zone may extend farther south of the outcrop area than is the case with sinkholes. Remedial treatment under the Main Reef Road occurrence near Longdale Township had to be applied for a distance of 60 m south of the outcrop.

A cavern of similar shape and dimensions, about 6 m high, is known to exist directly beneath the M2 Motorway at a depth of about 180 m below the surface immediately west of Sauer Street. The Main Reef had been mined here to a stoping width of two

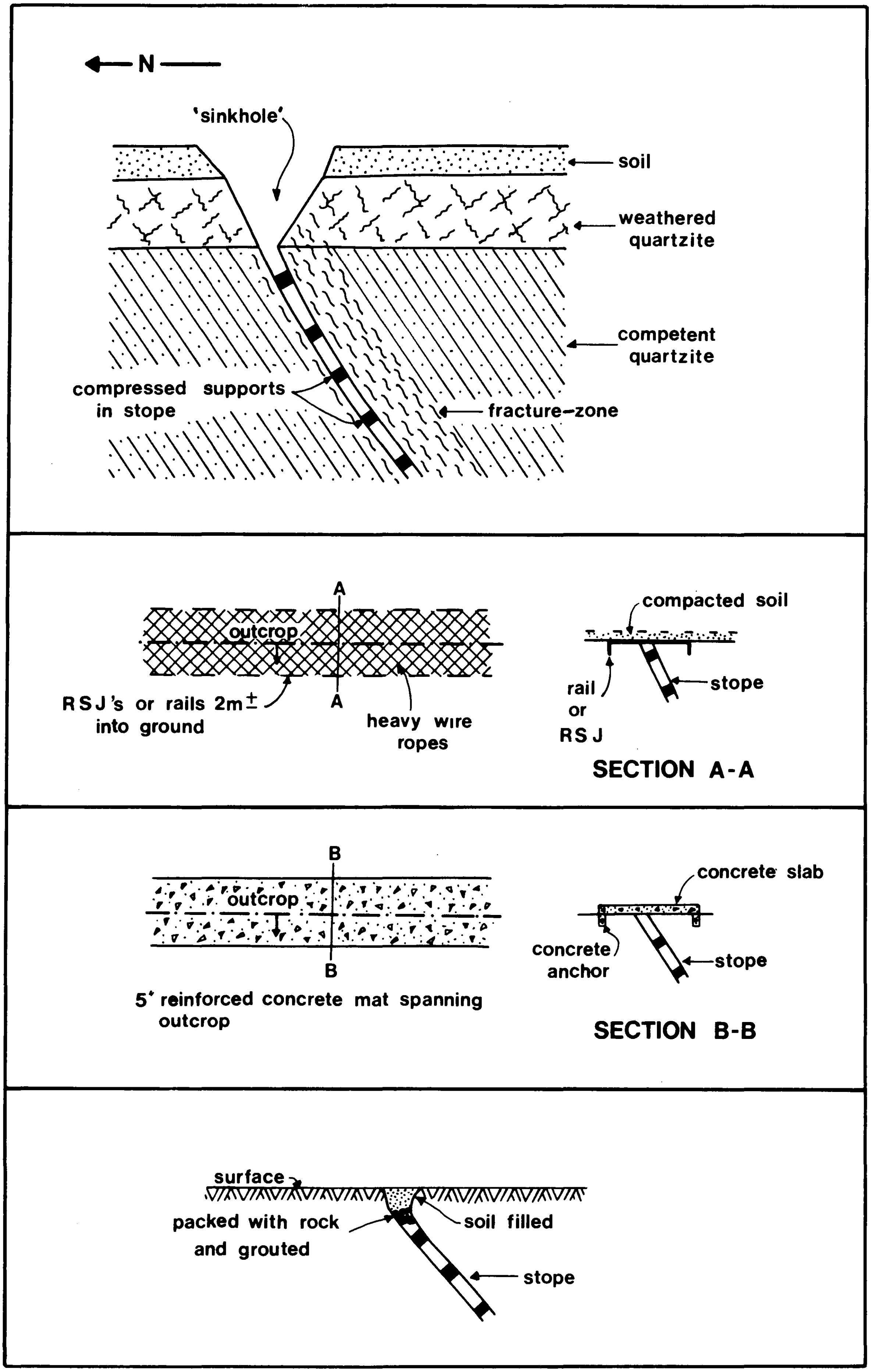


FIGURE 5/27 : 'Sinkhole' development above stope in outcrop area and suggested preventive treatment (After F.G. Hill) metres and the reef at this depth dips about 30° to the south. Frittering of the fresh but highly fractured hanging wall resulted in the formation of the vault-shaped cavern, the fallen material having moved down the stope to greater depth. There is no record of surface subsidence associated with this particular feature, and the great depth below the motorway did not warrant the filling of the cavern. This type of feature is common on the Central Rand, and develops within a large range of mining depths.

3. Subsidence related to the development of tension-fractures

Reference has already been made to the development of tension-fractures through the hanging wall and extending right up to the surface. This type of failure may result in severe disruption at the surface if it is accompanied by differential settlement and the formation of a 'step' at the surface. The fulcrum in the stope about which rotational movement takes place may be the intersection of a fault, a dyke or an extensive width of 'unpay' which has been left *in situ*. Vertical displacement accompanying this type of failure is enhanced by a steep angle of dip at the outcrop as illustrated in the occurrence at Nourse Mines described in Case History 11. Where a shallower angle of dip is involved, as in the case of the subsidence at Rand Leases (example 11 mentioned earlier), a substantial horizontal component of movement may also be observed.

This type of subsidence may take place a long time after completion of mining activity, as was the case at Nourse Mines.

4. Normal subsidence of hanging wall due to stope closure

As already discussed, the most widespread type of subsidence is associated with stope closure in the elastic or macro-plastic material constituting the hanging wall. Subsidence takes place at right angles to the plane of the reef, and the rate of subsidence decreases with time after mining activity has been discontinued. Rejuvenated mining activity in the area may result in further subsidence at the surface.

From a practical point of view it is only differential subsidence that affects engineering structures. Differential subsidence of a small order of magnitude may be encountered above areas which have been undermined at shallow depth. In this case the fracture-zone extends right up to the surface and macroplastic behaviour dominates in the subsiding mass. Where greater depths of mining are involved and the fracture-zone does not extend to surface there is no danger of differential settlement being experienced at surface - except as a result of the influence of faults and dykes. This is due to the fact that the rock mass above the fracture-zone moves downward *en masse*. The amount of subsidence is dependent on the total stope closure which, in turn, is dependent on such factors as support, stoping width, depth and number of reefs mined.

CASE HISTORY 7

MAIN-BIRD SERIES

CAVERN DEVELOPMENT IN HANGING WALL QUARTZITE BELOW MAIN REEF ROAD NEAR LONGDALE TOWNSHIP, JOHANNESBURG

Historical background and geological setting

An unusual case of surface subsidence associated with cavern development in the underlying rock took place during the early 1960's beneath a section of the Main Reef Road near Longdale Industrial Township. The affected area was in the section between Springbok Road and Commando Road on the western boundary of the Johannesburg Municipal area.

Though it crossed the outcrops of the Main Reef and the South Reef, the Main Reef Road through this area had been hurriedly proclaimed, in 1927, in preparation for the visit of the Prince of Wales. The South Reef in this area had been mined in 1908, but a crown pillar had been left intact for a distance of 17 metres along dip immediately below the road, and for a width equal to the width of the tarred surface of the road. A number of ore-pillars about 3 metres square had also been left to support the hanging wall of the South Reef stope, though in a somewhat haphazard fashion. Furthermore, a local section of Main Reef and Main Reef Leader (with the thin intervening 'Black Bar') had been considered as 'unpay' and had therefore not been mined out beneath the road. Consequently there appeared to be no liklihood of the road suffering damage from mining subsidence or from any other cause. Indeed the road had specifically been routed to cross the outcrops at this point in view of the absence of undermining at shallow depth.

The reefs locally dip to the south at an angle of 40^0. The South Reef had been mined on either side of the road and down dip below the crown pillar to a stoping width of about one metre.

In 1959 the Croesus Gold Mining Company commenced reclamation mining on the South Reef and the composite Main Reef and Main Reef Leader in the area, but no mining was done immediately below the affected section of the road. However, the reclamation mining, which continued until 1967, included the extraction of a substantial pillar of South Reef between No 3 Level and No 4 Level situated below the bend in Main Reef Road (see Figure 5/28).

In about 1960 it was observed that large cracks, both transverse and longitudinal, had opened in the tarred surface of the road over a distance of about 60 metres southwards from where the road crossed the line of outcrop of the South Reef. It was found, too, that the road had subsided in this area by as much as one metre. The subsidence occupied an area of about 0,2 ha. Though the immediate cause of the subsidence was not understood, it was decided to isolate the affected area by closing off that section of Main Reef Road between Springbok Road and Commando Road and re-routing traffic along these two roads.

In order to determine the cause of the subsidence an exploratory winze was driven through the middle of the South Reef crown pillar below the road. It was hoped that this winze would provide access to the underlying stope. It was found, however, that the winze opened into a 10 metre high, vault-shaped cavern which had developed in the hanging-wall quartzite, and the crown of which was no more than 4,5 metres below the tarred surface of the road. Continued rock-falls from the roof of the cavern prohibited entry into it or into the stope streching down-dip from the bottom of the cavern (see Figure 5/28). It was observed that the hanging-wall quartzite was oxidised and decomposed into a friable material locally known as 'red rock' down to No 3 Level, i.e. to a depth of about 90 metres below surface. Below this level it was in the form of undecomposed hard 'blue rock'. The cavern, which was situated at the toe of the crown pillar, had formed by 'frittering' of the decomposed hanging-wall rock. Rand Water Board pipelines along the road were found to be leaking as a result of the surface subsidence above the cavern crown, and the 'frittered' material was being washed down the stope into the deep mine workings. Frittering was seen still to be taking place at the time of the inspection from the bottom of the exploratory winze, and it was clear that it would be too dangerous to enter the cavern in order to effect remedial measures.

Level observations taken on 35 pegs in the road surface between October 1965 and January 1967 showed a further maximum subsidence of 61 mm (peg DS 19 on Figure 5/28).

Remedial measures

It was also clear that any filling material introduced into the cavern via the exploratory winze would simply gravitate down the stope to great depth, and there could never be certainty that all voids would be filled. Some means would have to be devised to form a plug within the 'throat' of the stope at the lower end of the cavern, in competent rock, and to create barriers in the stope on either side of the cavern, and then to fill the cavern with a settable compound. All of this work, however, would have to be achieved without workmen entering the cavern or the old mine workings.

The technique finally adopted was one devised by Ross Parry-Davies, at that time with the Cementation Company (Africa) Limited. Known as the 'Porcupine' technique, it involved the drilling of 100 mm diameter boreholes at 0,6 m centres around the perimeter of the affected area down to competent rock at the 'throat' of the stope, and the insertion of 'porcupines' through the boreholes. These resemble giant wire bottle-brushes. They are fabricated from a length of tube into which are inserted a series of 'quills' made from high tensile steel wire. The over-all outside diameter of the quill array when extended is about 500 mm but, when inserted into the 100 mm diameter borehole, the quills fold against the tube thus enabling them to be accommodated. On entering the stope void the quill array springs outwards and, with a series of such 'porcupines' at closely spaced intervals spanning across the width of the stope and socketed into the foot-wall at the bottom of the boreholes, an entanglement of wires is created along the stope.

Porcupines were inserted not only as a horizontal mesh into the throat of the stope but also down the dip of the stope on either side of the cavern, thus creating lateral barriers. Vertical boreholes 150 mm in diameter were then drilled from the surface to enter the stope above the basal porcupines and heavily 'doped' concrete containing fibrous material was dropped down the holes. The fibrous material bridged across the porcupine quills and prevented the concrete aggregate from falling through the porcupines. In this manner bulkheads were created at the bottom and on the sides of the cavern. Finally a very lean grout of cement and mine dump sand with bentonite and fly-ash additives was injected into the stope, thus completely filling the cavern.

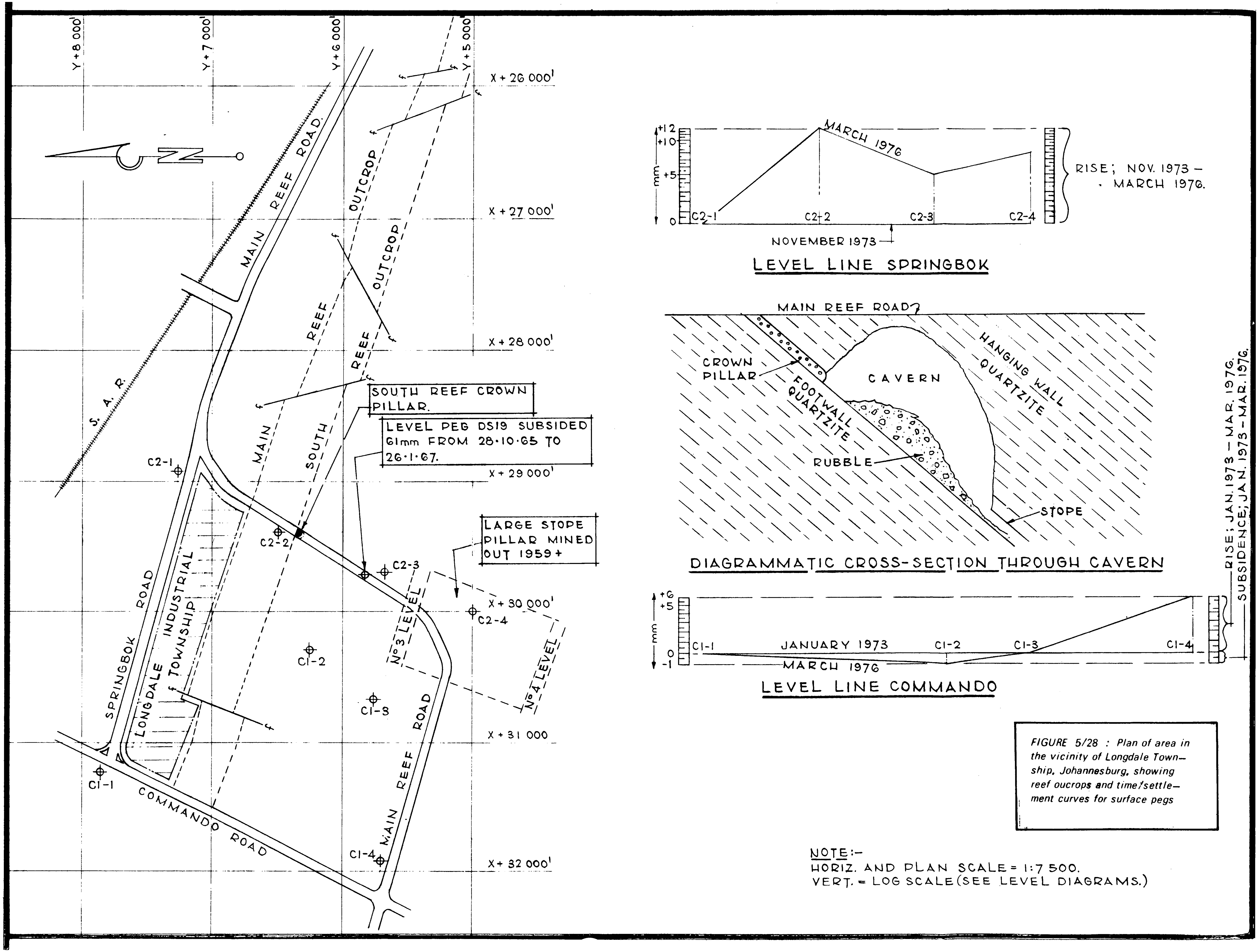


FIGURE 5/28 : Plan of area in the vicinity of Longdale Township, Johannesburg, showing reef oucrops and time/settlement curves for surface pegs

NOTE:–
HORIZ. AND PLAN SCALE = 1:7 500.
VERT. = LOG SCALE (SEE LEVEL DIAGRAMS.)

The drilling and grouting contract was commenced in August 1968 and was completed in June 1969. The total cost of R187 000 was borne by the Johannesburg City Council. Grout intake was 5 600 cubic metres. Fourteen check-holes drilled through the grouted area failed to locate any unfilled voids. The affected section of the road was resurfaced in March 1970 and opened once more to traffic. Level observations were taken on forty pegs up until June 1971 and showed not the slightest movement. In fact, as shown in Figure 5/28, an independent set of surface level observations conducted between 1973 and 1976 showed a rise of 5 mm in the treated area (peg C2-3) - possibly due to slight heaving of bentonite in the grout? Or slight settlement of the datum peg?

CASE HISTORY 8

MAIN - BIRD SERIES

MINING SUBSIDENCE ASSOCIATED WITH A MAJOR DYKE IN THE VICINITY OF CROWN INTERCHANGE, M2 - MOTORWAY JOHANNESBURG

> 'Even gold mining has to be considered in relation to ground subsidence in some areas. Johannesburg has already been mentioned in connection with the recent change to deep mining for its famous gold ore. In earlier days, this was found almost at the surface and within the limits of what is now the great modern city. The old workings have long since been unused but they are still there. They necessitate severe restrictions of modern building in a wide strip of land (the old reef) running through the city from east to west immediately to the south of the main city centre. Ground movements still take place, and so this area has limited capability of development. It was, however, ideally located for road construction when a great system of freeways was planned in the early sixties. Land for the east-west freeway, most conveniently located (as can be imagined) to serve central Johannesburg, was acquired at a relatively low cost. From the planning point of view this was all to the good, but the design and construction of this modern highway presented most unusual problems. Ground movements over the old workings had to be included as definite possibilities, and yet a wide overhead modern highway had to be constructed with certainty of performance. Adding to the complications is the fact that some gold still remains in some of the old shallow workings. Although uneconomical to remove now, it must be left available for removal in case the price of gold should go up! ..... The engineering solution is one of the greatest interest - the supporting piers, which weigh as much as 600 tonnes, are so mounted on bearings that, if and when necessary, they can be jacked either vertically or horizontally to compensate for any ground movement that may displace them. Few of the motorists who now use this fine highway can ever realise upon what an unusual structure they are riding.' Robert F. Leggett (1973), in 'Cities and Geology.'

The part of the Johannesburg East-West Motorway (or 'M2') known as Crown Interchange is situated immediately to the west of the point where the 'M1' (North-South Motorway) meets the 'M2'. The area occupied

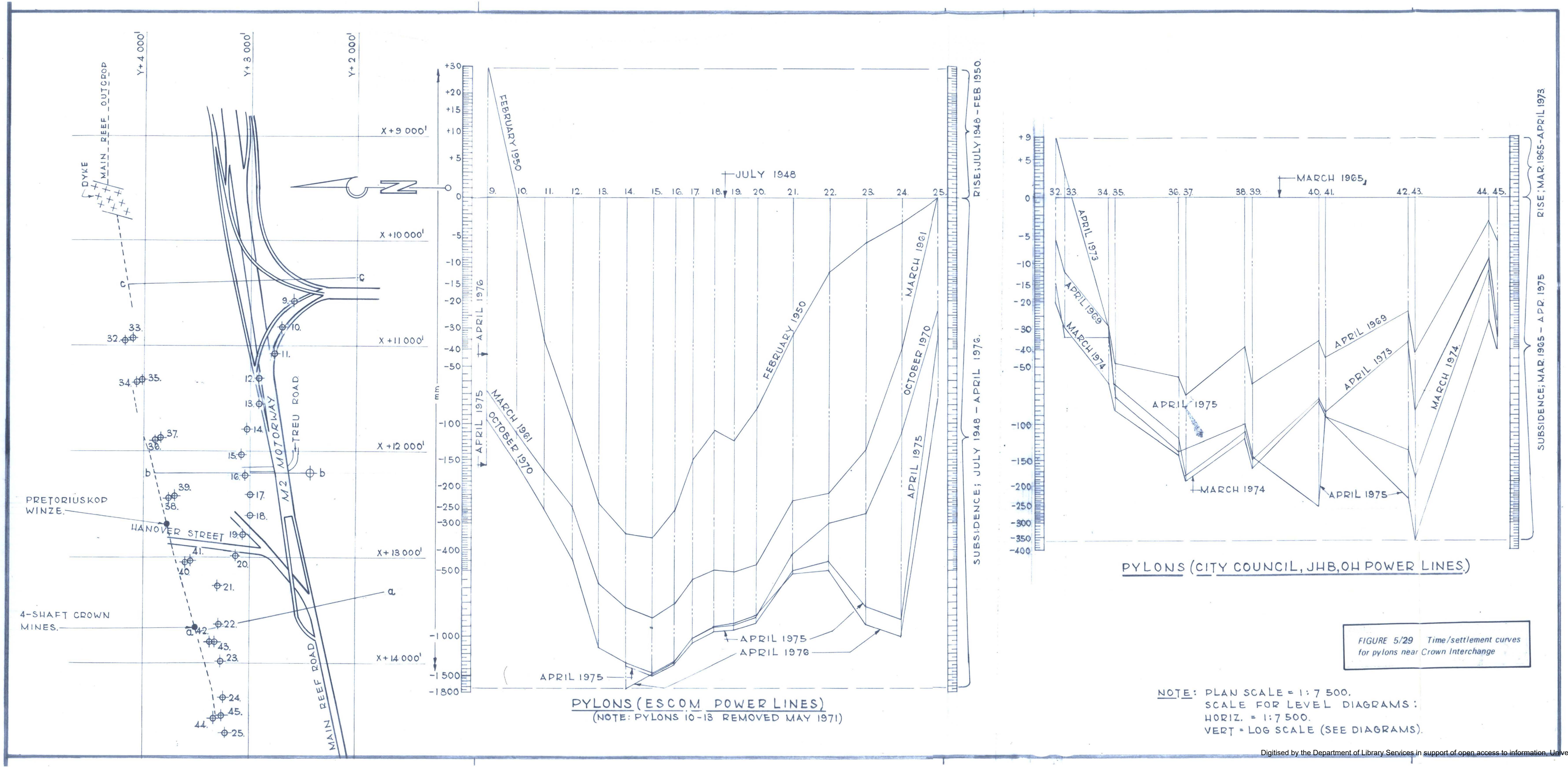


FIGURE 5/29 Time/settlement curves for pylons near Crown Interchange

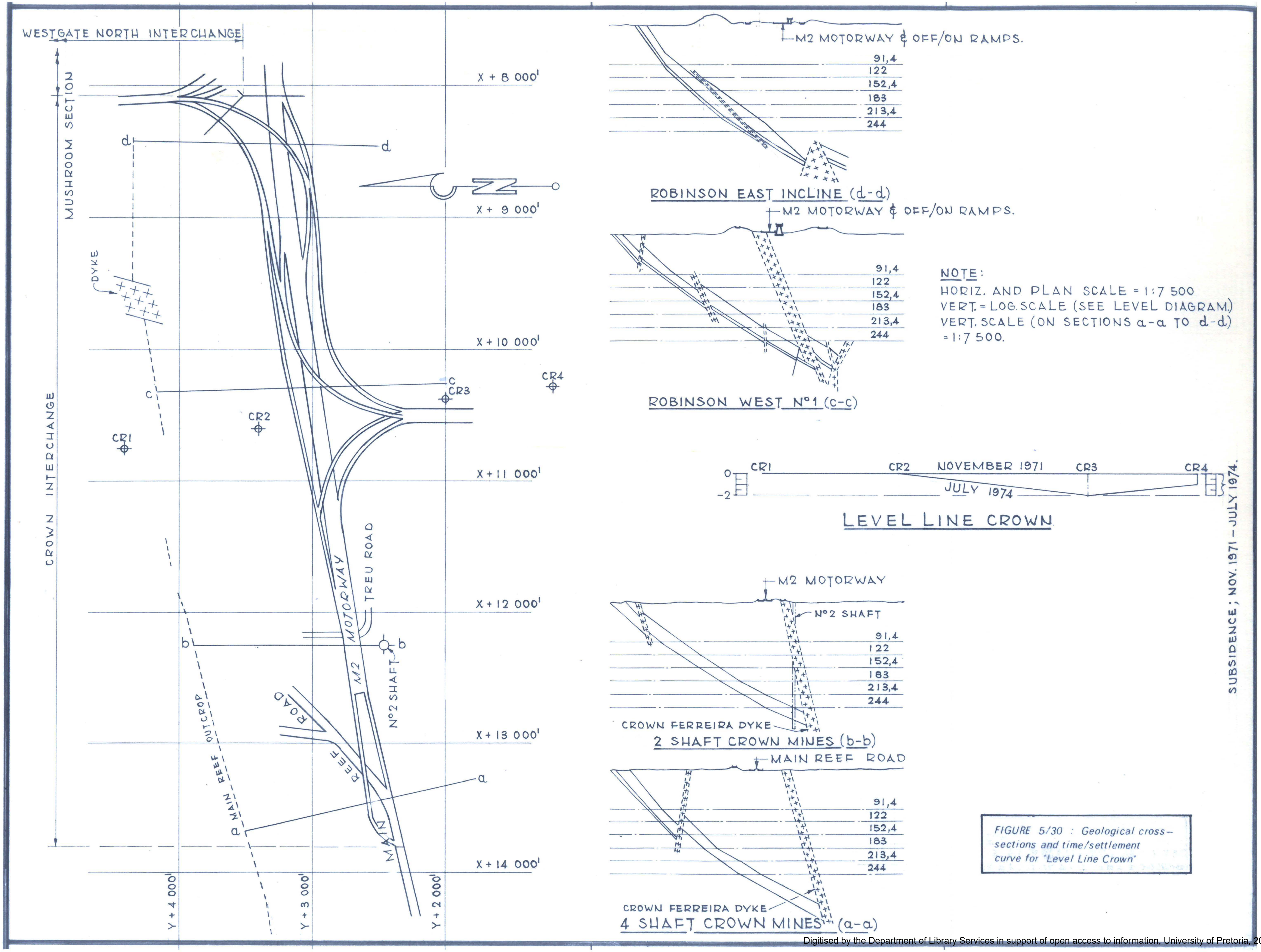


FIGURE 5/30 : Geological cross-sections and time/settlement curve for 'Level Line Crown'

FIGURE 5/31 Time/settlement curves for bridges at Crown Interchange by the bridges of Crown Interchange is of particular interest in the study of mining subsidence as level observations have been recorded on a number of structures in this area during the past thirty years, and surface movements are still taking place at the present time. And although to date there has been no need to apply the jacking procedure mentioned by Robert Leggett for the compensation of movements due to mining subsidence, it seems very likely that at least one of the bridges at Crown Interchange may have to be jacked up in the near future.

Records of level observations on pylons, motorway bridges and ground-pegs in the area are plotted in Figures 5/29 to 5/33. It will be seen that certain of these observations extend southwards from the immediate foot-wall of the Main Reef to points where the depth of undermining on the South Reef exceeds 244 metres.

LOCAL GEOLOGY

Geological cross-sections through the area from north to south are shown in Figures 5/13 to 5/16. For ease of reference these cross-sections are reproduced on a smaller scale in Figure 5/30. It will be seen that the three major reefs (Main Reef, Main Reef Leader and South Reef) dip to the south at the outcrop at angles ranging from 43^0 to 56^0 and that they all flatten out to angles of 35^0 or less by a depth of about 200 m.

Wherever the reefs are displaced by faults, all of which are occupied by dykes, the upthrown sides are on the south. The largest of the dykes is the Crown-Ferreira dyke. It occupies a reverse strike-fault and thus dips towards the upthrown side. As it is decomposed to great depth there are no outcrops of the dyke-rock and it has not been possible to map the surface trace of the dyke with any degree of accuracy. Although the trace of the northern contact of the dyke as encountered in the mine workings is well established (Figure 5/31) the attempt at mapping the dyke is largely subjective as it is based on upward extrapolation. No attempt has been made to extend the mapping of the dyke eastwards from bridge 83. De Beer's (1965) map shows the Crown-Ferreira dyke crossing the younger Robinson dyke in this vicinity without any displacement. But it will be seen from the cross-section in Figure 5/16 that the Crown-Ferreira dyke - which *may* be presumed to be represented here by

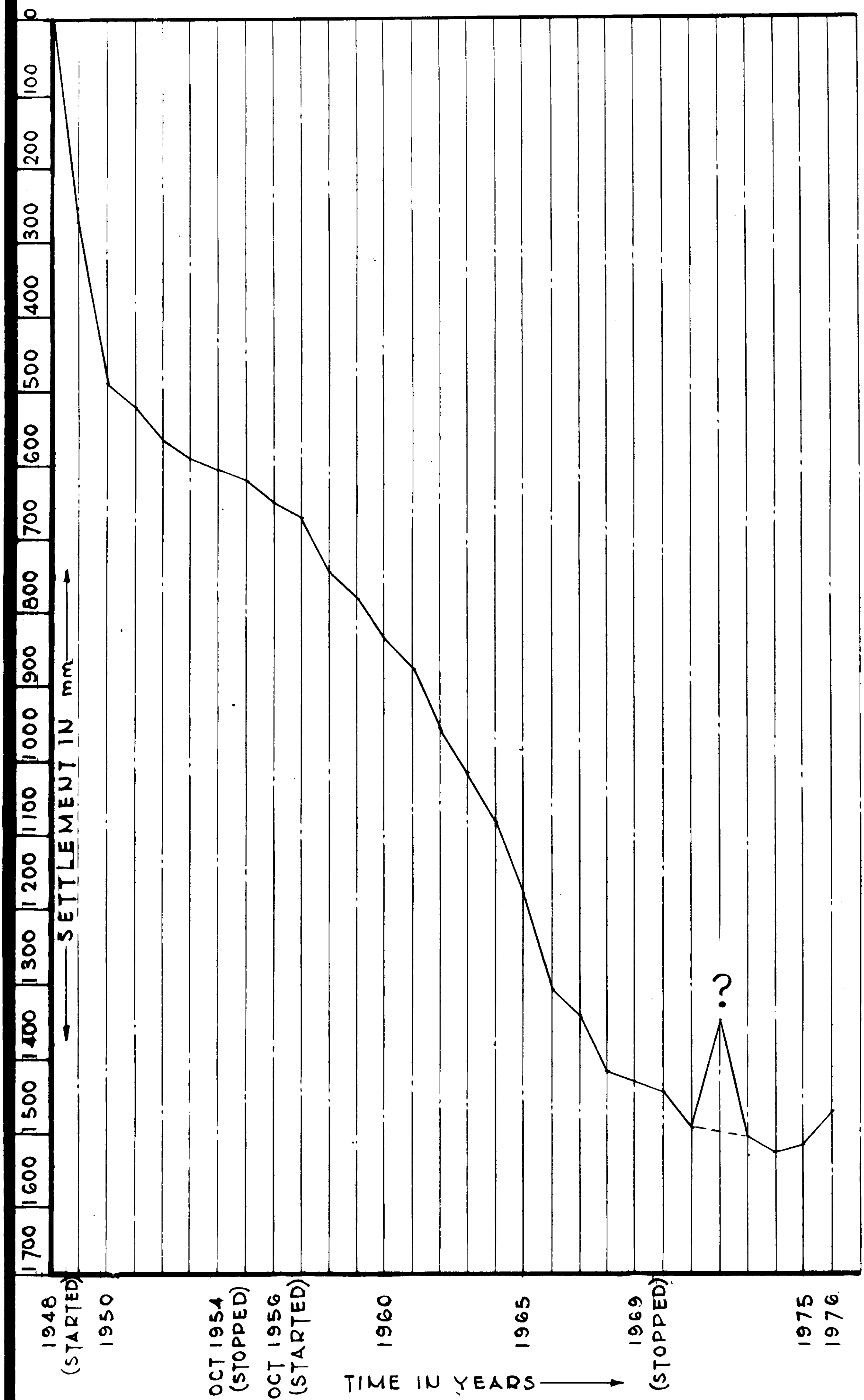


FIGURE 5/32 : Time/settlement curve for pylon 15 near Crown Interchange

Vertical scale 1 : 750
Horizontal scale 5 mm=1 year

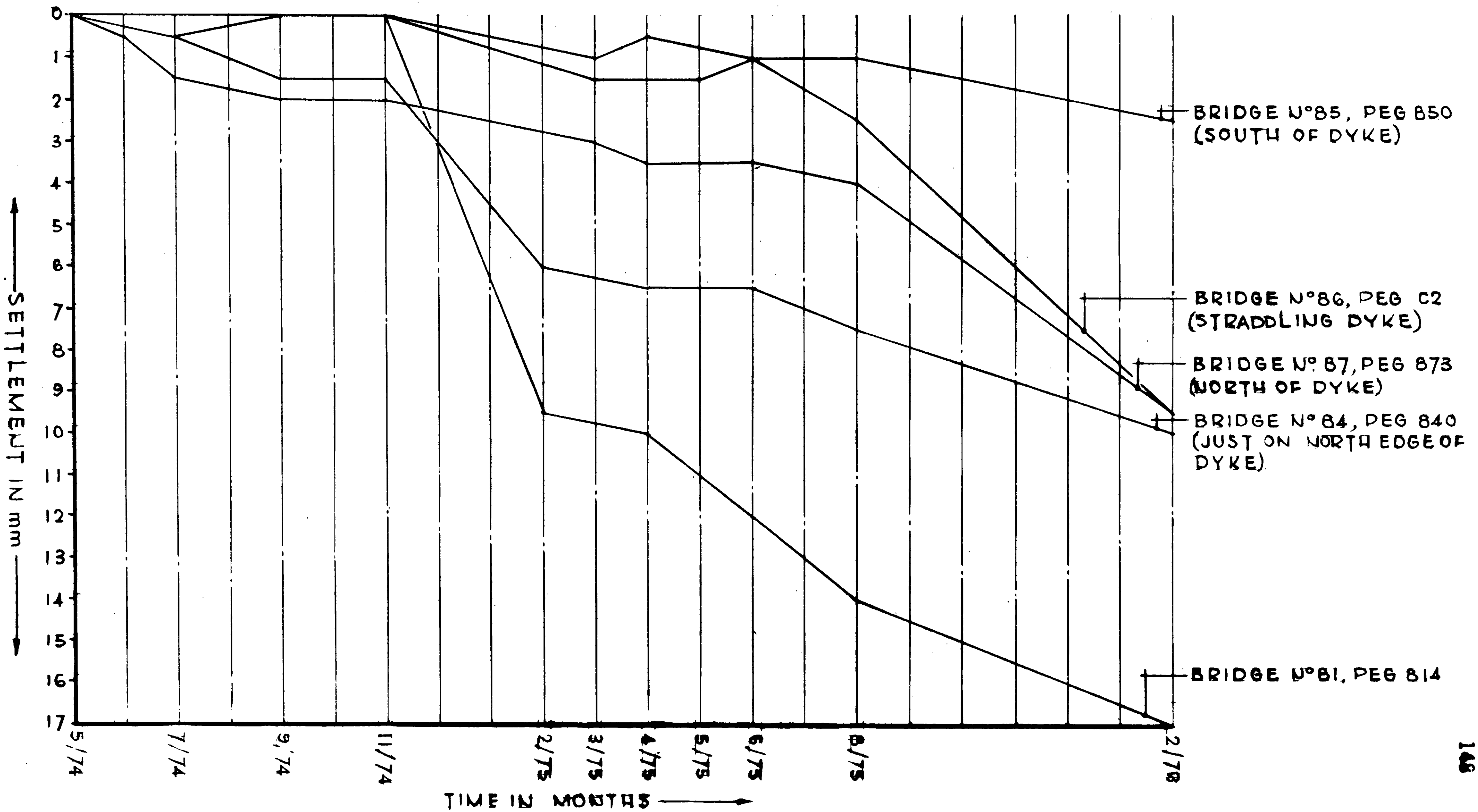


FIGURE 5/33 *Time/settlement curves for selected pegs on bridges at Crown Interchange* the triangular shaped intrusive body below 244 m - appears to have been displaced southwards in the area east of the Robinson dyke. This supposition will be seen to have significance in relation to the interpretation of subsidence records on bridge 81.

HISTORY OF MINING

Mining from the outcrop commenced in 1888 on the Main Reef Leader and the South Reef. In places where the Main Reef and the Main Reef Leader were found to be contiguous small portions of the former reef were extracted as well. By the time mining was stopped around the period 1908 to 1912 these reefs had been mined out to depths of more than 300 m below surface in places.

Mining was recommenced by Crown Mines Limited in 1947 on the richer parts of the Main Reef. Permission for this mining had to be granted by the Government Mining Engineer as certain roads, including the Main Reef Road, Hanover Street and Treu Road (see Figure 5/29), all carrying heavy traffic at the time, were situated above the affected area. In addition, Hanover Street carried a tram route. The GME granted permission for mining under this built-up area but took steps to minimise the subsidence which would inevitably take place on surface, and to try and make it as uniform as possible. Consequently he imposed conditions on the manner in which the stoping was to be done. The underground packing had to be systematic and a certain percentage of removed rock had to be replaced underground by rock-packs and walls. Stope widths were restricted to less than 2,5 m and no mining could be done within 30 m of the surface. Certain areas below Escom pylons and City Council pylons (see Figure 5/29) were not to be mined out.

Mining activity continued until October 1954 by which time large portions of the Main Reef had been removed to a depth of about 300 m below surface. Reclamation mining on the remaining patches of the Main Reef and Main Reef Leader was again undertaken during the period October 1956 until the end of 1969.

In 1972 Crown Mines Limited commenced a programme of restoration work in the stopes involving the installation of rock-packs to a depth of 100 m below important structures which had been erected on the surface and below areas which were to be proclaimed for township development. At the same time reclamation mining was recommenced. This involves the removal of loose ore from the old stopes and the extraction of pillars of ore which had been left in place to provide support. Most of the reclamation mining which is being undertaken at the present time is within the Main Reef to the east and west of No 4 shaft, from No 1 level (21 m below surface) upwards, but not right up to the surface, and downwards to No 3 level (69 m below surface). The Main Reef is very wide in this area and had been partly mined before. The current mining involves the extraction of additional 'slices' of the reef from within the old stopes.

The Main Reef, Main Reef Leader and South Reef have now all been mined out in the area below Crown Interchange. Each of the three stopes varies in width from 1,2 m to 1,8 m. The average combined stoping width is about 4,3 metres. But there are patches where the stoping width of the Main Reef alone exceeds six metres.

Underground inspections of the old mine workings in the vicinity of the Robinson East Incline Shaft (i.e. at the eastern side of the area under consideration) were carried out during 1965 by the City Engineer's Department and their consultants and also by the Government Mining Engineer's Department. These inspections were undertaken as part of the feasibility study for the planning of the M2 motorway. The following extracts from the reports on these inspections are of particular interest:

> 'The stopes are to a large extent closed with the stonewall packing crushed, yet further closure can take place and there were isolated areas where observers could see (from a drive on the Main Reef) right up into the Main Reef Leader workings. Such timber packing as had been utilised was rotten and at this stage must offer little or no support to the hanging.'
>
> 'Caves, usually in the proximity of dykes, some of them having the dimensions of 200 ft by 150 ft and 20 ft high and 76 ft by 270 ft and 35 ft high, were observed.'
>
> 'Settlement of the hanging wall is complete in some places but in others it is still taking place. It does not follow automatically that the effect of such

> subsidence has already reached the surface. Some cavities may still be left in the hanging wall of the South Reef due to the doming effect resulting from sagging of the strata. In such areas subsidence may still occur in the future.'

The influence of the Crown-Ferreira dyke in relation to the cracking of houses due to mining subsidence

The positions of fifteen old mine houses in the area, all but one of which have now been demolished, are shown on Figure 5/31. The colour code indicates which of these houses were cracked and which of them had no cracks. It will be seen that the houses which were either moderately or severely cracked straddled the northern contact of the Crown-Ferreira dyke. The cracking was obviously caused by differential movement on this contact. The house which was situated entirely upon the dyke was not cracked. Nor were the houses situated on the quartzite immediately north of the dyke. Level observations on pylons near to these houses show, as will now be discussed, that very considerable subsidence took place in this area north of the dyke. The fact that these houses did not crack, although they subsided by an amount of the order of one and a half metres, indicates that this movement was *en bloc* and not differential.

Discussion of level observations on pylons

In 1947 the Government Mining Engineer instituted a system of levels which had to be measured every three months on Escom pylons 9 to 25 and on City Council pylons 32 to 45. The positions of these pylons are shown on Figure 5/29 together with the level observations which are plotted on a logarithmic scale. It will be seen that Escom pylon 15, which was situated about 110 m north of the Crown-Ferreira dyke and which was undermined at a depth of about 165 m by the South Reef stope, settled 360 mm during the first 20 months of level observations. During the same period pylon 25, situated virtually on the Main Reef outcrop, did not move; and pylon 9, situated to the south of the Crown-Ferreira dyke, rose by an amount of 30 mm. The intermediate pylons settled by intermediate amounts. The phenomenon of the increase in elevation of pylon 9 during this period has been referred to as the *fulcrum effect*: just as when a writing pad is bent over the edge of a table the part above the table flexes upwards, so the strata immediately under the surface of the ground, when made to sag down over the edge of the subsiding zone, lift a little in advance of that zone.

During the next 26 years, from February 1950 to April 1976, pylon 15 settled a further 1 140 mm, making a total subsidence of 1,5 m as from July 1948. The time/settlement curve for this pylon is plotted in Figure 5/31, and it will be seen that there is a clear relationship between periods of mining activity and rate of subsidence. After 1969 only small patches of Main Reef were mined in the general area, but no reef was mined from immediately beneath the pylon: the subsidence curve flattens out after this date and even shows a slight tendecy to rise after 1974. The reading for 1972 is anomalous and probably represents an error in survey.

Level observations on the City Council pylons are available only for the ten year period from March 1965 to April 1975. As may be seen from Figure 5/29, these pylons, which carry overhead power-lines, are all situated immediately to the south of the Main Reef outcrop, with the exception of pylon 44 which is situated on the immediate foot-wall of this reef. The maximum subsidence, of 360 mm during this period, was experienced by pylon 43. Again the *fulcrum effect* is evident in pylons 32 and 33.

Discussion of level observations on M2 bridges at Crown Interchange

Construction of the bridges on Crown Interchange commenced at the beginning of 1970 and the contract was completed in June 1973. In May 1974 the first of a series of precise level observations was made on pegs installed at intervals in the bases of each of the bridge columns and referred to stable bench-marks on the foot-wall. The positions of the bridges and of the level pegs established on them are shown in Figure 5/31 together with time versus settlement curves plotted to date for each of them. From an examination of all the data given in Figure 5/31 the following observations emerge:

Bridge 87

The bridge spans across the Main Reef Road at the western end of the motorway and is founded on quartzite to the north of the Crown-Ferreira dyke. The level records show that the bridge settled about 10 mm over the 21 month period after levelling observations were commenced. The settlement is fairly uniform from one end of the bridge to the other and the rate of settlement is fairly constant at about 0,5 mm per month (see Figure 5/33).

Bridge 86

This bridge, which spans across Treu Road, straddles the northern contact of the Crown-Ferreira dyke. It shows a surprisingly uniform settlement of 10 mm over 21 months, and a reasonably uniform rate of settlement after June 1975 of 1 mm per month (see Figure 5/33). It will be seen from the detail of this bridge, given as an inset on Figure 5/31, that the row of level pegs for which the levels have been plotted is situated on the western side of the bridge. Insignificant movements have been measured on the eastern side of the bridge and these have not been plotted here.

Bridge 85

This bridge spans across Treu Road south of the Crown-Ferreira dyke. It appears to be tilting slightly towards the north-west at a constant rate. The slight rise of peg 853 between May 1974 and August 1975 must be ascribed to the tilting effect of the rigid concrete structure rather than to a fulcrum effect of the underlying strata.

Bridge 84

The greater length of this bridge is situated to the south of the Crown-Ferreira dyke: level observations on this part of the bridge showed a uniform settlement of only 1 or 2 mm over the period May 1974 to August 1975 and a cessation of movement after that. But near the north-western end of the bridge it crosses the dyke, and the last length of decking rests on a concrete support which is founded on compacted mine dump sand above the quartzite immediately north of the dyke. This support (peg 840) settled 10 mm over 21 months and the rate of settlement will be seen from Figure 5/33 to have been fairly constant at about 0,5 mm per month. It will only be known whether this settlement is due to consolidation of the mine dump sand or to progressive closure of the underlying stopes after further level observations have been made and plotted.

Bridge 83

The north-eastern end of this bridge is probably situated on the Crown-Ferreira dyke: the rest of the bridge is founded on quartzite south of the dyke. The whole structure appears to be subsiding to a very slow and steady rate.

Bridge 82

This bridge, founded entirely on quartzite south of the Crown-Ferreira dyke, is also subsiding at a very slow and steady rate.

Bridge 81

The movements taking place on this bridge give cause for concern. The location of the bridge in relation to the Crown-Ferreira dyke is somewhat questionable, as discussed earlier, but is is clear from the cross-section in Figure 5/16 that the bridge is situated above a portion of the South Reef stope which is 'wedged' between two major dykes, the northern one of which occupies a reverse fault with a very low angle of dip. Level observations on the bridge show a remarkably uniform rotation towards the south-west, pivotting from the north-eastern end. As shown in Figure 5/33, the south-western end of the bridge (peg 814) settled at an alarmingly rapid rate of more than 3 mm per month during the period November 1974 to February 1975, after which the rate of settlement decreased to about 0,5 mm per month. If future observations show a continuance of subsidence at this rate it may well be that the enormous hydraulic jack, which to date has only been used to adjust vertical and lateral displacements which arose from construction defects, will have to be used to counteract the effect of mining subsidence.

CASE HISTORY 9

MAIN-BIRD SERIES

STABILISATION OF STOPE BENEATH EXISTING BUILDING : HAAK'S GARAGES, 26 SAUER STREET, OPPOSITE HALL STREET, JOHANNESBURG

Historical background and geological setting

During the early days of gold mining in Johannesburg before the turn of the last century, the old Ferreira Mine worked the ground on which Haak's Garages and Bell's Motor Parts Company were subsequently situated. As shown in Figure 5/34 the South Reef outcropped on the surface just south of the common boundary between these two properties. The strike of the reef was approximately east-west and the southerly dip was about 80^0, flattening gradually in depth (see cross-sections in Figures 5/19, 5/20 and 5/21). Locally the South Reef terminated in the west against the Crown-Ferreira Dyke and in the east against a near vertical dip-fault striking approximately north-south. As shown in Figure 5/35 the reef was mined out to a stope width of about one to two metres.

As some stage after the completion of local mining operations the wedge of foot-wall ground between the stope and the dyke slumped into the stope, probably as a result of the seepage of water along the decomposed upper portion of the dyke, producing a deep trough up to about 10 metres wide. This large hole was subsequently back-filled with earth and rubble.

At the time that Haak's Garages were to be erected in about 1937, the architects were faced with the problem of supporting the building over the old rubble-filled stope. Initially it was planned to support the columns of the building on reinforced concrete beams spanning the stope from north to south, with the northern ends of the beams supported on the dyke in Bell's Motor Parts Company property and the southern ends supported on the hanging wall quartzites. In the event, however, only two such bridging beams were constructed, and both of them at the western side where the dyke and the stope converge. The remaining length of the stope was spanned longitudinally by means of a thick concrete slab extending about 45 metres eastwards to beyond the dip-fault. The bearing

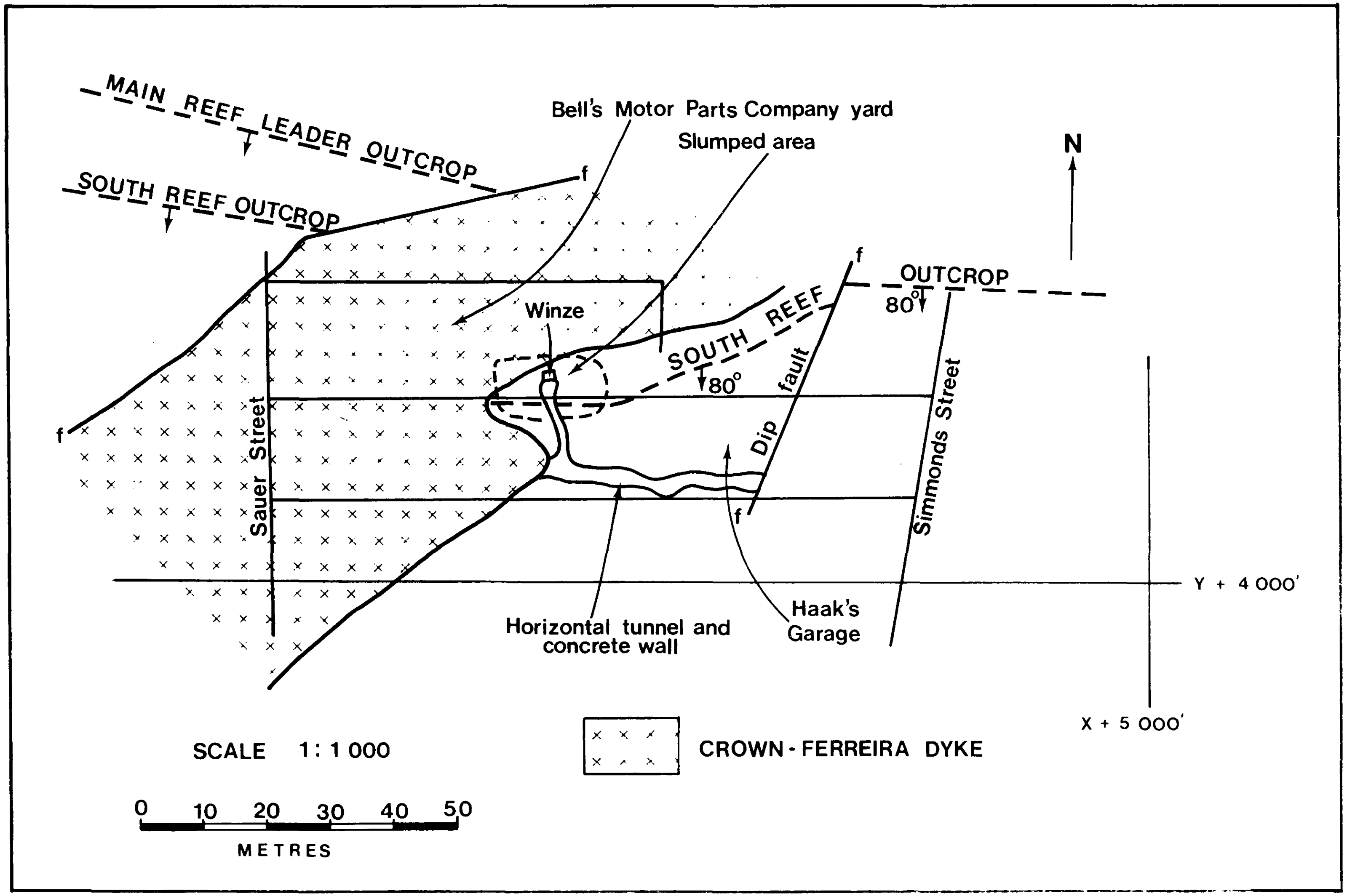
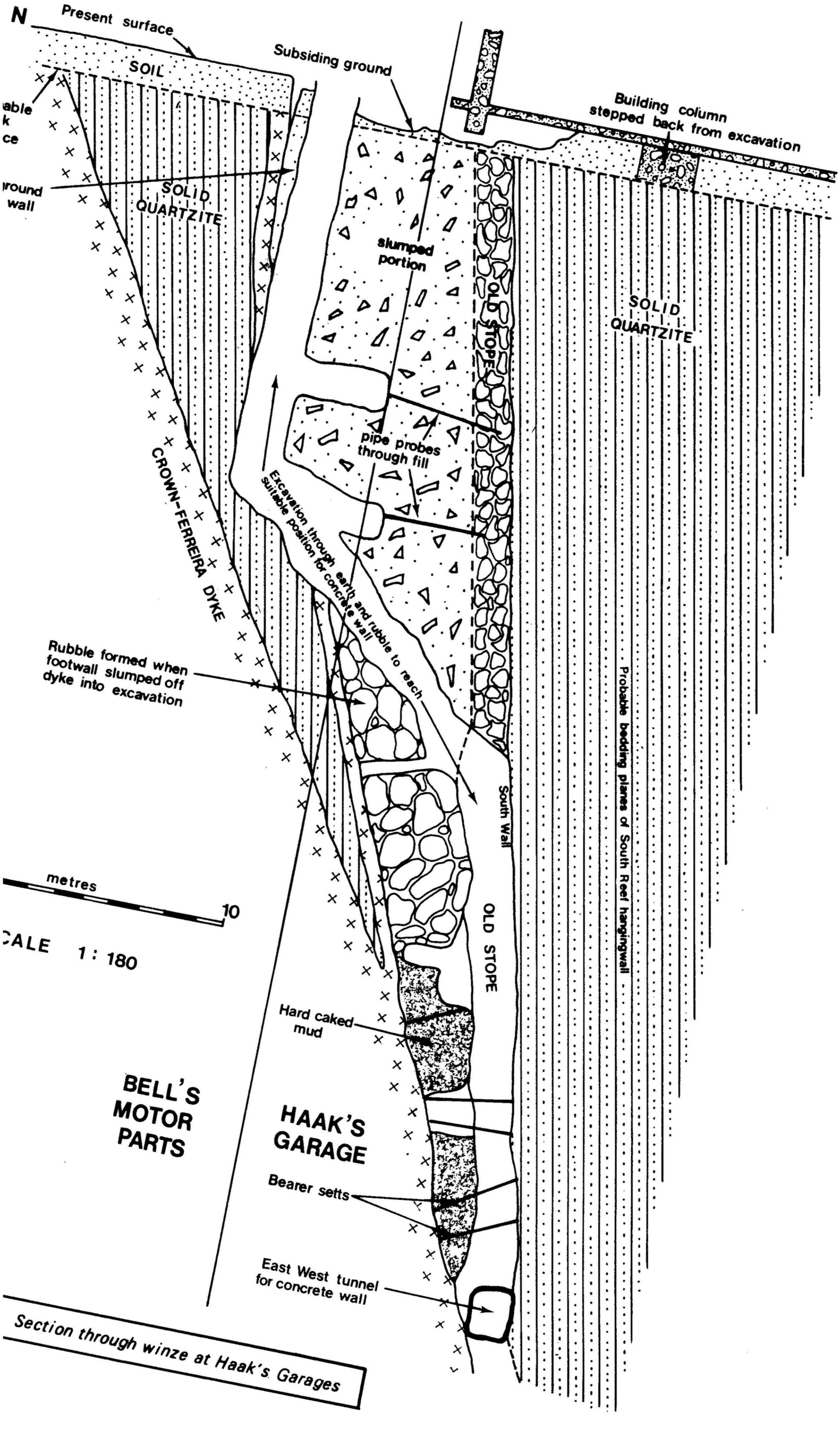


FIGURE 5/34 : Plan showing position of slumped area at Haak's Garages, Johannesburg in relation to Crown-Ferreira dyke



Section through winze at Haak's Garages pressure of the wall over this section was minimised by the installation of windows: the roof and upper floor were carried on columns founded on the hanging wall quartzites, as shown in Figure 5/35.

With the passage of time, and probably as a combined result of sub-surface erosion by water seepage, decomposition of timber props in the stope, earth tremors and vibration caused by trams and other traffic in Sauer Street, the stope filling started moving downwards into the mine in May 1952. If this had been allowed to continue unchecked it would undoubtedly have resulted in substantial quantities of fill being swallowed down the stope, as in fact happened on adjoining properties both to the east and to the west of the area. The immediate threat was the possibility of a catastrophic 'sinkhole' developing within the old stope by the sudden falling away of the fill. In an occurrence of this type on the adjoining property on Sauer Street to the north of Bell's Motor Parts Company, within the stope of the composite Main Reef and Main Reef Leader, large quantities of second-hand motor spares had been swallowed into the mine workings. Furthermore, should the stope have become emptied in this fashion even for a moderate distance, Haak's Garages would have been left poised on the brink of an overhanging precipice. Such an eventuality would carry with it the threat of slope failure within the hanging wall quartzite and the consequent destruction of the building. It was clear that remedial measures would have to be instituted without delay.

Remedial measures

It was decided that a reinforced concrete plug or 'wall' inserted within the old stope and extending from the Crown-Ferreira Dyke in the west to the dip-fault in the east would solve the problem. Such a wall would effectively prevent further erosion or subsidence of the stope filling down into the mine and would, at the same time, provide adequate support for the fill. Naturally the wall would have to be inserted at a depth where the stope width had not been increased by foot-wall failure, and where the dyke-rock was sufficiently unweathered to provide a sound western abutment to the wall. It was hoped that suitable conditions of this sort would be encountered at shallow depth, but this proved not to be the case.

Unlike similar projects which had been successfully achieved at shallow depth, the situation here proved to be considerably complicated by the extreme width of the collapsed section of foot-wall near the surface. It was impractical to insert the cut-off walling within the collapsed section as conditions here were extremely unstable and the walling would have had to be excessively wide. The only alternative was to sink a vertical winze against the intact foot-wall, followed by an inclined winze through the collapsed material to intersect the old stope, and then to excavate down dip through the stope filling to a point 46 metres below surface: it was only at this depth that the undisturbed 'throat' of the stope was encountered and that conditions were sound enough for the installation of the concrete walling (see Figure 5/35).

The awkard configuration of the excavation down to this depth necessitated three-stage hand-hoisting of the excavated material which very considerably retarded the progress of the work. It was also necessary to proceed with great caution for fear that the loose material originally used to fill the old excavation might at any moment have fallen away into the deeper part of the stope and wrecked the timber framework installed in the winze to retain this material and thus to protect the workmen, particularly in view of the fact that the timber framework had no firm base to support it. It was thus a relief when a stope width was reached where "bearer setts", i.e. strong timbers wedged between the solid dyke material on one side and the hanging wall quartzite on the other, could be installed to support the timber framework above.

A horizontal east-west tunnel had next to be excavated along the length of the 'throat' of the old stope. From the bottom of the shaft the tunnel had to extend for a distance of 6 metres towards the west, 3 metres of which was excavated into the dyke-rock to ensure adequate abutment support for the concrete wall. To the east the tunnel had to be driven for a distance of 33,5 metres before intersecting the dip-fault and establishing the eastern abutment. The tunnel had also to be driven with great caution because, although the side walls were flanked by solid rock, both the roof and the floor were in loose fill. The width of the old stope along the length of the tunnel varied between one and two metres, averaging one and a quarter metres. Under the circumstances it was decided to make the wall two metres high. The tunnel was accordingly excavated to this height and subsequently completely filled with concrete. A concrete mix of 1:2:4 was used and the wall was strongly reinforced with '60 pound' rails and burnt hoisting rope threaded through eyebolt rods which had been grouted half a metre into the rock on either side of the tunnel (Figure 5/36). Drainage through the wall was provided by the installation of pipes of 50 mm diameter at intervals of 3 metres.

When the concrete wall was completed the shaft and winze were backfilled with the excavated material which was compacted as well as possible. There were probably cavities within the fill which would have developed during the period of subsidence but, judging by the nature of the ground encountered during the course of shaft sinking and tunnelling, they were likely to be of insignificant dimensions which would not give rise to further major settlement of the surface. This prediction has proved true to date.

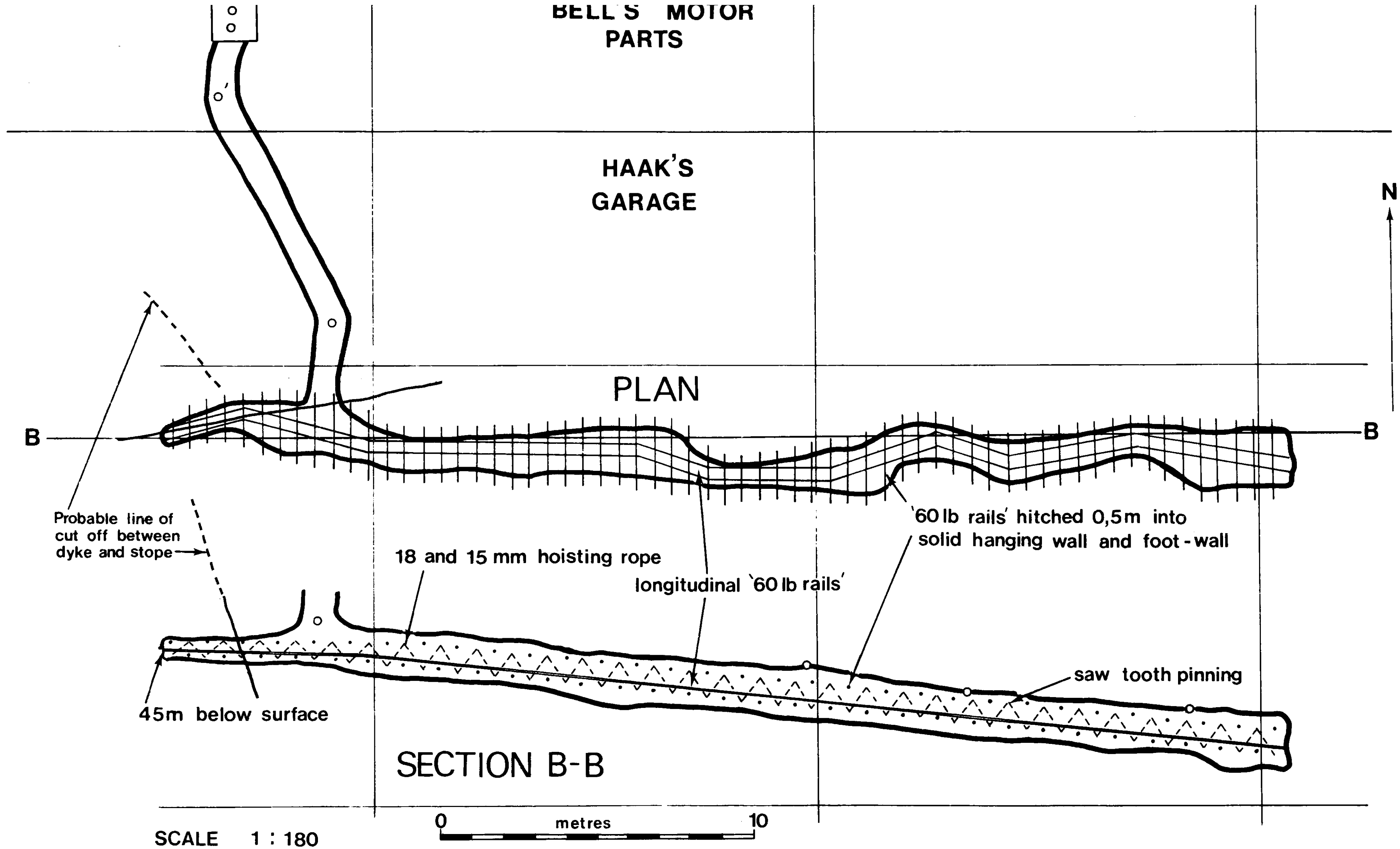


FIGURE 5/36 : Plan of, and section through, tunnel below Haak's Garages

CASE HISTORY IO

MAIN-BIRD SERIES

STRUCTURAL DAMAGE IN A BUILDING CAUSED BY DIFFERENTIAL SETTLEMENT AS A RESULT OF UNDERMINING AT A DEPTH OF 175 METRES

SUNLIGHT HOUSE, CORNER ROSSETTENVILLE ROAD AND HEIDELBERG ROAD, STAND 57 VILLAGE MAIN, JOHANNESBURG

Sunlight House is a three-storey reinforced concrete frame structure. It was built in 1937. The building suffered severe distortion and cracking some time after its completion. In 1954 a structural engineer was commissioned to ascertain the cause of the damage and to determine whether further damage could be anticipated.

Investigation disclosed that a maximum differential settlement in excess of 330 mm had taken place between the eastern and western sides of the building. Columns were found to be out of plumb by more than 50 mm on one or both of their principal axes. Many of the beams had been sheared off their supporting columns. Brick panel walls were badly cracked and substantial distortion had taken place in door and window frames (see Figure 5/37).

Mining subsidence was suspected. Examination of mining records revealed that, with the exception of a portion under the western side of the building, the area was undermined at a depth of 175 m. Level observations and horizontal measurements taken during 1956 and 1957 showed that settlement of the eastern portion of the building relative to the western portion was still taking place, though at a very much slower rate than must have been the case during the preceding period of the life of the building.

It was therefore concluded that most of the settlement must have taken place over a relatively short period of time. Further examination of the mining records in the Mining Commissioner's office showed that large portion of the underlying Main Reef had been mined after 1949 following the revaluation of sterling. It seemed likely, therefore, that most of

FIGURE 5/37 *A view of Sunlight House taken from the east in Heidelberg Road : differential settlement shown by horizontal line*

(Photo by Watermeyer, Legge, Piesold & Uhlmann) the movement resulted from stope closure triggered off by this subsequent reclamation mining. Nevertheless it was puzzling that differential settlement of so large an order should have been experienced at surface in view of the relatively great depth of undermining. Furthermore, it was puzzling that other buildings in the vicinity did not exhibit structural damage to anything like the same extent.

Further elucidation of the phenomenon has recently come to light in the geolocial cross-section made available to the writer by Mr L.J. Blackman, Mine Surveyor of Village Main Gold Mining Company (1934) Limited, and reproduced in Figure 5/22. It will be seen that all three of the underlying stopes are displaced by an oblique fault occupied by a narrow dyke and situated immediately below Sunlight House. In the words of W.G. Pyne-Mercier (1970), former Government Mining Engineer:

> 'When mining occurs at greater depths or when reclamation on a large scale is undertaken the presence of dykes and faults causes instability of the surface to an extent which is difficult to assess and for which it is equally difficult to make adequate provision when buildings are erected.'

CASE HISTORY II

MAIN-BIRD SERIES

SUDDEN SUBSIDENCE OF HANGING WALL ON STEEPLY DIPPING OUTCROP: NOURSE MINES LIMITED

At 7.30 a.m. on Sunday morning 3rd May 1942, the ground surface over an area of nearly half a hectare immediately south of the South Reef oucrop workings near the eastern boundary of Nourse Mines Limited suddenly subsided by an amount of two metres. The Underground Manager's house (house A in Figure 5/38) was situated near the middle of the area that subsided.

The area of subsidence was bounded on the northern side by the outcrop workings of the Main Reef Leader and the South Reef and on the eastern and western sides by dykes. On the southern sides a 'line of fracture' appeared as a stepped displacement in the surface, as may be seen from Figure 5/39. Farther south from the line of fracture a number of tension cracks appeared in the ground, up to half a metre wide, and the width of these cracks diminished farther southwards.

According to the Inspector of Mines who conducted the enquiry, structures in the subsided area suffered remarkably little damage and the surface showed hardly any disturbance, except in the immediate vicinity of the line of fracture. Very tall fir trees retained their normal erect position, small brick outbuildings showed no cracking, and lawns and gardens remained virtually undisturbed. Even a concrete fish pond in the subsided area did not crack. However, the house within the subsided area (house A) and the two houses immediately south of the line of fracture (houses B and C) suffered substantial damage. In house A the pannelling of a brick-lined timber and corrugated iron extension had some large cracks. Windows in the southern wall of the framework were badly distorted as a result of uneven subsidence at that point. The brick walls of the northern portion of the building showed only minor cracks. Houses B and C and their outbuildings, south of the line of fracture, were badly cracked. The three houses had to be evacuated and demolished.

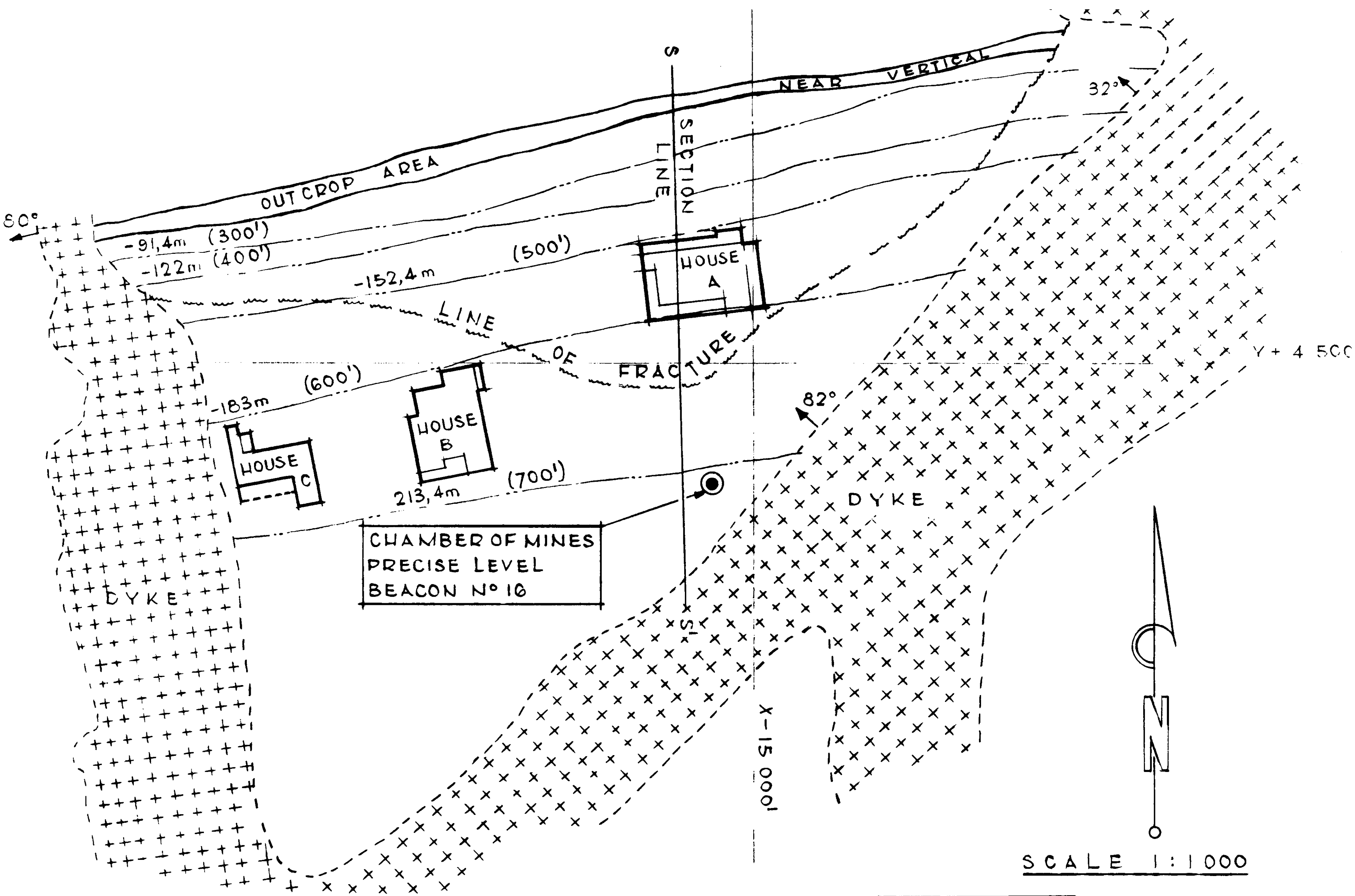


FIGURE 5/38 : *Plan of area south of reef outcrops at Nourse Mines showing extent of surface subsidence*

FIGURE 5/39 : *'House B' at Nourse Mines photographed from the north after the subsidence of 3rd May 1942 showing the line of fracture*

As shown in Figure 5/40, the reefs have an excessively steep outcrop dip of 85^0 to the south. They start to flatten out only below the No 6 Level at a depth of 183 m below surface. Apart from some drive-pillars and a few stope-pillars the South Reef and the Main Reef Leader had been entirely mined out by the year 1905, i.e. more than 35 years prior to the subsidence. The stoping width on each of these reefs was about one metre and the exceptionally narrow middling between them varied from 0,6 to 1,5 m. Apart from the few pillars mentioned above the only supports consisted of untreated timber props.

The subsidence resulted in complete closure of the underground workings between the two boundary dykes down to No 3 Level, and partial closure from No 3 to No 7 Level. The only work still in progress in the vicinity at the time of the subsidence was the development of a cross-cut on No 3 Level to the South South Reef. Fortunately the subsidence took place on a Sunday morning otherwise the miners engaged on this development work would have been trapped. Fortunately too, the line of fracture at the surface did not pass through any of the houses, as this would inevitably have resulted in at least partial collapse of any building which was so situated with possible tragic consequences to the occupants.

It will be seen from Figure 5/40 that the eastern dyke dipped steeply towards the workings and would have been intersected along much of the length of the workings at about No 6 Level, i.e. at the depth where the reefs started flattening out. The subsidence was thus clearly due to a simple slope failure under the weight of the largely unsupported hanging-wall wedge over the entire depth of the very steeply inclined stope. Shear failure would presumably have taken place along the 'assumed plane of fracture' between the dyke at the bottom and the line of fracture at the top (see Figure 5/40). In view of the fact that the surface subsided by an amount equal to the combined stoping widths of the two reefs, it is unlikely that any further subsidence could be expected. This conclusion is substantiated by two independent sets of precise level records in the affected area. Maximum settlement of Chamber of Mines precise level beacon No 16 (see Figure 5/38) between 1969 and 1974 was only 6,9 mm. Maximum settlement of CED precise level beacon N2 between January 1973 and July 1974 was only 2 mm (see Figure 5/41).

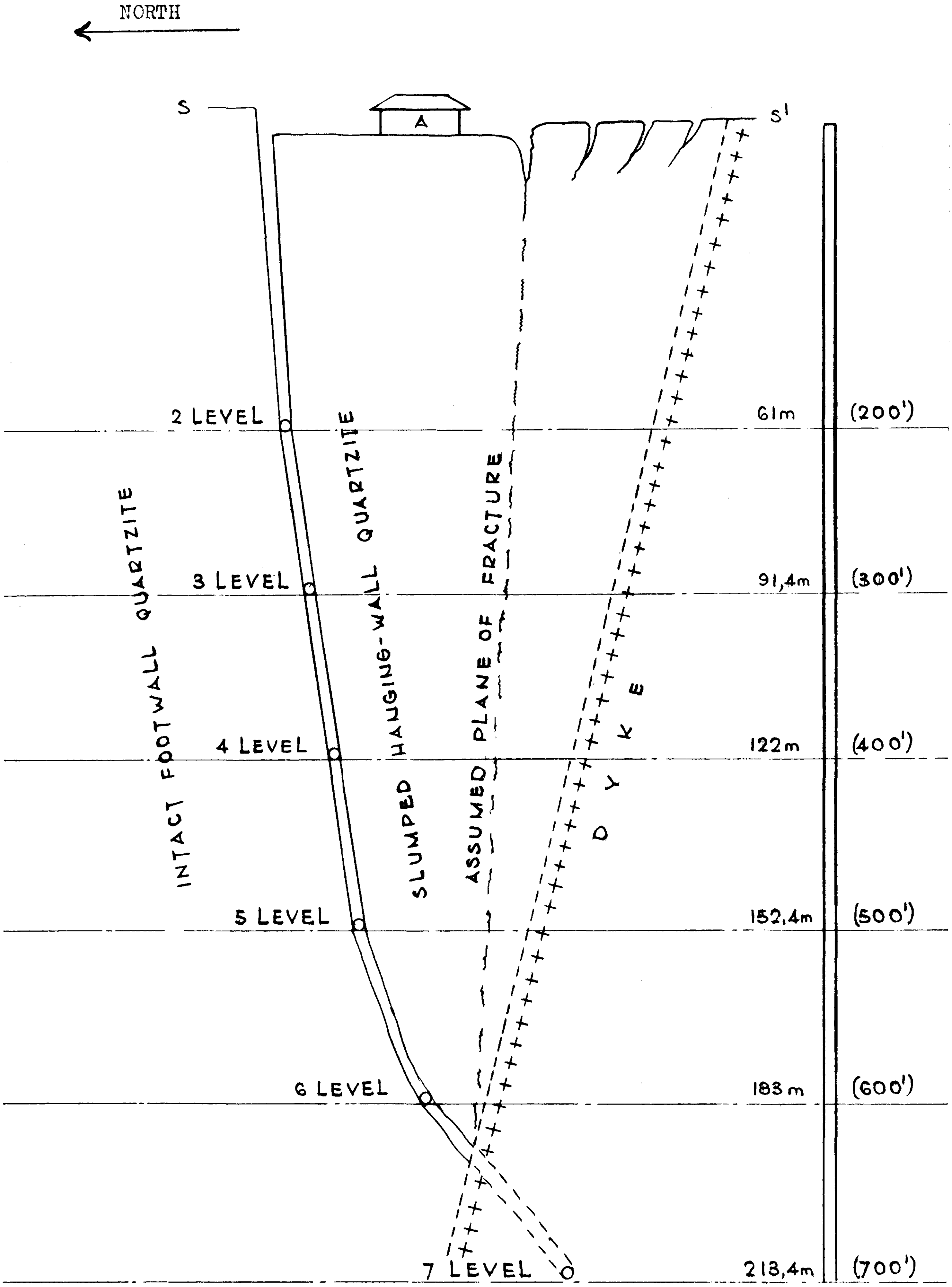
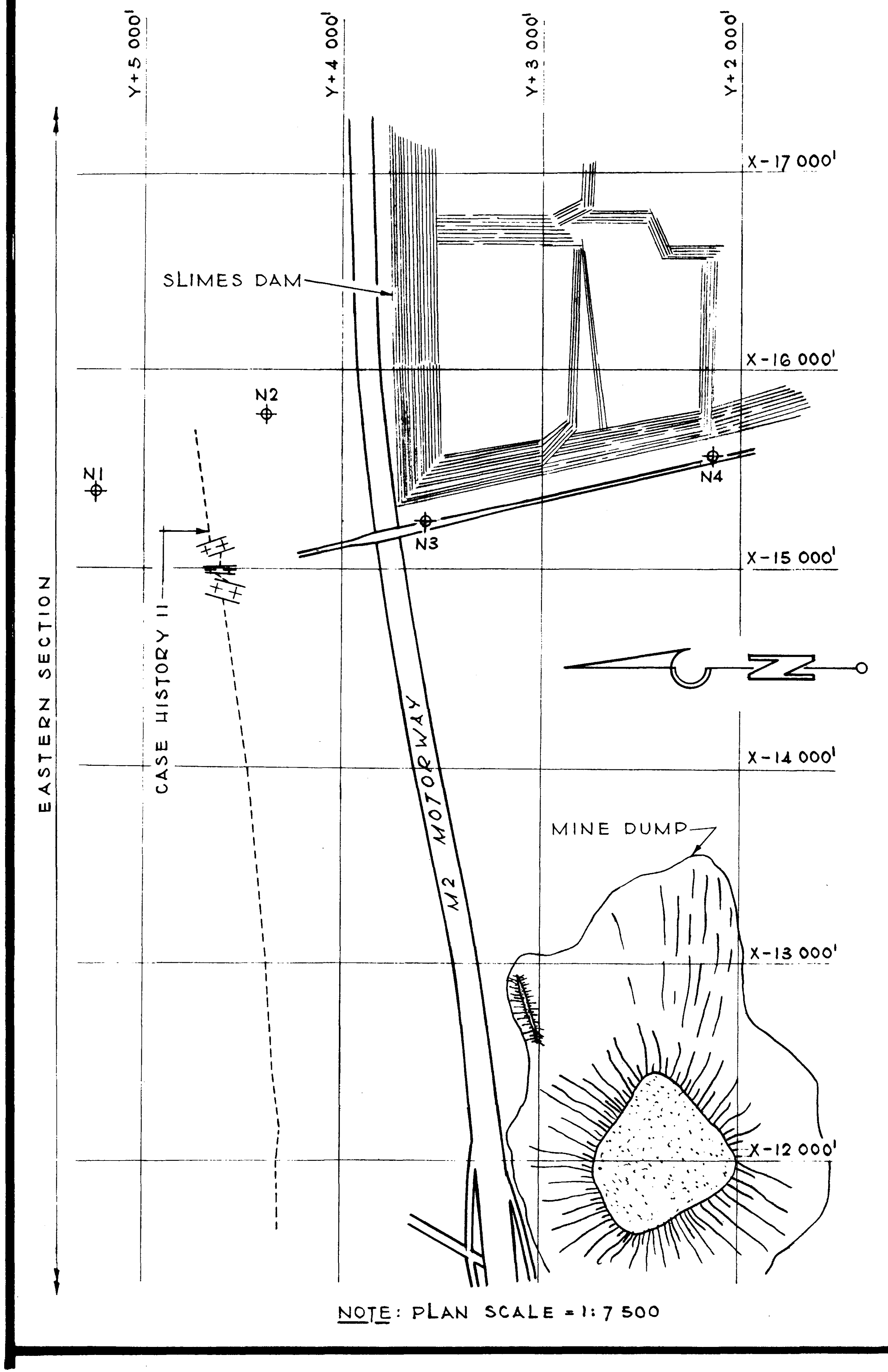
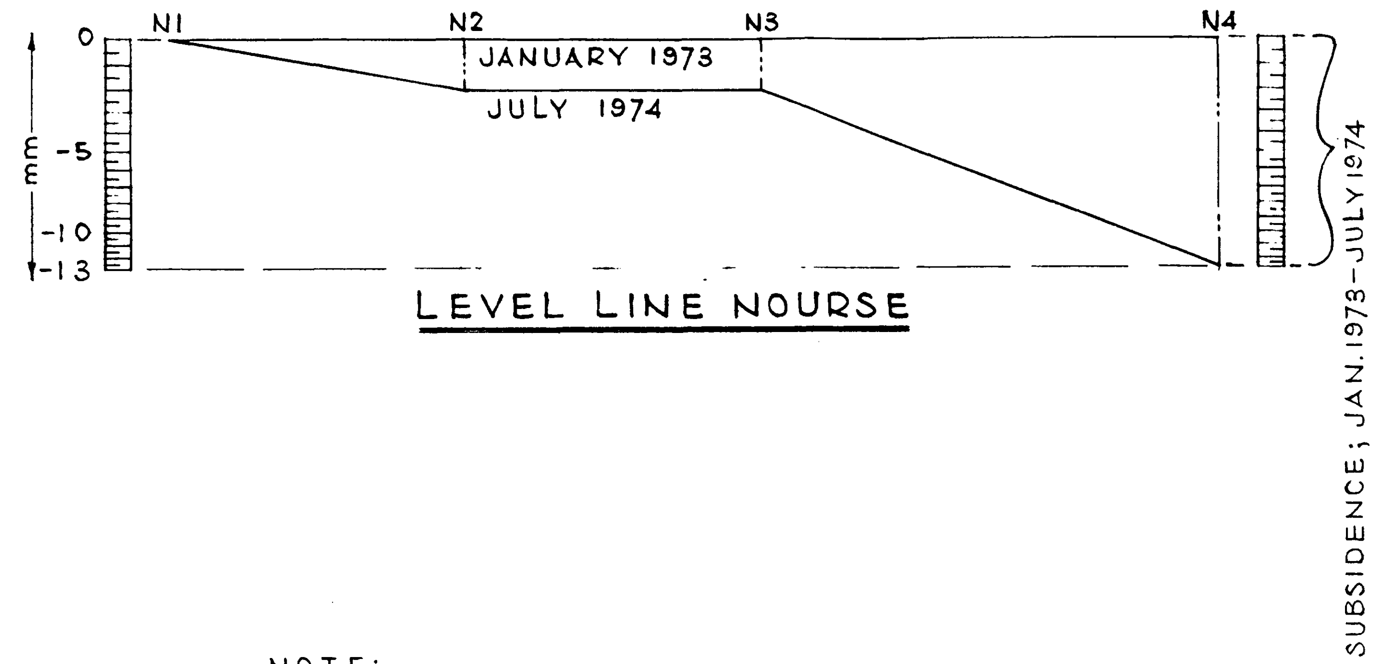


FIGURE 5/40 : Cross-section through subsided area at Nourse Mines





LEVEL LINE NOURSE

NOTE:
HORIZ. SCALE = 1:7 500
VERT. = LOG SCALE (SEE LEVEL DIAGRAM.)

LEGEND
MAIN REEF OUTCROP.
DYKE.

FIGURE 5/41 : Time/settlement curves for surface pegs on 'Level Line Nourse' in the vicinity of Nourse Mines

CASE HISTORY I2

MAIN-BIRD SERIES

MINING SUBSIDENCE ADJACENT TO DYKE: HOUSES IN CLEVELAND, JOHANNESBURG

Two mining houses were built many years ago on the eastern side of Cleveland Road, some 75 m south of the reef outcrop area. As shown in Figure 5/42, house A was built on a dyke which had been left intact, while house B, about 30 m away, was built on undermined ground. According to records at the Mining Commissioner's office the houses were originally built at about the same horizontal elevation. Figure 5/43, a photograph taken in 1969, shows that by that time house B had subsided some 2 metres relative to house A.

It would seem that settlement must have been reasonably uniform as there are few signs of cracking in house B, even at the present time. However, outbuilidngs which were nearer the dyke had to be rebuilt. Surface depressions on both sides of the dyke appear to become less pronounced with increasing distance from the outcrop area, indicating that subsidence diminishes as the depth to the underlying mine workings increases.

Three reefs were mined on either side of the dyke: the Main Reef, Main Reef Leader and South South Reef. Stoping widths were 1,8 m, 1,2 m and 1,2 m respectively. The depth to the shallowest stope under house B, that of the South South Reef, is approximately 90 m while that to the deepest stope, of the Main Reef, is 120 m.

EARTH-TREMORS

Reference has already been made to the occurrence of earth-tremors associated with mining activity on the Central Rand.

As far as can be ascertained the first earth-tremor was felt in Johannesburg in 1894, only eight years after the start of gold mining. By 1908,

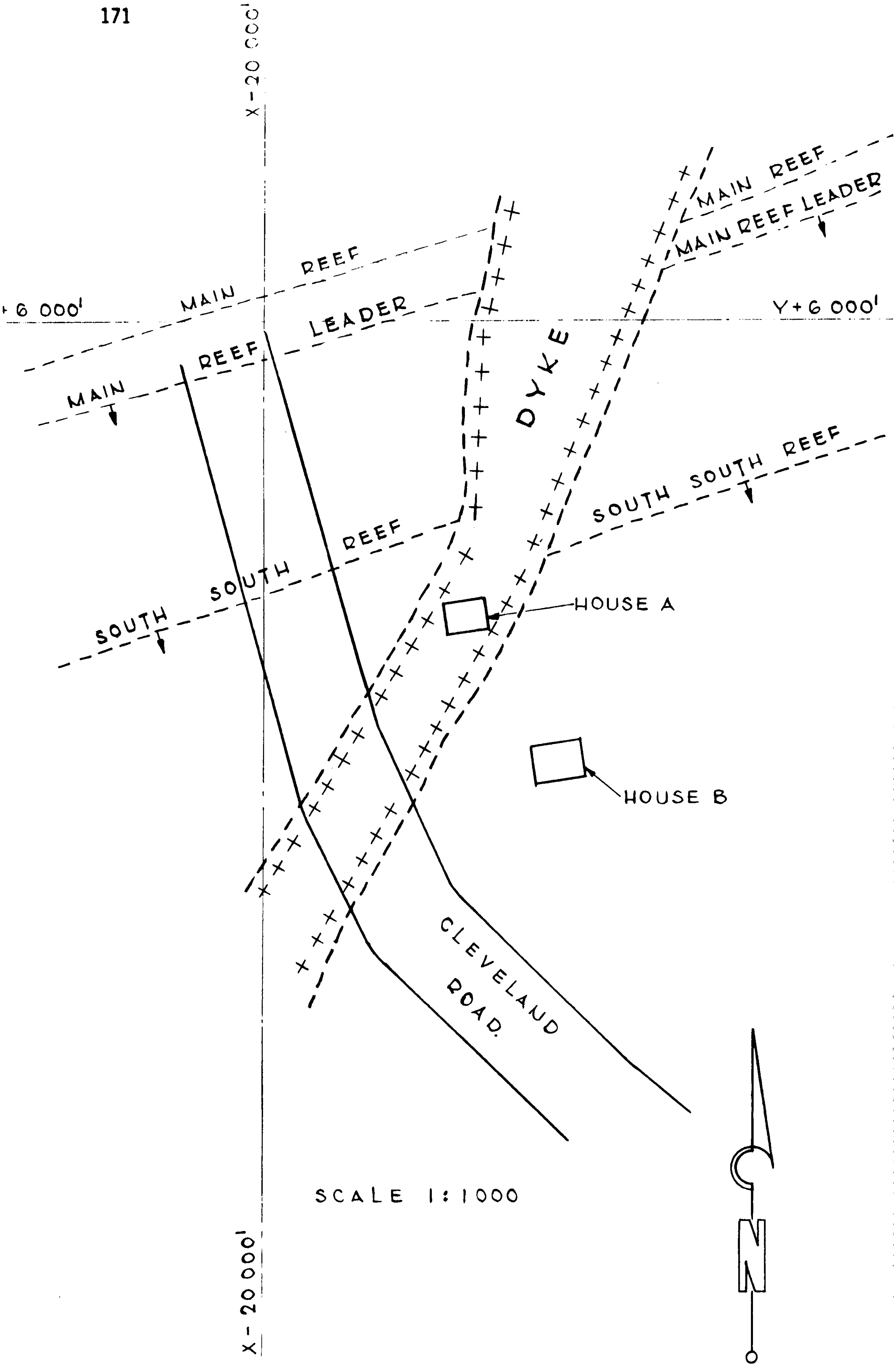


FIGURE 5/42 : Plan of area immediately south of reef outcrops in Cleveland, Johannesburg

House A on intact dyke and house B on undermined ground.

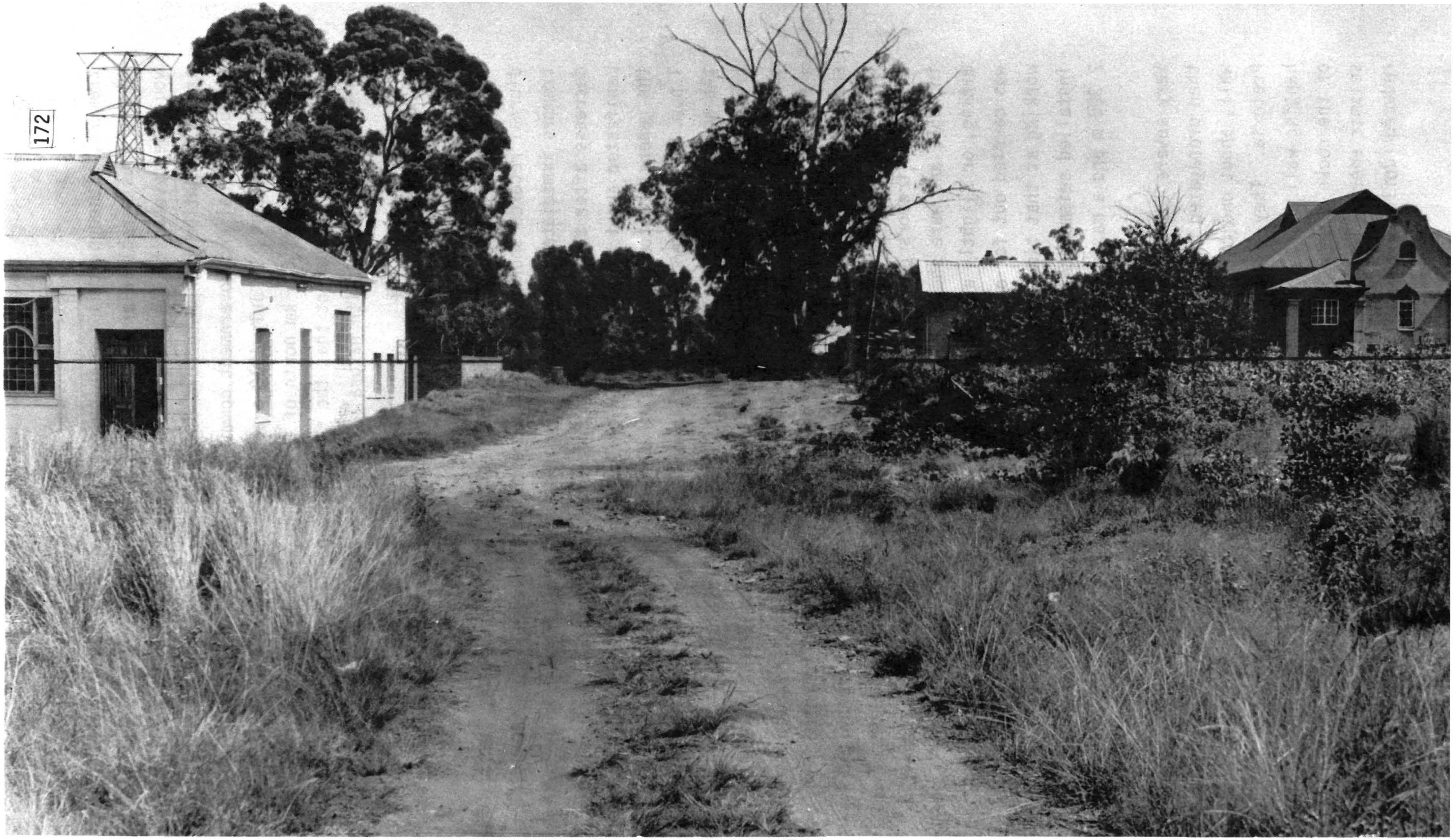


FIGURE 5/43 : Two old mine houses in Cleveland, Johannesburg : house 'A' on the right is situated on the dyke while house 'B' on the left is just off the dyke, the dyke running behind the house ; the amount of subsidence can be judged by the difference in ground level between the two houses, house 'A' being situated at original ground level as indicated by horizontal line.

(Photo 1969 by Watermeyer, Legge, Piesold & Uhlmann) tremors, although still few in number, were of sufficient intensity to awaken considerable alarm and several columns of the local newspaper would be devoted to the report of a new shock. It was feared that the tremors might be the precursors of an imminent catastropic earthquake, and both local and government committees were commissioned to investigate the matter. In July 1910 a seismograph was installed at the Union Observatory about 6,5 km north of the mining-area in Johannesburg, and it soon became apparent that the tremors emanated from the mines and were not associated with natural seismic instability (Wood, 1914).

A statistical analysis of seismographic records of the Union Observatory published in 1939 proved conclusively that blasting in the mines was by far the most effective *trigger* in initiating a tremor (Gane, 1939). The diurnal records showed a sudden increase in the relative frequency of tremors immediately after 2 p.m. on weekdays, and a roughly exponential decrease after that time. The increase was absent on Sundays and was manifested about an hour earlier on Saturdays. This was explained by the general custom of the mines to blast at about 2 p.m. on weekdays and 1 p.m. on Saturdays: yet the seismograph was too far from the mines for the actual blasts themselves to be felt by the instrument. The recorded tremors were interpreted as seismic waves resulting from the release of accumulated stresses in the rock. The stresses build up in the rock as a result of the removal of material by mining, and are released in shear-failure in the form of instantaneous brittle fracture by the trigger action of blasting (Gane *et al*, 1946). The focal depths from which the tremors emanated were studied by the Bernard Price Institute in 1950, and were shown to be in close proximity ~~to~~ of the level of active mining, or slightly above this level (Gane *et al*, 1952). This study was carried out for one particular mine, where the greatest depth of working at that time was 2 750 m, and the records of six specially installed seismometer stations indicated a mean depth of focus of 2 300 m and a maximum of 2 900 m.

Very severe tremors are felt on the mines themselves and the waves are then propagated mainly northwards through the solid ground of the footwall, about one third of the tremors being recorded as far north as Pretoria. They are propagated particularly freely along dykes. On the hanging-wall side, south of the outcrop, the generally fractured nature of the rocks appears to be responsible for dissipation of much of the seismic energy, and the intensity of the shocks is usually considerably reduced (Joffe, 1957).

The number and the intensity of tremors in Johannesburg increased at a slow but relatively steady rate after the inception of mining activities. It was generally believed that after 1950 they decreased both in number and in intensity and this was ascribed to the decrease in mining activity immediately south of the city. A more recent analysis of recorded seismic events at the Republic Observatory conducted by Grobbelaar (1970) reveals, however, that they reached a maximum in 1953, and only decreased after 1965, as shown in Figure 5/44. It is also evident from Figure 5/44 that the decrease in seismic shocks is not proportional to the decrease in the tonnage of ore extracted. The dots in Figure 5/44 represent the total production of ore at the nine mines on the Central Rand, from the Witwatersrand Gold Mining Company in the east to the Durban Roodepoort Deep Limited mine in the west. From 1950 to 1968 the tonnage decreased from 16 741 000 to 5 761 000, i.e. a reduction of 65 per cent. Seismic events decreased from 4 504 in 1953 to 2 851 in 1968, i.e. a reduction of 27 per cent. This would suggest that seismic activity may continue on the Central Rand even after the final cessation of mining activity.

Tremors are now accepted by local residents as a matter of course, and it is only visitors to the area who find them particularly startling. Masonry buildings near the mining area suffer damage, and it is accepted as inevitable that crakcs will reopen and chimneys will topple down after repairs have been effected.

Reference was made earlier to the extraction of the Turf Shaft pillar at Robinson Deep during 1957 to 1959, and to the subsidence of the surface caused by this operation. While the subsidence itself caused no damage to structures at surface, a 'very very strong' earth-tremor, which was attributed to the extraction of the pillar, was recorded on 30th May 1959. Severe damage was suffered by buildings on either side of the entire length of Eloff Street Extension in a belt bounded on the east by the Grahamstown Dyke and on the west by another dyke. Damage included the breaking of windows, buckling of steel frameworks, diagonal and 'X-cracking' in brickwork panels of reinforced concrete frame structures and, in some cases, collapse of such panelling. The frameworks of concrete structures, however, remained intact (Pyne-Mercier, 1970).

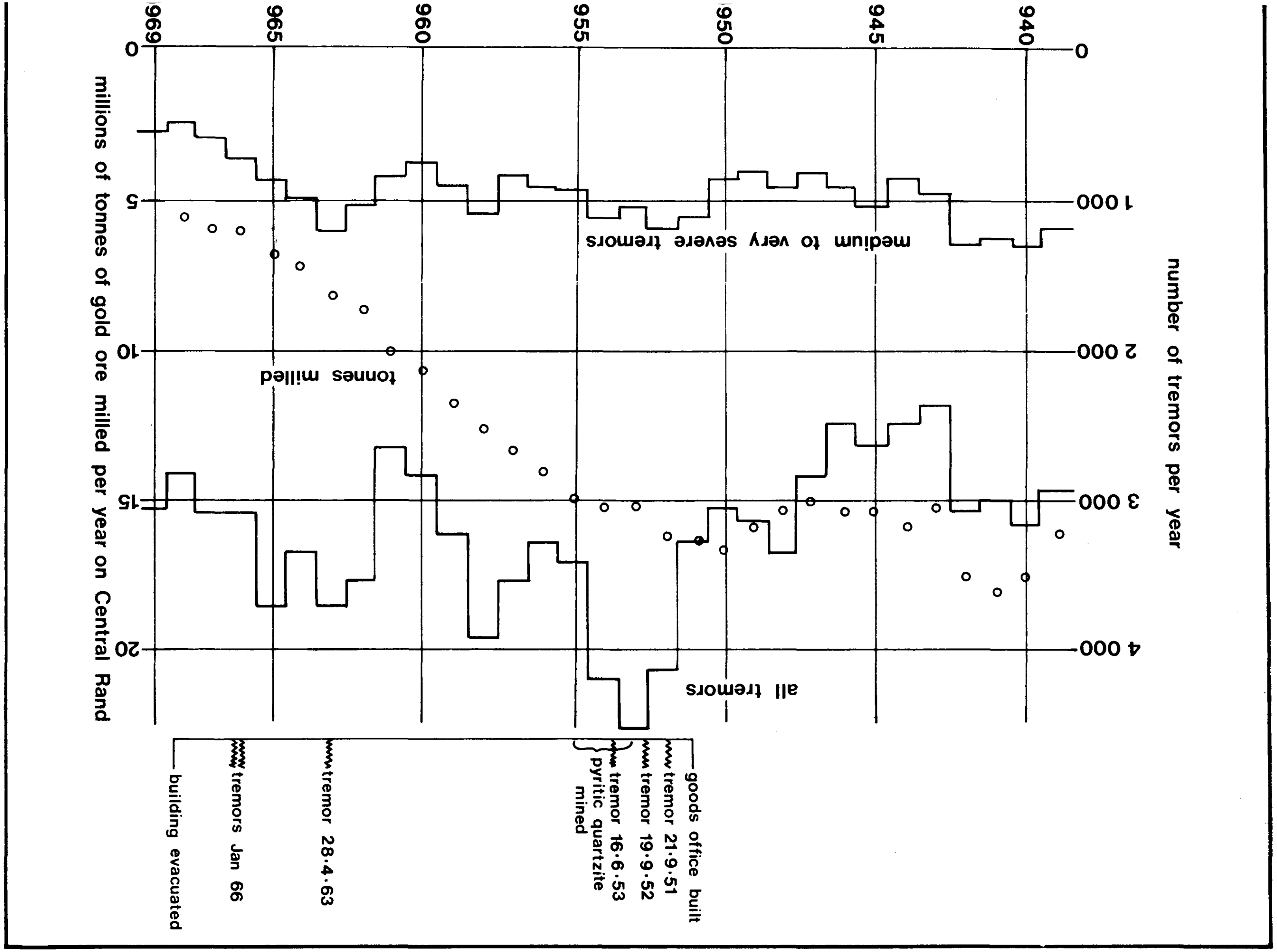


FIGURE 5/44 : *Seismic events in Johannesburg from 1940 to 1969 in relation to tonnage of ore milled on the Central Rand and summarising the history of the Kazerne Goods Office*

The only concrete-framed structure to have suffered severe damage from tremors is the South African Railways Goods Office at Kazerne, Johannesburg (see Case History 13). Nevertheless the question has been raised as to whether the exceptionally tall or 'highrise' buildings at present being constructed or proposed in Johannesburg will be more susceptible to damage by earth-tremors than existing buildings. This question has been examined by Ockleston (1968), who has observed that the main characteristics of Johannesburg tremors are as follows:

1. The tremors are essentially transverse seismic waves: particles in their path may oscillate in any direction normal to their direction of travel.

2. In the centre of the city the horizontal component of ground movement predominates over the vertical component for most tremors and, since buildings are more vulnerable to horizontal than to vertical loading, the relevant vibrational characteristics are those relating to horizontal oscillation.

3. Frequency of the ground motion due to tremors varies from 5 to 25 cycles/second but is usually between 10 and 20 cycles/second.

4. In the central city area the maximum amplitude of the ground oscillation due to tremors usually varies from 0,001 mm to 0,1 mm. Only one or two of the several hundred tremors that are experienced each year in Johannesburg have amplitudes of 0,1 mm or more, the maximum ever recorded being 0,3 mm.

5. The tremors are of short duration, seldom lasting much longer than a second. Typically the amplitude of the ground oscillation builds up to a maximum during the first cycle or two and then decays logarithmically.

6. Ground oscillation due to tremors is of higher frequency, smaller amplitude and shorter duration than is the case with deep-seated tectonic earthquakes.

From analogue computer studies Ockleston (1968) shows that a highrise building would be expected to suffer a maximum deflection of about 2,4 times the maximum ground displacement. Since the greatest amplitude of ground movement ever recorded due to a Johannesburg tremor is 0,3 mm, the deflections of a highrise building would be less than 1 mm. As an extreme case he also considered, instead of a rapidly decaying ground oscillation, a steady simple harmonic motion of amplitude equal to the maximum ground displacement. He found that for this case the maximum deflection of a tall building would be just under 2 mm. He considered that maximum deflections of 1 or 2 mm would have a negligible effect on the stability of a highrise building, and that earth-tremors in Johannesburg should therefore not give rise to serious design problems.

CASE HISTORY I3

MAIN-BIRD SERIES

EFFECT OF EARTH TREMORS ON GOODS OFFICE BUILDING, SOUTH AFRICAN RAILWAYS, KAZERNE, JOHANNESBURG

During 1950-1951 a large four storey building with basement was erected at the railway goods depot at Kazerne. It consists of a concrete-framed structure with brick pannelling. The building was opened in August 1951, and put into use as the Goods Office Administrative Block.

The following month the brick panelling suffered considerable damage during a severe earth-tremor. Repeated eposodes of this nature during the ensuing 17 years resulted in the building being finally evacuated and put out of use at the end of May 1968, though it has not been demolished.

It is of interest to note the progressive deterioration in the structure over these years, and the following account records the nature of the damage suffered as a result of some of the more intense tremors.

Tremor on 21st September, 1951:

Cracks up to 30 mm wide appeared in the outside face-brick walls and the inside stock-brick pannelling. The concrete frame remained undamaged (Jackson and Meijer, 1954). On 23rd September the Sunday Express reported: 'Not a single office in the R350 000 building escaped damage in the severe earth-tremor that shook Johannesburg on Friday evening.' The cracks were subsequently repaired.

Tremor on 19th September, 1952:

Numerous cracks appeared in the stock-brick panelling forming the inside walls. A section of face-brick panelling on one of the outside walls was displaced horizontally causing it to jut out about 50 mm beyond a corner of the building. Portion of another outside wall was similarly displaced about 25 mm. Minor damage was sustained by the parapet wall above the main roof level. The concrete frame was apparently not damaged. As the loosened brickwork of the cavity walls was a hazard and might easily have been dislodged by a further tremor, it was stripped off and the affected corners were rebuilt in solid, non-cavity brickwork (Joffe, 1957).

Tremor on 16th June, 1953:

Small plaster cracks appeared and a few of the old cracks reopened. No repairs were done.

Tremor on 28th April, 1963:

In addition to further damage to brickwork, beams and floor slabs of reinforced concrete were cracked. Columns remained undamaged. On the ground floor, cracks in the beams and corresponding cracks in the slabs appeared mainly at mid-span and near columns: this pattern was repeated on the first and second floors and to a lesser extent on the top floor. The parapet wall on the roof was very badly damaged. Much of the brickwork, including the parapet wall, was broken down and rebuilt. With repairs completed it was considered that the building was in no way unsafe for further usage.

Tremor on 16th January, 1966:

Widespread cracking appeared throughout the building. The worst damage took place in the upper floors, where distortions suffered by the lift motor-house put the lifts out of action. Further cracks opened in reinforced concrete members, and some cracks in the interior brickwork on the top floor were as much as 75 mm wide. Fortunately this tremor took place at about 1 a.m. on a Sunday morning, when nobody was in the building. The perimeter brick wall of an adjacent site fell down.

Tremor on 20th January, 1966:

The lift house was damaged further, and plaster and brickwork were dislodged, mainly in the entrance hall on the ground floor. The computer housed in the building was put out of action.

By this time the personnel occupying the building were understandably somewhat panic-stricken and would run outside every time there was even a minor tremor. It was feared that 'tremor happy' people might be hurt in stampeding down the stairways and it was decided to abandon the top two floors of the building. On 31st May, 1968, the remaining floors were evacuated and the building was put out of use entirely.

The history of seismic events in the area related to the development of cracks in the building is summarised in Figure 5/44.

Geology of the site

The Kazerne Goods Depot is underlain by the Main-Bird quartzites which dip southwards at about 30^0. Three auriferous horizons are present in the quartzites at depths 455 m to 485 m below the Goods Office as shown in Figure 5/24. The upper two of these, the South Reef and the Main Reef Leader, were completely mined out more than 50 years ago, each leaving a stope width of about 1,5 m. The third horizon, the Pyritic Quartzites, was mined locally during the years 1953-55, leaving an additional stope width of 4 m directly beneath the northern and central portions of the building.

An investigation carried out in 1968 in the vicinity of the abandoned building (Wiid, 1968) has revealed that the Main-Bird quartzites are locally overlain by a variable thickness of residual soil formed by advanced decomposition of either a thin diabase or a remnant of Bird Amygdaloidal Lava. It is in this residual soil that the building is founded. The residual soil is exposed in road and subway cuttings in the vicinity, and was encountered in trial-holes augered at each corner of the building and in two trial-holes dug through the basement floor of the building. It consists of a stiff to very stiff silty clay, estimated to have a safe bearing capacity of 400 to 500 kPa at a depth of 3,5 m to 4 m below surface level, which was the depth at which the spot footings of the building had been founded. As the maximum bearing pressure imparted to these footings by the column loads is 200 kPa it is clear that the damage sustained by the building is not related to overloading of the soil.

In order to ascertain whether the damage sustained by the building could have been associated with differential settlement as a result of mining subsidence, a computer programme was devised to simulate the underlying mining configuration as from 1951. In creating this model the following assumption were made:

(i) That elastic deformation alone would take place in the hanging-wall strata; and
(ii) that the hanging-wall strata would behave as an intact and homogeneous mass.

The analysis showed the differential settlement across the building site after 1951 to be of negligible proportions. This finding was confirmed by a precise levelling survey within the building itself.

An unusual feature of the cracking pattern observed in the building was the presence of X-cracks within brick panels confined between beams and columns. A typical example of such cracking is shown in Figure 5/45. This type of cracking is a characteristic feature of earthquake damage in buildings. For comparison with Figure 5/45, an example of identical X-cracking in a building panel which resulted from the severe earthquake of 27th March, 1964, in Anchorage, Alaska, is shown in Figure 5/46. This unique evidence is the final proof that the damage suffered by the Kazerne Goods Office was produced by seismic shocks.

Figure 5/47 shows the situation of the building in relation to the Village Main step-faults and the diabase dykes encountered in the underground workings, and it is this situation, coupled with the presence of at least 25 m of residual soil below founding level, that is thought to have been responsible for the particular susceptibility of the building to tremor damage. It is known that seismic waves are propagated particularly well along dykes and faults and that their effect in causing structural damage is particularly pronounced in buildings founded in deep soil. A further local factor is the relatively shallow depth of origin of the tremors most severely felt at Kazerne.

Of paramount significance is the fact that along length of 18 haulage of the Village Main Mine, situated directly beneath the building, was observed to have collapsed at the time of the severe tremor at 1 a.m. on 16th January 1966: the south wall of the haulage appeared to have moved northwards against the north wall.

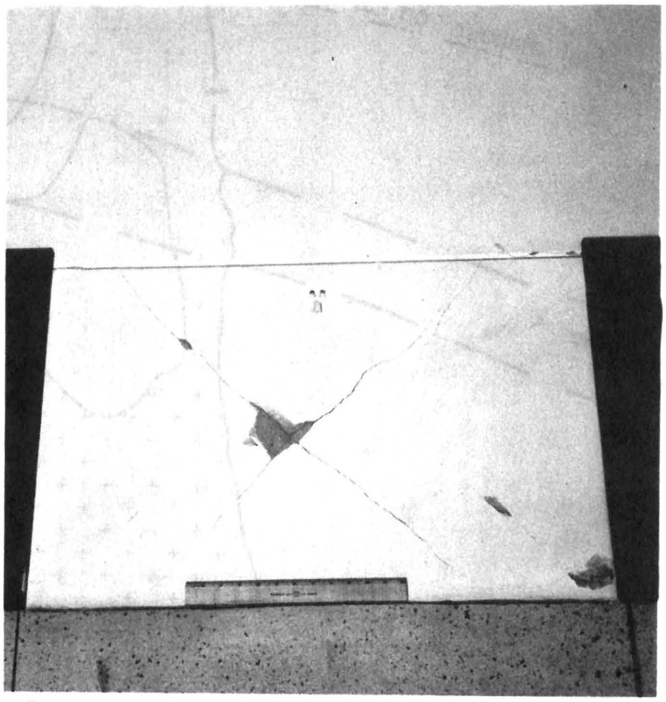


FIGURE 5/45 : Typical example of X-cracking in brick panel within beam and column frame ; Kazerne Goods, Office, Johannesburg

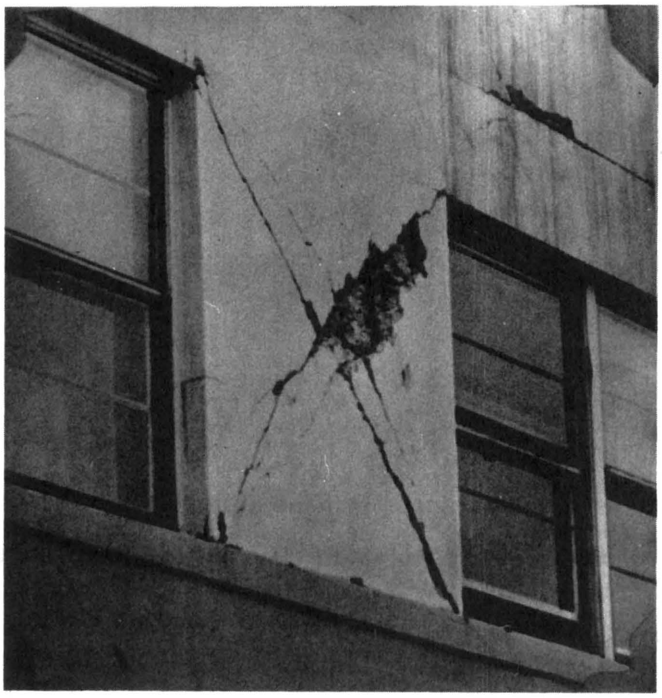
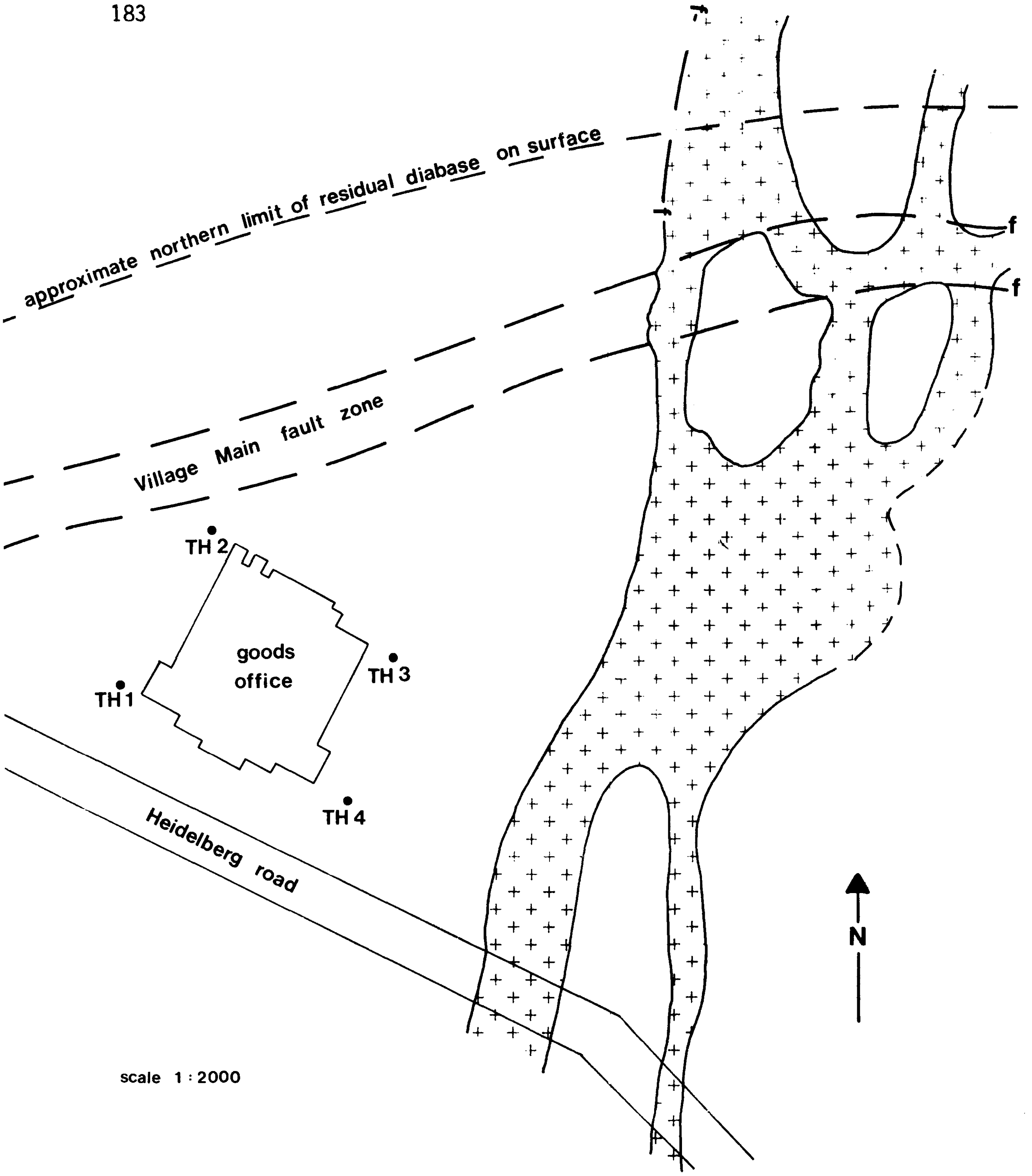


FIGURE 5/46 : Typical example of X-cracking induced by seismic waves in panel within beam and column frame : building in Anchorage, after earthquake of 27.3.1964



f- —— -f faults

+ + + diabase dykes as encountered in mine workings

Trial-hole 1 : Main-Bird quartzite encountered below residual diabase soil at depth 25m

Trial-holes 2 & 4 : residual diabase to 20m

Trial-hole 3 : residual diabase to 30m

FIGURE 5/47 : Situation of Kazerne Goods Office, Johannesburg, in relation to dykes and Village Main step-faults

6 VENTERSDORP SYSTEM

As shown in Figure 6/1, outcrops of the Ventersdorp System within the Republic of South Africa fall into three distinct climatic zones which may conveniently be referred to as semi-arid, sub-humid dry and sub-humid moist (Schulze, 1958). As a result of the different weathering and soil-forming processes operating in these zones, three distinct, though gradational, types of soil profile have developed on the Ventersdorp lavas*: these in turn gives rise to three distinct types of foundation conditions, characterised by shallow rock founding, heaving and high compressibility, respectively.

SEMI-ARID ZONE

Annual rainfall < 500 mm
Thornthwaite moisture index -40 to -20
Weinert's climatic N-value 5 to 10

Sound lava outcrops over much of the area occupied by this zone. Where residual soils are developed they are seldom more than about 2,5 m deep, are often densely cemented by calcrete and usually contain large boulder-like weathering spheroids of hard lava. Adequate founding is thus available for most engineering structures at shallow depth, if not at surface.

Problems only arise where structures with abnormally high loadings are concerned. As this zone forms part of the Mealie Belt the need has arisen for the construction of grain silos which, owing to uneven loading when only one row of bins is full coupled with toe pressures caused by wind

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* This is, of course, a very broad generalisation. According to the National Data Bank for Roads, CSIR, eight distinct land patterns have been defined for South African occurrences of the Ventersdorp System bearing the GeoCEP Indices: Ventersdorp Lava 211, 212, 214 222, 233, 262, 333 and 392. There are also two further definitions for occurrences outside of the Republic namely Ventersdorp Lava 423 and Ventersdorp Shale 352.

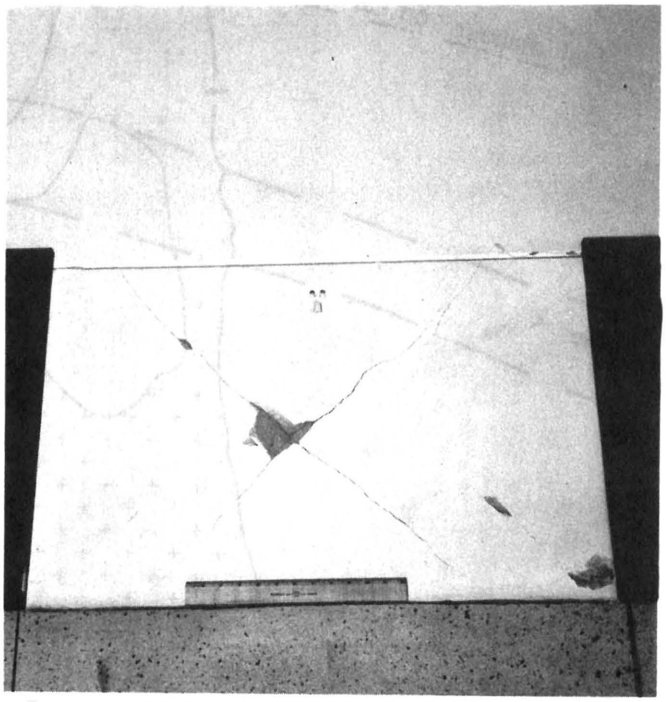


FIGURE 5/45 : *Typical example of X-cracking in brick panel within beam and column frame ; Kazerne Goods, Office, Johannesburg*

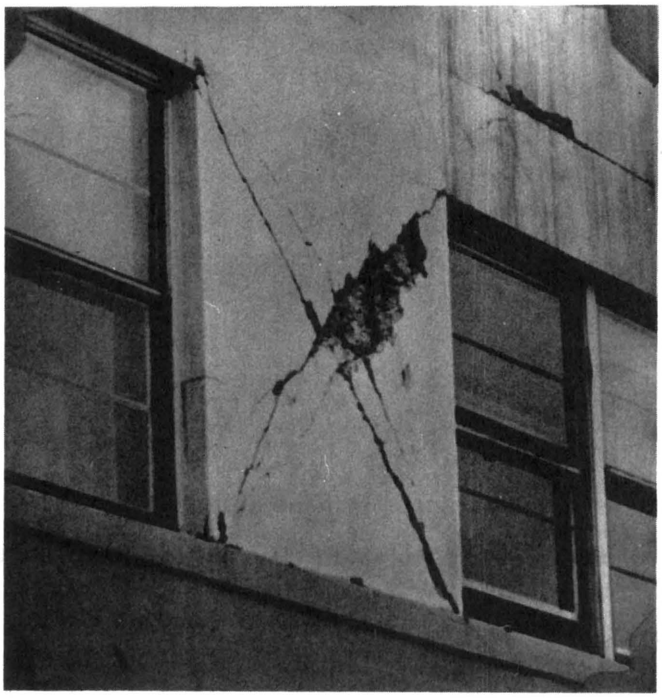


FIGURE 5/46 : *Typical example of X-cracking induced by seismic waves in panel within beam and column frame : building in Anchorage, after earthquake of 27.3.1964* loads, often require bearing capacities of the order of 750 to 1 000 kPa for spot footings or 550 kPa for raft foundations. While the sound lava itself is capable of supporting loads far higher than these, account has to be taken of relatively soft pyroclastic materials, in the form of tuffs, breccias and agglomerates, which are often present as large lenses and pockets within the lavas. The delineation of these pockets can only be achieved by a closely spaced grid of diamond-drill holes, preferably located in accordance with the finding of appropriate geophysical surveys.

This problem was first encountered during construction of a silo complex in Delareyville in 1959. A site investigation had been carried out by the writer during 1957: nine diamond-drill holes had been put down on the 120 m x 35 m site, and all had encountered hard lava below the proposed founding level of 4,5 m. When the excavation had been opened up to this depth, however, a patch of relatively soft pyroclastic material was found to occupy an area of about 340 square metres on the floor of the bulk excavation. The presence of this soft patch, which extended to a depth of 7 metres below surface, had not been revealed during the site investigation: none of the exploratory boreholes had been located within it. However, it may well have been detected had a seismic or electrical resistivity survey been carried out. A volume of some 850 cubic metres of the soft material had to be excavated and replaced by mass concrete before construction could proceed, much to the writer's embarrassment.

SUB-HUMID ZONE

Annual rainfall 500 to 750 mm
Thornthwaite moisture index -20 to 0
Weinert's climatic N-value 2,5 to 5,0

Although there are some outcrops of sound lava in this zone, a blanket of residual soil is the general rule. The thickness of residual soil is variable, but rock is commonly encountered within a depth of 12 m below the surface. The upper metre or two of residual soil is red or reddish brown in colour, contains abundant ferruginous concretions, and may be densely cemented by ferricrete. The pebble marker overlying the residual soil is often found to be densely ferruginised, except in topographic situations where there is a substantial thickness of transported soil in the profile.

Below the red ferruginised zone the profile is occupied by yellow clayey silt which usually merges with depth into an olive green, somewhat coarser textured material. This colour sequence, from red through yellow to olive green, is characteristic of most well-drained residual soils developed on igneous rocks of basic or intermediate compostion in a subtropical environment. The red colour is imparted by ferric oxides such as hematite, the yellow colour by hydrated ferric oxides such as limonite, and the olive colour by chloritised minerals and possibly ferrous compounds. Broadly speaking the predominant clay-mineral in the upper horizons of the profile is kaolinite, followed at depth by montmorillonite, with chlorite and sometimes vermiculite occurring in the lower horizons. Although a gross over-simplification, a useful rule of thumb associates kaolinite with the red, montmorillonite with the yellow, and chlorite with the green horizon (Brink, 1955).

Experience at a number of places, such as Klerksdorp, Sannieshof, Alberton, Kempton Park and on the Klipriviersberg south of Johannesburg (see Figure 6/1) has shown that heaving conditions must be expected on the residual lava soil profile described above. This is illustrated by the indicator test values for a number of samples of residual soil from the Alberton municipal area given in Table 6.1, and from the proposed township of Meyersdal situated on the Klipriviersberg given in Table 6.2. Mean values of the Atterberg Limits and clay contents are given in these tables for certain prominent Land Facets within two different Land Patterns. It is of interest to note that, while there may not be a significant variation in mean clay content from one Land Facet to another, there is a marked increase in activity, and hence in potential heave, down the topographic slope from crestal areas, through sideslopes to gullies.

However, where a dense ferricrete of substantial thickness (1 to 2 m) has developed in the upper horizons of the soil profile, the effects of heaving on normal structures founded on the ferricrete will usually be minimal. Such cases often involve a difficult choice of foundation design between the alternatives of shallow founding on ferricrete and deep

LAND FACETS		CRESTAL AREAS				SIDESLOPES				GULLIES			
INDICATOR TEST:		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	CLAY CONTENT	LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	CLAY CONTENT	LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	CLAY CONTENT
		LL	PI	LS	$<2\mu m$ %	LL	PI	LS	$<2\mu m$ %	LL	PI	LS	$<2\mu m$ %
MAXIMUM	x_M	56	28	13	36	78	49	19	55	82	56	18	54
MINIMUM	x_m	35	5	5	5	31	3	4	7	36	14	9	14
MEAN	$\bar{x}$	44	17	9	22	48	23	11	21	62	37	12	32
NUMBER OF TEST DATA	n	13	13	13	13	93	93	93	93	14	14	14	14
STANDARD DEVIATION	s	7,26	6,25	2,16	7,99	10,10	9,03	2,87	7,31	12,39	13,48	2,95	13,27
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,165	0,362	0,240	0,371	0,210	0,393	0,266	0,346	0,200	0,364	0,247	0,415

TABLE 6.1 : Indicator test data for residual soils occurring within three prominent Land Facets of the Ventersdorp Lava 222 Land Pattern in Alberton

LAND FACETS		SIDESLOPES				GULLIES			
INDICATOR TEST		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	CLAY CONTENT	LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	CLAY CONTENT
		LL	PI	LS	<2μm %	LL	PI	LS	<2μm %
MAXIMUM	x_M	69	39	16	49	65	34	15	24
MINIMUM	x_m	33	6	5	6	44	14	6	19
MEAN	$\bar{x}$	49	18	10	20	56	25	11	21
STANDARD DEVIATION	s	8,64	8,26	2,79	10,10	9,49	8,81	3,88	2,63
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,178	0,452	0,277	0,503	0,169	0,355	0,361	0,124
NUMBER OF TEST DATA	n	41	41	41	41	4	4	4	4

TABLE 6.2 : Indicator test data for residual soils occurring within two prominent Land Facets of the Ventersdorp Lava 214 Land Pattern at Meyersdal on the Klipriviersberg

founding on hard lava. When high loadings demand the latter choice, the nature of the 'hard' lava may present a further problem. The lava may be jointed and present a *blocky* appearance, as is commonly the case in the Isando industrial area outside Johannesburg, where a bearing capacity of the order of 500 kPa may usually be safely assumed for piers or end-bearing piles. But in other areas the lava may be found to exhibit *spheroidal weathering*: what may appear to be solid bedrock at the depth of auger refusal, may in fact be a large spheroid of hard lava surrounded and underlain by clayey silt. Where heavy structures are concerned, or parts of structures which can tolerate little or no settlement, solid rock may first have to be proved under individual foundation bases by appropriate exploratory drilling.

SUB-HUMID MOIST ZONE

Annual rainfall > 750 mm
Thornthwaite moisture index 0 to 20
Weinert's climatic N-value 1,25 to 2,50

Only a small area of the Ventersdorp System is present in this climatic zone but, as it occupies a graben valley striking east-west through the centre of Johannesburg city (De Beer, 1965), the behaviour of its residual soils is of major concern in foundation engineering. The location of the graben (and of the lavas forming the Klipriviersberg) in relation to strata of the Witwatersrand System, is shown in the diagrammatic cross-section given in Figure 6/2. Where it passes through the central city area, the lava-filled graben varies from about ½ to 1½ km in width with the SAR reserve and Johannesburg railway station in the middle of it. Within the last few years some of the tallest and heaviest buildings in the Republic have been erected in this small but rapidly developing area: consequently considerable effort has been directed to the study of engineering properties of the soils.

The depth of decomposition is far greater here than elsewhere in the Ventersdorp lavas. From the accumulated records of over a hundred soil profiles it has been established that, while the minimum depth of refusal to a power-driven auger is 7 m, many holes may be augered to depths of 25 m or 30 m without refusing. Such an excessive depth of residual soil

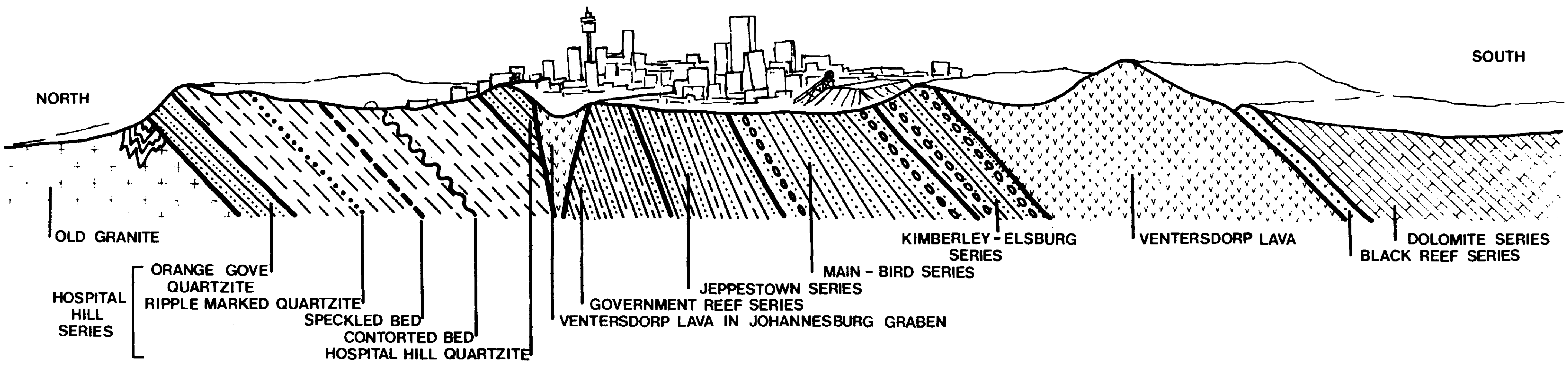


FIGURE 6/2 : Diagrammatic geological cross-section through Johannesburg showing the relationship between Ventersdorp lavas in the Johannesburg Graben and the Klipriviersberg and the strata of the Witwatersrand System cannot be accounted for on the climatic basis alone: it seems clear that the soils also occupy a favourable topographic situation within the shallow valley immediately below the continental divide, where they have been protected against erosion.

Soft red clayey silts or clay-silts extend to an average depth of 12 m and to a maximum depth of 25 m: yellow residual soils are present below this and often persist to a depth of over 30 m. The hard lavas encountered at greater depth are usually excellent carriers of ground-water, a yield of 20 000 litres per hour commonly being available at depth of 75 to 100 m; but the water-table is seldom encountered in holes of less than 25 m depth.

A well-developed pebble marker of vein quartz and quartzite gravels, varying in thickness from 100 mm to over 3 m but with an average thickness of 300 mm, overlies the residual soil. The blanket of transported hillwash soils overlying the pebble marker, and often exhibiting a collapsible grain structure, has an average thickness of about 3 to 4 m but may extend to depths of as much as 15 m along parts of the northern boundary of the graben. Ferruginisation of the upper horizons has only been observed in about ten per cent of the recorded soil profiles, so that shallow founding is out of the question on the vast majority of building sites.

On many of the sounder geological formations throughout the country the pebble marker serves as a convenient indication of the presence of adequate founding materials immediately below it. This is because residual soils, on the whole, possess a higher bearing capacity than transported soils. One of the notable exceptions to this general rule is provided by the residual Ventersdorp lavas in Johannesburg. As far as present knowledge extends, it would appear that they are the most highly compressible and most rapidly consolidating residual soils on the Highveld, with the exception of residual dolomite in the form of 'wad'. Table 6.3 gives a summary of a number of test data on these soils. It will be seen that both the Compression Index and the Coefficient of Consolidation are excessively high, and that for most structures it is from these, rather than from the strength characteristics, that safe founding pressures must be determined. Further, comparison of mean values for the red and yellow residual soils indicates the more highly compressible nature of the former, and confirms that these soils owe their unusual characteristics to their advanced degree of decomposition and leaching.

ENGINEERING PARAMETERS		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	PERCENTAGE CLAY	PERCENTAGE SILT	DEGREE OF SATURATION	APPARENT COHESION kPa	ANGLE OF SHEARING RESISTANCE	SPECIFIC GRAVITY	BULK DENSITY kg/m^3	DRY DENSITY kg/m^3
		LL	PI	LS	%<2µm	%<60µm >2µm	S_r	Cuu kPa	$\emptyset_{uu}$	G_s	γ kg/m^3	γD kg/m^3
MAXIMUM	x_M	76	36	15	24	88	83	207	26^0	2,93	1 762	1 153
MINIMUM	x_m	30	13	3	5	15	60	31	9^0	2,60	1 425	1 073
MEAN	$\bar{x}$	56	20	7	12	66	71	109	17^0	2,73	1 586	1 121
NUMBER OF TEST DATA	n	22	22	22	7	8	6	17	16	9	7	6
STANDARD DEVIATION	s	11,72	5,92	2,89	7,86	28,31	7,39	8,08	5,15	0,12	7,67	1,97
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,204	0,285	0,412	0,647	0,425	0,104	0,508	0,294	0,042	0,075	0,024

TABLE 6.3 : Engineering parameters for residual soils occurring within the Ventersdorp Lava 211 Land Pattern of the Johannesburg graben

ENGINEERING PARAMETERS		VOID RATIO	VOID RATIO : RED SOIL	VOID RATIO : YELLOW SOIL	COMPRESSION INDEX	COMPRESSION INDEX : RED SOIL	COMPRESSION INDEX : YELLOW SOIL	MOISTURE CONTENT (above 15 m)	MOISTURE CONTENT : RED SOIL (above 15 m)	MOISTURE CONTENT : YELLOW SOIL (above 15 m)
		eo	eo red only	eo yellow only	C_c	C_c red only	C_c yellow only	w (%)	w (%) red only	w (%) yellow only
MAXIMUM	x_M	2,11	2,11	1,40	0,88	0,88	0,70	49	49	41
MINIMUM	x_m	0,82	0,82	0,98	0,17	0,17	0,28	22	22	35
MEAN	$\bar{x}$	1,18	1,20	1,24	0,52	0,53	0,48	35	34	38
NUMBER OF TEST DATA	n	24	17	6	22	17	5	22	18	4
STANDARD DEVIATION	s	0,31	0,30	0,15	0,21	0,22	0,18	5,67	5,94	2,94
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,265	0,252	0,120	0,399	0,410	0,379	0,155	0,173	0,077

TABLE 6.3 Continued

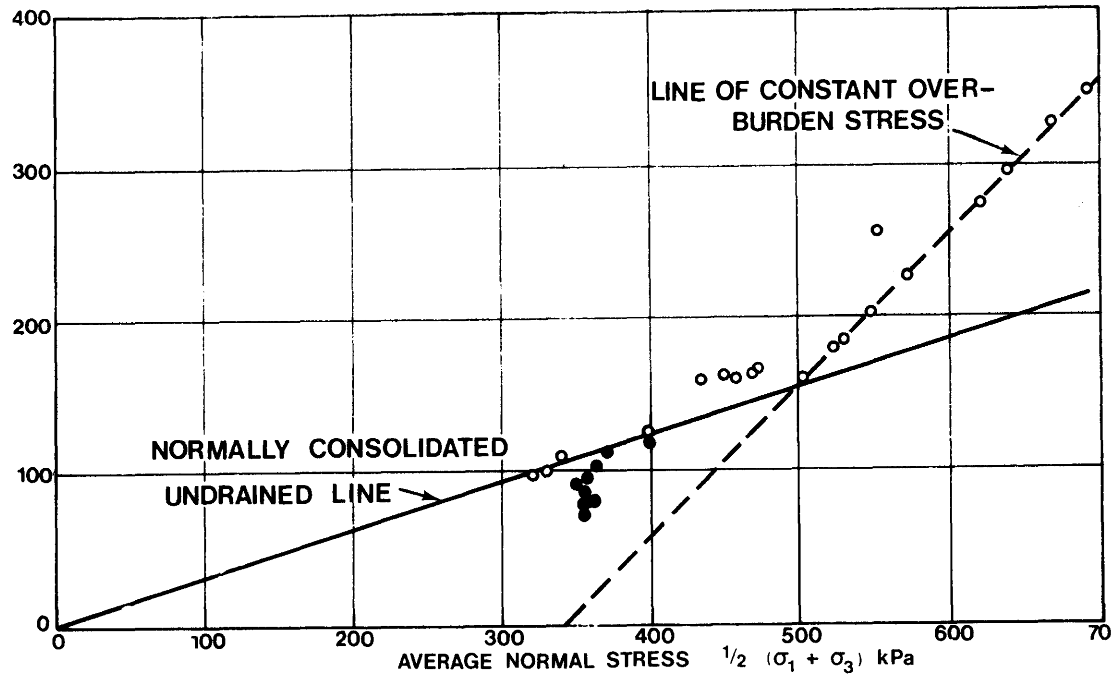
ENGINEERING PARAMETERS		MAXIMUM PRECONSOLIDATION PRESSURE Pc kPa	MAXIMUM PRECONSOLIDATION PRESSURE : RED SOIL Pc red only kPa	MAXIMUM PRECONSOLIDATION PRESSURE : YELLOW SOIL Pc yellow only kPa	COEFFICIENT OF CONSOLIDATION C_v mm^2/min	COEFFICIENT OF CONSOLIDATION : RED SOIL C_v red only mm^2/min	COEFFICIENT OF CONSOLIDATION : YELLOW SOIL C_v yellow only mm^2/min	OVER-CONSOLIDATION RATIO O C R	OVER-CONSOLIDATION RATIO : RED SOIL O C R red only	OVER-CONSOLIDATION RATIO : YELLOW SOIL O C R yellow only
MAXIMUM	x_M	1 408	1 408	871	2 580	2 580	2 046	7,3	7,3	7,2
MINIMUM	x_m	86	86	245	194	516	194	1,0	1,0	2,0
MEAN	$\bar{x}$	418	414	433	1 290	1 548	645	3,8	3,7	4,2
NUMBER OF TEST DATA	n	22	17	5	8	5	4	23	17	6
STANDARD DEVIATION	s	3,16	3,37	2,65	1,45	1,28	1,45	1,98	1,90	2,33
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,723	0,778	0,585	0,726	0,525	1,415	0,519	0,517	0,559

TABLE 6.3 Continued

So advanced is the degree of decomposition in these soils that the strength sometimes even drops below the normally consolidated strength envelope, as indicated in Figure 6/3; i.e. from the point of view of shear strength, the soil sometimes behaves as it if were under-consolidated (Blight, 1963). Figure 6/3, which also shows the correlation between bulk density and shear strength under overburden pressure, illustrates the fact that as decomposition proceeds the density of the residual soil decreases. It is also clear from this curve that the bulk density of this soil may be used as an approximate guide to its shear strength.

Many buildings erected on residual lava in Johannesburg have been founded on deep augered or replacement piles but, with the growing need for the provision of underground car parking facilities within the last decade, increasing use has been made of raft foundations under deep basements. This practice permits the use of higher pressures which are compensated by the mass of the soil removed. For example, excavation of a 10 m deep basement in this soil with its mean bulk density of about 1 600 kg/m^3 results in a pressure relief of 160 kPa; the bearing capacity of the soil therefore has to cater only for that part of the foundation bearing pressure in excess of 160 kPa, and settlement calculations would thus be based only on pressures in excess of this figure. If such calculations still indicate excessive settlements, as in the case of highrise buildings on restricted sites, piling may have to be resorted to in addition to deep basements.

There is no recorded case of heave having occurred on these soils. This might appear surprising in view of their relatively high indicator test results - compared for example with those of the expansive residual soils developed on the Ongeluk andesites in Pretoria: however, they appear to be maintained at a sufficiently high moisture content, doubtless as a result of their favourable topographic situation within an ancient valley, to inhibit their potential heaving capacity. This is confirmed by their mean value of 71 for the degree of saturation. The low density of these soils would be a further factor mitigating against the development of heaving characteristics. Their unusually high silt content, however, make them prone to a different kind of heave, namely *frost heave*. This is a matter of no general concern under the prevailing climate, but artificial inducement of frost heave in these soils below cold-storage warehouses can have very grave consequences, as illustrated by Case History 15.



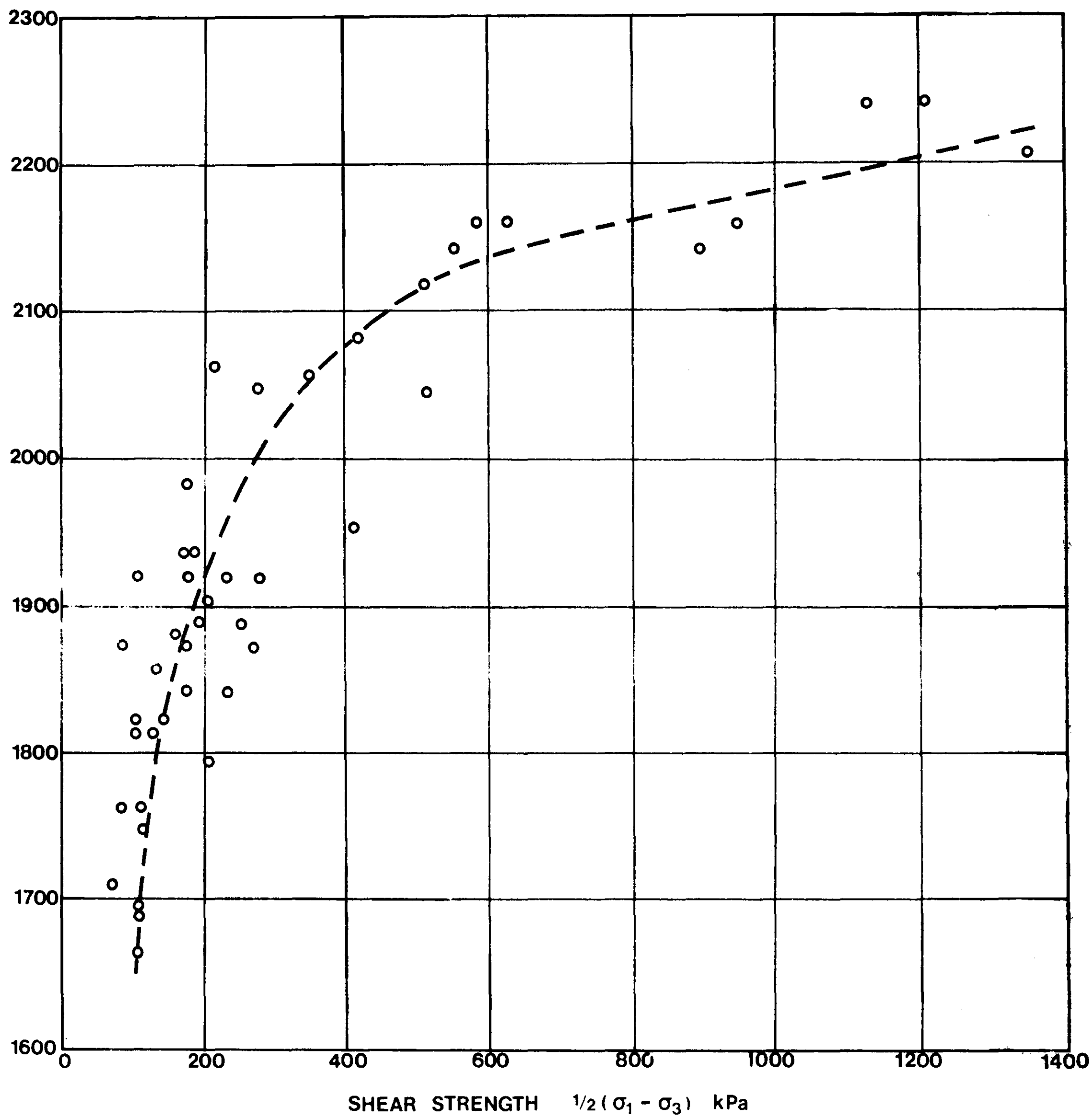


FIGURE 6/3 Strength and density tests on residual Ventersdorp lava from Johannesburg Graben (After Blight, 1963)

Founding conditions are particularly challenging where buildings straddle the faulted contacts between the Ventersdorp System and the Witwatersrand System on both the northern and the southern confines of the Johannesburg graben. Whereas the Witwatersrand System is here represented for the most part by competent quartzites and shales, the Ventersdorp lavas in juxtaposition with them are represented by the highly compressible residual soils described above. The resulting problems in foundation design are possibly nowhere better illustrated than in the buildings occupying the southern portion of the Witwatersrand University Campus in Milner Park, as described in Case History 16.

CASE HISTORY I4

RESIDUAL VENTERSDORP LAVA : SUB-HUMID MOIST ZONE

FOUNDATION FOR EASTERN KAZERNE PARKING GARAGE, HARRISON STREET, JOHANNESBURG

A site investigation for the parking garage was carried out early in 1960. As initially envisaged, the structure was to be four-decked reinforced concrete frame, with column loadings of the order of 180 tonnes and with columns spaced at 9 m intervals in rows 8 m apart.

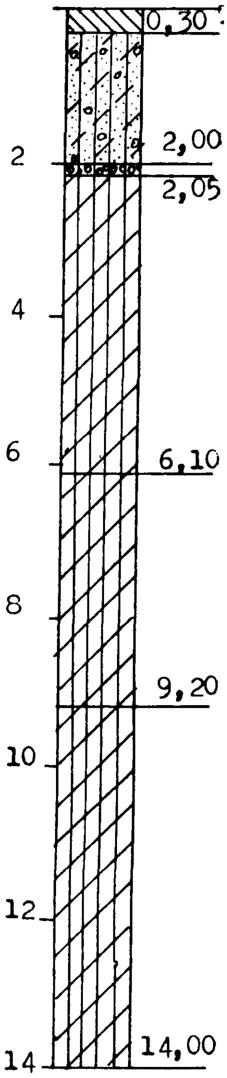
A generalised soil profile for the site is given in Figure 6/4.

Triaxial shear tests on undisturbed samples indicated a shear strength of 35 kPa at a depth of 3 m (Figure 6/5), and of 70 kPa at a depth of 9 m. Consolidation tests showed high compressibility at shallow depth and a considerable reduction in compressibility with increasing depth (Figure 6/6). Values for the Coefficient of Compressibility (C_v) varies from 1 930 to 5 160 mm^2/min which, although lower than the average for residual lava in Johannesburg, indicated that settlement would take place rapidly.

Consideration was given to three different methods of founding:

(i) Spread footings: Using the value of 35 kPa for shear strength as determined at 3 m depth, it was calculated that the initial bearing capacity for 3 m square bases founded at this depth would be 145 kPa, increasing to over 200 kPa due to rapid consolidation after loading. However, the consolidation curves indicated that a settlement of about 120 mm could be anticipated under a load of 145 kPa, which would result in a differential settlement of more than 25 mm. The use of spread footings was consequently ruled out.

(ii) Under-reamed *cast-in-situ* auger piles: Using the value of 70 kPa as determined for the shear strength at 9 m depth, it



Tarmac

Moist, brown, in profile reddish brown mottled yellow, firm, intact, slightly clayey, sand-silt, with ferruginous concretions and scattered angular vein quartz gravels; hillwash.

Concentration of angular vein quartz gravels in matrix as above; pebble marker.

Slightly moist, brown, in profile brownish red, soft to firm, intact, clay-silt with decomposed amygdales; residual ventersdorp andesite.

As above, but yellowish red in colour and firm in consistency.

As above, but yellow in colour and becoming progressively firmer with increasing depth; fresh amygdales below 13 m.

Bottom of 750 mm diameter Caldweld auger-hole; not at refusal; water-table not encountered.

ABAB
17/3/1960

FIGURE 6/4 : Generalised soil profile for Eastern Kazerne Parking Garage site, Harrison Street, Johannesburg

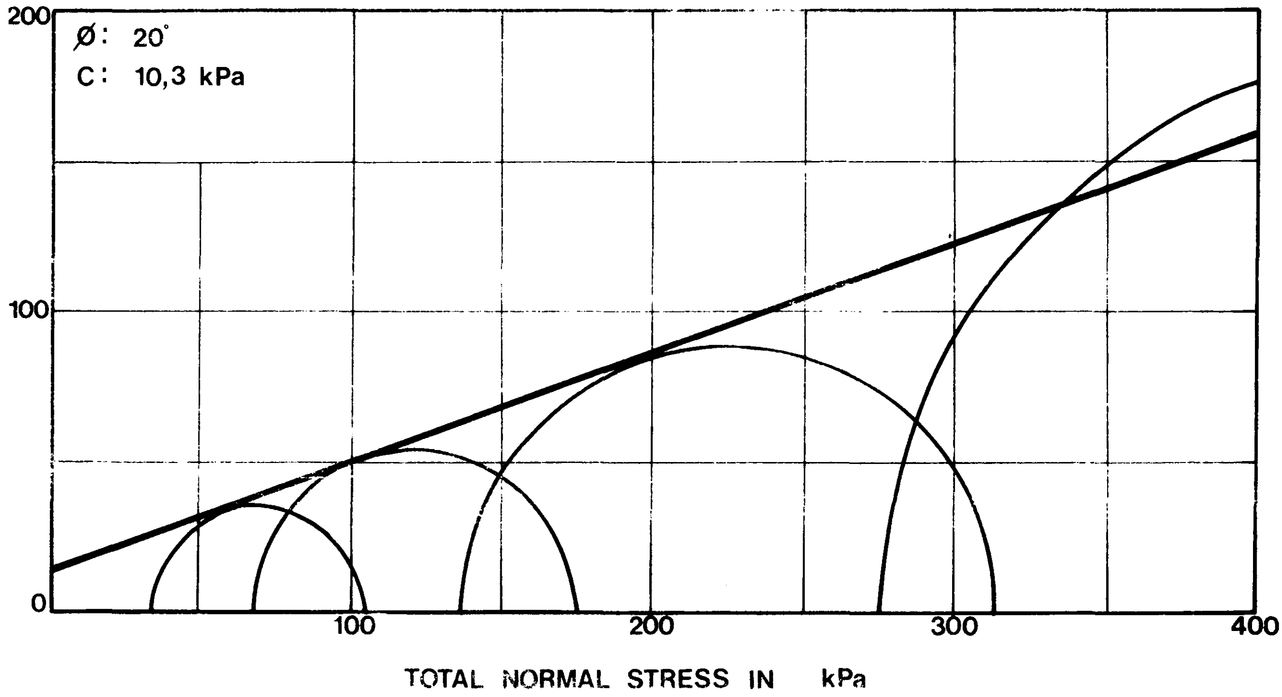


FIGURE 6/5 : Saturated, consolidated, undrained trixial shear test results for sample of residual lava taken at a depth of 3 m at Eastern Kazerne Parking Garage site, Johannesburg

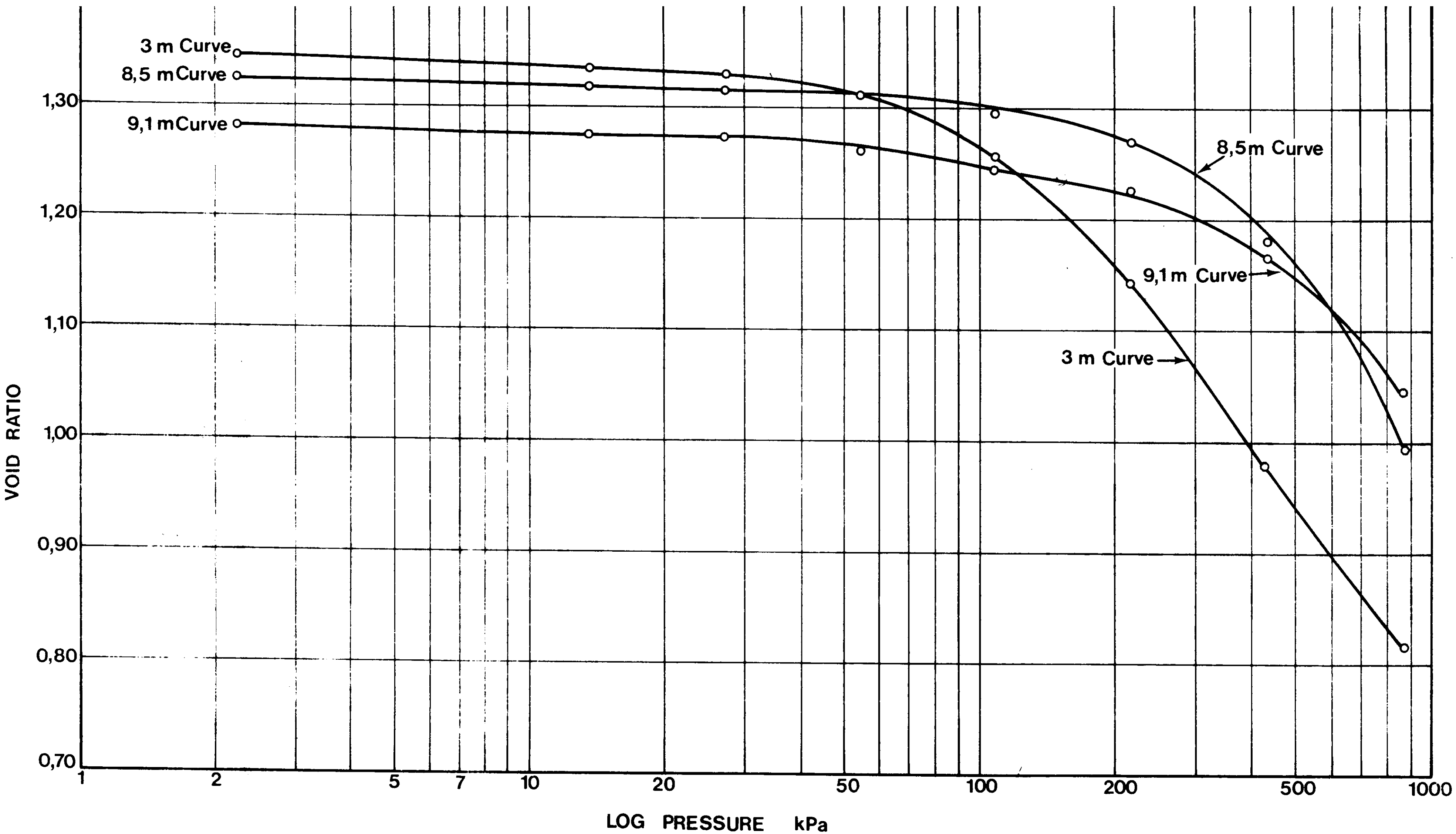


FIGURE 6/6 : Consolidation curves for samples of residual Ventersdorp lava taken from different depths at Eastern Kazerne Parking Garage site, Johannesburg was calculated that the initial bearing capacity for piles at this depth would be 400 kPa. At this loading, a settlement of 40 mm was calculated for a single pile under-reamed to 2,5 m diameter and carrying 180 tonnes. However, two factors would combine to reduce the actual settlement to less than this amount: skin friction on the pile shafts and expected decrease in compressibility below a depth of 9 m. Consequently differential settlement would be minimal and, owing to the high Coefficient of Compressibility, most of the settlement would take place during construction.

(iii) Driven *cast-in-situ* displacement piles: Displacement piles, each carrying 60 tonnes, were considered as an alternative to auger piles. It was not possible to predict founding depths for such piles on the basis of the tests which had been carried out, but from experience in the area it was assumed that piles driven to a set at about 12 m would safely carry 60 tonnes each. The action of forming an enlarged concrete bulb at the base of the pile has the effect of compacting the surrounding and underlying soil which, combined with the skin friction developed along the shaft, would adequately increase the bearing capacity of the soil. It was estimated that settlement on a group of three of these piles would be of the same order as that of a single 180 tonne auger pile.

The final design made provision for five parking levels - a ground floor slab plus four suspended decks - which resulted in increased column loadings varying from 250 to 350 tonnes. The most economical method of founding proved to be the use of a combination of 90, 60 and 15 tonne displacement piles at an average founding depth of 13 m. Loading tests on 60 tonne and 15 tonne piles indicated a maximum settlement of 5 mm and 0,5 mm respectively. Auger piles, under-reamed to 3 m diameter, had to be used along either side of a buried sewer which was found to run through the centre of the site. A total of 407 displacement piles and 12 auger piles were installed.

The structure was completed in 1962 and no apparent differential settlement has taken place.

CASE HISTORY I5

RESIDUAL VENTERSDORP LAVA : SUB-HUMID MOIST ZONE

FROST HEAVE UNDER COLD-STORAGE WAREHOUSE, NEWTOWN, JOHANNESBURG

In the year 1900 a cold-storage room was built in Johannesburg on the site of an abandoned brickfield in residual Ventersdorp lava. The room measuring 16,5 m by 15 m was built in a basement excavation 3,5 m deep, with brick walls and timber beams and columns. The columns were supported on concrete footings and the floor was insulated with a layer of granualted cork between timber sheathings, as shown in Figure 6/7. Fifteen years later a similar room was built adjacent to the first, but using a framework of structural steel. Further refrigeration rooms were erected at ground level on top of the basement rooms at this time, and these were in turn topped by a loft for archive storage. Floor loading in the basement rooms was about 15 kPa and foundation pressures about 300 kPa. The storage rooms were refrigerated with an ammonia plant operating at temperatures between -18^{0}C and -12^{0}C (0^{0}F and 10^{0}F).

Heaving of the basement floors into a dome-shaped pattern was first noticed in 1940. Distortions became progressively more pronounced during the following years, and the heaving movements accelerated alarmingly during 1954. By the end of 1954 the total heave under the centre of the warehouse was about 500 mm and the differential heave between the centre and the periphery walls about half this figure. The building was approaching a dangerous condition, with rafters pulling out of their sockets on the tops of the walls, beams being lifted off their supporting columns, doors jamming and internal walls badly cracked, structural steelwork warping and rivets snapping. The loft walls were leaning outwards and appeared to be about to topple over. The photograph reproduced in Figure 6/8 shows the state of one of the basement floors at this stage. The fact that the dome-shaped distortion of the floor had been transmitted upwards through the ground floor to the loft indicated that the column footings had also heaved, though to a somewhat lesser extent than the basement floors.

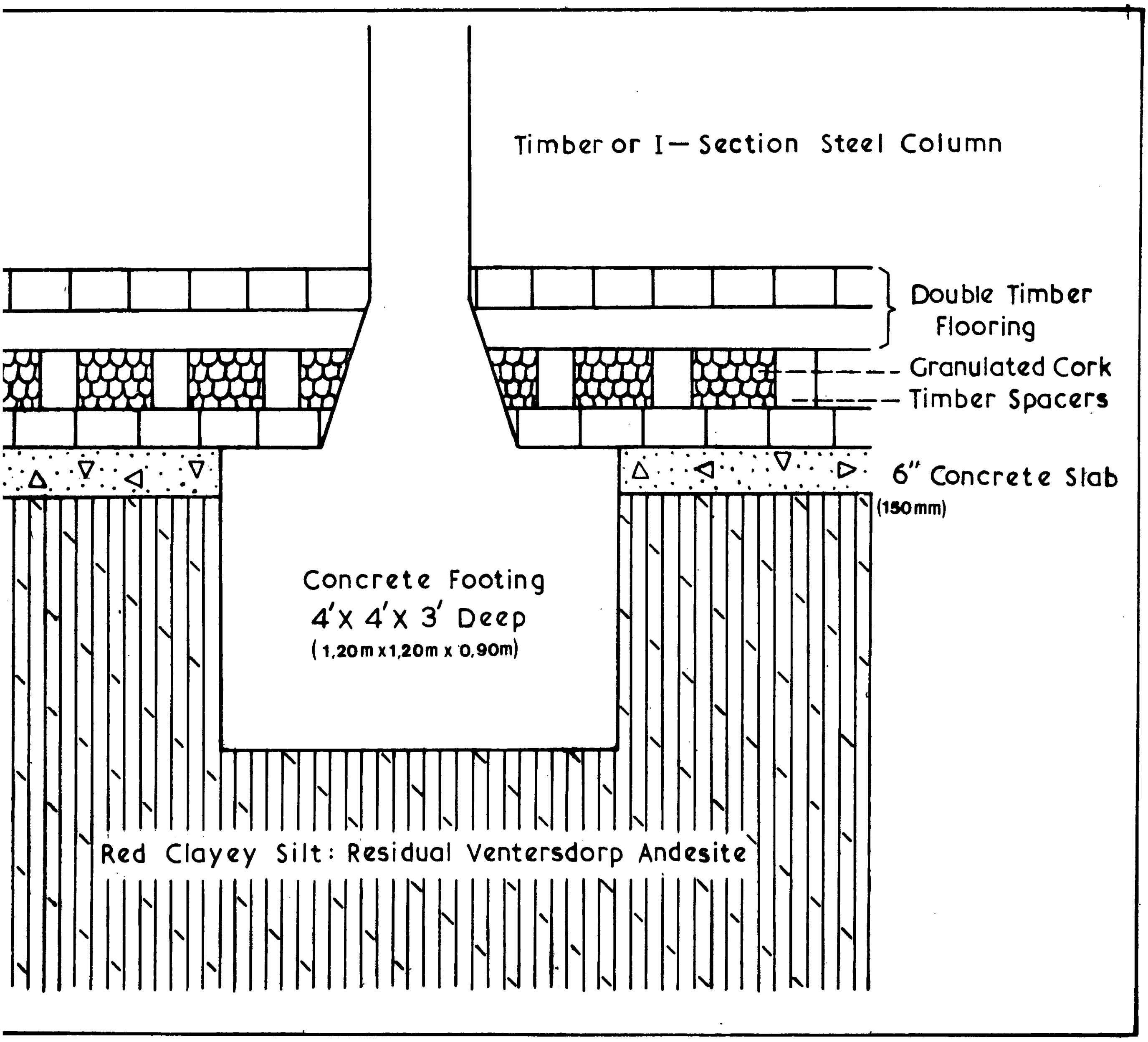


FIGURE 6/7 : Insulation below floor of cold-storage room built in Johannesburg in 1900

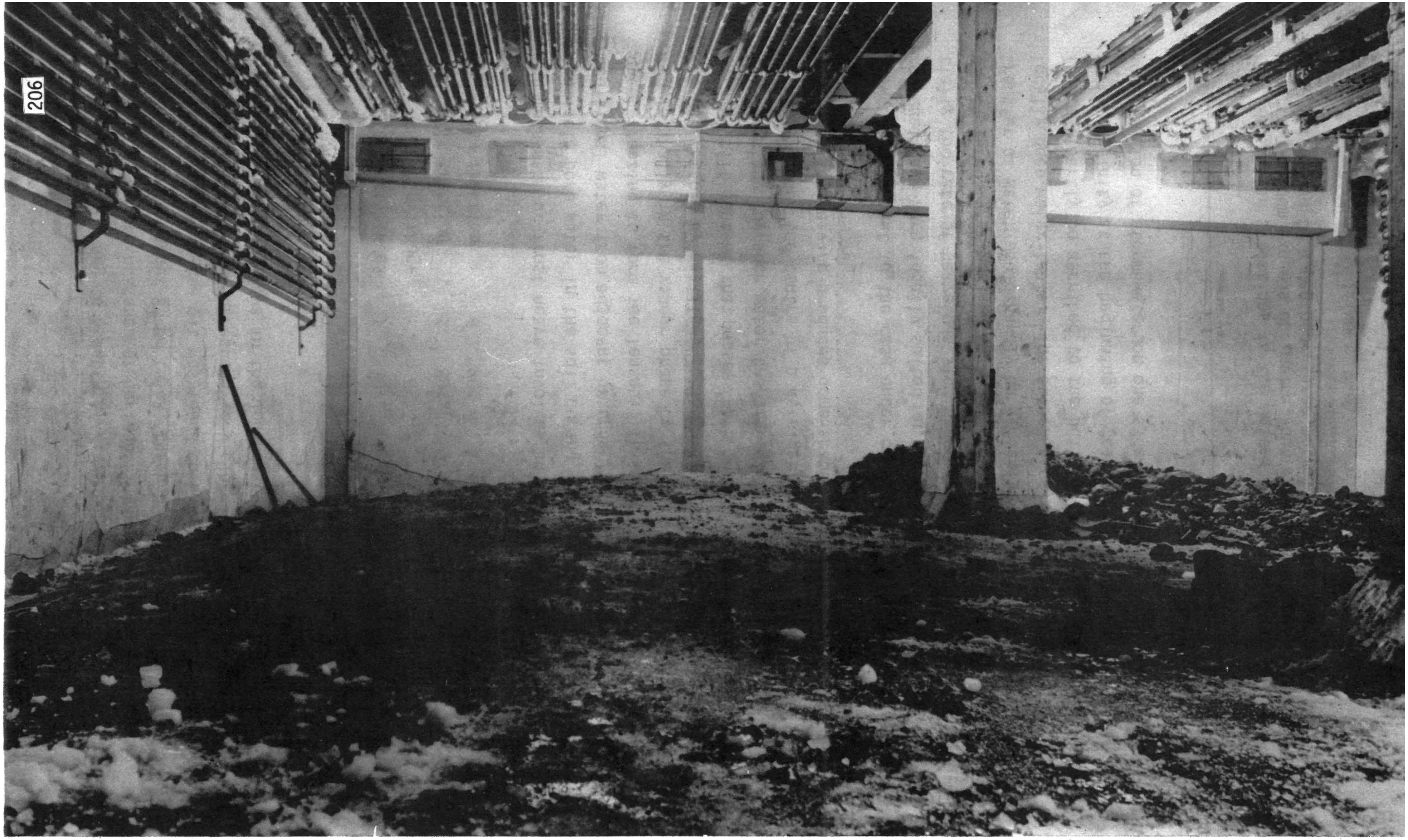


FIGURE 6/8 : Dome-shaped distortion in floor of basement cold-storage room produced by 250 mm differential frost heave ; Newtown, Johannesburg

(Photo taken 1946)

An inspection-pit dug to a depth of 2,5 m beneath one of the basement rooms revealed the presence of horizontal layers and lenses of ice in the soil, mostly about 25 mm thick, throughout the total depth*. A sample of soil from the pit was found to have the following characteristics, all of which correspond very closely to the mean values for residual Ventersdorp lava in the Johannesburg graben as given in Table 6.3:

Liquid Limit	:	59
Plasticity Index	:	19
Linear Shrinkage	:	6
Silt fraction	:	59%
Clay fraction	:	22%

The mechanism of frost heave involves an increase in volume of the available pore water in the soil during freezing, and the attraction of additional water from adjacent or underlying soil masses and its subsequent freezing to form ice lenses. This additional water may be attracted laterally in the vapour phase by thermo-osmosis, or vertically from the water-table by capillary action. It is only soils with a sufficiently high silt content that possess both the high permeability and capillarity necessary for such water transfer. The freezing isoline in the soil is deeper under the centre of a refrigerated room than under the perimeter, and hence the heaving produces dome-shaped distortions.

It would seem that in the case under consideration the properties of the residual lava were ideally suited for the promotion of frost heave. Initially the rate of heave was slow presumably due to the fact that thermo-osmotic transfer of moisture is a slow process, and that capillary transfer of water would hardly be effective from what appears to be a locally deep water-table. The sudden acceleration in heaving during 1954 may partly be ascribed to the building of large railway embankments in the vicinity at the beginning of that year, which results in the accumulation of storm-water on one side of the building; partly due,

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* In similar situations in the United States the depth of ice formation seldom exceeds one metre, but one case is on record where ice had formed to a depth of three metres (Krynine and Judd, 1957).

also, to insulation breakdown and rapid transfer of freezing once the floors had cracked. The total frost heave of 500 mm observed towards the end of 1954 must rank among world records.

Remedial measures undertaken early in 1955 involved the ripping up of the basement floors, excavation of the frozen soil by jack-hammers to a depth of one metre (i.e. to the base of the column footings), back-filling with coarse gravel and reconstructing the floor with an insulation of solid cork, 200 mm thick, sandwiched between two concrete slabs. The loft walls were demolished and replaced with flexible panelling. Slow thawing of the ice lenses below the column footings resulted in the beams gradually coming to bear on the columns again.

These measures have proved to be partially effective and the refrigerated rooms are still in use. Heaving continues to take place in the dividing wall between the two basement rooms, however, and more radical remedial measures will undoubtedly have to be applied in due course. The problem appears to have been adequately solved in a block of refrigerated rooms built elsewhere on the site in 1948: here the floors were suspended with a one metre clear gap above the soil, and air is kept circulating in this gap by a draught induced by a tall chimney. No frost heave has been experienced in this block to date.

CASE HISTORY I6

FAULTED CONTACT BETWEEN VENTERSDORP LAVA AND HOSPITAL HILL QUARTZITE

SOUTHERN PORTION OF WITWATERSRAND UNIVERSITY CAMPUS, JOHANNESBURG

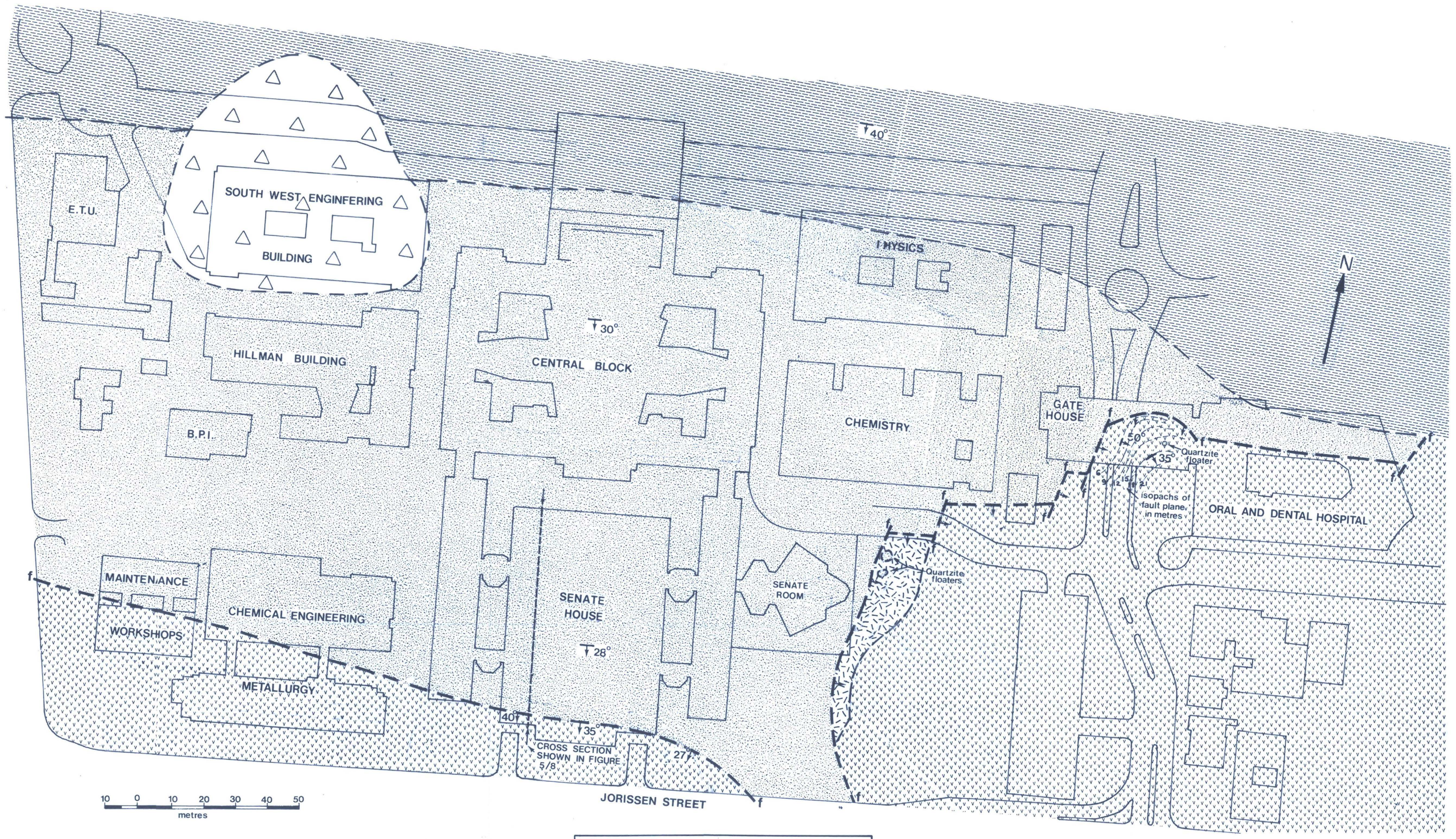
As may be seen from the geological map in Figure 6/9, four large buildings on the southern part of Witwatersrand University Campus straddle the northern fault of the Johannesburg Graben: the Oral and Dental Hospital, Gate House, Senate House and the Chemical Engineering and Metallurgy Building. The northern portions of each of these buildings are founded on the hard Hospital Hill quartzite, and the southern portions on Ventersdorp lavas which are decomposed into soft and highly compressible residual clayey silt throughout their entire depth.

The Oral and Dental Hospital, built during 1951-1952, is founded throughout on spot footings. The southern portion of this building suffered substantial initial settlement as a result of consolidation of the residual lava during and immediately after construction and, judging by the continual cracking of the rigid tile finishes which are a feature of the building, settlement finally ceased only during 1955. A current proposal to extend the building by adding extra storeys to the existing structure is thus receiving very cautious consideration. An exploratory borehole drilled on the southern side of the building in 1976 showed the presence of residual soil to a depth of 42,5 m: below this depth the lava is in the form of highly sheared very soft rock. It is thus clear that underpinning of the southern half of the existing structure would be a very costly procedure.

Column loads of the southern portions of the other three buildings mentioned above are all carried on *cast-in-situ* augered piles founded on the faulted slope of the quartzite at refusal depth. At Gate House the piles vary in length from 2,5 m to 23 m, the founding depths increasing in a south-easterly direction.

LEGEND

- Rubble-filled old quarry (more than 6m deep in places) in this general area*
- Breccia — VENTERSDORP SYSTEM
- Lavas — VENTERSDORP SYSTEM
- Quartzite — HOSPITAL HILL SERIES, LOWER DIVISION WITWATERSRAND SYSTEM
- Shale — HOSPITAL HILL SERIES, LOWER DIVISION WITWATERSRAND SYSTEM



* South West Engineering Building built on strip footings founded on rubble fill : note subsidence of steps on northern side and of walls on western side

FIGURE 6/9 : Geology of southern portion of campus, University of the Witwatersrand, Johannesburg

Isopachs of the founding surface as plotted on Figure 6/9 indicate that one of the columns is carried on a pile founded on a 'floater' of quartzite within the residual soil: the building was erected in the late 1960's and no perceptible settlement has taken place.

When levelling of the site for Senate House was already fairly far advanced by the beginning of 1971, the writer was commissioned to undertake an investigation to determine the position and nature of the faulted contacts. The excavation provided reasonably good exposure at the commencement of the investigation: nevertheless, it proved necessary to put down 36 augered trial-holes and 8 inspection-trenches in order adequately to delineate structure contours of the quartzite surface below the wedge of residual lava and to determine the nature of the materials on either side of the faulted contact. The residual lava was found to attain a maximum thickness of 11 m on the south-western corner of the building site. Further to the east, under Lawson's Building, a trial-hole had penetrated through a depth of 25 m of residual soil without reaching refusal (De Beer, 1965). The soil was basically described as a moist, red, soft, clayey silt containing sheared amygdales.

Blasting on the major portion of Senate House has revealed orthoquartzite with a consistency varying from soft rock to very hard rock, and characterised by a medium-grained 'sago texture' imparted by well-rounded clear quartz grains set in a fine-grained opaque matrix and also by the presence of fuchsite, a bright green chromium-muscovite mineral. Minor shale partings intercalated in the quartzite indicated the possibility of slope stability problems on the northern side of the excavation adjacent to the Central Block.

In several of the trial-holes augered through the residual lava to refusal on the quartzite, a layer of decomposed sericite-talc schist, in the form of firm silt, was encountered immediately above the faulted contact with the quartzite. This probably represents a metamorphic derivative from a shale parting in the Hospital Hill Series, and metamorphism presumably took place as a result of shearing stresses on the major strike-fault situated largely within the incompetent shale parting. Similar material was also encountered in a fault-plane within the quartzite itself: a large block of it in the western face of the excavation threatened the safety of workmen and had to be carefully prised out.

Immediately to the east of the Senate House site, a narrow strip was found to be occupied by a brecciated rock which was too hard to be penetrated by the Calweld auger-bucket. Two large quartzite 'floaters' were situated within the breccia, and a great many smaller quartzite fragments of varying dimensions made up a large part of the breccia mass. The balance of the rock mass was found to consist of silica-rich amygdaloidal lava with the consistency of soft to very soft rock. It was apparent that the high silica content had inhibited chemical decomposition of the breccia. Auger refusal in this strip nowhere exceeded 3 m and was more commonly encountered at depths less than one metre.

'Floaters' of quartzite were encountered in the residual lavas near all the faulted contacts: some of these were found to be decomposed into friable sand, but most had the consistency of hard rock. It was recommended that all augered pile holes on the southern side of the Senate House site be probed by percussion drilling to a depth of 3 metres below refusal depth to make sure that no piles would be founded on floaters underlain by compressible residual soil. The same caution was to be exercised in augered pile holes which could encounter refusal on the massive quartz veins which were also present within the residual soil.

The structural interpretation which emerged from the investigation was complex, as may be seen from Figure 6/9. Where exposed in the excavation for Senate House, the Hospital Hill quartzite dipped to the south at an angle of 28^0. Towards the south of the site the dip became somewhat steeper as the major strike-fault was approached. Structure contours showing the thickness of soil cover in this area -both residual lava and residual sericite-talc schist - showed a southerly dip of the fault-plane at angles varying from 27^0 to 40^0.

The trace of the major dip-fault to the east of Senate House was established with a reasonably good degree of accuracy. The faulting pattern here and farther to the north-east towards Gate House represents a series of parallel step-faults displacing the major strike-fault: the upthrow-sides of the dip-faults on the east in each case. It will be seen that the dip-faults are shown as younger than the displaced segments of the strike-fault; however, it is thought likely that all the faulting was broadly contemporaneous. The extrusion of the Ventersdorp lavas is also interpreted as being broadly contemporaneous with the faulting, as this would account for the presence of quartzite blocks detached from the fault-scarps being incorporated as 'floaters' in the lavas and breccia. Subsequent rejuvenation of faulting is evidenced by the highly sheared nature of the lavas in many places, and the stretching out of amygdales within these. While all these events are thus conceived as being broadly contemporaneous, their sequence in detail is interpreted as follows:

(i) formation of graben, with the major E-W strike-fault representing the northern edge of the graben, developed largely within the incompetent shale parting and producing a schistose structure within it;

(ii) development of dip-faults displacing the major strike-fault into segments;

(iii) extrusion of lavas into the graben, engulfing quartzite blocks below the fault-scarps, and assimilating considerable quantities of silica below the dip-fault west of Senate House to produce the chemically resistant breccia;

(iv) reactivation of shearing movements along most of the faults, producing shear in the solidified lavas.

The diagrammatic cross-section below Senate House given in Figure 6/10 illustrates the foundation design which was adopted. Where auger refusal in the pile shafts for the southern rows of columns was encountered at lesser depth than had been predicted from the isopachs, percussion probes 3 m deep were put down to prove adequate founding on solid quartzite. On the northern side of the excavation a number of retaining walls span between the footings and the beams on top, as illustrated in Figure 6/10. Thirteen permanent rock anchors, about 8 m long, were inserted through the retaining walls and into the quartzite. These support the earth pressure reaction from the fill: they form part of the structural system and result in a reduction in the thickness of the retaining walls.

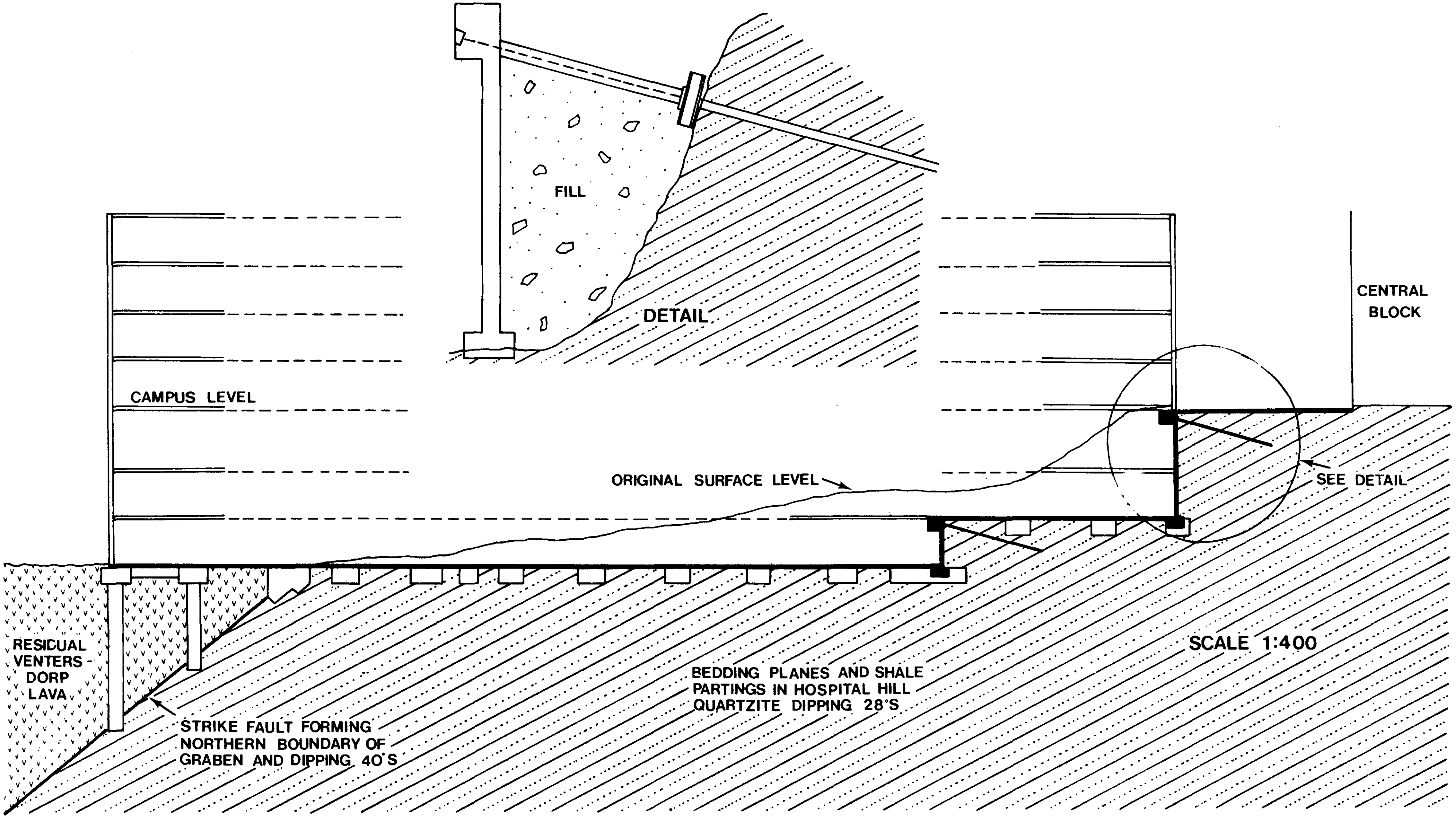


FIGURE 6/10 : North-south cross-sectiom below Senate House, University of the Witwatersrand

7 TRANSVAAL SYSTEM

BLACK REEF AND DOLOMITE SERIES

INTRODUCTION

The Transvaal dolomites have a notorious reputation. Many front-page newspaper headlines have been devoted to the devastation caused by sinkholes and other subsidences during the past fifteen years. During this period 38 people have lost their lives by being interred in sinkholes. Serious damage or complete obliteration of buildings and other structures experienced on the dolomites in the Transvaal has been more severe than on any other geological formation in Southern Africa. Apart from the development of sudden and catastrophic sinkholes, there is also the problem of gradual subsidence of the surface during the formation of dolines and, in addition, the problems of founding any structures on highly compressible wad which is frequently present in the dolomite formation. While these karst features are not confined to the Transvaal dolomites, nor to the time span of the past fifteen years, it is the accelerated development of these phenomena on the Far West Rand during this period that has caused grave local concern and world-wide interest. Consequently this chapter will deal mainly with the Far West Rand area.

As reference will be made throughout this chapter to *sinkholes* and *dolines*, it may be as well at the outset to define these terms with specific reference to their characteristic features as developed on the Far West Rand:

> A *sinkhole** is a subsidence which appears suddenly, and sometimes catastrophically, as a cylindrical and steep-sided hole in the ground. It is usually, but not always, circular in plan, and may be up to 125 m wide and 50 m deep. If the dimensions exceed 45 m in diameter and 30 m in depth, it is regarded as a *large sinkhole* (Foose, 1967).

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* In accordance with current usage in engineering literature in South Africa the spelling *sinkhole* has been used throughout this chapter, rather than "sink-hole" or "sink hole".

A *doline* (or compaction subsidence) is a surface depression which appears slowly over a period of years. It may be circular, oval or linear in plan. Where circular or oval it may be up to two or three hundred metres in diameter; where linear, up to a kilometre long. It may attain a depth of up to about 12 metres. The periphery of a doline is characterised by the presence of tension cracks within a zone of shear. There are a few cases on record where a sinkhole has appeared within a doline.

On the credit side, mention will also be made in this chapter of the engineering uses of dolomite. Also on the credit side it should be mentioned, in passing, that if it were not for the existence of the dolomites in the Transvaal, the valuable hominid remains from Makapansgat, Sterkfontein, and Swartkrans (Figure 7/1), which assist in piecing together the history of the evolution of man, would never have been preserved.

DISTRIBUTION AND STRATIGRAPHY

Strictly speaking the problems associated with the Black Reef Series should be discussed before those of the Dolomite Series: This, because the Black Reef Series is stratigraphically older than the dolomite. However, as the distribution of the Black Reef Series is very much more limited than that of the Dolomite Series, and as the engineering problems associated with the former are far less grave than, and largely similar to, those associated with the latter, it is more convenient to deal with the two together.

Figure 7/1 shows the distribution of the dolomites in the Transvaal. The series outcrops over an area of some 15 500 sq km. The rocks of the Black Reef Series have a more limited distribution, forming a narrow outcrop encircling the areas occupied by the Dolomite Series. One important exposure of Black Reef strata to which reference will be made is the so-called West Rand Anticline (and its western extension into the Hartebeesfontein Anticline) which strikes east-west in the area immediately north of the black rectangle in Figure 7/1 which represents the Far West Rand.

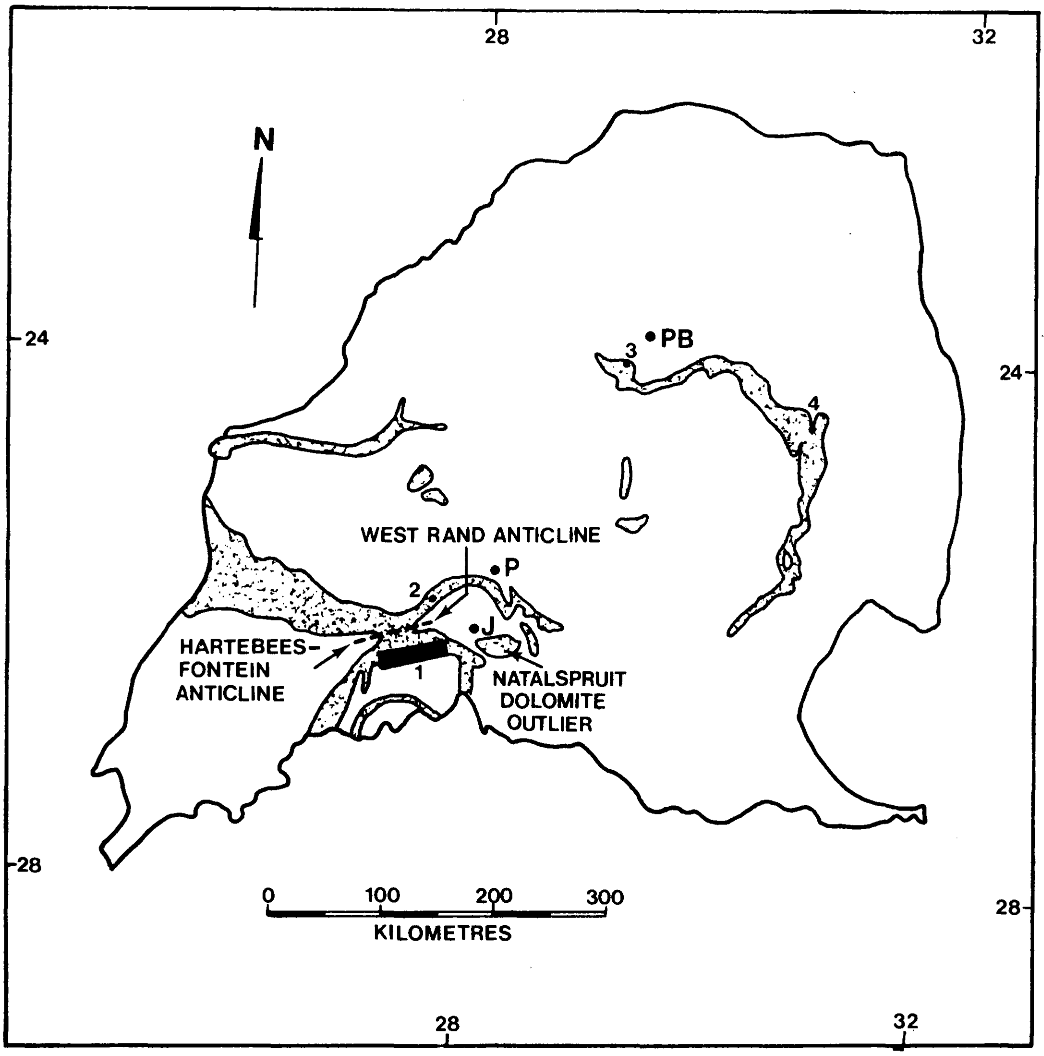


FIGURE 7/1 : Distribution of the Dolomite Series in the Transvaal

1 : Far West Rand
2 : Sterkfontein Caves
3 : Makapansgat Valley
4 : Echo Caves

J : Johannesburg; P : Pretoria;
PB : Pietersburg

The stratigraphy of the Black Reef and Dolomite Series has recently been described in terms of stratotypes by Button (1973) and by Eriksson and Truswell (1974). Button separates the Black Reef from the Malmani dolomite in the Eastern Transvaal where the former has a thickness up to 500 m. In the Central Transvaal, however, the Black Reef Series now becomes one of nine formations which collectively constitute the Malmani Subgroup. Certainly from an engineering point of view the inclusion of the Black Reef Series with the Dolomites must be regarded as a logical innovation, particularly in view of the fact that at least as far as founding conditions on *wad* are concerned, the two sequences of strata belong together.

The stratigraphic sequence described by Eriksson and Truswell is for an extensive area north-west of Johannesburg. The Black Reef Formation here has a thickness of 24 metres and consists of a basal conglomerate overlain by 5 m of quartzite followed by 10 m of carbonaceous shales. These are, in turn, overlain by a layer of wad, 2 m thick, which probably represents a residue from a greater thickness of dolomite. The carbonaceous shales contain two intercalated dolomite horizons which weather to a chocolate colour. Further shales are present above the wad horizon and again a layer of dolomite is intercalated within these shales.

The remaining eight formations of the subgroup have a total thickness of just over 1 400 metres and consist essentially of various types of dolomite, recrystallised in places, and with most of the formations containing chert horizons. Of great interest is the recognition of the Fountains Formation (formerly known as the "Giant Chert" breccia) representing a residual concentration of chert rubble on an eroded surface following subaerial exposure (Button, 1974; Eriksson, 1971). The base of this formation represents an unconformity. The probable nature of the breccia prior to its silicification on surface into solid rock is well illustrated by the vast thicknesses of 'chert rubble', residual from the weathering of the dolomites, which are so commonly observed during the course of site investigation work in dolomite terrain. The subsurface characteristics of the Fountains Formation differ from those generally seen on outcrop in that the brecciated chert is embedded in a matrix of carbonaceous shale (Eriksson, 1972). Button has recorded the presence of manganese wad in the matrix of this formation in the Eastern Transvaal.

The stratigraphy of the Dolomite Series in the Far West Rand is relatively simple, and comprises a series of alternating layers of dolomite and chert with a total thickness of about 1 200 metres. Figure 7/2 shows a composite stratigraphic column, compiled by Eriksson (1972) from the logs of four deep boreholes in the Far West Rand area.

The chert bands vary in thickness from a few centimetres up to about a metre; they increase in frequency and thickness towards the top of the succession where they account for some 20 per cent of the total volume. The topmost member of the succession, the Fountains Formation, is a chert breccia of very variable thickness (from 10 to 100 metres). It is not surprising that the thickness of this member is so variable as it represents cemented chert rubble; indeed the present-day chert rubble, developed as a residue from the weathering of dolomite, is also known to vary in thickness between such wide limits.

Dolomite is a carbonate of calcium and magnesium, and the relative proportions of these two compounds have been shown by chemical analysis to approximate to the chemical composition of the pure mineral dolomite (Eriksson, 1971). Only in the lowermost fifty metres of the succession is there a sharp increase in the calcium content and the rock here has the composition of a dolomitic limestone. This is illustrated in Figure 7/3, in which the vertical distribution of the Ca/Mg ratio is shown for the whole succession from 60 metres below the base of the Fountains Formation to the base of the Black Reef Series. This is based on chemical analyses of 85 samples of dolomite selected from the core of Borehole UD 15 (see Figure 7/9).

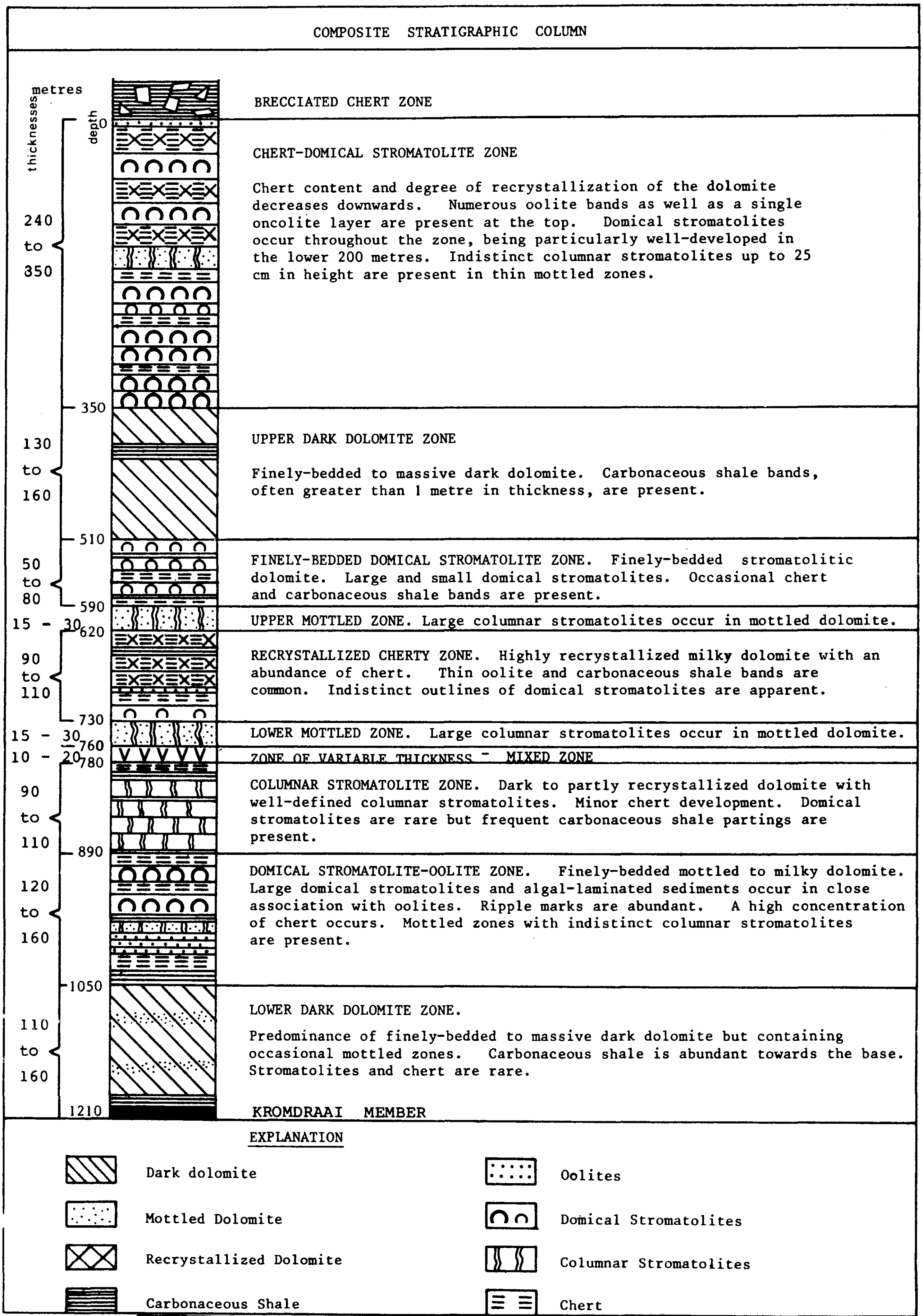


FIGURE 7/2 : Composite stratigraphic column through Dolomite Series in the Far West Rand (After Eriksson, 1972)

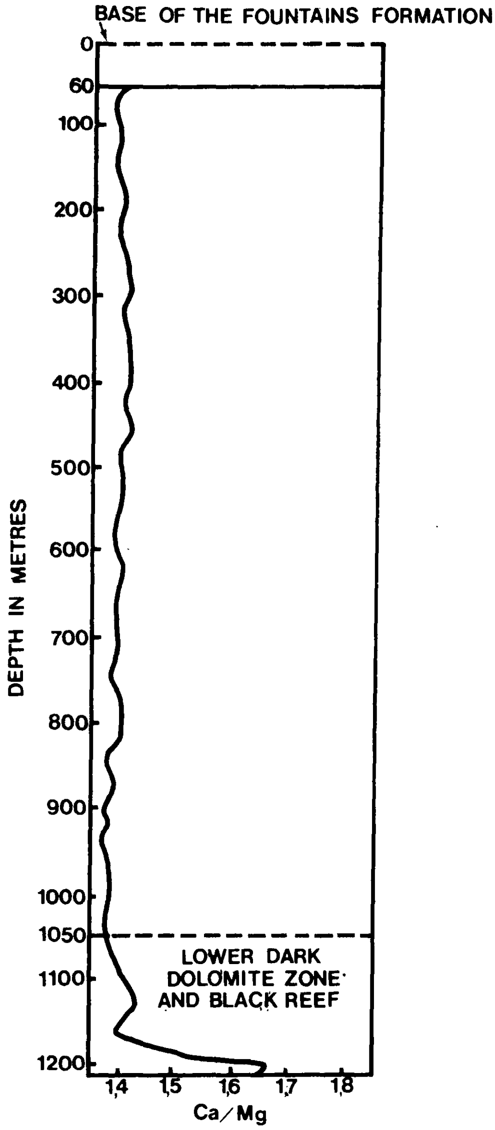


FIGURE 7/3 : Vertical distribution of Calcium/Magnesium in the Dolomite Series of the Far West Rand

(After Eriksson, 1971)

EARLY HISTORY OF KARST PHENOMENA AND FOUNDATION PROBLEMS ON THE DOLOMITES

The first published mention of karst phenomena in the Transvaal dolomites, and specifically of sinkholes, was made by Penning (1884). During the South African War, Denys Reitz hid his whole commando from the British in a sinkhole in the Gatsrante behind the Doornfontein mine. The Gatsrante were indeed so named by the Voortrekkers on account of the abundance of karst features within these hills (De Kock, 1964). Sinkholes are thus not phenomena which developed only during the past fifteen years. Further examples of these features dating to the period prior to recorded history are the Wondergat, near Slurry, which has been probed by divers to its total depth of 65 metres, and the sinkhole which forms the entrance to the Wolkberg Caves in the Northern Transvaal. Indeed there are many other examples of "paleo-sinkholes" throughout the dolomites, not only in the Transvaal but also in the Northern Cape and in South West Africa.

In 1937 sinkholes formed during the re-location of the Germiston-Pretoria railway line (Jennings, 1966 a), and in 1938 the south abutment of the Fountains Viaduct in Pretoria subsided into a compressible layer of wad.

Fountains Viaduct

It would be appropriate here to mention further details of the subsidence of the Fountains Viaduct, as the investigation and correction of this failure perhaps played a significant part in confirming the interest which Professor J.E. Jennings had already acquired in foundation engineering: an interest which has earned his reputation as the doyen of Soil Mechanics in South Africa.

The viaduct spans across the contact between the upper horizons of the Dolomite Series and the lower beds of the Timeball Hill Stage of the Pretoria Series. Prior to construction, a site investigation conducted by means of a jack-hammer two metres long, (the longest pneumatic steel bit available at that time), revealed the presence of a minimum of two metres of hard chert below the south abutment. After construction of the bridge there were no signs of failure, until the approach embankment was built on the southern side. At this stage the arches of the bridge started to crack. Plaster of Paris plaques were installed on both sides of the larger cracks and these indicated that the cracks were opening up at a rapid rate. Jennings, who was in the employ of the South African Railways at that time, was instructed to investigate the 'bad concrete'. When he saw the nature of the cracking pattern he maintained that it had been caused by settlement; he was told, however, that the bridge had been built on solid rock.

Levelling observations on steel pegs which Jennings inserted along the length of the bridge, using the northern abutment as a bench-mark, showed a rotation of three millimetres of the southern section of the bridge due to settlement of the south abutment within one week of observation. A jumper-drill hole put down next to the abutment then revealed that the two metre chert horizon was underlain by a further two metres of wad, below which the hole penetrated solid chert and dolomite for a considerable depth.

The abutment was underpinned by excavating the wad in sections down to a depth of four metres, and backpacking the excavation with bags of a dry sand and cement mix which was then wetted in place. No further movement has been recorded, and the bridge is still in daily use.

Sinkholes at Waterkloof and Swartkops Air Stations

In 1953 a swimming bath at the Waterkloof Air Station outside Pretoria was drained instantly when a sinkhole appeared near the deep end. At about this time too, an ablution block at Swartkops Air Station tilted to an angle of 15^{0} as a result of the development of a sinkhole, caused by a leaking pipe, at one end of the building.

Apart from the numerous sinkholes which formed in the Far West Rand during the 1960's, and which will be discussed later, there were several occurrences elsewhere in the dolomites, of which it is worth recording three significant examples.

Sinkhole in the Natalspruit Dolomite Outlier

A sinkhole appeared at the bottom of a gravel pit next to the Johannesburg-Durban Road (T3-11) during February 1964, at a point 20 kilometres north-west of Heidelberg, and near the centre of the Natalspruit Dolomite Outlier (Figure 7/4). The gravel pit was in the form of a trench about four metres deep and parallel to the eastern side of the road; it had no surface drainage outlet. Rainwater accumulating in the pit was sufficient to cause subsurface erosion of the unconsolidated soils into a cavern, 18 metres deep, situated between the pit and the road (Figure 7/5).

The sinkhole and the cavern were examined by the writer in the company of colleagues from the National Institute for Road Research. Figure 7/6 shows the entrance to the cavern from the bottom of the sinkhole, and Figure 7/7 is a schematic soil profile from the surface down to this level. It is of interest to note that a layer of wad, about 1 metre thick, was present around the dolomite pinnacles: this material had a moisture content in the range 188 to 203 per cent.

The cavern had walls of pinnacled dolomite. The roof, of residual diabase and in places of chert rubble, was vault-shaped with arches spanning between the buttresses of pinnacled dolomite.

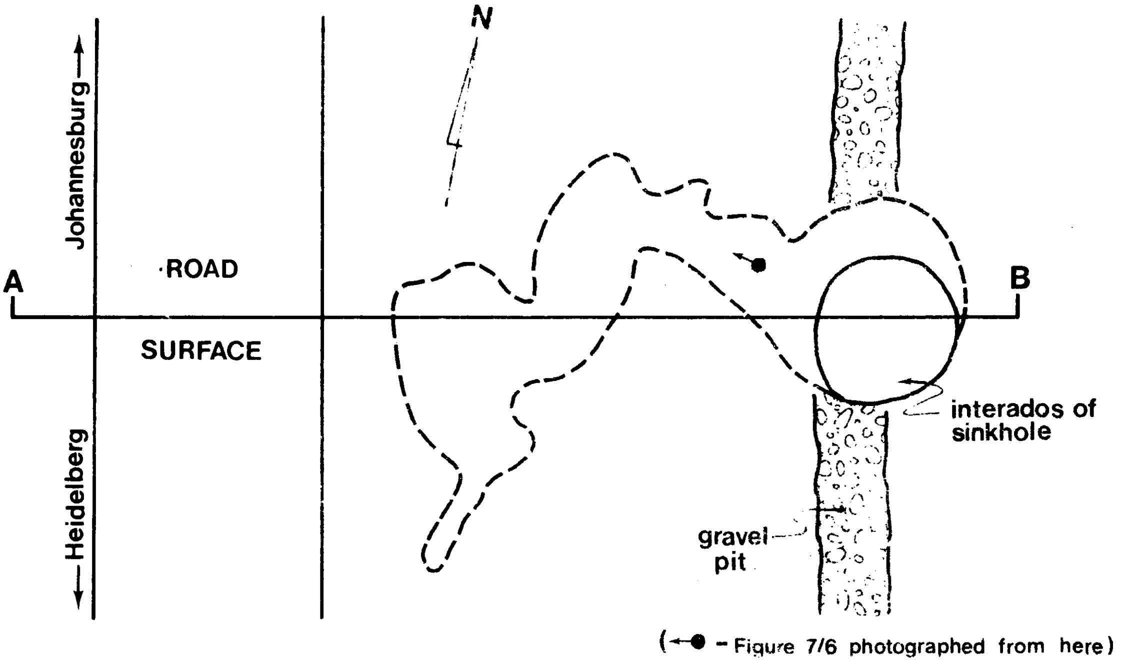
Sinkhole at Vogelstruisbult, near Kuruman

Figure 7/8 is an aerial view of a large isolated sinkhole which appeared in November 1967 in the dolomites of the Kaap Plateau, on the farm Vogelstruisbult, a few kilometres south-west of Kuruman. The sinkhole was 50 metres deep and the walls were of chert rubble throughout. A funnel-shaped cavity into a dolomite-bound slot was visible at the bottom of the hole.

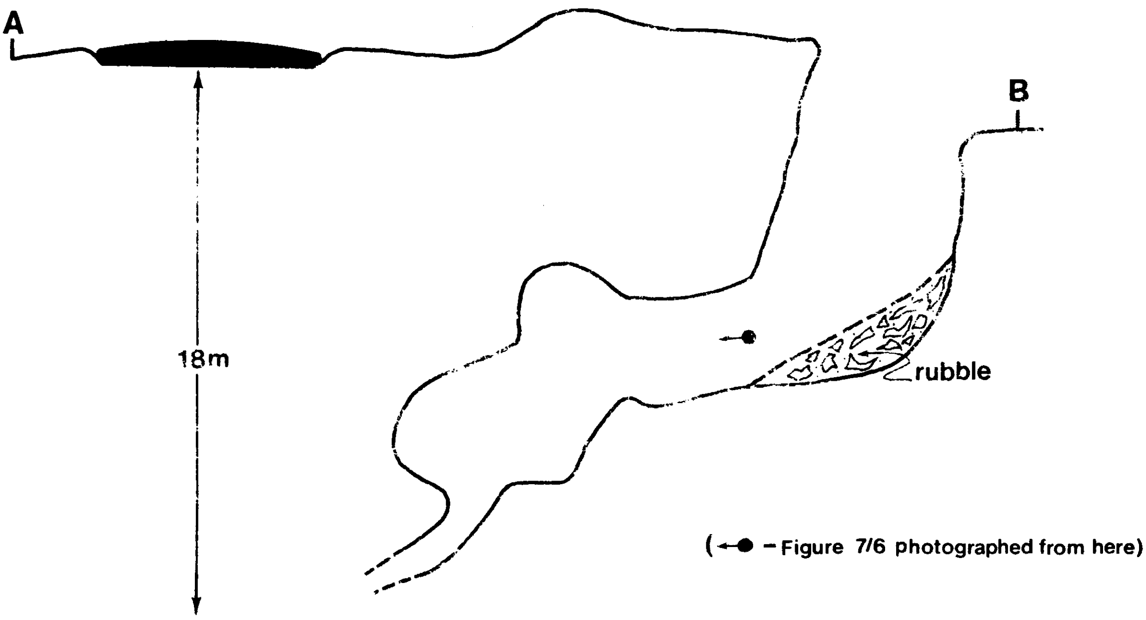
The sinkhole developed at the blind end of a shallow elongated drainage channel in which water flowed during periods of heavy rain. The channel can be seen at the top centre of the photograph in Figure 7/8.

It is possible that a 'paleo-sinkhole' was formerly present at the same spot, and that the 1967 event represented a re-opening of the sinkhole. During 1962 the owner of the farm had drilled, unsuccessfully, for water on the very spot where the sinkhole subsequently appeared. During this time there was a heavy rainstorm and the drilling rig disappeared into a hole 'the size of a motor car'. The rig was subsequently recovered and the hole was filled in with stones. During subsequent visits to the

FIGURE 7/4 : Entrance to sinkhole in gravel pit next to Johannesburg – Durban Road, 20 km north-west of Heidelberg



PLAN



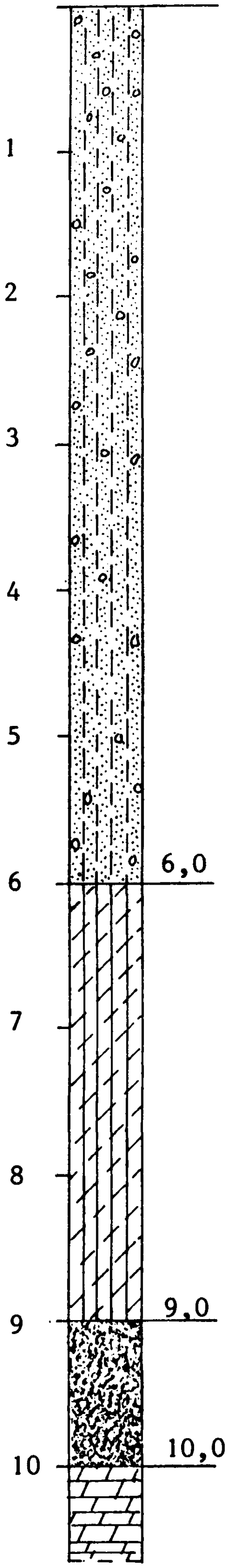
SECTION ON A – B

SCALE 1 : 475 ±

FIGURE 7/5 : Plan and section through sinkhole and subsurface cavern near the middle of the Natalspruit Dolomite Outlier, next to Johannesburg-Durban Road, 20 km north-west of Heidelberg

FIGURE 7/6 : Entrance to cavern from bottom of sinkhole in gravel pit next to Johannesburg —Durban Road, 20 km north-west of Heidelberg

JOHANNESBURG-DURBAN ROAD (T3-11)



Subangular, coarse, medium and fine chert gravels in a matrix of reddish brown silty sand; residual chert rubble.

Grey mottled yellow, firm to stiff, clayey silt; residual diabase. (X-ray analysis showed the presence of kaolinite and quartz).

Wet, very soft, wad; residual dolomite.

Dolomite pinnacles (with debris, wad and voids between pinnacles). Veins of talcose material in the dolomite.

FIGURE 7/7 : Schematic soil profile of western face of sinkhole next to Johannesburg–Durban Road from surface down to level of cavern entrance

FIGURE 7/8 : Large isolated sinkhole on the farm Vogelstruisbult near Kuruman

Note drainage channel disgorging into sinkhole at top centre of photograph area the owner became convinced that the ground was hollow underneath, as he heard 'rumbling echoes'. The final collapse took place during a period of heavy rains when the drought broke in the summer of 1967.

Sinkhole in Verwoerdburg

In 1970, three workmen lost their lives while attempting to stabilise the soils at the bottom of a sinkhole by means of vibration compactors. The sinkhole had formed in a suburban street as a result of leakage from a water main. A further subsidence of the unconsolidated materials at the bottom of the hole during remedial treatment resulted in the interment of the workmen.

FAR WEST RAND

The main claim to fame of the Far West Rand is that the mines in this area produce 20 per cent of the annual production of gold in the world, and that its West Driefontein Mine is the largest single producer of gold in the world.

Also, during the past fifteen years the Far West Rand has been the scene of the most dramatic occurrences of sinkholes, dolines and related phenomena anywhere in the world. But before recording the most significant of these occurrences it is necessary to describe in some detail the geological setting within which they developed and, indeed, are continuing to develop. This will be done be describing the local stratigraphy and its influence on the surface topography, followed by a geological history and, finally, a description of the significant structural and geohydrological characteristics of the area.

General Sequence of Strata

The surface geology of the West Wits Line is shown in Figure 7/9. The 'sub-Transvaal geology', as it would appear if the cover of younger rocks down to the base of the Transvaal System were removed, is shown in Figure 7/10. Both of these maps are based on the work of W.P. de Kock (1964).

From the oldest to the youngest, the following geological formations are represented in the area:

1. The Lower Division of the Witwatersrand System, outcropping along the Hartebeesfontein Anticline (an extension of the West Rand Anticline) in the north-eastern corner of the area covered by the maps, and consisting of slates and quartzites with conglomerate bands and a lava bed (the Jeppestown Amygdaloid).

2. The Upper Division of the Witwatersrand System, consisting of quartzites with auriferous conglomerate bands and a bed of sandy slate (the Kimberley shale).

3. The Ventersdorp System consisting of amygdaloidal lava, volcanic breccia and tuff, with subsidiary sediments.

4. The Black Reef Series (at the base of the Transvaal System) consisting of conglomerate, grit, quartzite, carbonaceous shales and lenses of impure dolomite and wad, the whole succession varying in thickness from 6 to 30 metres.

5. The Dolomite Series (middle member of the Transvaal System) occupying most of the northern half of Figure 7/9. As described above, the series consists of a maximum of some 1 200 metres of dolomite with interbedded chert bands.

6. The Pretoria Series (top member of the Transvaal System) covering the southern half of Figure 7/9. The series consists of a number of thick quartzite and shale bands with one thick bed of andesitic lava (the Ongeluk lava); the succession has been intruded by a number of remarkably persistent diabase sills.

7. Syenite and diabase dykes of Pilanesberg age, trending slightly east of north and forming impervious barriers to underground water, which divide the dolomite of the Wonderfontein Valley into a number of ground-water compartments. The width of the dykes varies from about 6 to 60 metres; the Bank dyke, which is vertical, is 45 metres wide.

8. The Karoo System represented by a number of outliers occupying depressions in the dolomite south of the Wonderfontein Spruit, and consisting of tillite, shale and lenticular seams of coal. (The nature of the outlier on which the Driefontein Brickworks is situated is described in greater detail in Case History 18).

9. A thin surface layer of naturally transported soils of Pleistocene and Recent age, consisting of brown peat and alluvial gravels deposited by the Wonderfontein Spruit, and red surface sands, mainly hillwash derived from the quartzite of the Pretoria Series, but possibly locally redistributed by wind action. These surface materials, together with the upper horizons of chert rubble formed as a residue from the weathering of dolomite, are in places extensively cemented into a discontinuous blanket of hardpan ferricrete.

Topography

The relationship between topography and geology in the Far West Rand is illustrated in Figure 7/11, which is a diagrammatic cross-section from north to south through the area, with a greatly exaggerated vertical scale. It will be seen that the Wonderfontein pediment, the flat area flanking the course of the Wonderfontein Spruit, has been carved entirely within the wedge of gently dipping dolomite. The upper strata of the Dolomite Series contain much hard and insoluble chert which, together with the uppermost layer of chert breccia known as the Fountains Formation (or formerly as the Giant Chert), forms the low hills to the south of the pediment known as the Gatsrante. Resistant quartzites of the Pretoria Series form the Gatsrand range immediately to the south of the Gatsrante. The low rise to the north of the Wonderfontein pediment is formed by the Hartebeesfontein Anticline on which a thin veneer of resistant Black Reef rocks cover the Basement-granite.

Geological History

In order properly to grasp the nature and significance of the geological factors affecting movements on the surface associated with the dewatering of the dolomite compartments, it is necessary to understand the geological events which gave rise to the stratigraphic succession in which these movements have become manifest. The following geological history and cross-sections are based very largely on the work of Eriksson (1971).

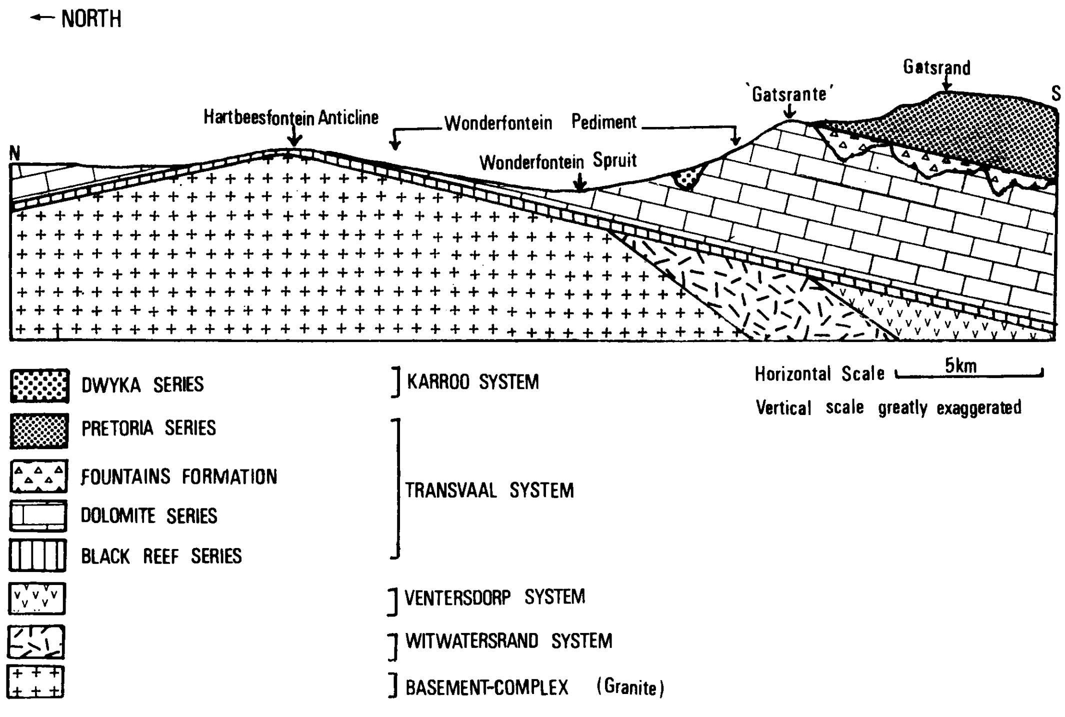


FIGURE 7/11 : Diagrammatic cross-section through Far West Rand area showing relationship between geology and topography

The oldest rocks in the area are granites of the Basement-complex which forms the floor of the oval, synclinal basin into which were washed sands, muds and gravels that were to form the strata of the Witwatersrand System. As shown in Figure 7/12, the basin extended from Witbank to Bloemhof, a distance of some 560 km, and from Rustenburg to Senekal, about 280 km.

In the Far West Rand area the Witwatersrand strata were tilted by tectonic movements and partly planed off by erosion before lavas were extruded onto them in Ventersdorp times (Figure 7/13):

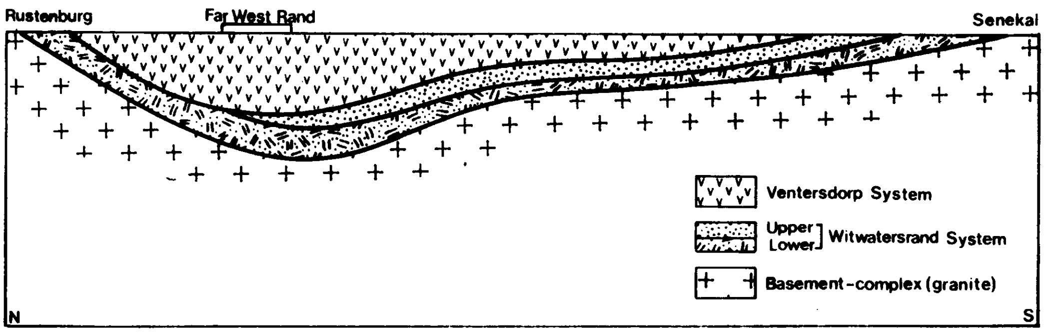


FIGURE 7/13 : North–south cross-section through Witwatersrand Basin in Ventersdorp times

Uplift of the Johannesburg-Pretoria Dome and the Hartebeesfontein Anticline which took place in post-Ventersdorp and pre-Transvaal times, again produced local tilting of the rocks (Figure 7/14):

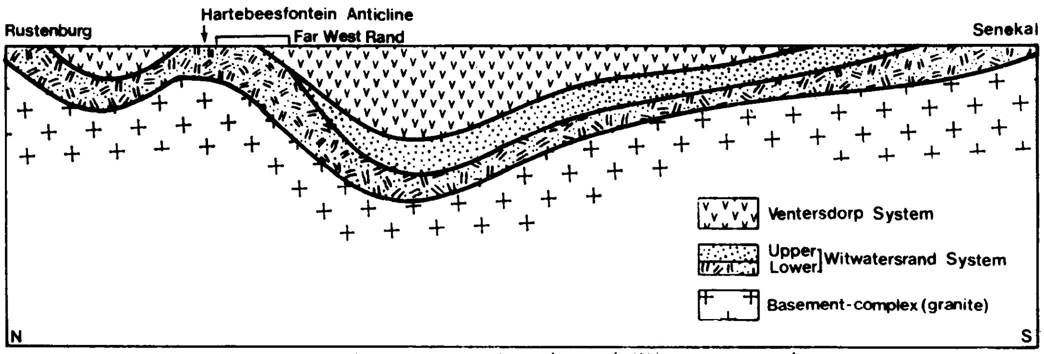


FIGURE 7/14 : North–south cross-section through Witwatersrand Basin in post-Ventersdorp times

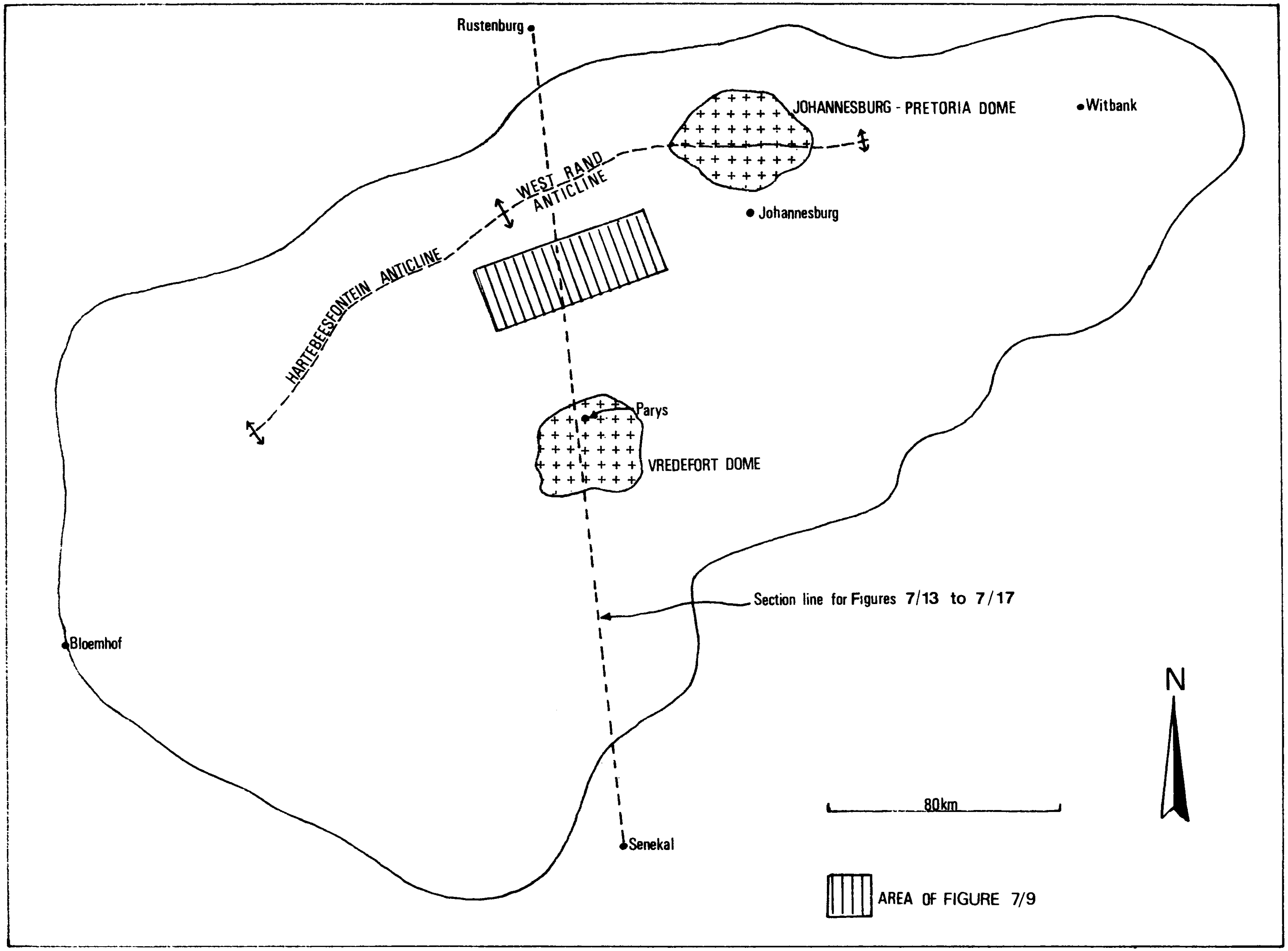


FIGURE 7/12 : Outline of Witwatersrand Basin showing location of Far West Rand

(After Brock and Pretorius, 1964)

The major period of erosion which ensued resulted in the development of the large 'Transvaal' basin, extending from Senekal to north of Pietersburg. Detritus and sediment which was to form the Black Reef Series were laid down in this basin some 2 300 million years ago. The waters occupying the basin were rich in bicarbonates and silica which had been leached from the decomposed rocks of the Basement-complex and the Ventersdorp lava.

Deposition of the Dolomite Series from these waters took place by chemical and organic (algal) precipitation. The algae responsible for much of the precipitation are presented in the rock today by their fossilised stromatolitic structures. Limestone is considered to have been the original precipitate and the dolomite and chert represent secondary replacement of the limestone. As may be seen from Figure 7/2, chert is present throughout the stratigraphic column except in the Lower and Upper Dark Dolomite Zones and in the Columnar Stromatolite Zone.

The attitude of this thick succession of chemical and biochemical sediments in relation to the older stratigraphic units is shown in Figure 7/15:

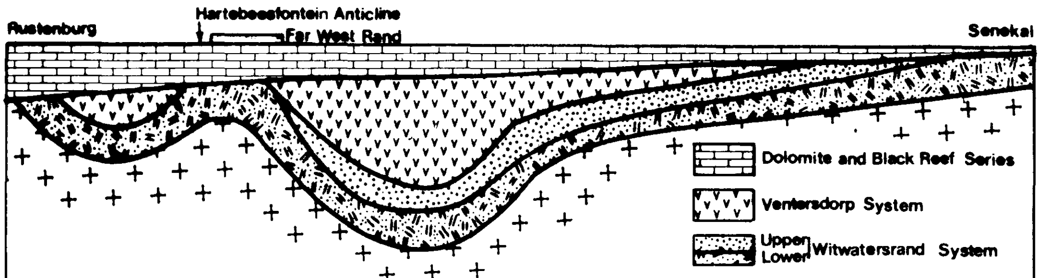


FIGURE 7/15 : North–south cross-section through Witwatersrand Basin during Dolomite times

After deposition of the Dolomite Series, but before the deposition of the sediments of the Pretoria Series, major uplift of the Johannesburg-Pretoria Dome and the Hartebeesfontein Anticline took place. A major period of erosion ensued during which vast amounts of dolomite were taken into solution while the insoluble chert and other substances were residually concentrated, later to form the brecciated chert zone (Fountains Formation) upon which sediments of the Pretoria Series were deposited (Figure 7/16):

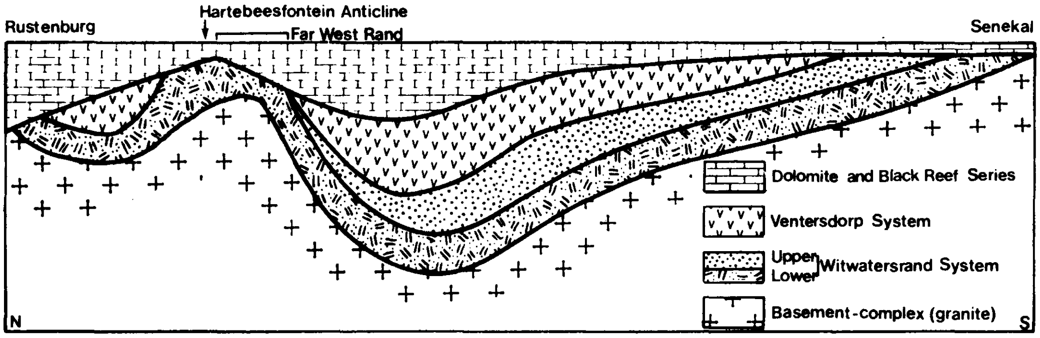


FIGURE 7/16 : North–south cross-section through Witwatersrand Basin in post-Dolomite times

Following the deposition of the Pretoria Series, further reactivation of the Johannesburg-Pretoria Dome and the Hartebeesfontein Anticline took place, this time accompanied by uplift of the Vredefort Dome. An ensuing period of weathering and erosion, lasting for about 1 600 million years, resulted in the removal of vast amounts of rock matter from the Transvaal System in this area, leaving a distribution of the formation much as we find it today (Figure 7/17):

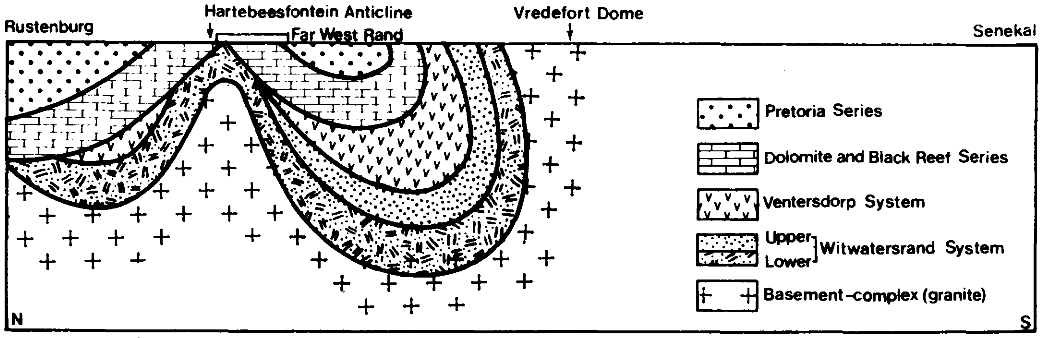


FIGURE 7/17 : North–south cross-section through Witwatersrand Basin in pre-Karoo times

The topography of the Far West Rand was at this stage probably very similar to the present topography, with the Gatsrante flanked to the south by a plateau formed by rocks of the Pretoria Series, and to the north by the broad pediment developed in the dolomite and extending across the alluvial plain of the Wonderfontein Spruit to the granite ridge capped by hard rocks of the Black Reef, which probably even then represented the surface of the Hartebeesfontein Anticline. Being a soluble rock, the dolomite had been severely weathered during this period and, as will be discussed more fully later, the weathering extended to great depths along planes of easy access to water such as fault-planes and fracture-zones. Thus the dolomite contained deep subsurface valleys which were choked with chert rubble and other insoluble residue from the weathering of dolomite. Indeed such deep buried chasms, formed within more recent times, are again characteristic features of the dolomite of this area today.

During the Carboniferous period the widespread ice-sheet glaciation resulted in scouring away of surface soil and soft rock. In the Far West Rand area, chert rubble occupying the subsurface valleys along fault-zones would have been particularly susceptible to removal by glacial scouring. When the ice finally melted, the cleanly scoured and steep-sided valleys and depressions in the dolomite became the receptacles for glacial debris or till. It is for this reason that we now find Dwyka Tillite preserved at the bottom of some of the deeper ice-scoured chasms in the dolomite.

Following the glacial episode, a very large area of the interior plateau basin of Southern Africa again became subjected to depositional conditions which resulted in the laying down of the Karoo strata. Since the Far West Rand lay along the northern flank of the main depositional basin, the deposition of Karoo strata here was sporadic and thin. Slow accumulation of fine-grained sediments, under a humid, tropical environment which supported a lush flora, resulted in the formation of an interbedded succession of shales and coal seams during Upper Dwyka, and later during Ecca times. These strata were deposited preferentially in the chasms and depressions, some of which were already partially occupied by tillite.

Various cycles of erosion in post-Karoo times led to renewed planation of the area and the formation of an extensive pediment cutting uniformly across both dolomite and Karoo strata in the area flanking the Wonderfontein Spruit (Brink and Partridge, 1965). The remnants of Karoo strata that are left as outliers would seem to have moved progressively downwards into sockets produced by continuous weathering of the dolomite. This is evidenced by the fairly steep angles of dip to be seen, for example, in the Driefontein quarries (Figure 7/18), and by the logs of 'continuous sample boreholes' BH 1 and BH 2 at Driefontein, in which dips of up to 45^0 were recorded (see Case History 18).

Faults and Fractures in the Dolomite

In Figure 7/10, De Kock shows the major faults which affect the disposition of the Witwatersrand beds. From east to west these are the Panvlakte Fault, the Witpoortjie Fault, the Bank Break bounded by faults on its eastern and western limbs, and the Mooi River fault situated west of the area covered by Figure 7/10. It will be seen that these major faults trend approximately north-south, parallel to the tension-fractures which are now occupied by the dykes shown in Figure 7/9.

De Kock (1964) states that some of the ".... pre-Transvaal faults have manifested themselves in post-Transvaal times but seldom to any marked degree". A notable exception is the Bank Fault which not only displaces the Witwatersrand System horizontally by some 8 km as can be seen from Figure 7/10 but has also been active in post-Transvaal times. This fault displaces Timeball Hill quartzite on the Gatsrand as well as rocks of the Dolomite Series as it extends northwards under the Driefontein Brickworks. From the nature of the displacements, it is clear that the upthrown block lies on the eastern side of the fault. It must be pointed out, however, that more than one fault plane is concerned here. It has been observed by Pelletier (1937) that the "Bank Fault is in all probability a zone of faulting comprising a number of parallel dislocations and not a simple break". Indeed exploration boreholes and gravimetric surveys have confirmed that we are dealing here with *en echelon* faulting within the dolomites, having a north-south orientation.*

The dolomite has suffered other intensive faulting and fracturing in

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* Personal communication from J.F. Wolmarans, Chief Geologist, Gold Fields of South Africa Limited (1976)

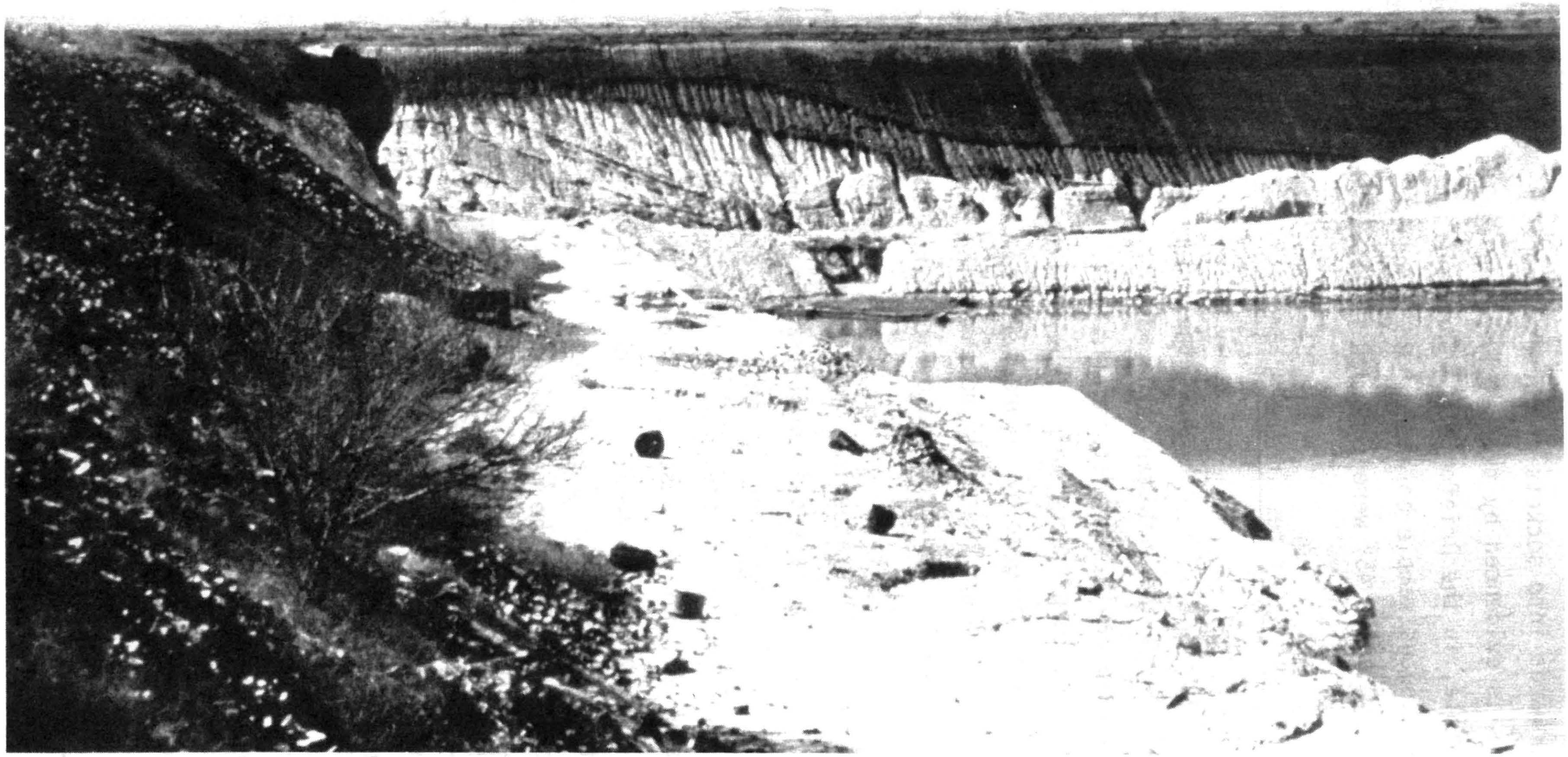

FIGURE 7/18 : Dip of strata in Karoo outlier within the Dolomite Series at Driefontein Brickworks addition to the pre-Transvaal faults which were reactivated to greater or lesser extents in post-Transvaal times. On most of the post-Transvaal faults there has been only minor displacement, but it is faults of this type that penetrate right down through the dolomite and into the underlying Witwatersrand rocks where they present a water hazard in the mine workings (De Kock, 1964).

Three dominant fault directions, represented by subsurface valleys, have been recognised by Dr O.L. Papendorf using gravimetric data from surveys conducted by the Geological Survey. The first of these directions, termed the Bank 'Break', is illustrated on the strike frequency diagram (Figure 7/19) and trends 10^0 to just over 20^0 west of north. A second set of fractures varies in strike direction from 10^0 to 45^0 east of north, but most frequently strikes 20^0 to 40^0 east of north. This set of fractures, termed the Witpoortjie 'Parallel', is thus roughly 50^0 to the Bank 'Break' with a maximum value of just over 60^0. An intermediate set of valleys strikes mainly 5^0 to 10^0 east of north. Noteworthy is the extent of leaching in these valleys as is evidenced by the large numbers of sinkholes and depressions associated with them.

From the above discussion, and with reference to Figure 7/19, it can be concluded that the faults, fractures and joints tend largely to be arranged at approximately 60^0 to one another. These directions of fracturing are consistent with the chief stress directions involved in the margin of the dolomite basin of this region, consequent upon the rise of the Hartebeesfontein Anticline (Figure 7/17) and are compatible with a stress field in which the major principal stress acted in a roughly N 10^0 E direction. This conclusion accords with that of L.F. Pienaar (1971) based on very extensive and thorough field work in the area.

As will be seen from Figure 7/9, some of the major tension fractures which are parallel to the N 10^0 E direction of major principal stress are occupied by dykes. This direction also coincides with the intermediate set of valleys (Figure 7/19), while the concentration of sinkholes provides further evidence of this being a direction of tension.

Geohydrology of the area

Reference has been made repeatedly to the "compartments" in the dolomites of the Far West Rand; a more detailed explanation of the nature

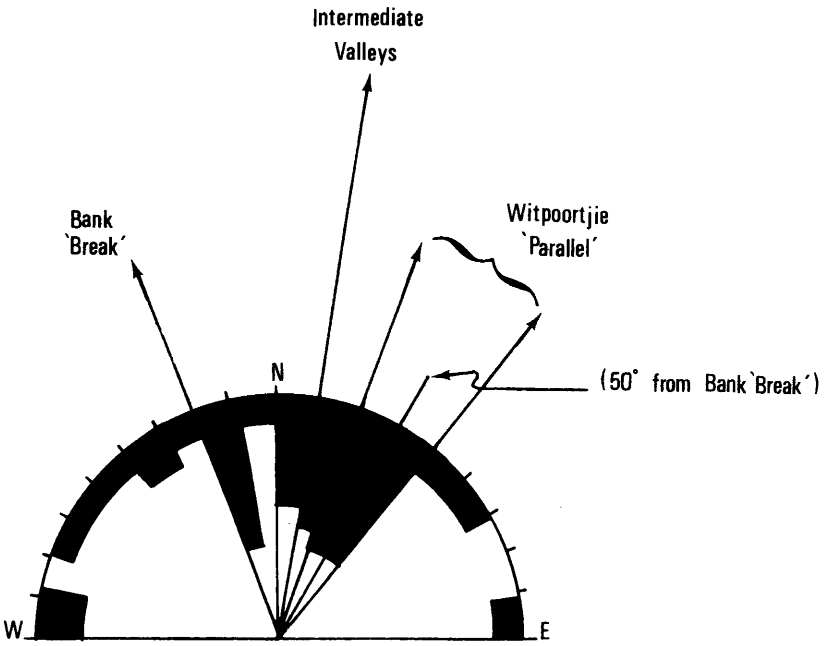

FIGURE 7/19 : Strike frequency diagram of faults in the Far West Rand

From a compilation by Dr O.L. Papendorf, 1971 of these compartments follows. It has already been stated that a number of igneous intrusions, in the form of dykes trending slightly east of north, divide the dolomite beneath the Wonderfontein pediment into a number of ground-water compartments - it should now be added that the Basement-granite of the Hartebeesfontein Anticline forms the ground-water barrier at the northern end of these compartments, and that the rocks of the Pretoria Series overlie the compartments to the south. Ground-water is contained in each compartment in a network of inter-connected caverns and faults and joints widened by solution, some of which are also partially occupied by insoluble chert, wad, etc. The natural level of the water-table in each compartment tends to be nearly horizontal, and is controlled by the level of the spring, or 'eye' which emerges at the surface near where the Wonderfontein Spruit intersects the dyke on the western side of each compartment. This situation has aptly been likened to a "gigantic freshly filled and tilted ice-tray" (Schwartz and Midgley, 1975).

When mining started in the area in the early 1930's, difficulty was experienced in dealing with the vast quantities of water in the cavernous dolomite through which the shafts had to be sunk down to the underlying auriferous conglomerates. Initially unlined shafts were sunk through the dolomite in the Venterspost Compartment while the water-bearing fissures were being sealed in advance at progressive depths by means of cementation (Allen and Crawhall, 1937). Shafts sunk in the early 1950's in that portion of the West Driefontein Mine situated within the Oberholzer Compartment (see Figure 7/21) were lined. There was no conscious attempt during these initial stages to 'dewater' the compartments: the mines were equipped with large pumps capable of dealing effectively with water that found its way through unsealed fissures into the underlying drives and stopes.

As pointed out by Cousens and Garrett (1969) water would not have proved an embarrassment to mining operations, other than in the sinking of shafts through the dolomite, if it were not for the presence of tension-faults (of post-Transvaal age) which cut through the dolomite and down into the Witwatersrand rocks. These faults act as channelways connecting the water reservoirs in the dolomites with the mine workings below. Within the Venterspost Compartment such tension-faults are more numerous in the Venterspost area than in the Libanon area: in the Oberholzer Compartment they are abundant in the West Driefontein area (see Table 7.2 later).

Eventually the Venterspost Mine, and later the West Driefontein and other mining companies, had to adopt the policy of deliberately dewatering the compartments and disposing of the vast quantities of water via lined canals traversing their own or neighbouring compartments and proceeding to irrigation areas or elsewhere as advised by the Department of Water Affairs.* The Venterspost Compartment was the first to be dewatered in this way: active pumping from the Venterspost Mine commenced in 1935/36 after serious inrushes of water containing wad and other materials had been experienced in the shafts during 1934/5. Water pumped from the Venterspost Compartment was initially discharged into the Wonderfontein Spruit in the adjacent Bank Compartment and, as progressively larger quantities of water had to be dealt with when development extended along strike in the underlying workings, up to 45 megalitres were being pumped out daily by the late 1940's and a maximum of 56,8 megalitres by the early 1960's. With the consequent lowering of the water-table the amount of water that had to be pumped out decreased after this time, and the present rate of pumping is only about 30 megalitres per day, which is about 30 per cent more than the yield of the Venterspost eye when it was still flowing.

The Oberholzer Compartment was the next to be dewatered (early 1960's) followed, more recently, by commencement of dewatering in the Bank Compartment (early 1970's). The ground-water levels in each of the three compartments as in 1966 (prior to the commencement of dewatering in the Bank Compartment), are shown in Figure 7/20, which is a diagrammatic cross-section along the Wonderfontein Spruit based on published work by Enslin (1967), Enslin and Kriel (1968) and Bezuidenhout and Enslin (1969). Figure 7/21, showing ground-water contours in the compartments as in 1966, is based on the publication by Enslin and Kriel (1968).

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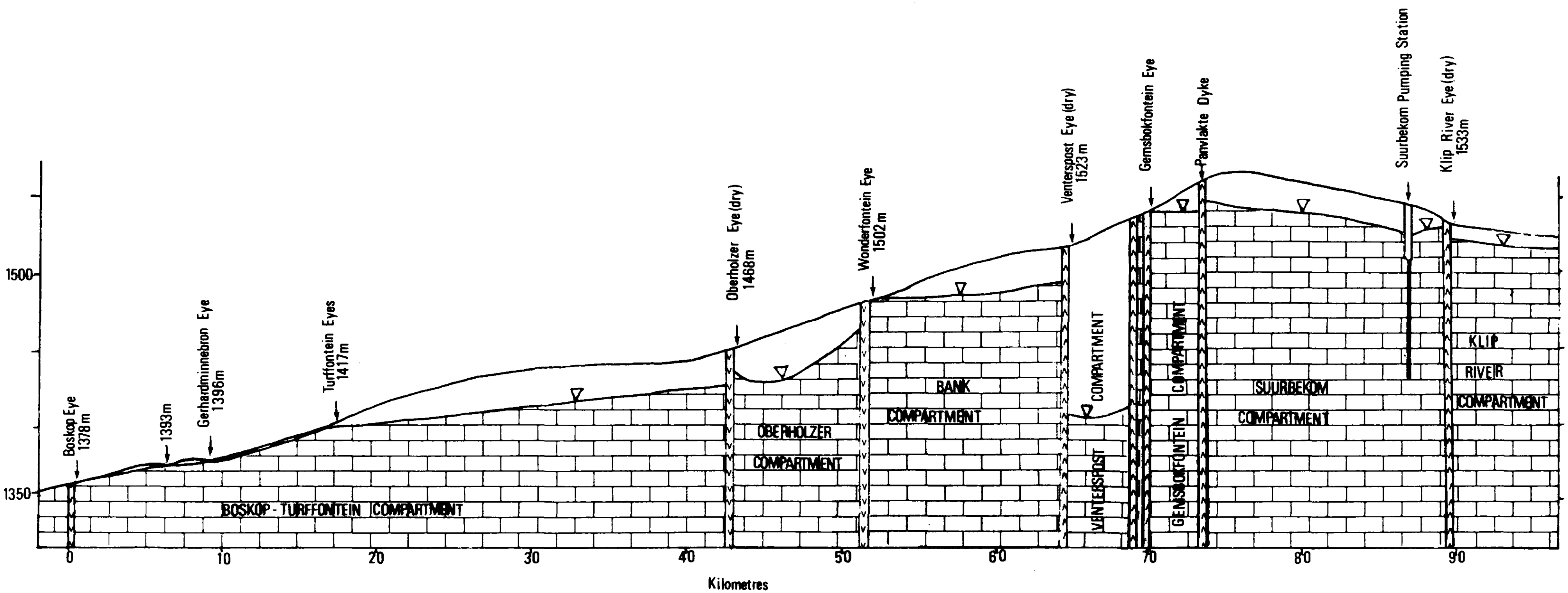
* The terms 'dewatering' and 'dewatered compartment' should be defined here. Dewatering starts the moment the quantity pumped from underground exceeds the natural rate of replenishment. The compartments are never completely dewatered. The term 'dewatered compartment' is applied when the rate of ingress of water into the mine workings has diminished to a quantity equal to the rate of replenishment. (Personal communication from J.F. Wolmarans, 1976).



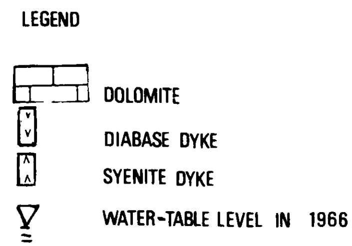
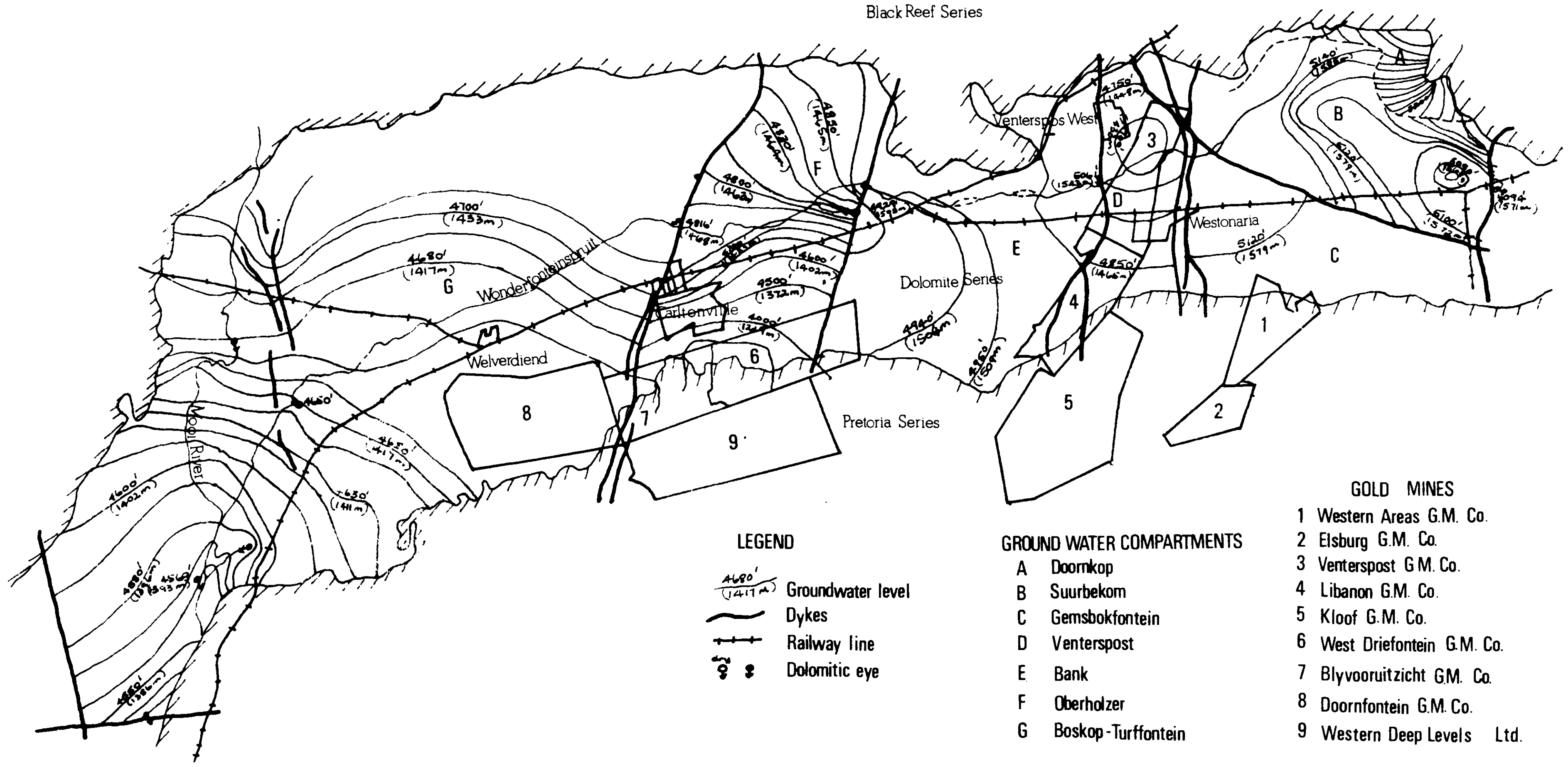


FIGURE 7.20 Geohydrological section through dolomitic compartments of the Far West Rand as in 1966

(After Enslin and Kriel, 1968)



SCALE 5 miles 8 km

FIGURE 7/21 : Ground-water contours in dolomitic compartments of the Far West Rand as in 1966

(After Enslin and Kriel, 1968)

Some salient details about the three dolomitic compartments which have been affected by large-scale pumping are given in Table 7.1 below: the eyes of all three compartments are, of course, now dry - their average daily yields refer to the period prior to the commencement of large-scale pumping.

NAME OF COMPARTMENT (AND ITS EYE)	SURFACE AREA OF COMPARTMENT IN SQUARE KILOMETRES	ELEVATION OF EYE ABOVE SEA LEVEL IN METRES	AVERAGE YIELD OF EYE IN MEGALITRES PER DAY
Venterspost (Venterspost Eye)	54,4	1 540	20,9
Bank (Wonderfontein Eye)	156,7	1 502	49,1
Oberholzer (Oberholzer Eye)	153,8	1 468	54,1

Table 7.1 : Average yields of eyes on the Far West Rand

Data concerning the maximum quantities of water pumped by mines on the Far West Rand, based largely on published records of Cousins and Garrett (1969) and of Taute and Tress (1971) are given below in Table 7.2. It will be seen that the decade of maximum dewatering was from 1960 to 1970:

COMPARTMENT	MINE	MAXIMUM QUANTITY PUMPED (in mega-litres per day)	YEAR
Venterspost	Venterspost	56,8	1961
Venterspost and Bank	Libanon	13,0	1968
Oberholzer	West Driefontein Blyvooruitsig (recirculating)	145,5 60 ±	1963 1961
Boskop-Turffontein	Doornfontein	15,9	1960
Bank	East & West Driefontein	340,0	1970

Table 7.2: Maximum quantities of water pumped from their workings by certain mines on the Far West Rand

It will also be seen that the rate of pumping from the Venterspost Compartment in 1961 was more than doubled in the case of the Oberholzer Compartment in 1963, and that that figure was more than doubled again in the case of the Bank Compartment in 1970.

The average annual rainfall in the area is 660 mm, of which about 10 per cent, or approximately 66 megalitres per square kilometre per annum, percolates through to the water-table (Cousens and Garrett, 1969). It has been estimated by Schwartz and Midgley (1975) that the capacity of the Bank Compartment is 2 million megalitres (i.e. roughly ten times the capacity of Haartebeestpoort Dam and nearly equal to the capacity of Vaaldam). While the writer has produced evidence to indicate that this may be an overestimate (Brink, 1975), there is little doubt that their figure is of the right order of magnitude. This illustrates the contention of Enslin and Kriel (1968) that "large quantities of water are stored in the unconsolidated material, in particular where the dolomite has been leached extensively and these materials fill 'geological valleys' which have been found to extend to depths as much as 200 metres below the surface".

At this stage, and against this geological background, it would be appropriate briefly to outline the salient events in the more recent history of the area relating to the accelerated development of karst features which have a surface manifestation. This will be followed by an account of the mechanisms responsible for the development of solution cavities in the dolomite, and of sinkholes and dolines in the overlying soils.

<u>Sequence of events relating to mining and the accelerated development of sinkholes and dolines in the Far West Rand</u>

1910 : The Pullinger brothers attempted to sink a shaft through the dolomites in the Venterspost Compartment not far from the Wonderfontein Spruit. The shaft was lined with cast-iron casings, 5 m in diameter, imported from Germany. By the time a depth of 23 metres had been reached, the steam pumps were no longer capable of dealing with the inrush of water. By controlling the ingress of water with the use of two electric pumps the shaft was later taken down to 30 m, but even these pumps were 136 kilolitres per hour below the required capacity. The shaft became flooded and had to be abandoned (Allen and Crawhall, 1937). For many years it was used as the main local source of water, and it is still in existence today.

1934	:	The Venterspost Gold Mining Company Limited was inaugurated and two shafts were sunk in the Venterspost Compartment, using cementation to seal water-bearing fissures and to stabilise deposits of mud and wad in the upper zones of the dolomite (Allen and Crawhall, 1937).
1935 - 1949	:	Mining continued in the Venterspost Compartment, with sealing of water-bearing fissures where necessary.
Early 1950's	:	The first attempts at lowering the water-table in the Venterspost Compartment were made by sinking three wells in the bed of the Wonderfontein Spruit. Lined shafts were sunk through the dolomite in the Oberholzer Compartment.
1957 - 1959	:	Water was pumped from the Oberholzer Compartment by the West Driefontein Mine at a rate of 84 megalitres per day, which was equivalent to the rate of inflow into the mine workings and 30 megalitres per day more than the average yield of the Oberholzer eye.
1960	:	Workings at the West Driefontein Mine were extended along strike, causing a greater inflow of water. As a result 145,5 megalitres had to be pumped out daily. This water was led out into the adjacent Boskop-Turffontein Compartment to the west via a concrete-lined canal. The water-table was systematically lowered and the Oberholzer eye stopped flowing.
12th December 1962 (6.10 a.m.)	:	A three-storey crusher plant at the West Driefontein Mine was engulfed by the formation of a sinkhole, 55 m in diameter and at least 30 m deep, which claimed the lives of 29 people (Figure 7/22). No trace of the building or its inhabitants was ever seen again. There was practically no warning: the building had shown only very small movements (not uncommon on the dolomites) and the collapse took

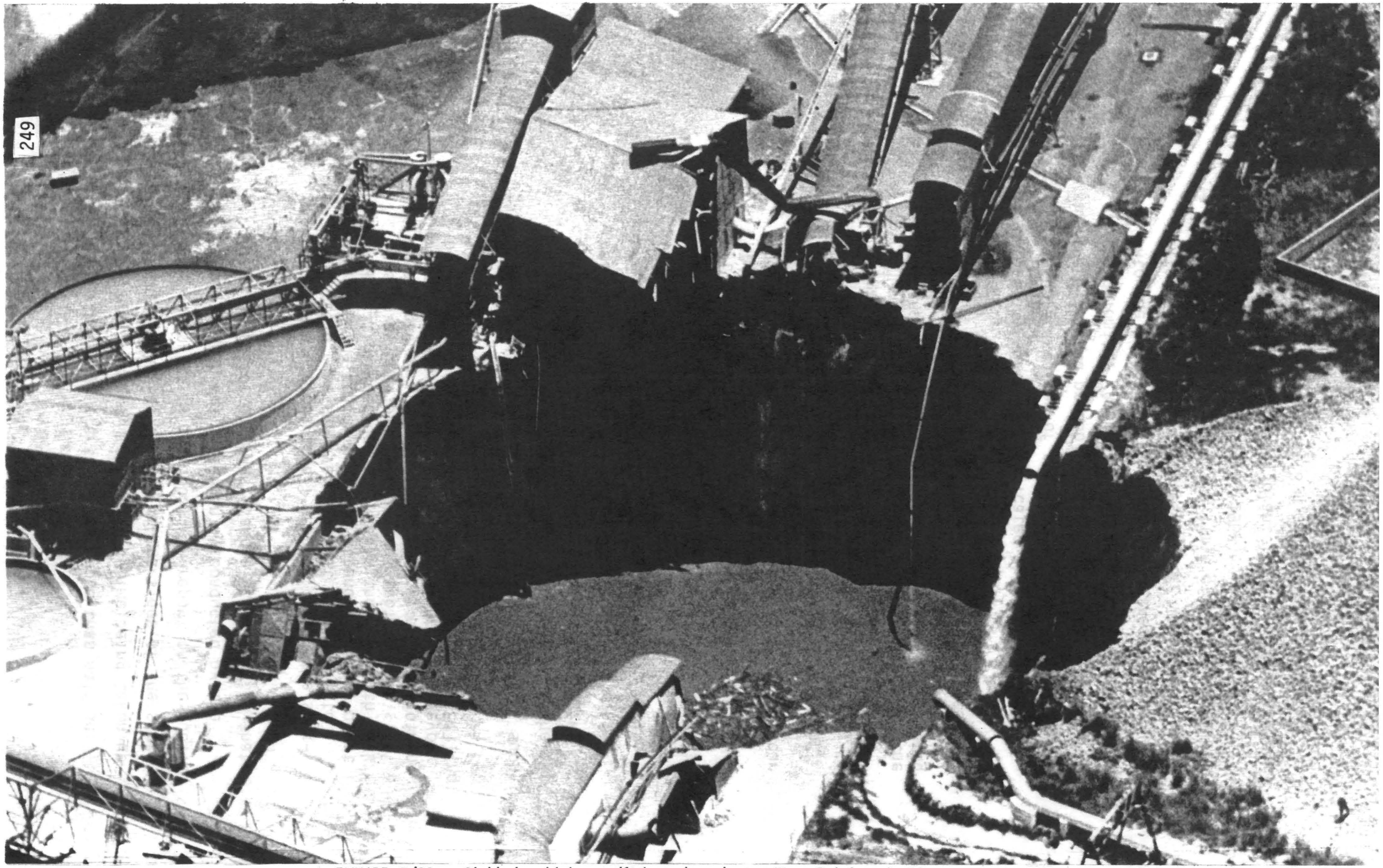


FIGURE 7/22 : Sinkhole which engulfed crusher plant at West Driefontein Mine on 12.12.1962

(Note that rupture of the water pumping main from the mine shaft was a consequence and not a cause of the sinkhole) place suddenly. Certainly not more than half an hour was involved, most of this time being taken up in the further falling-in of the sides of the sinkhole (Jennings *et al*, 1965). A report on the public enquiry which was subsequently conducted by the Chief Inspector of Mines (Bennie, 1963) states that:

"As it was obvious to all than no one in the sinkhole could have survived and that any effort to retrieve the bodies would be attended by very great risks, it was decided to fill in the hole with rock as soon as possible in order to stabilise the still moving ground. Broken waste rock from the adjoining rock dump was loaded by bulldozers onto a conveyor belt erected to discharge into the sinkhole. Filling was completed by 10th Janaury 1963."

Discussing the possible causes of the sinkhole, such as lowering of the water-table, the weight of the three-storey building and plant, vibration of the heavy machinery, and tremors resulting from mining operations, the same report includes the following interesting observation:

"It is known that the water-table in dolomitic areas fluctuates several inches twice daily due to lunar and solar attraction, in the same way as tides occur in the sea. It seems likely that the effect on a large mass of ground such as that forming the roof of the sinkhole in question, could precipitate a collapse when the combined effect is at its greatest or least. It might be a coincidence that it was full moon the day prior to the accident and that the moon and sun were diametrically opposed, one setting and one rising, at the time of the accident resulting in a minimum of vertical attraction by these heavenly bodies."

A gravity survey had indicated that there was a subsurface valley of substantial magnitude beneath the plant and probably coincident with a fault in the dolomite. A borehole which had been drilled into the buried valley a hundred metres north of the site had disclosed the presence of 'weathered formation' extending beyond the limit of the borehole at a depth of 117 m (Bennie, 1963). Prior to the erection of the crusher plant 171 boreholes had been drilled on the site at 3 m centres and to depths varying from 9 m to 15 m. Cement grout had been injected into these boreholes to consolidate the ground. In addition the whole surface of the area around the plant for a distance of about 60 m had been concreted or tarmacadamised in order to prevent water from seeping into the ground and endangering the building. The enquiry conducted by the Chief Inspector of Mines thus concludes that "the information available to the management, prior to the disaster, in respect of the plant appeared to be insufficient to condemn it. In the circumstances reasonable efforts appear to have been made to secure it and to ascertain whether it was endangered. No blame is attached to any person for the disaster". (Bennie, 1963).

1963 : There was concern at this time about the progressive development of large *dolines* in the Oberholzer Compartment. 'Schutte's Depression' near Carltonville was so named from the fact that Mr Schutte's house, situated within this doline, had to be demolished. The doline was 180 m in diameter and eventually attained a depth of over 8 m (Figure 7/23). Boreholes put down from the bottom of the depression showed the presence of compressible materials extending well below the depth of the original water-table. A larger doline was developing in a residential suburb of Carletonville at Lupin Place. Some twenty houses were situated within the feature. Houses near the centre eventually subsided about 5 metres, but the subsidence was so uniform that the houses were not cracked. Those built within the zone of shear on the periphery of the doline were subjected to extreme differential settlement and were totally destroyed. Boreholes within the doline again showed the presence of compressible materials extending well below the original level of the water-table, but no voids were encountered. While the safety of the houses in the central part of the doline was thus reasonably

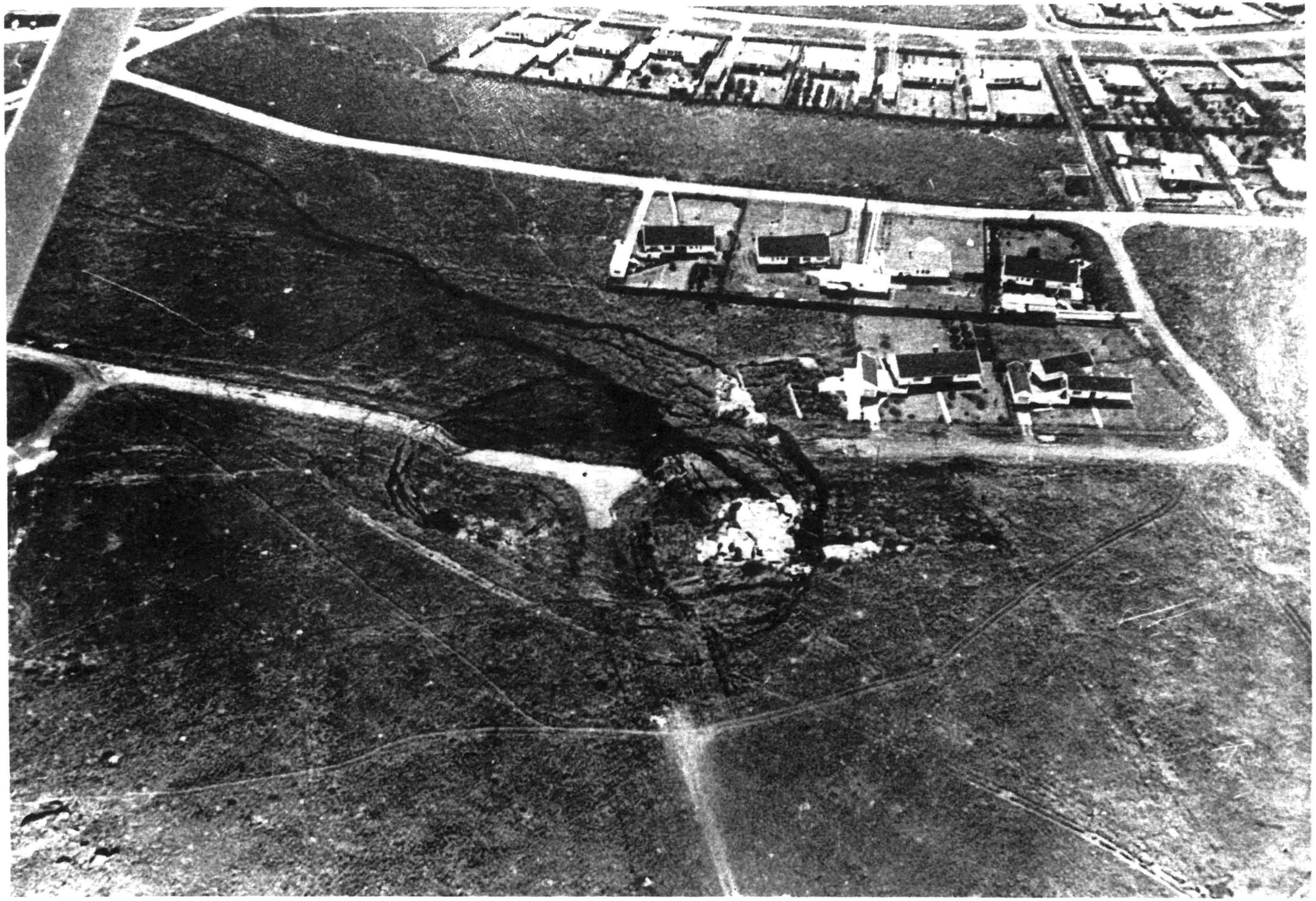


FIGURE 7/23 : Example of doline : Schutte's Depression near Carltonville ensured, progressive difficulty was experienced with the functioning of the sewers and other services leading from the depression. By 1967 all the houses had been demolished and the doline filled with mine slimes.

3rd August 1964 : A sinkhole of slightly larger dimensions than the one which engulfed the West Driefontein crusher plant appeared in a mining village at Blyvooruitzig Mine (Oberholzer Compartment) in the middle of the night and claimed the lives of the Oosthuizen family of five as their house suddenly dropped more than 30 metres. Three other houses which were situated on the edge of the initial collapse also fell in within a short period of time as the sides of the sinkhole caved in, their occupants making dramatic escapes (Figure 7/24). Subsequent enquiry revealed that there had been leakage from water pipes in the area where the sinkhole appeared.

September 1964 : By September, 1964, more than 2,5 million tonnes of waste slimes had been pumped down a sinkhole which had appeared earlier in the West Driefontein slimes dam (Figure 7/25).

October 1964 : During a period of heavy rain a sinkhole 21 m in diameter, fell in to a depth of 15 m in the heart of suburban Westonaria (this time in the Venterspost Compartment) causing alarm among the local inhabitants. No houses were affected. Heavy rains continued for the next few weeks, and cracks opened in the surface of the ground in various parts of the compartment.

1966 : During the period December 1962 to February 1966, eight *large* sinkholes (more than 45 m in diameter and 30 m deep) had opened up in the Oberholzer Compartment within the area where the cone of depression formed in the water-table by pumping from the West Driefontein Mine had reached or exceeded a

FIGURE 7/24 : Sinkhole which swallowed house at Blyvooruitzig Mine on 3.8.1964
(Photo by Die Transvaler)

FIGURE 7/25 : Sinkhole at West Driefontein slimes dam depth of 160 m below surface (Figure 7/26). The largest of these appeared in open veld on Carletonville Extension 8 in February 1966: it was 125 m in diameter and 50 m deep. In addition, 122 smaller sinkholes had opened up elsewhere on the Far West Rand. This latter observation was made by L.F. Pienaar (1971) from a study of sinkhole occurrences on aerial photographs which had been taken in 1948, 1961, 1963 and 1966. In addition to the relatively few sinkholes visible on the 1961 photographs, a further 122 were visible on the 1966 photographs and, of these, 107 had formed between 1961 and 1963. (The frequency of sinkhole development dropped off after 1966, until dewatering commenced in the Bank Compartment towards the end of 1969).

28th January 1966: On this date, the Minister of Mines stated in the House of Assembly that a total of 309 houses had been evacuated and 145 demolished in the Far West Rand since the beginning of 1962. Several commercial and public buildings had also been evacuated or demolished. The total estimated damage to buildings and other structures, including replacement costs, had amounted to R14 035 700, and the installation of protective measures in the area had cost R4 433 000 (*Hansard*).

26th October 1968: An unprecedented inrush of water estimated at 360 megalitres per day, flooded a stope in the eastern (No 4 shaft) section of West Driefontein Mine. This stope was situated below the Bank Compartment, access to the stope having been made via a drive which penetrated the Bank dyke separating the dewatered Oberholzer Compartment from the undewatered Bank Compartment. The inrush of water was largely associated with the 'Big Boy Fault' but was triggered by the settlement of the hanging wall of the stope. The West Driefontein pumps were not capable of coping with the colossal volume of water involved. About 1 200 men were in the affected area of the

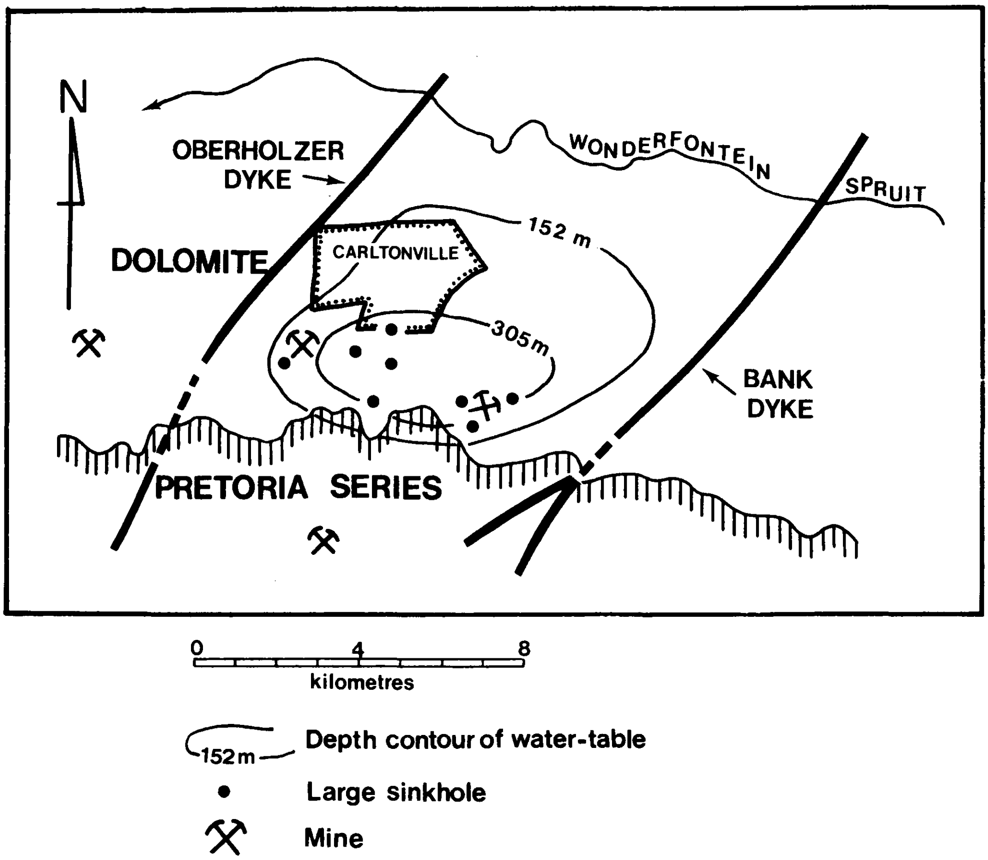


FIGURE 7/26 : *Location of eight large sinkholes which appeared in the Oberholzer Compartment between December 1962 and February 1966 within the cone of depression of the water-table* (After Foose, 1967) mine at the time: miraculously they all escaped safely to surface. After an epic battle had been waged against the raging torrents for 23 days, two concrete plugs were successfully installed on the mine's 10 and 12 Levels, isolating the No 4 shaft area from the western part of the mine, i.e. effectively sealing water from the Bank Compartment and stopping further flow into the Oberholzer Compartment. The valves bleeding water through the plugs were closed off on 18th November 1968, and water in the eastern (Bank Compartment) section of the mine returned to the original level of the water-table in about three days (Cousens and Garrett, 1969; Taute and Tress, 1971). But the Bank eye had stopped flowing.

December 1968 : It was now clear that the Bank Compartment would have to be drained in order to regain entry into the flooded workings within the compartment. The mine requested permission to dewater from the Department of Water Affairs and this was granted on 17th December 1968.

December 1968 - May 1969 : After the experience of surface subsidences following the dewatering of the Oberholzer Compartment, the immediate consideration was the safety of the people living on the Bank Compartment. Consequently, during this period, intensive gravity surveys were conducted to delineate 'safe from less safe' areas. In addition, 2 162 level pegs were established on surface, and monitoring of the level of the water-table was undertaken in 78 boreholes in the compartment.*

December 1969 : Residents in the vicinity of Bank (a town within the compartment), were given one month's notice to

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* Press announcement released on 11th December 1969 by the Department of Information at the request of the Department of Mines.

vacate the area. By the end of January 1970 the area was vacated and one of the two provincial roads traversing the compartment was closed to normal traffic.

January - July 1970 : The total quantity of water pumped out of the Bank Compartment by both the West Driefontein Mine and the East Driefontein Mine during this period rose rapidly month by month to a maximum daily average of 340 megalitres by July 1970.

The inflow and the quantity pumped gradually declined after this date (Taute and Tress, 1971). Serious subsidences of the surface, in the form of sinkholes and dolines, and the opening of tension-cracks in the ground, were observed in various parts of the compartment and particularly in the vicinity of Bank, which more than justified the timely decision to evacuate this part of the compartment. Kilns at the Driefontein Brickworks near the middle of the compartment started cracking badly and had to be abandoned by the end of the year.

24th October 1970: A sinkhole appeared on this Saturday afternoon at the tennis courts of the miners recreation centre at Venterspost, swallowing part of the clubhouse and one spectator, Mr Karl Nortje. Four people playing a game of tennis at the time, and three others in the clubhouse who narrowly escaped, said they heard a rumbling noise like a tremor, followed by a loud crack. The initial tremor was followed by another and there were two further subsidences of the sides of the sinkhole. That night a nightwatchman at the clubhouse heard periodic rumblings throughout the night. It is believed that the formation of the sinkhole was triggered off by an earth-tremor.

January 1971 : The remaining provincial road crossing the Bank Compartment from Carletonville to Westonaria was also closed to normal traffic.

4th April 1971 : A sinkhole appeared in a bowling green at Ventersport recreation centre, 60 m from the tennis court sinkhole. The sinkhole was 26 m deep, and shaped like a bottle-neck: at the surface it was 9 m in diameter but, at a depth of 19 m, it belled out to 15 m in diameter. During a helicopter flight over the area it was seen that this sinkhole and the tennis court one were in a straight line with two further sinkholes which had appeared some time earlier in the open veld near by.

August 1971 : At the end of August 1971, a borehole near the pumping area in the Bank Compartment showed that the water-table had been lowered here by some 300 m. Other boreholes in the western part of the compartment also showed a substantial drop. The water-level in boreholes in the eastern part of the compartment, however, had only dropped an average of 10 m. It was suspected that a semipervious barrier, the exact nature and location of which were unknown, divided the compartment roughly into two halves. However, more recent drawdown contours do not indicate the presence of such a barrier*. Subsidence phenomena on surface continued, and still continue, to move progressively eastwards across the compartment as the water-table drops. As the water is being discharged into the Boskop-Turffontein Compartment, there has been a substantial rise in the water-table in that compartment and the flow from the Turffontein eye has almost doubled from 7 megalitres to 13 megalitres per day (Taute and Tress, 1971). The total cost of dewatering from the Driefontein mines up to this time was over sixteen million rand; to this must be added a sum of nearly twenty-one million rand representing the estimated loss of profits to West Driefontein during 1968/9 as a result of the flooding (Taute and Tress, 1971).

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* Personal communication from J.F. Wolmarans (1976).

Early 1973 : A sinkhole appeared below the southern weir of the Donaldson Dam in the Gemsbokfontein Compartment. This dam is one of the understandably very few dams of any size on the Transvaal dolomites. It had been constructed before mining commenced at Venterspost in the early 1930's, and had ostensibly been sited on the Suurbekom dyke which strikes NW - SE. In fact, however, it would seem that this dyke lies just to the east of the dam wall. Leakage problems had been experienced earlier at the dam and a clay lining had been laid within it. The sinkhole, about 10 m across and 7 m deep was subsequently filled with waste rock from a near-by mine.* It should be mentioned in passing that the Boskop Dam, which receives, via the Mooi River, the water which has been discharged in the Gemsbok-Turffontein Compartment, is invulnerable: it is situated on the Pretoria Series directly above a dyke which is intrusive into the underlying Dolomite Series.

9th April 1975 : The latest episode in this chronicle of dramatic events concerns the appearance of a sinkhole 20 m wide and 7 m deep, under the railway line near Bank. The driver of a moving passenger train spotted the hole too late to stop the train. Three coaches were derailed and two passenger coaches were left suspended on the rails above the sinkhole. Passengers fled from the train unhurt. The railway line through the Bank Compartment had only remained open to goods traffic since dewatering of the compartment commenced. Passenger traffic had been diverted. Constant maintainance, exploratory drilling and 24-hour patrols had been carried out by the SAR: ironically the sinkhole appeared, only 8 days after the route had been reopened to passenger traffic.

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* Personal communication from the Manager of Venterspost Mine (1976).

The inescapable conclusion to be drawn from the above historic account is that the accelerated development of surface subsidence is directly related to artificial lowering of the water-table. Further, it may be broadly true to conclude that the severity and the extent of surface subsidence is related to the *rate* of the lowering of the water-table: least severe and widespread in the Venterspost Compartment*, more severe in the Oberholzer Compartment and, though it may yet be too early to judge, there is mounting evidence of the greatest severity and extent being in the Bank Compartment. Furthermore, as will be discussed later, there have been virtually no large sinkholes formed within living history in the vast areas of the Transvaal dolomite in which the water-table has been lowered very gradually by natural agencies and is now both deep and static: practically all sinkholes developed in such areas have been small, and have formed in situations in which pinnacled bed-rock outcrops or is present at shallow depth below surface. It seems clear, therefore, that the slower the rate of the drawdown of the water-table, whether by natural or artificial means, the more effectively will consolidation take place in the compressible materials being drained. Consolidation results in higher shear strength and a greater resistance to erosion, both of which factors will militate against the formation of sinkholes.

Finally, as is expounded at greater length later, it is the writer's contention that *the water table must be regarded as the base level of subsurface erosion in dolomite terrain*, in the same way that the ocean is regarded as the base level of superficial erosion from the continents.

WEATHERING OF DOLOMITE

Before the unique mode of weathering of the dolomite can be considered, it is necessary to point to some unusual aspects of the chemistry of this rock.

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* This statement must be qualified by drawing attention to the very large number of sinkholes in the valley of the Wonderfontein Spruit at West Rand Garden Estates. This local concentration of sinkholes is assocaited with the shallow depth of the original water-table in this area and the highly cavernous nature of the dolomite.

Figure 7/27 shows the vertical distribution of total manganese, a minor constituent of the dolomite. It will be seen that concentrations of mangenese of more than 1,5 per cent are present in the 'Upper Dark Dolomite Zone' at a depth of about 300 to 450 metres below the top. Elsewhere in the succession the average manganese content is considerably less, except in the 'Lower Dark Dolomite Zone' where the content is just over one per cent and in the Black Reef where it exceeds three per cent.

Other constituents, such as iron and sodium, are also present in small quantities in the dolomite, but particular attention has been drawn to the total manganese content as this has a bearing on the formation of wad, to which further reference is made later.

Solution of Dolomite

The presence of carbon dioxide in ground-water greatly increases its solvent ability. Rain-water already contains a small amount of atmospheric carbon dioxide in solution when it falls on the surface, but as it percolates through the soil it becomes more richly charged with carbon dioxide as the concentration of this gas may be as much as 90 times greater in 'soil air' than in the atmosphere. Thus ground-water may be regarded as very weak carbonic acid:

$$H_2O + CO_2 = H_2CO_3$$

Dolomite is a particularly compact and impervious rock with a porosity of less than 0,3 per cent. However, owing to the highly developed network of joints, tension-fractures and faults in the rock, rain-water can readily percolate through the rock mass. Dolomite goes into solution in the weakly acidic percolating ground-water in the form of bicarbonates:

$$\underset{\text{Dolomite}}{3\,CaCO_3.2MgCO_3} + \underset{\text{Carbonic Acid}}{5\,H_2CO_3} = \underset{\text{Calcium bicarbonate}}{3\,Ca(HCO_3)_2} + \underset{\text{Magnesium bicarbonate}}{2\,Mg\,(HCO_3)_2}$$

Ground-waters pumped from the dolomite or emerging at 'eyes' have a very consistent concentration of dissolved materials consisting of about 90 per cent bicarbonates and 10 per cent silica. The silica content indicates that partial solution of chert also takes place. An indication of

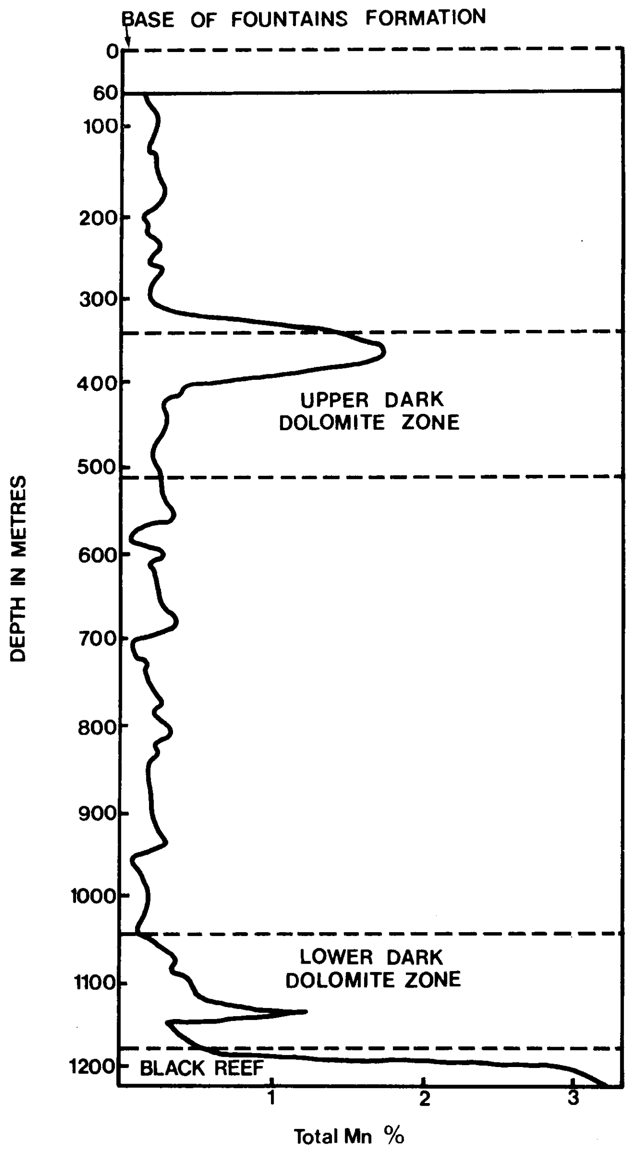

FIGURE 7/27 : Vertical distribution of manganese content in the dolomite rock of the Far West Rand

(After Eriksson, 1971)

the rate at which solution takes place may be gained from A.L. du Toit's (1954) calculation that the Gerhardminnebron Eye, flowing normally at a rate of about 57 megalitres per day, removes over ten tonnes of dolomite in solution each day.

Extensive observations within caves throughout the Transvaal dolomites and elsewhere, coupled with a study of the voluminous geomorphological literature relating to karst areas (too numerous to refer to here), has led the writer to the following conclusion regarding the solution of carbonate rocks:

Above the water-table, i.e. in the *vadose zone*, solution results in the widening of joints and fractures. Between adjacent slots formed in this way the pillars of rock stand as pinnacles rounded off by solution. Insoluble slabs of chert are squeezed together as the intervening layers of dolomite are dissolved away, and the residuum sags between the rock pinnacles to form the characteristic structure so often exposed in excavations where the depth to bedrock is shallow (Figure 7/28).

As joints and fractures in the dolomite are generally steeply inclined or even vertical, it stands to reason that the *slots** which develop from them by solution should be similarly inclined, which is in fact observed to be the case. Naturally the presence of less soluble or insoluble bands

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* An extensive search in the geomorphological literature relating to karst features has failed to provide a single descriptive term to denote what Thornbury (1954) describes as a 'solutionally enlarged joint' of vertical or near-vertical inclination. Brain (1958) introduced the term *aven* for a feature of this sort into the South African literature by stating that 'the result of ..... solvent action is to widen the joint plane and to produce what is known as an aven. In the Transvaal dolomite, avens are characteristically widest where they connect with the cavern roof, but taper upwards towards the surface.' However, Moore's Dictionary of Geography (1949) defines aven as 'a term applied in France, mainly in the Caucusses region, to a Sink Hole'. In the absence of a generally accepted term the word *slot* will be used throughout this chapter to denote a vertical or near-vertical joint, fissure, or fault-plane which has been widened by solution. Though not an entirely satisfactory term, it at least has the merit of conveying the geometric concept which is required for a simple exposition of the writer's hypothesis concerning the mechanism of the development of sinkholes.

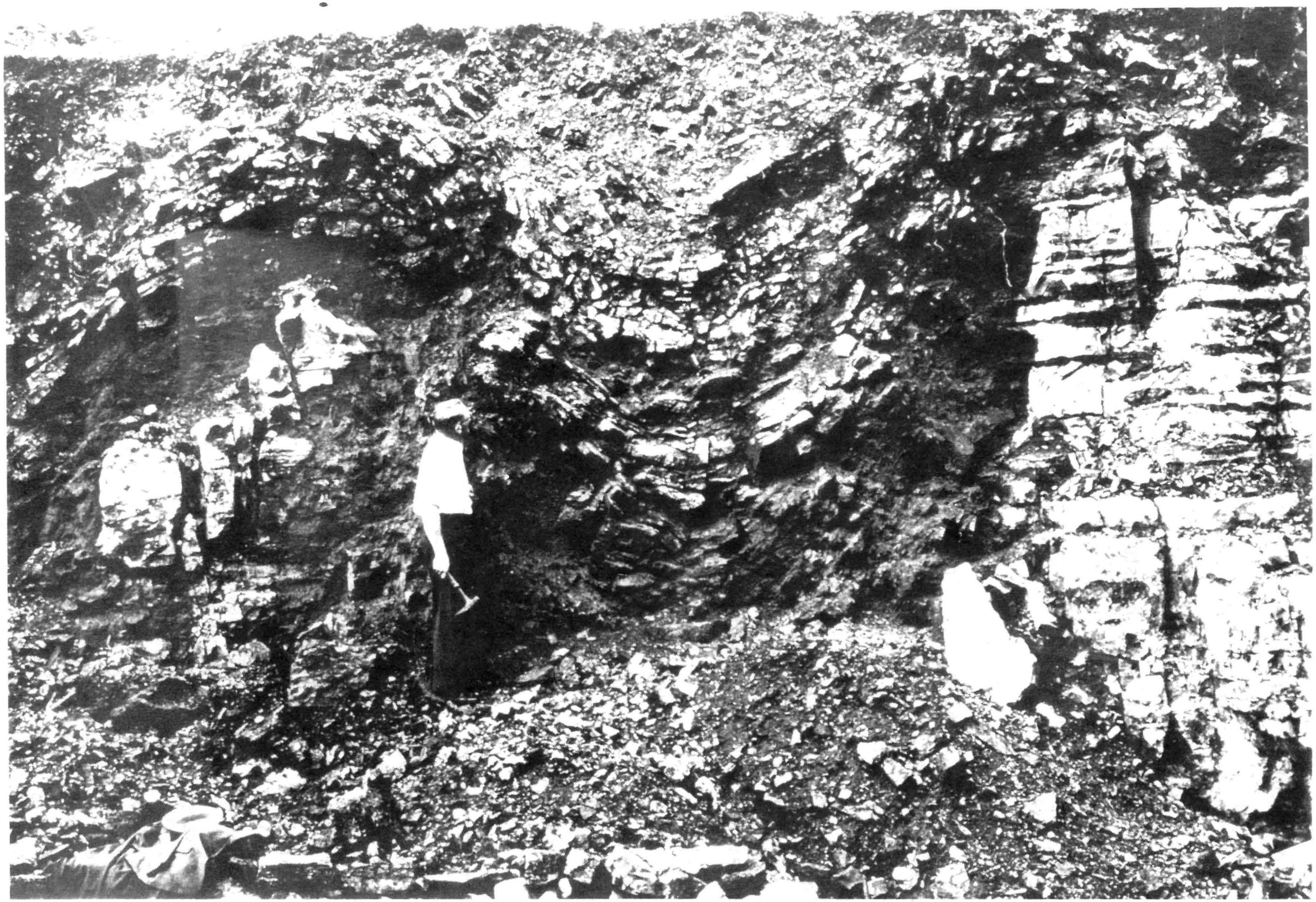

FIGURE 7/28 : Chert residuum sagging between pinnacles of dolomite in excavation for ore-bin at Vaal Reefs like layers of chert produce irregularities so that the slots do not always have smooth and flat sides, but over-all they do tend to follow the direction of dominant flow of water, which is vertical or near-vertical.

Most solution, however, takes place below the water-table, in the *phreatic zone*. Phreatic solution manifests itself in two ways:

(i) immediately below the level of the water-table, where the water is more acidic than deeper down, large horizontal caverns are corroded into the rock, and

(ii) at depth within the phreatic zone, where widening of fissures by corrosion continues to take place.

Water in the phreatic zone is not static. Dissolved bicarbonates are continuously being removed by the slow migration of water along joints, obeying the general hydraulic laws in the form of a flow-net (Figure 7/29). The flow follows ordinary flow-net theory representing the course of seepage through a jointed rock mass, but the net tends to flatten out as solution enlarges the flow-paths near the phreatic surface: i.e. the horizontal component of flow (k_x) soon exceeds the vertical component (k_y) and solution takes place mainly along the horizontal flow direction. Thus the flow of CO_2-charged water results in the development of a network of interconnected caverns in the zone immediately below a water-table which has been static for any appreciable period of geological time (Jennings, 1966a).

Cycles of erosion and planation resulting in such long periods of static water-table lead to the concentration of solution-cavities at particular subsurface elevations. Subsequent lowering of the level of the water-table through the incision of streams in a new cycle of erosion leads to the exposure of such cavities in the vadose zone above the water-table and to the solution of new cavities below the new static level of the water-table. Many of the caves present in the elevated Gatsrante to the south of the Wonderfontein valley probably belong to ancient cycles of erosion dating back to pre-Karoo times; although the West Driefontein cave, some 125 metres in depth, is of various ages, the lowest levels corresponding to the African cycle of erosion under which the pediment of the Wonderfontein valley was carved. Numerous caverns belonging to this cycle of erosion are known in the area and have relatively recently

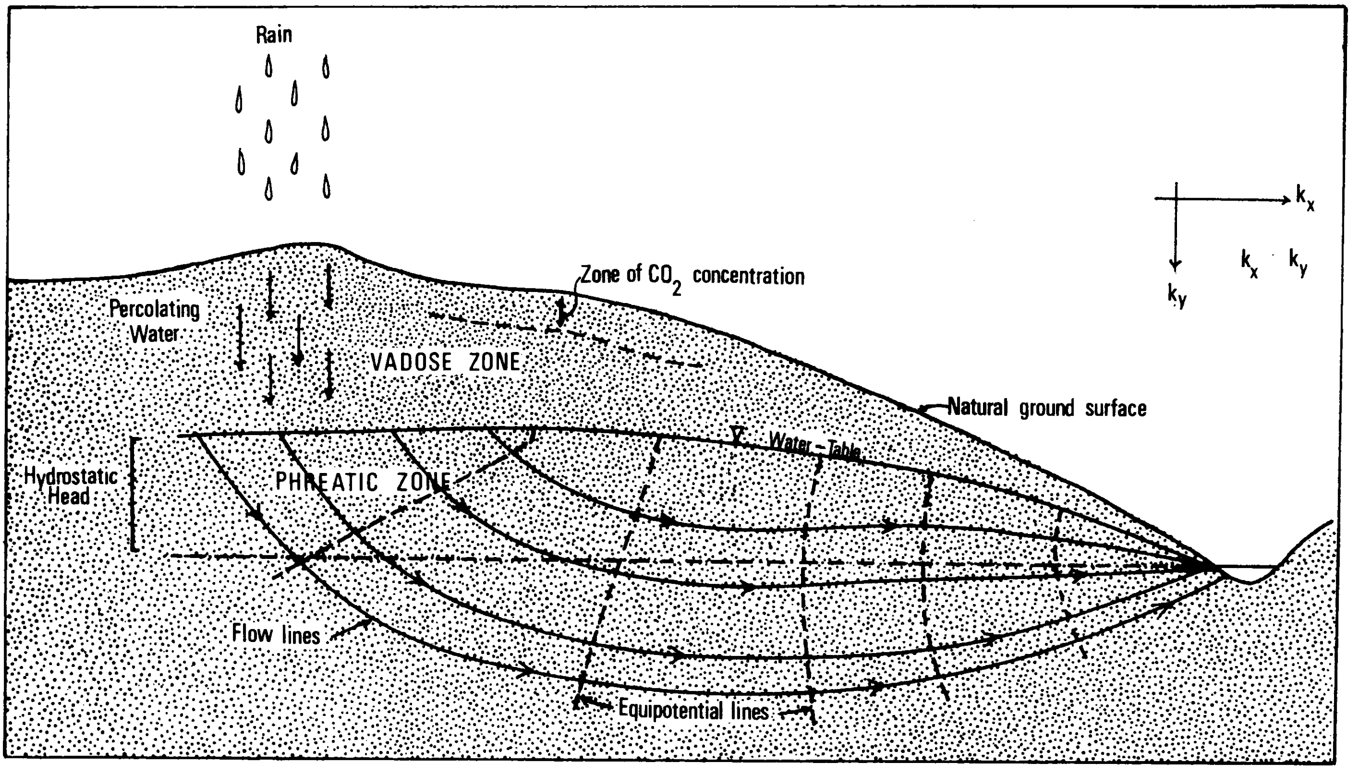

FIGURE 7/29 : Flow-net for water percolation below the water-table in dolomite

(The vertical scale, and hence the gradient of the water-table, are greatly exaggerated; the actual hydrostatic head in the dolomites is very small

(Largely after Jennings, 1966 a)

- during the last few million years - entered the vadose zone through lowering of the water-table corresponding to the African cycle by about 30 metres under the influence of minor post-African incision of the Wonderfontein Spruit (Brink and Partridge, 1965).

Solution thus results in the formation of vertical slots in the rock both above and below the water-table, and the formation of horizontal chambers at the level of a static water-table.

Both vadose and phreatic solution is most effective in faults and tension-fractures, owing to the greater ease of movement of water in such zones. Steep-sided subsurface valleys thus become corroded into the dolomite, in places to depths as great as 150 metres, and these are choked with the residue of insoluble materials.

As solution of dolomite, both in the vadose and phreatic zones, is dependent on the number and the spacing of discontinuities (bedding planes, joints, tension-fractures and faults) and, to a lesser extent, on the composition of the rock, the resulting bedrock surface is frequently highly irregular and characterised by the presence of pinnacles. As solution is particularly effective immediately below a static water-table, and as the level of the water-table is controlled by the level of the local 'eye' within the compartment, the major solution caverns are seldom very deep below the surface and usually within the upper 100 metres below the ground surface.

The formation of wad and other insoluble residues

The main insoluble residues left from the weathering of dolomite are the following:

(1) *Chert* (SiO_2) which remains almost completely intact in the form of slabs. After prolonged exposure chert tends to weather to a hard white rock, or sometimes to a friable white material with a consistency and appearance of chalk. The chert breaks up principally by mechanical action as it loses the support of the underlying carbonate rocks removed in solution. Chert residuum is usually in the form of angular gravels or of slabby boulders, anything from a centimetre to six metres in length. A small amount of chert dissolves in ground-water where this is alkaline or has a high pH.

(2) Iron from the layers of ferrugious dolomite oxidises above the water-table to form *hematite*. This material, together with insoluble clay residues, is very easily eroded downwards and is often found in fissures and caverns into which it has been transported by moving ground-water. Such deposits of 'red unctuous clay' are frequently referred to in the geomorphological literature relating to karst terrain in both dolomites and limestones; for example Bretz (1953).

(3) An insoluble and highly compressible material which is developed during the weathering of dolomite is the substance known as *wad*, or manganiferous earth. Chemical analysis has shown wad to consist of manganese and iron oxides with minor impurities (see Table 7.3) and it is clear that the source of the manganese and iron is within the dolomite rock where these metals are present in small quantities in their divalent states in the dolomite lattice (Eriksson *et al*, 1975).

While wad is encountered in small quantities in any cave in the dolomite and in any accumulation of chert rubble within the zone of weathering, and particularly at any contact between dolomite and other rock types, such as dykes or sills, its presence in large quantities would seem to be confined to restricted zones within the dolomites. Large quantities of wad are always present where the 'Lower Dark Dolomite Zone' is exposed in the zone of weathering and the residual soil has not been eroded away. Large quantities of wad are also encountered in the Black Reef Series immediately below this zone, e.g. in Randfontein and at Swartkops, north-west of Johannesburg. Where the weathering residue of these horizons has been protected from erosion on remnants of the African erosion surface, the manganese may be converted by intense leaching (pedogenesis) into nodular or hardpan manganocrete, often in the form of psilomelane or pyrolusite. Such deposits may extend to depth of 18 m or more below the surface. They have been fairly extensively mined along, and immediately adjacent to, the axis of the Hartebeesfontein Anticline between Ventersdorp and Randfontein. It is of interest to note that these deposits are largely confined to the Black Reef Series and the immediately overlying Lower Dark Dolomite Zone. It seems reasonable to conclude, therefore, that the higher percentage of manganese in the dolomites of these formations, as shown in Figure 7/27 is responsible for the presence of these large quantities of wad and manganocrete. If this is so, then large quantities should also be present in the 'Upper Dark Dolomite Zone' (Figure 7/27), and this would seem to be the case, for example in the weathered zone below Karoo outliers in Westonaria (Bezuidenhout and Enslin, 1969). This is indeed also the case below the Karoo outlier at the Driefontein Brickworks (see Case History 18).

Table 7.3 gives the chemical analyses of two samples of wad and four samples of dolomite. One sample of wad and one sample of dolomite are from the Black Reef Series. The other sample of wad, and the remaining three samples of dolomite are from the Lower Dark Dolomite Zone of the Dolomite Series. It will be seen that wad in the Black Reef Series tends to be more highly ferruginous than that in the Lower Dark Dolomite Zone of the Dolomite Series.

This observation can be directly related to the higher FeO/MnO ratio of the parent dolomite in the Black Reef Series (a ratio of nearly 2) than that in the Lower Dark Dolomite Zone (a ratio of less than 1).

Wad is the most highly compressible residual soil known to occur on the Highveld. The extraordinary engineering characteristics of this material as determined on eight samples are summarised in Table 7.4.

ANALYSIS IN TERMS OF OXIDES	BLACK REEF SERIES		LOWER DARK DOLOMITE ZONE			
	DOLOMITE (1)	WAD (2)	WAD (3)	DOLOMITE (4)	DOLOMITE (5)	DOLOMITE (6)
	%	%	%	%	%	%
CaO	27,20	-	-	25,59	29,44	29,34
MgO	15,12	-	-	18,78	19,32	18,79
Al_2O_3	0,43	3,13	2,29	1,17	0,03	0,47
MnO	3,66	-	-	1,47	3,73	3,05
MnO_2	-	16,46	42,32	-	-	-
FeO	7,26	-	-	2,63	2,17	1,78
Fe_2O_3	-	64,83	19,70	0,30	-	-
SiO_2	4,18	4,71	26,64	2,86	0,68	3,31
CO_2	42,15	-	-	47,20	44,63	43,26
H_2O	-	10,87	9,05	-	-	-
TOTAL	100	100	100	100	100	100
RATIOS:						
FeO/MnO	1,98			0,46	0,58	0,58
Fe_2O_3/MnO_2		3,94	0,46			

Wad (2) overlies dolomite (1) of the Black Reef Series at Swartkops, NW of Johannesburg.

Wad (3) overlies dolomite (4) of the Dolomite Series from the Oaktree intersection on the Johannesburg-Ventersdorp road.

Samples 3 and 4 analysed by G.R. Davies (unpubl. Honours project, Univ. Wits. 1974).

Samples 1,2,5 and 6 analysed by General Superintendence Company (Pty) Limited for Eriksson and Truswell (1974).

Table 7.3 : Chemical analyses of wad and dolomite

PARAMETERS		1	2	3	4	5	6	7	8
	SAMPLE DEPTH (m)	CAVE SAMPLES				3,5	10,2	FISSURE	9,7
<2µm	<2µm CONTENT %					28	31		
LL	LIQUID LIMIT %					47	48		96
PI	PLASTICITY INDEX %					28	5		8
LS	LINEAR SHRINKAGE %					14	4		12
e_o	INITIAL VOID RATIO	2,7	3,8	3,4	4,0	4,6	3,9	9,6	4,7
Cc	COMPRESSION INDEX	0,98	1,00	0,78	1,00	1,80	1,26	0,57	1,46
Pc	PRECOMPRESSION PRESSURE (kPa)	153	316	670	86	700	359	190	193
Po	IN SITU EFFECTIVE STRESS (kPa)					46			83
OCR	OVERCONSOLIDATION RATIO					15			2,32
C'c	COMPRESSION INDEX AT P_o (Schmertmann, 1953)	0,05	0,05	0,06	0,07	0,05	0,06	0,03	0,07
SG	SPECIFIC GRAVITY OF SOLID PARTICLES	3,21	3,21	3,21	3,21	3,34	3,47		1,63 (?)
γb	BULK DENSITY (kN/m^3)	13,76	13,35	13,27	9,91	13,85	14,62		8,58
w	WATER CONTENT %	105	105	85	57	168	110		202
γd	DRY DENSITY (kN/m^3)	6,71	6,57	7,22	6,30	5,81	7,00		2,85
C_v max	COEFFICIENT OF CONSOLIDATION	380	451	380	380				
Cv min	(mm^2/minute)	34	213	279	213				
Cv mean		294	343	297	320				

Samples 1 to 4 : Cave in Bank Compartment
Sample 5 : Lower Dark Dolomite Zone : Natalspruit Dolomite Outlier
Sample 6 : Black Reef Series : Randfontein
Sample 7 : Fissure in quarry near Daleside
Sample 8 : Lower Dark Dolomite Zone : Natalspruit Dolomite Outlier

Table 7.4 : Engineering parameters determined on eight samples of wad

MECHANISM OF SINKHOLE FORMATION

The mechanism responsible for the development of sinkholes as envisaged by the writer, after extensive observations of the spatial geometry of solution caverns in the dolomites, is illustrated in Figure 7/30.

Diagram A depicts the 'equilibrium situation' of the residual soils in relation to a water-table which has been static for a long period of time. A series of horizontally disposed and interconnecting chambers has been formed by solution of the dolomite immediately below the water-table. Vertically or near-vertically disposed tension-fractures have been widened into 'slots' by solution in percolating vadose waters above the water-table and by phreatic water at depth below the horizontal chambers. The caverns are occupied by water; the slots above them may be choked with residual soils. The highly irregular dolomite surface is overlain by a mantle of residual chert rubble of variable thickness. Chert slabs within the rubble sag between the abutments of rock pinnacles. Rain-water percolating through the permeable chert rubble enters the phreatic reservoir without erosive effect.

In diagram B the water-table has been lowered to a level below the horizontal chamber. Water from a leaking pipe or some other source, or even concentrated rain-water seepage, has penetrated to the surface of the dolomite and thence downslope to the nearest slot. The soil choking the slot has been eroded from the mouth of the slot upwards, by a process of headward piping not dissimilar to the piping process known to take place in earth-dams. A temporary arch has formed in the residual soil at the top of the slot.

In diagram C successive arch collapse has taken place in the roof of the vaulted void within the chert rubble. In each arch the weight of the overlying soil is transmitted to the pinnacle abutments by arching thrusts. Successive arch collapse is activated by further erosion at the base of the arch. As may be seen in Figures 7/6 and 7/35, a veneer of highly erodible wad is commonly present on the sides of the dolomite pinnacles. In some situations arch collapse may be activated by a loss in shear strength of the material at the base of the arch as it becomes wet. In either case, the collapsed debris is eroded down the slot and into the underlying chamber. The last arch, which may be stable for a

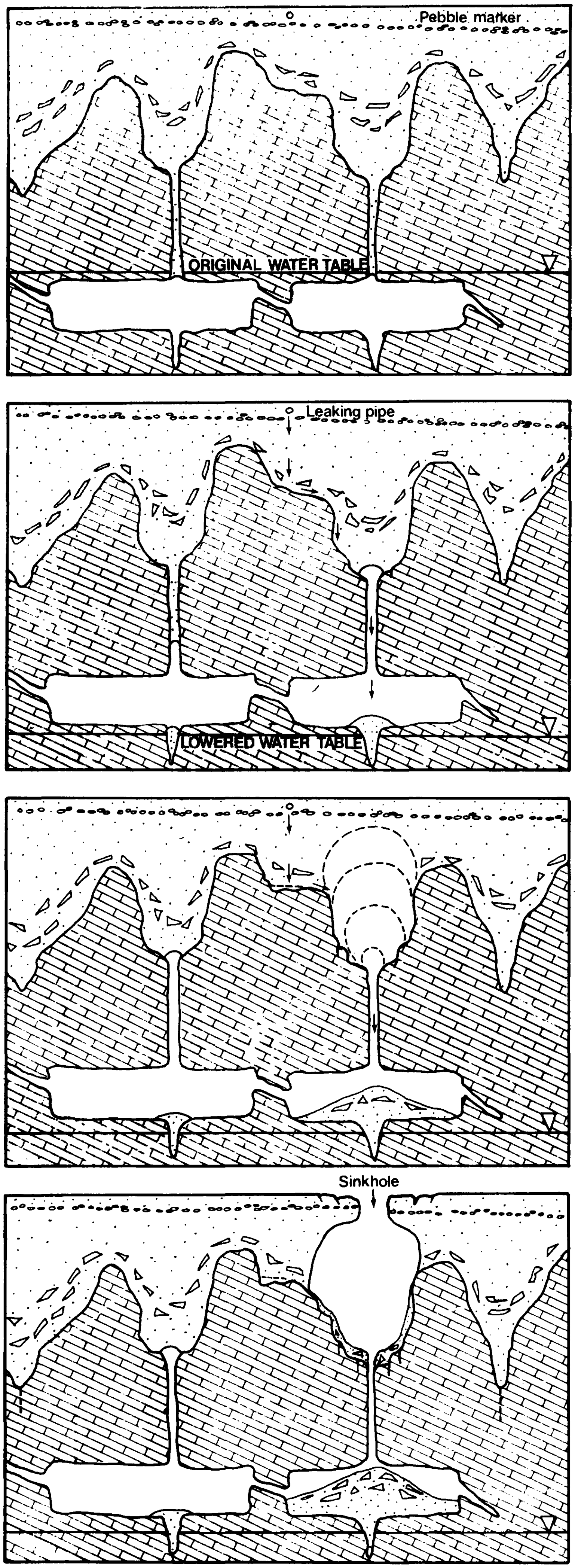

Diagram A

Equilibrium situation before lowering of the water-table.

Diagram B

After lowering of the water-table. Active subsurface erosion; slot is flushed out by a process of headward erosion.

Diagram C

Progressive collapse of roof of vault, possibly temporarily arrested by ferrugi-nised pebble marker.

Diagram D

Collapse of last arch to produce a sinkhole surrounded by concentric tension-cracks.

FIGURE 7/30 : Mechanism of the development of a sinkhole considerable period of time, is often supported by a near-surface layer of hardpan ferricrete or a densely ferruginised pebble marker of high shear strength. It must also be pointed out that, in the absence of strong layers at shallow depth, the stresses in arches approaching the surface are lower than those for smaller arches at depth, owing to the reduced overburden pressure. In such cases even the sandy topsoil may form the crown of the last arch with nothing but grass roots to provide cohesion. The writer has examined several such vaults where the crown was no more than about 150 to 200 mm thick and yet remained stable for several months. In one such case, in the dolomites south of Pretoria, a soil crown about 200 mm thick was subjected to a plate loading test to determine its bearing capacity.* Using a loading plate 250 mm in diameter, Mr Derek Pike of the Geological Survey applied progressive loadings by means of a water tank, and observed that failure took place by the punching through of the loading plate at an applied pressure of 215 kPa.

In diagram D the crown of the last arch had collapsed to form the sinkhole. In cross-section the sinkhole at this stage is like a bottleneck. Small sinkholes may retain this shape for a considerable period of time. Figure 7/31 shows a section which was accurately measured in such a sinkhole at West Rand Garden Estates in the Venterspost Compartment.

Larger sinkholes tend rapidly to develop concentric tension-cracks in the soil around the collapsed crown and further collapses take place within a short period of time to produce a cylindrical cross-section.

In most cases sinkholes are circular in plan. This fact provides geometrical corroboration of the theory of successive 'arch', or better 'dome' collapse: the intersection of the horizontal surface plane with the dome-shaped void is a circle.

Clearly, then, a number of interdependent conditions are necessary before a sinkhole can form. Five such conditions were listed in 1965 by Jennings, Brink, Louw and Gowan. Briefly, they were as follows:

1. There must be adjacent rigid material to form abutments for the roof of the void. These are provided by the dolomite pinnacles or

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* Personal communication from D.R. Pike (ca 1971).

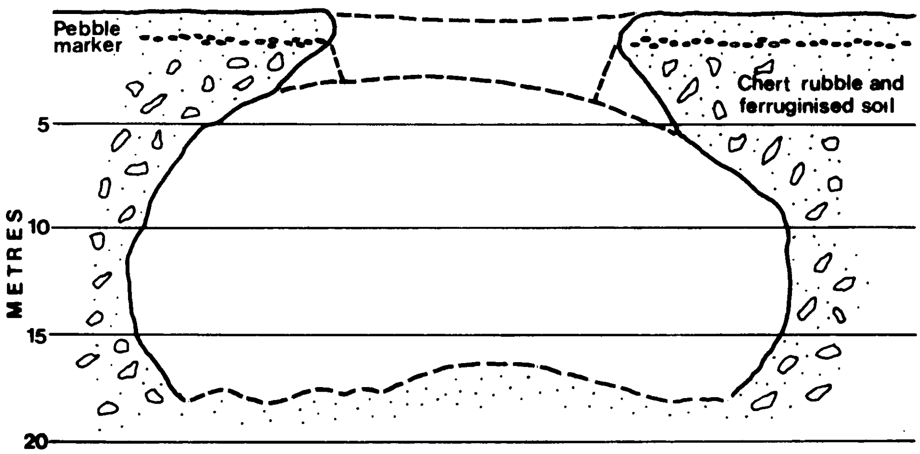

FIGURE 7/31 : Surveyed section through small sinkhole at West Rand Garden Estates showing collapse of crown only

(After Jennings, [illegible])

sides of the steep-sided subsurface canyons (i.e. slots). The span must be appropriate to the strength of the bridging material since, with a span which is too large, the arch cannot form.

2. A condition of arching must develop in the residuum, i.e. a part or all of the vertically acting self-weight must be carried by arching thrusts to the abutments. Complete arching will have taken place when the vertical stress along the intrados is zero.

3. A void must develop below the arch in the residuum.

4. A reservoir must exist below the arch to accept the material which is removed to enlarge the void to substantial size. Some means of transportation for the material, for example flowing water, is also essential.

5. When a void of appropriate size has been established in the residuum, some disturbing agency must arise to cause the roof to collapse. The void will move progressively upwards towards the surface. A common agency causing collapse is water in the arched material which leads to loss of strength or washing out of critical binding or keying material. This provides the trigger which initiates the collapse leading ultimately to the sinkhole.*

While still maintaining the validity of the above essential conditions for sinkhole development, the writer wishes here to stress the importance of subsurface erosion consequent upon rapid lowering of the water-table. This aspect is mentioned in condition 4 listed above, but not sufficiently emphasised.

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* In addition to the triggering action of water entering the arch material, and so causing a loss of strength, Jennings (1966 a) subsequently recognised three further triggering mechanisms, viz:

(i) earth-tremors causing vertical and lateral accelerations, thus resulting in externally applied body forces to the materials of the arch;

(ii) ground movements resulting from subsidences associated with mining; and

(iii) surface loading of a vibratory nature, particularly where the energy of vibration is great and sustained.

The significance of excess seepage water was recognised by Enslin and Smit as early as 1955. Referring to sinkholes in the dolomites south of Pretoria (prior to the Far West Rand era), they wrote as follows:

> "It has been observed that all sinkholes occurred as a direct result of an excess of water which seeped down and disturbed the equilibrium of the critically balanced arched bridges of loose material covering the network of near-surface cavities in the dolomite. The excess water originated from leaking pipes, storm waters and even from overwatering of gardens. It is quite clear that many slumps of sinkholes could have been prevented if care had been taken to check the local accumulation of water" (Enslin and Smit, 1955).

Again, in 1963, Donaldson emphasised the significance of subsurface erosion:

> "Where man's operations result in a concentrated percolation of water down into the soil, a water-table will form on top of the dolomite unless it finds a fissure into which it can flow. Once this occurs, the possibility exists for the material round the opening in the dolomite surface to be eroded and washed into the fissure, so forming a small cavity. Further erosion will cause the cavity to enlarge into a cavern. The roof will start falling in and, if the flow of water is sufficient, the fallen material will be washed down the fissure leaving an enlarged cavern. If the flow is not sufficient to remove all the debris, the cavern will not increase in size, but will move progressively upwards through the soil. In either case the stage will be reached where the arching soil roof will be unable to sustain its load and will collapse dramatically, to reveal a sinkhole. Circumstantial evidence in support of this contention is provided by the relatively small amounts of debris found at the bottom of sinkholes" (Donaldson, 1963).

In a sketch accompanying these observations (very similar to that shown in Figure 7/30), Donaldson illustrated the 'cavern' referred to above as a vaulted void within the soil and rubble above a fissure (or slot).

Donaldson goes further and draws attention to the significance, in terms of subsurface erosion, of a lowering of the water-table:

> "It should be noted here that, if there is a permanent water-table above the top of the dolomite, the effect of percolating ground water on the flow through fissures in the dolomite will be very small and erosion is unlikely. However, if water is being pumped out of the dolomite from below a fissure, the flow through that fissure will be increased and may cause erosion."

Evidence from the Far West Rand has fully substantiated this contention. With few exceptions sinkholes have been demonstrably associated with the ingress of excessive quantities of water into the soil mantling a dewatered compartment. The greatest abundance of sinkholes is within the former flood-plain of the Wonderfontein Spruit. The excessive quantities of water here are mainly from natural surface concentration of rainwater. In some cases here, too, it has been from leakage waters from disrupted concrete canals. The largest sinkhole on the Far West Rand which has not been filled in and is thus still available for inspection today, is a crater 100 m in diameter and 40 m deep, which appeared in 1957 at West Rand Gardens Estates near Ventersport Mine offices. It is situated on the pediment immediately above the flood-plain of the Spruit. The point of ingress of the water which eroded more than 300 000 cubic metres of soil and rubble through a narrow slot in the underlying bedrock is represented by the ruptured concrete irrigation furrow on the southern perimeter of the sinkhole. The slot through which this enormous volume of material was eroded is no longer visible, having been choked by rubble which has subsequently collapsed from the sides of the crater. But it was visible when the writer inspected the sinkhole shortly after it opened up, i.e. shortly after the collapse of the last arch, and it was seen to be no more than about 2 metres wide.

In other cases the source of erosive water has been from leaking water mains, reservoirs, canals, french drains, storm-water drains, etc.. In very few cases has there been no obvious local source of water at the surface, and in these isolated cases it is likely that the contours of the bedrock were such as to create local concentration of rain-water seepage.

There is thus ample evidence to support the writer's contention that subsurface erosion is the prime factor in sinkhole development, and that the water-table is the base-level for such erosion.

In this regard it should be mentioned, finally, that the vast majority of the sinkholes which have developed during the past 15 years have been confined to the pediment and flood-plain.

It is below these landforms that the effect of the artificial lowering of the water-table has been critical. Sinkholes in the Gatsrante, however, formed during an earlier period of subsurface erosion related to the natural lowering of the pre-existing water-table during post-African incision of the Wonderfontein Spruit. These 'paleo-sinkholes' have thus been high and dry for tens of thousands of years. Few new ones have formed here during the current, artificially induced, cycle of subsurface erosion.

That the water-table in former times was at a substantially lower level, even below the pediment and flood-plain, is evidenced by the observation that a great many sinkholes (perhaps the majority) which have formed on these landforms as a direct result of dewatering have been ascribed to the re-activation of palaeo-sinkholes which had been completely filled with transported materials and subsequently obscured by a superficial blanket of hillwash. Wolmarans has frequently referred to red soils exposed to great depths as being characteristic of such reactivated palaeo-sinkholes.* The only proven method of locating palaeo-sinkholes on the pediment and the flood-plain is by drilling a close grid of shallow boreholes through the cover of transported soil. By this means isopachs of the transported soils can be constructed and these show the potential danger spots.* Furthermore recognition of the pebble marker and horizons of hardpan ferricrete in the boreholes can be used to distinguish between basin-shaped palaeo-dolines (see Figure 7/33) and funnel-shaped palaeo-sinkholes: these horizons are absent from the latter.

This technique is now applied as a standard method to locate these structures in such places as Westonaria and Carletonville.

Large sinkholes

As previously defined, a *large sinkhole* is one which has dimensions exceeding 45 m in diameter and 30 m in depth. There have been relatively few of these in the Far West Rand; maybe a dozen, at most a score. The location of eight of these, which appeared prior to July 1966 in the Oberholzer Compartment, are shown in Figure 7/26.

Referring to these, Foose (1967) writes as follows:

> "The unusual size of the large sinkholes that have formed makes it difficult to visualise an opening within uncon-

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* Personal communications from J.F. Wolmarans (1976).

> solidated debris of adequate size to accept all the material at the time of collapse ... I suggest that large sinkholes, and possibly even smaller ones, involve the development of multiple arches between rock pinnacles, close enough to one another so that, as they grow larger, they may suddenly coalesce, thereby increasing the span of the arch beyond its ability to support the load. Experimental studies have shown this not only to be feasible but also to provide the most likely explantion for giant sinkholes. This would help to explain both the large amount of existing void into which debris could collapse and the rapidity with which the debris moves downwards through multiple openings. After collapse of several of the large sinkholes, continued downward movement of the debris near the base of the sinkhole has been observed."

Jennings (1966 b), in an unpublished report, had suggested the same possible mechanism for the development of sinkholes larger than 60 m in diameter. He states:

> "A possible mechanism of rapid propogation to form a sinkhole from a deep-seated feature is that of the collapse of a series of multiple arches. The mechanism ...... has been sucessfully demonstrated on a model in the laboratory. It should be appreciated that for the 'multiple-arch' hypothesis to apply in practice it is necessary to have void volumes which are large enough to receive the fallen and bulked material".

Jennings goes on to observe that "It is, of course, also necessary that the multiple arches exist in nature", and then admits that "such conditions have not been discovered ...."

This hypotehesis of the multiple arch probably originated with Dr Kenneth Knight who conducted the laboratory experiments at the University of the Witwatersrand which are referred to by both Jennings and Foose. Jennings relates (personal communication) how Dr Knight used the analogy of a vaulted roof of a cathedral composed of multiple arches, drawing attention to the fact that the whole roof would collapse if a single arch support were to be removed.

It would seem that the hypothesis arose from two considerations. The first is that it is difficult to conceive of chert rubble, a very loose material, having the strength to support a large arch for any length of time and therefore the need, in Jennings' words, to postulate a process of "rapid propogation to form a sinkhole from a deep-seated feature ..." The second, in Foose's words, is that it is "difficult to visualise an opening within unconsolidated debris of adequate size to accept all the material at the time of collapse".

Based on the observations of the mechanism of sinkhole development already outlined, the writer cannot support the multiple-arch theory, and specifically for the following reasons:

1. As a general observation it may be stated that in the case of most *small* sinkholes, bedrock pinnacles are near the surface, or, in some cases, even outcropping. In other words, the top of the vertical slot is virtually at the surface. An example of such a sinkhole, which developed next to the Potchefstroom road immediately north of Blyvooruitzig Mine as a result of leakage from a Rand Water Board main, is shown in Figure 7/32. In some places a number of such small sinkholes are found to lie on a straight line. Attention has already been drawn to the straight alignment of four small sinkholes at the Venterspost recreation centre; the tennis court hole, the bowling green hole and two further sinkholes in the open veld near by. It is clear that such alignment follows the surface (or near-surface) trace of a shallow slot, or solution-widened fissure.

   The converse is true for large sinkholes. Bedrock is deep, the abutment pinnacles and associated slot being buried beneath a cover of chert rubble the thickness of which is equal to, or greater than, the depth of the sinkhole (i.e. more than 30 m). While Foose's observation that 'after collapse of several of the large sinkholes, continued downward movement of the debris near the base of the sinkhole has been observed' is true, this is because the relatively small amount of material constituting the last arch often hides the throat of the underlying slot from view; it may later be exposed by further erosive flushing. But in one case, at least, this was not so. As stated earlier, the writer observed a *single narrow slot* at the bottom of the 1957 sinkhole at West Rand Garden Estates shortly after the giant crater had opened. There was no question, here, of multiple draw-off points.

2. Also as stated earlier, there is ample evidence that the last arch is frequently arrested in the shallow layer of hardpan ferricrete or the ferruginised pebble marker. While loose chert rubble ob-

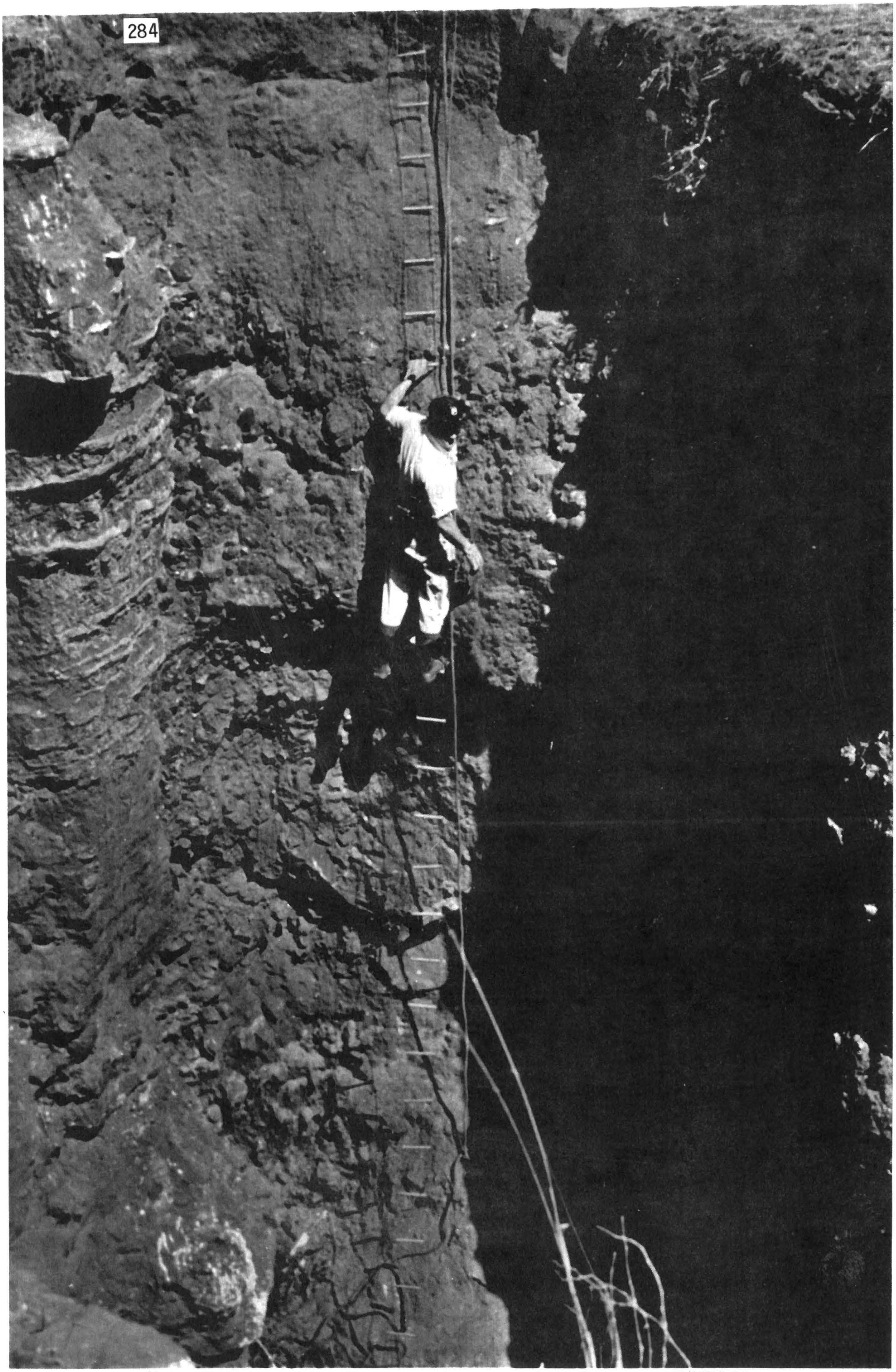

FIGURE 7/32 : Small sinkhole between outcropping dolomite pinnacles caused by leakage from water-main next to Potchefstroom viously does not possess the strength to support a large span, the ferricrete apparently does. The laboratory experiments conducted by Knight, and quoted by Foose and Jennings, do not meaningfully simulate the situation in nature. The experiments were conducted in a transparent tank in which a layer of dry sand was overlain by a layer of moist sand. Plugs at the bottom of the tank were opened to allow the cohesionless dry sand to flow out, like sand from an hour-glass, simulating a draw-off point in a pinnacle-bound slot. Arches developed in the overlying, slightly cohesive, moist sand representing the chert rubble. By progressive arch collapse the vaulted voids rose to the surface of the tank and coalesced to produce the final breakthrough to the surface, representing the large sinkhole.

In the writer's view, this model, while demonstrating very well the progressive 'onion-peel' collapse of successive arches, does not accurately simulate natural conditions. It might well have done so if a material of greater strength had been used to represent the superficial horizon of strong ferricrete. In that case a single arch would probably have sufficed to produce a large, circular breakthrough at surface.

Thus, if the last arch in strong material stands up long enough for the collapsed debris below it to be flushed through the slot, the enormous void thus created in the unconsolidated material is clearly of more than adequate size to accept debris from the final collapse with insignificant decrease in its final volume.

3. Finally, and again as stated earlier, sinkholes - and *particularly* large sinkholes - are circular in plan. Spatial geometry clearly demands a single arch (or dome) span. Multiple arches would produce a pattern of interlinking circles in plan. The nearest approach to such a pattern is that observed in parts of the flood-plain of the Wonderfontein Spruit where contiguous, and relatively small, sinkholes merge to form an irregular outline on surface. But even here the pattern is more like a *line* of partially intersecting circles, produced from a series of draw-off points along a single slot.

THE MECHANISM OF DOLINE FORMATION

As discussed earlier, preferential solution of dolomite, in both the vadose and phreatic zones, takes place along discontinuities in the rock mass. Preferential solution is most intense within the shear-zones of tension-faults which, in the Far West Rand, trend mainly in a direction N 10^0 E, i.e. the direction of the 'intermediate valleys' shown in Figure 7/19. Where these tension-faults comprise a number of closely spaced parallel dislocations, the resulting shear-zones are so vulnerable to chemical weathering by solution that wide subsurface valleys, up to 200 m deep, are corroded into the rock mass. Valleys of this kind extend to considerable depth below the level of the original water-table, and are occupied by the insoluble residues of chert rubble, red unctuous clay and wad. Corrosion of the dolomite resulting in these features has been active over long periods of geological time: naturally, as corrosion proceeds, the surface subsides to form the elongated and enclosed depressions, or natural dolines, which characterise most areas of karst topography. Where ample supplies of hillwash are deposited over the karst topography, as in the Far West Rand, the surface depressions will tend to be filled in and the surface thus levelled off.

Such a situation is illustrated in diagram A of Figure 7/33. Here the surface of the hillwash deposit forms a level plain, but the sagging of the pebble marker indicates the former existence of a natural doline. The buried subsurface canyon is largely occupied by potentially highly compressible material such as saturated wad below the water-table 'Floaters' of rock, representing former dolomite pinnacles now detached from the bedrock by basal corrosion, may be present within the unconsolidated debris. These may range in size from a few cubic metres up to a thousand cubic metres or more. The subsurface valley continues to grow gradually wider and deeper by continual slow corrosion but, as far as surface subsidence is concerned, the situation may be regarded as virtually static or in a state of equilibrium.

As illustrated in diagrams B, C and D, accelerated reactivation of the doline becomes apparent at the surface as the water-table is progressively lowered through the unconsolidated debris. As Jennings (1966 a) points out, it can be shown that lowering of the water-table is roughly equivalent to the placing of a surcharge loading at an elevation halfway

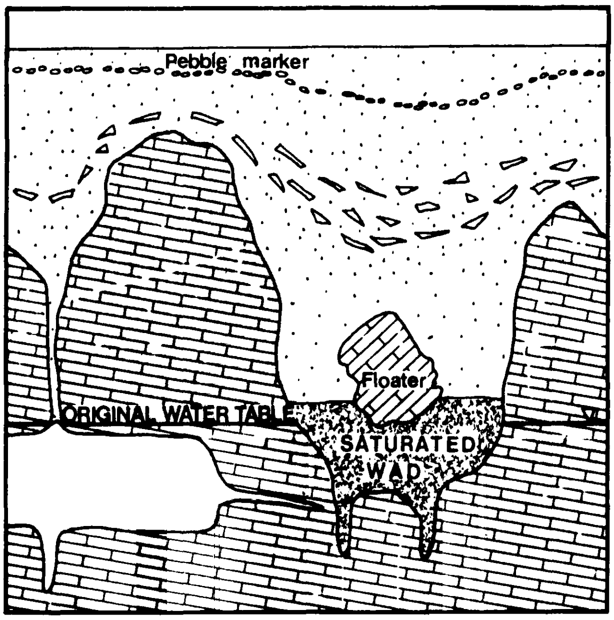

Diagram A

Equilibrium situation before lowering of water-table. 'Palaeo-doline' not apparent at surface but indicated by sagging chert rubble and pebble marker.

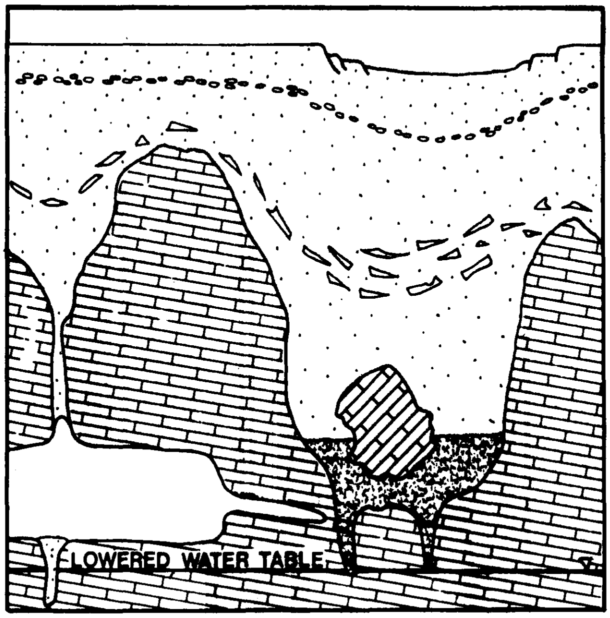

Diagram B

After lowering of water-table. Reactivated doline development becomes apparent as a surface subsidence caused by consolidation of wad. Periphery of doline characterised by shear-zone and tension-cracks.

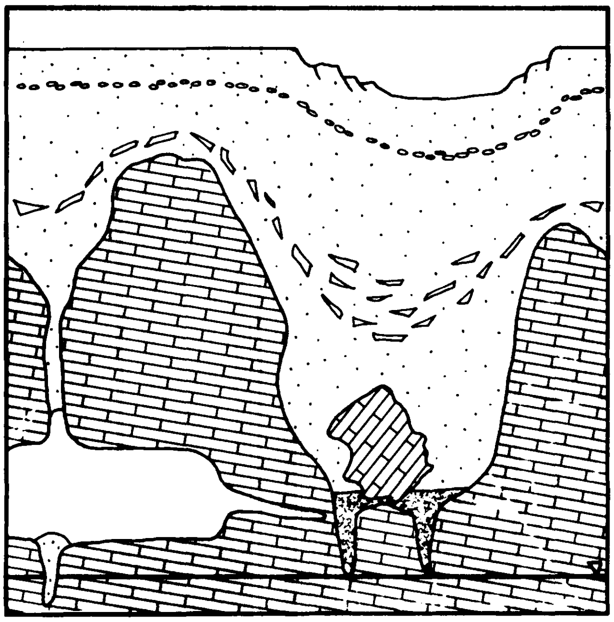

Diagram C

Progressive consolidation of wad causes progressive subsidence on surface.

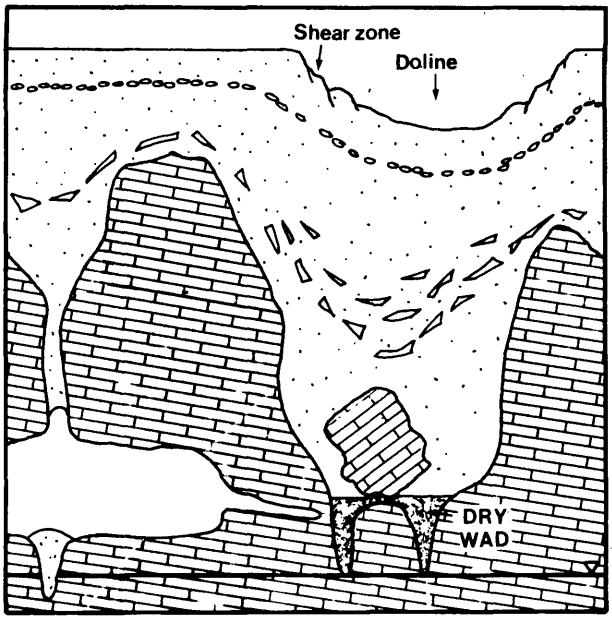

Diagram D

Final equilibrium situation: wad completely consolidated and doline development complete.

FIGURE 7/33 : Mechanism of the development of a doline

between the initial and the final water-table levels; i.e. loading being that which would be exerted by a head of water equivalent to the drop in the water-table. Thus, for example, if the water-table were lowered by 100 m, the effect would be the same as placing a load of 100 x 9,81 = 981 kPa (or nearly 1 MPa) at the mean elevation of the old and the new levels of the water-table.

As water in the voids in materials below the original level of the water-table drains out, settlement follows Terzaghi's consolidation theory. The amount of settlement will depend on the compression index (C_c) of the consolidating material, and the rate of settlement on its coefficient of consolidation (C_v). It will be seen from Table 7.4 that wad has a mean C_c greater than 1, and a mean C_v greater than 300 mm^2/ minute. It follows, then, that where there is a substantial thickness of wad below the original water-table, the amount of settlement will be excessive and the rate of settlement will be extraordinarily rapid. This accords with the observations of the Carletonville dolines of the early 1960's described earlier: up to 12 m of surface subsidence in three years. It remains only to be stated that large quantities of wad were in fact encountered below the original water-table in all exploratory boreholes drilled within dolines on the Far West Rand.

Diagram D of Figure 7/33 illustrates the development of surface tension-cracks in the peripheral shear-zone, and explains why buildings which are situated within a doline show little or no cracking while those straddling the shear-zone are completely destroyed.

To conclude with the words of Foose (1967):

> "One may expect maximum subsidence where there is a maximum thickness of unconsolidated debris and where maximum lowering of the groundwater surface occurs".

DELINEATION OF POTENTIAL SINKHOLE AND DOLINE AREAS BY GEOPHYSICAL EXPLORATION

Geophysical exploration involves the measurement of some factor at the ground surface to predict the presence of features buried beneath the surface. As discussed by Enslin and Smit (1955) and later by Jennings (1966 a), a variety of geophysical methods have been applied experimentally in attempts to delineate potential sinkhole and doline areas. Methods which have proved of little or no use in this regard are electrical resistivity and seismic refraction. The electromagnetic method has proved of limited use in the detection of solution-widened joints (i.e. slots), particularly where these are not deep below the surface and are completely choked with mud. Likewise the use of magnetometer traverses has proved of limited use insofar as they are able to detect the presence of dykes and to delineate the extent of diabase or dolerite sills intruded into the dolomite. Intrusive sills are particularly common on the East Rand, and their detection and delineation are important: the presence of thick intrusive sills will obviously inhibit sinkhole formation, and may render large areas of the Dolomite Series suitable for development.

The most promising geophysical technique, however, is the gravimetric method, and specifically in relation to the delineation of subsurface contours of dolomite bedrock relative to the level of the original water-table (Bezuidenhout and Enslin, 1969; Enslin and Smit, 1955). The success of the gravimetric method in this regard is based upon the fact that there is sufficient difference between the specific gravity of solid dolomite (2,85) and that of chert rubble and other unconsolidated debris (1,7 to 2,4) to produce recognisable gravity anomalies related to the mean depth of bedrock. The observations do not distinquish between 'floaters' and bedrock, nor do they precisely define the outlines of dolomite pinnacles and they cannot be interpreted with any certainty to show subsurface voids in the chert rubble or caverns in the bedrock: but, when the Bouguer values have been calculated, and corrections based on observed depth to bedrock in selected boreholes have been applied to eliminate regional trends, the observations indicate the presence of deep subsurface valleys very clearly. Using a closely spaced grid of observation stations the gradients of the buried valley sides may also be deduced with sufficient accuracy to predict at least the potentially most dangerous situations for sinkhole development, all other factors (such as sources of water ingress) being equal.

In a specific application to conditions in the ground-water compartments of the Far West Rand, Bezuidenhout and Enslin (1969) indicate how corrections can be applied to -

".... reduce the zero gravity contour to correspond to lines along which the sub-surface of solid dolomite coincides with the original groundwater table. Positive gravity contours thus indicate areas where the solid dolomite is above the original ground-water table, i.e. areas which will not be affected by dewatering. Negative gravity contours indicate where the original ground-water table is above the solid dolomite, i.e. areas which will be subject to slow subsidence with dewatering; and the zones with the close-spaced gravity contours indicate the scarps adjoining the valleys."

The same Authors go on to claim that

"By more sophisticated analysis and interpretation of the gravity contours it is possible to delineate the zones which are potential sinkhole areas, with fair accuracy. The accuracy of the interpretation is further increased by drilling boreholes at strategic points. It has thus been possible by the scientific interpretation of data obtained by gravimetric surveys supplemented by boreholes to delineate safe and potentially unsafe areas and to advise accordingly".

As far as the accurate delineation of areas of potential doline formation are concerned, the writer would agree entirely with this claim. These are the areas of 'negative gravity contours' as defined above. But the historic evidence of sinkhole development in the Far West Rand, even in areas which had been covered by a close grid of gravity stations, and even during the past five years, indicates that this claim must be regarded with reservations as far as the prediction of individual sinkholes is concerned. This is the inevitable conclusion if the writer's concept of the mechanism of sinkhole development is true. However close a grid of gravity stations may be, and however accurately all the crucial 'slots' may have been detected, the gravimetric method - and any other conceivable geophysical method for that matter - can never predict where at some future time a leak may develop in a buried pipe or a water-bearing structure to produce the erosive power essential to the development of a local sinkhole.

Nonetheless the gravimetric survey remains the best geophysical method so far applied to indicate areas where development should either be prohibited, or engineering design should be such as to cope with anticipated troubles. In interpreting the gravity surveys, cognizance must always be taken of the depth of the water-table and the liklihood of it remaining static or being artificially lowered. In this regard, three different situations need to be considered:

(i) Dolomite terrain in which the water-table is being, or is likely to be artificially lowered:

Areas showing 'negative gravity contours' are suspect for doline formation. Such areas, which may also be broadly referred to as gravity 'lows', must also be regarded as suspect for the formation of large sinkholes in situations where the water-table is drawn down through unconsolidated materials. Gravity 'highs', where pinnacled dolomite either outcrops or is present as shallow suboutcrop, are areas where small sinkholes may be anticipated.

(ii) Dolomite terrain in which the water-table is static and shallow, i.e. solution chambers are filled with water and are likely to remain so:

As the water-table is the base-level for subsurface erosion, and all the large cavities which could act as receptables for eroded materials are below the water-table, there is no possibility for large sinkholes developing in the gravity 'lows', or elsewhere for that matter. However, as there may be smaller solution-widened fissures above the water-table, small sinkholes may develop in the gravity 'highs'. As long as the water-table remains static there is no possibility of doline development.

(iii) Dolomite terrain in which the water-table is static and deep, i.e. solution chambers are situated above the water-table:

This is the most common condition in the Transvaal dolomites. Gravity 'highs' must be regarded as areas where small sinkholes may develop. The rational conclusion which could be drawn from the mechanism of sinkhole development outlined earlier is that the gravity 'lows' should be regarded as suspect for the development of large sinkholes. In fact, however, to the writer's knowledge, there are no cases of large sinkholes having developed in such situations during recorded history, with the exception of the isolated sinkhole at Vogelstruisbult described earlier. This fact would tend to support the early contention of Enslin and Smit (1955) that

> "if the dolomite is covered by chert boulders, rubble or soil over a fairly large area and to a depth of 50 feet or more, slumping of the loose material into the deeper cavities can at the most cause a slow settling of the surface". But why have large sinkholes not developed in these situations? One possible explanation is that the thick mantle of dry unconsolidated materials above the long-static and deep water-table must act as a sponge which will absorb percolating water to the extent that the erosive power necessary for sinkhole development cannot be generated. If this is indeed the case, it is conceivable that a sufficiently large and continuous supply of excess percolating water could eventually initiate the development of a large sinkhole. But perhaps a more cogent explanation is the one stated earlier, namely that effective consolidation of compressible materials took place during the period of gradual drawdown of the water-table, resulting in materials with a sufficiently high shear-strength and resistance to erosion to inhibit sinkhole development.

It is thus clear that the gravimetric method, properly interpreted, is a valuable tool for the exploration of dolomite terrain, and it is heartening to record here that the Department of Local Government in the Transvaal, advised by Geological Survey, has stipulated that no township may be proclaimed on the dolomites unless a thorough investigation, including a 30 m grid of gravity stations followed by appropriate drilling of exploratory boreholes, has indicated to its satisfaction that local conditions are such that the liklihood of the formation of dolines and large sinkholes is reasonably minimal.

PROTECTIVE MEASURES

In terms of the local ordinance referred to above, no major development, or expansion of existing developments, is permitted to take place on the dolomites in the Transvaal without a thorough gravimetric survey, accompanied by appropriate exploratory drilling, having first been performed. However, even thorough investigation of this type seldom results in absolute confidence that every potential sinkhole has been located, except, for example, where the investigation has revealed the presence of intrusive sills of sufficient competence to inhibit sinkhole development. In addition, fairly extensive developments took place on the dolomites prior to the promulgation of this ordinance, including residential and industrial townships and mining installations. In few such cases had there been adequate site investigation - in most cases there had been none. Consequently the need has arisen, and particularly as a result of catastropic events during the past two decades, to seek means of protecting existing structures, and especially vital installations, built on the dolomites.

Based on the guidelines given by Jennings (1966 a) in his Kanthack Memorial Lecture, the protective measures currently practised may be summarised as follows:

1. Control of water

   Every effort must be made to prevent the ingress of excessive quantities of water into the soil near existing buildings by ensuring that water pipes and sewers do not leak, and that stormwater and other surface waste-waters are canalised and led to areas where the development of a sinkhole would not cause embarrassment. As it is difficult to provide sufficient flexibility in underground pipes to avoid all leakage, pipework has, in some instances, been kept on or above the surface to facilitate regular inspection and maintenance. Having emphasised the crucial role played by subsurface erosion in the development of sinkholes, the writer would again emphasise the vital importance of this aspect of protection.

2. Exploratory and protective drilling

   Boreholes drilled on a closely spaced pattern in the vicinity of important installations serve to disclose the presence of any voids in the residual soil and, if jumper-drills are used, to provide a record of resistance to penetration which might indicate the existence of arching conditions in the soil. They may also be used for the installation of depth points for level observations (i.e. telescopic bench-marks).

3. Observation of telescopic bench-mark behaviour

   The telescopic bench-mark, or TBM, was introduced as a protective, or rather warning, device in and around many buildings and installations on the Far West Rand in the mid-1960's. Basically a series of level pegs installed at various depths in a borehole and extending to the surface for observation, the TBM was a logical and practical consequence of the application of Jennings' theory of the successive collapse of rubble arches which culminates in the opening of a sinkhole. The device is illustrated in Figure 7/34. From the bottom of the borehole a long rod, the lower end of which is set in a block of concrete, extends to the surface. The rod is sleeved by one or more pipe casings, one inside the other, the lower ends of which are set in concrete plugs at progressively shallower depths, and the upper ends of which are provided with flanges at the surface to serve as observation points for changes in elevation of the concrete plugs.

In this way any decrease or increase in length of the vertical line represented by the borehole can be measured at surface at regular time intervals, and the movements can be plotted against time in graphical form. The graphs are scrutinised for any sudden changes that may be indicative of the development of subsurface arches. As illustrated in the idealised graphs in Figure 7/34, a shortening of the TBM line indicates that consolidation is taking place and a doline is developing; an elongation of the TBM line indicates arching conditions and the possibility of sinkhole development. If the inner rod falls away down the hole, this is simple and direct proof that an arch has collapsed above the depth of the deepest concrete plug and appropriate protective measures can immediately be initiated.

The expenditure of R4 433 000 on protective measures on the Far West Rand between 1962 and 1966, as reported by the Minister of Mines, was devoted mainly to the drilling of boreholes and the installation of TBMs.

Full-scale field tests were carried out during 1964/65 at the Pulik Caves near the Venterspost Mine, under the supervision of Professor Jennings, with the dual purpose of observing the mechanism of sinkhole development and monitoring the behaviour of TBMs (Jennings, 1966 a and 1966 b). Access to the caves was through an opening at the bottom of a sinkhole which had developed as a result of a

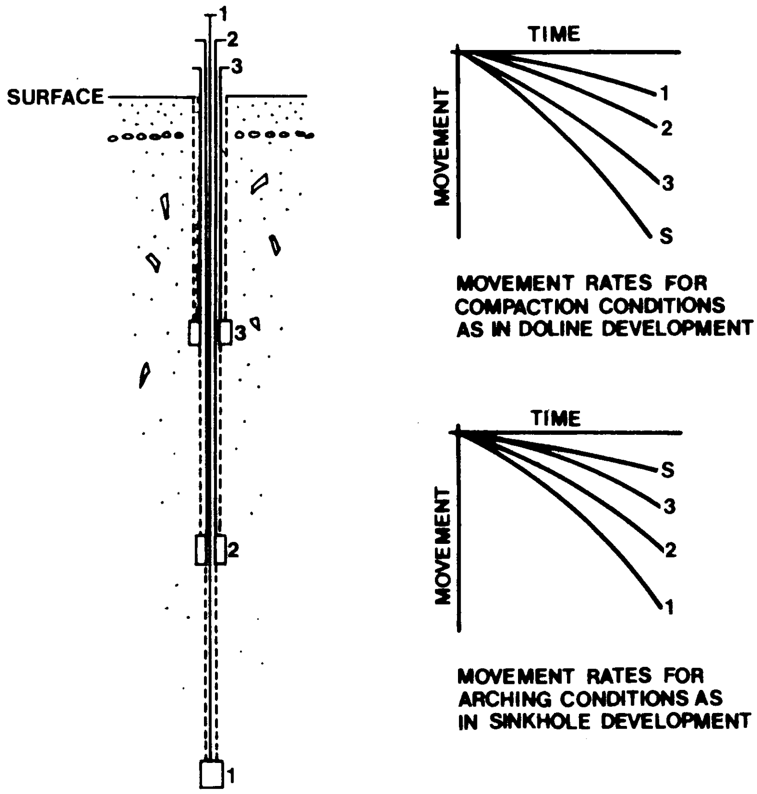

FIGURE 7/34 : The telescopic bench-mark (TBM)

(After Jennings, 1966 a)

leaking canal. The caves were in the form of two barrel-shaped voids developed at the top of two dolomite-bound slots, orientated at an angle of 60^0 to one another in plan. As shown in Figure 7/35 and diagrammatically in Figure 7/36, the sides of each cave were of dolomite covered by wad and with arched roofs composed of soil and chert rubble.

Briefly, the tests comprised the installation of a number of telescopic bench-marks and surface level pegs, followed by the artificial inducement of successive collapse of arches while observing the behaviour of the TBMs and surface pegs. Arch collapse was induced initially by introducing water down rows of raking boreholes which had been drilled for this purpose. As the arches rose towards the surface the voids became more stable and injection of water under pressure was necessary to induce further arch collapse. Later, further rows of boreholes were drilled vertically from the surface at one metre intervals to points near the quarter span positions and to within half a metre above the arched roofs, and charges of gelignite were exploded simultaneously at the bottom of these holes. This served to cause further collapse initially but, after a blast of four sticks of gelignite fired simultaneously in each hole had failed to produce further collapse, the experiment was abandoned. The roof of one cave was finally 6 m below ground level, the roof of the other 4 m and, at the 60^0 junction between the two, the roof was 5 m below surface. It is of interest that almost the greatest collapse took place at the junction of the two caves in a situation where no special efforts had been made to induce failure.

Clearly the collapse of the Pulik Caves was not a true simulation of the natural process: the collapse was aritificially induced, and the fallen material was not being removed by downward erosion through the basal slot. Nevertheless the experiment served to confirm the theory of successive arch collapse, and also illustrated both the effectiveness and the limitations of the TBMs. In the latter regard it must be recorded that it was only in the TBMs situated directly above the original arch (such as TBM B in Figure 7/36) that measured settlements increased with depth, i.e. the TBM lines became elongated. In some of the TBMs situated just beyond the arch (e.g. TBM A in Figure 7/36), an actual shortening of the

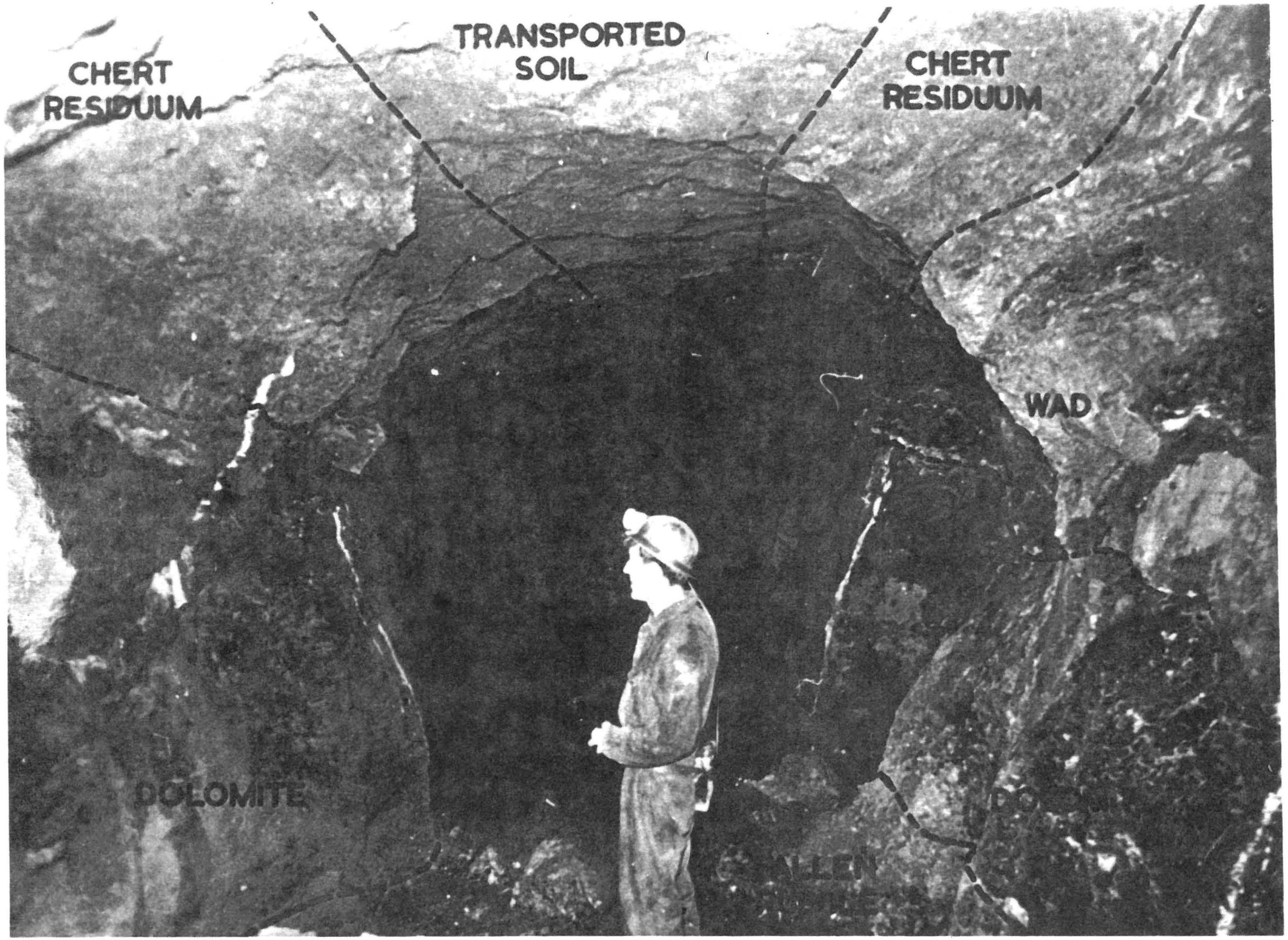

FIGURE 7/35 : Materials forming sides and roof of barrel-shaped void near entrance to Pulik Caves (Photo by J.E. Jennings)

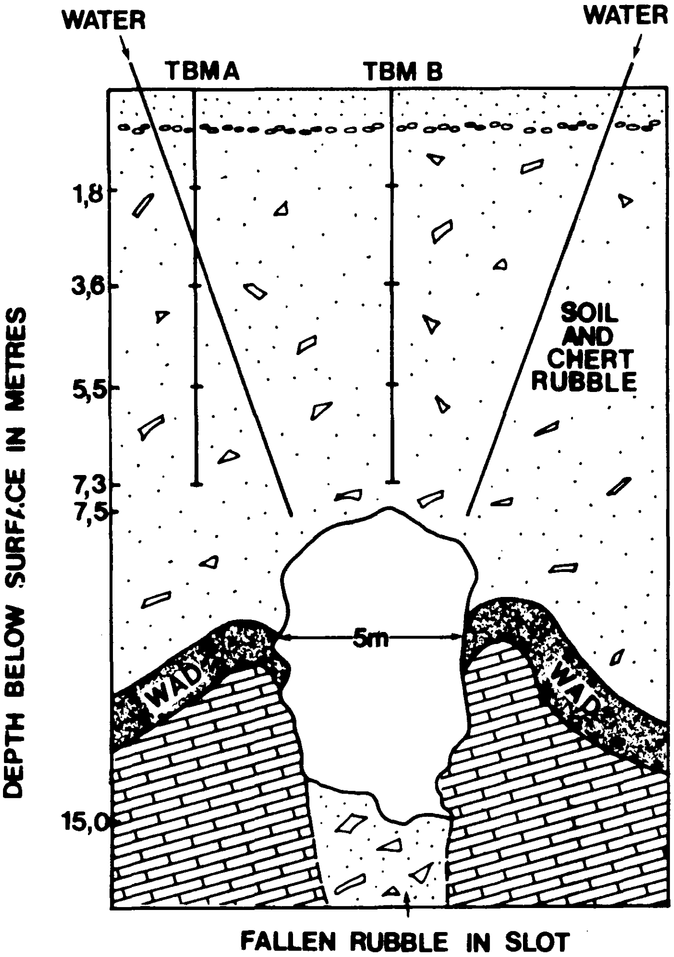

FIGURE 7/36 : Diagrammatic cross-section through Pulik Caves showing examples of telescopic bench-mark (TBM) positions vertical line was recorded, thus giving no indication of the arching condition near by. This observation stresses the necessity for locating these warning devices at close centres in areas which are to be thoroughly safeguarded.

4. Maintenance of buildings and other structures

Buildings erected on sites where gradual doline development is taking place can generally be considered as safe, except in the peripheral shear-zone. But regular maintenance has to be undertaken to re-level important structures where differential settlements have taken place, usually by jacking and wedging. The economic justification for remedial treatment of this type is a matter of judgement: some structures and installations do not warrant the expense, for others there is no alternative.

In conclusion the obvious should be stated. Prevention is safer and less costly than protection. Where thorough site investigation leaves doubt, it is better to build elsewhere - if at all possible.

ENGINEERING USES OF DOLOMITE

Having devoted the major part of this chapter to the hazards of building *on* dolomite, it would be unfair to conclude without some mention of the advantages of building *with* dolomite.

With a view to extending the use of dolomite (and limestone) as coarse aggregate in concrete, Fulton (1961) has collated the findings of a large number of research workers; he shows that, by comparison with concretes made from other acceptable aggregates, concretes made with dolomite aggregate have, in general, the following properties:

(i) a higher flexural strength;
(ii) an equal or greater compressive strength;
(iii) a more uniform resistance to wear;
(iv) a lower drying shrinkage;
(v) a greater durability; and
(vi) a lower thermal coefficient of expansion.

Concrete made with dolomite aggregate will also have:

(vii) a suitable resistance to freezing and thawing;
(viii) a comparatively high resistance to fire;
(ix) a satisfactory adhesion between the aggregate and the cement paste;
(x) a comparatively high density; and
(xi) a less variable modulus of elasticity than commonly expected.

Furthermore, it has been demonstrated by research conducted at the National Building Research Institute in the 1950's, and by subsequent observation in practice, that an effective solution to the problem of sewer corrosion is the use of dolomite aggregate in the manufacture of concrete sewer pipes. The reason for this is that, for a particular amount of sulphuric acid formed per unit area on the inner surface of the concrete pipe, the average rate of penetration is inversely proportional to its neutralisable base content (Stutterheim, 1954). Thus the rate of corrosion of sewer pipes made of concrete with dolomite aggregate is about one fifth or less of that for concrete made of non-calcareous aggregate; or in other words, the former lasts at least five times longer than the latter. The fruits of this research have been made generally available, and today there is hardly a country in the world where sewer pipes are not made in this way.

A word of caution is necessary regarding the use of dolomite as an aggregate in concrete or in sewer pipes. With its tendency to form wad, even on slight weathering, dolomite quarried for use as aggregate must be absolutely fresh. Fresh dolomite also provides a good basecourse crusher-run for road construction, but has too poor a 'polishing value' for use as aggregate in the bituminous carpet. It has been widely used, too, for railway ballast. An indication of the strength characteristics of fresh dolomite, and the range of variation to be expected, may be gauged from the following figures for Uniaxial Compressive Strength from tests conducted on samples from the Western Transvaal, near Mafeking:

Maximum Uniaxial Compressive Strength	:	256 MPa
Minimum Uniaxial Compressive Strength	:	51 MPa
Mean Uniaxial Compressive Strength	:	181 MPa

Number of tests conducted	:	6
Standard Deviation	:	77
Coefficient of Variation		0,425

Other significient uses of dolomite are in the manufacture of cement and as a flux in blast-furnace reduction of iron ores.

Finally, dolomite, particularly when closely interbedded with chert bands, is a popular decorative stone in rock-gardens, and commonly graces such artifices - even where it may appear incongruous to sensitive geological susceptibilities in areas where the underlying bedrock is of some other stratigraphic unit.

CASE HISTORY 17

BLACK REEF SERIES

(West Rand Anticline)

FOUNDATIONS FOR MILL AND SILO COMPLEX - RANDFONTEIN, TRANSVAAL

A preliminary site investigation was carried out in Randfontein during 1968 for a particularly large and heavy milling complex. The complex comprises a group of 63 hexagonal silos, 80 m high, covering an area of 90 m x 16 m, and a mill building of the same height covering an adjacent area of 46 m x 20 m immediately to the north of the silos. The silos are taller than any other in Africa (Figure 7/37).

Eight trial-holes were augered to refusal with a Hughes LDH 100 Digger and the soil profiles were recorded by the writer. The spiral auger refused in all of the holes on Black Reef quartzite at depths varying from 2,5 m to 10 m. The soil profiles down to the quartzite consisted of hardpan ferricrete overlying densely ferruginised shales. It was concluded that light structures associated with the complex could be founded on the hardpan ferricrete at pressures up to 300 kPa and that the heavy structure could be founded on the quartzite at pressures up to 3 000 kPa.

It must be pointed out, in fairness to the writer, that at this time (1968) no published work on the Black Reef Series mentioned the presence of *wad* within the stratigraphic succession.* Du Toit (1954), for example, makes mention only of conglomerates and 'dense, hard, and rather dark quartzites, thickly bedded, but with some shales, particurly at the very top just beneath the Dolomite ...' It was thus assumed that the ferruginised shales encountered in the trial holes represented the upper part of the Black Reef succession. Furthermore, it was established from the local mine that the series was underlain by sound rocks of the Witwatersrand System.

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* The first published mention of wad within the Black Reef Series, to the knowledge of the writer, was in 1974, in the work of Eriksson and Truswell.

FIGURE 7/37 : Mill and silo complex, Randfontein : the tallest silos in Africa.

These apparently reasonable conclusions were shattered at the beginning of 1970, when the writer had occasion to conduct a site investigation for an elevated water tower elsewhere in Randfontein, within a kilometre of the site for the proposed milling complex. The excavation for the structure revealed a sequence of shales and hard quartzites underlain by fine-grained conglomerates, a succession again thought to be entirely characteristic of the Black Reef Series on the West Rand Anticline. Consequently it was a great surprise when two boreholes drilled within the excavation revealed the presence of an horizon of wad, two metres thick, at a depth of 10 to 12 m below ground level. Undisturbed samples of the wad had a moisture content of 110%, a grain specific gravity of 3,47, a compression index of 1,26 and a void ratio of 3,89. It consisted mainly of clay with small percentages of silt and fine sand. As the presence of this material would have necessitated very costly foundation treatment, the elevated water tower was finally built elsewhere in the town where deep drilling revealed sound foundation conditions.

By this time the contractors had been appointed for the construction of the milling complex. It was suggested to them that a programme of deep drilling be undertaken to establish the sequence of strata below the quartzite on which the initial auger-holes had refused. Substantial thicknesses of soft materials were encountered but it was not possible to obtain cores of these materials with a diamond-drill. In order to examine the profile in detail, two exploratory shafts were drilled in the positions shown on Figure 7/38, with a Wirth L3 hard-rock drilling rig to depths of 24,5 m; owing to the extreme hardness of the quartzites, each hole took more than a hundred hours to drill. These shafts and a test-pit excavated near the north-western corner of the mill site, were examined by the writer, and revealed the following (generalised) sequence of strata:

0,0 - 5,0 m	Hardpan ferricrete
5,0 - 9,0 m	Ferruginised residual shale
9,0 - 10,0 m	'Upper' quartzite
10,0 - 11,7 m	Wad
11,7 - 12,0 m	'Middle' quartzite
12,0 - 13,0 m	Wad
13,0 - 14,0 m	'Lower' quartzite
14,0 - 14,5 m	Residual shale
14,5 - 15,5 m	Wad
15,5 - 24,5 m	'Founding' quartzite with bands and lenses of white shale

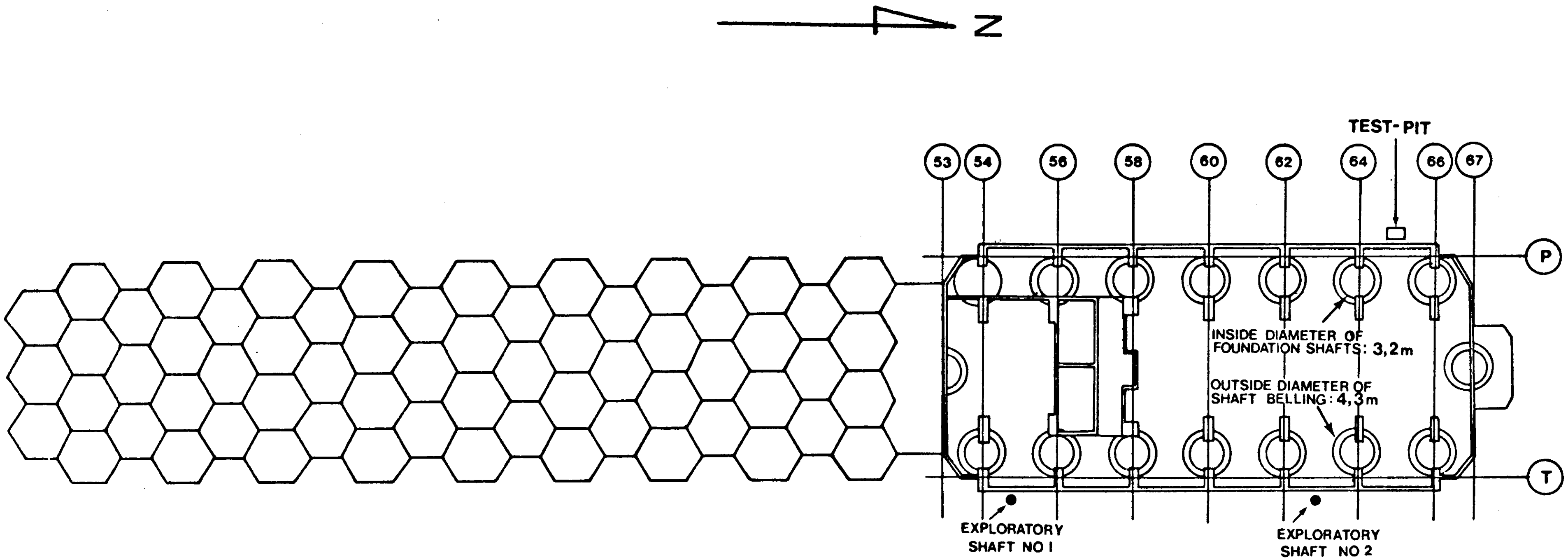


FIGURE 7/38 : Site plan showing layout of mill building and silo complex and positions of exploratory shafts, Randfontein

The initial auger-holes had, of course, refused on the 'upper' quartzite. Had the complex been founded on this layer, it would not have stood.

At this stage the contractors' design engineers decided to excavate the entire site to a depth of about 13 m and then to take individual footings down to the surface of the lowermost quartzite horizon, which consequently became known as the 'founding' quartzite. Blasting was required through the quartzite horizons, and the wad layers were simply scraped up by bulldozer.

When the bulk excavation was complete it was seen that the strata had a general dip of about 5^0 towards the north-east. The suboutcrop of the founding quartzite was thus shallowest on the southern side of the complex. This meant that the silos could be founded on a raft on this material at depths varying from about 13 m to 17 m, and at bearing pressures of the order of 500 kPa.

On the northern portion of the site, under the mill building, conditions were more complex. The founding quartzite was not only deeper here, owing to the general direction of dip, but the whole sequence of strata was found to be disrupted by a succession of four step-faults. These faults showed up clearly on the northern face of the bulk excavation, as can be seen in Figure 7/39. They all dip eastwards at about 65^0 and have downthrows in the same direction. They are thus clearly normal or tensional faults, as would be expected in the crestal zone of a major anticline.

A close examination of these faults revealed that, although they could be traced upwards into the ferruginised residual shale, they gradually died out in the overlying hardpan ferricrete. It was therefore inferred that the faulting was relatively old and had not been reactivated since the time of ferricrete formation on the African erosion surface. More likely the area has been tectonically stable since pre-Karoo times.

The founding quartzite, and the shale horizons within it, had been locally severely brecciated by the faulting. This fact coupled with the greater depth of the suboutcrop, necessitated the excavation of fourteen large-diameter foundation shafts for the mill building as shown in Figure 7/38.

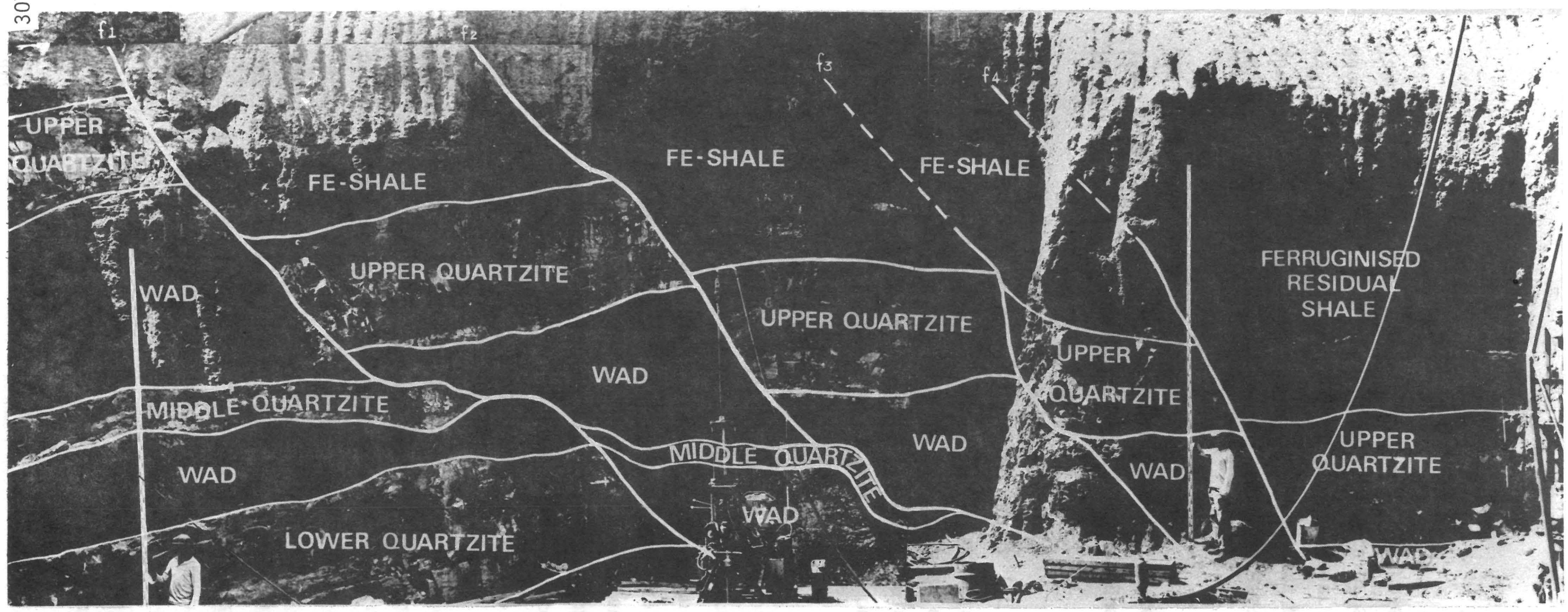


FIGURE 7/39 : Composite photograph of north face of the bulk excavation for mill building, Randfontein, showing displacement of Black Reef strata resulting from four step-faults

From what was observed of the faulting on the north face of the excavation and in the three northern shafts, a structural geologist, Mr E.J. Poole, concluded that the faults (numbered for convenience f1 to f4 from west to east) belong to a sympathetic or imbricate system of which fault f4 is the master displacement. It appeared, therefore, that a strong possibility existed that all four faults would gradually coalesce in depth, terminating in one main fault in the position of f4. This was supported by the observation that, in the shaft situated in the middle of the northern side of the bulk excavation (on line 67) the dip of fault f1 became flatter and ran into f2 which itself had attained an inclination suggesting that it would merge with f3 at further depth.

The foundation shaft on the north-eastern corner (on line 66) had to be excavated deepest owing to the very intense brecciation encountered between faults f3 and f4, which resulted in a substantial decrease in bearing capacity. The direction of strike of faults f3 and f4 was such that brecciated conditions were again, predictably, encountered in the three immediately adjacent shafts on the eastern side, i.e. on lines 64, 62 and 60. Sound founding material was, however, encountered in these shafts at progressively shallower depths towards the south as may be seen from Figures 7/40 and 7/41.

From an examination of the western face of the bulk excavation it appeared that only faults f3 and f4 were present. This was probably due to a coalescence of f1, f2 and f3. The magnitude of displacement had also decreased somewhat, and consequently the intensity of brecciation encountered in the zone between the two faults (in the shaft on line 62) was less than on the eastern side.

The foundations of the mill building were designed to exert pressures of the order of 1 500 kPa on the founding quartzite at the base of the belled-out foundation shafts.

The occurrences of wad in the Black Reef succession would seem to be quite extensive on the West Rand Anticline, and possibly elsewhere as well. More recent site investigations for other heavy structures in Randfontein have revealed a similar sequence of strata, and the wad layers in some places are even thicker than those encountered at the milling complex site.

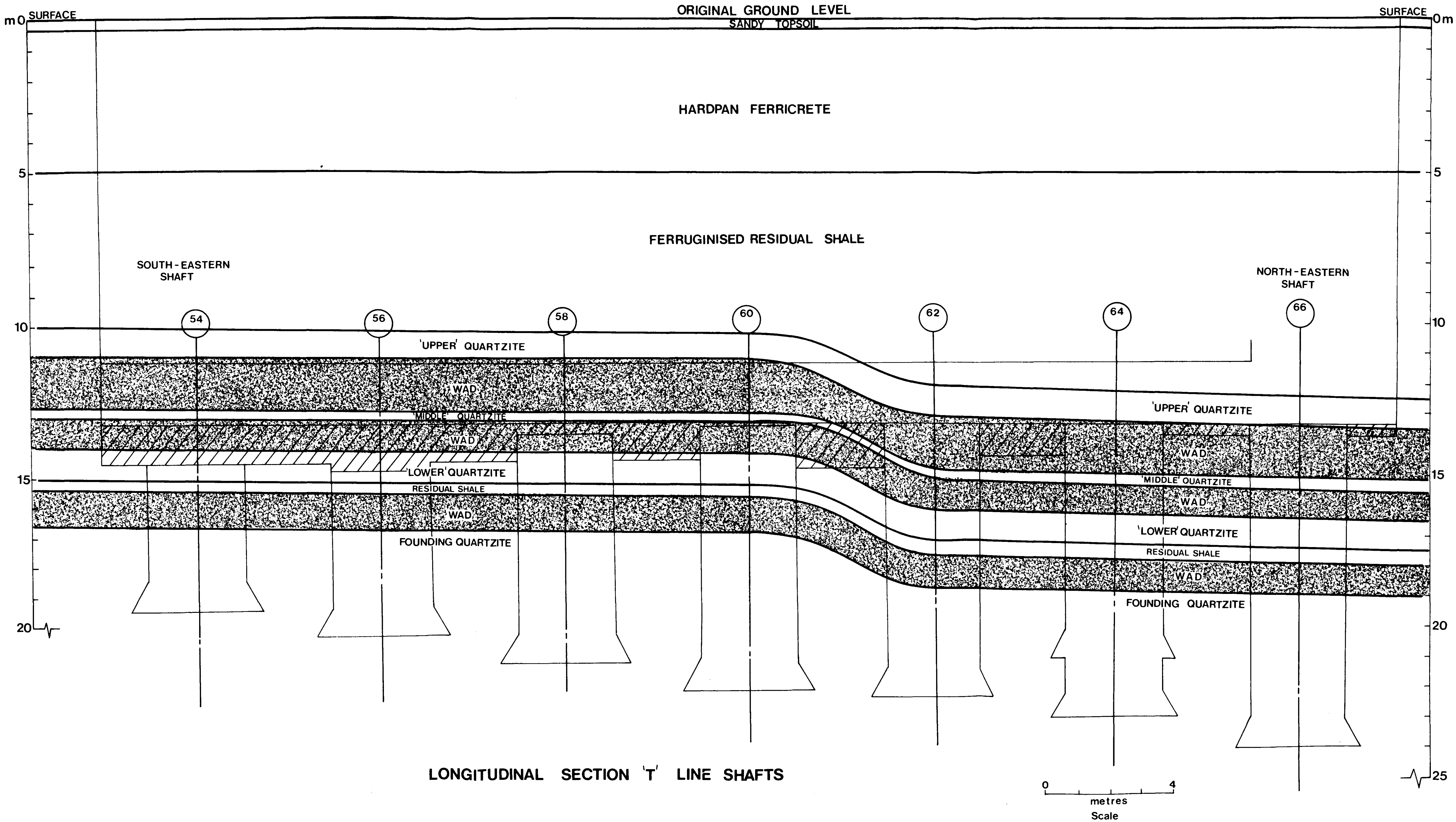
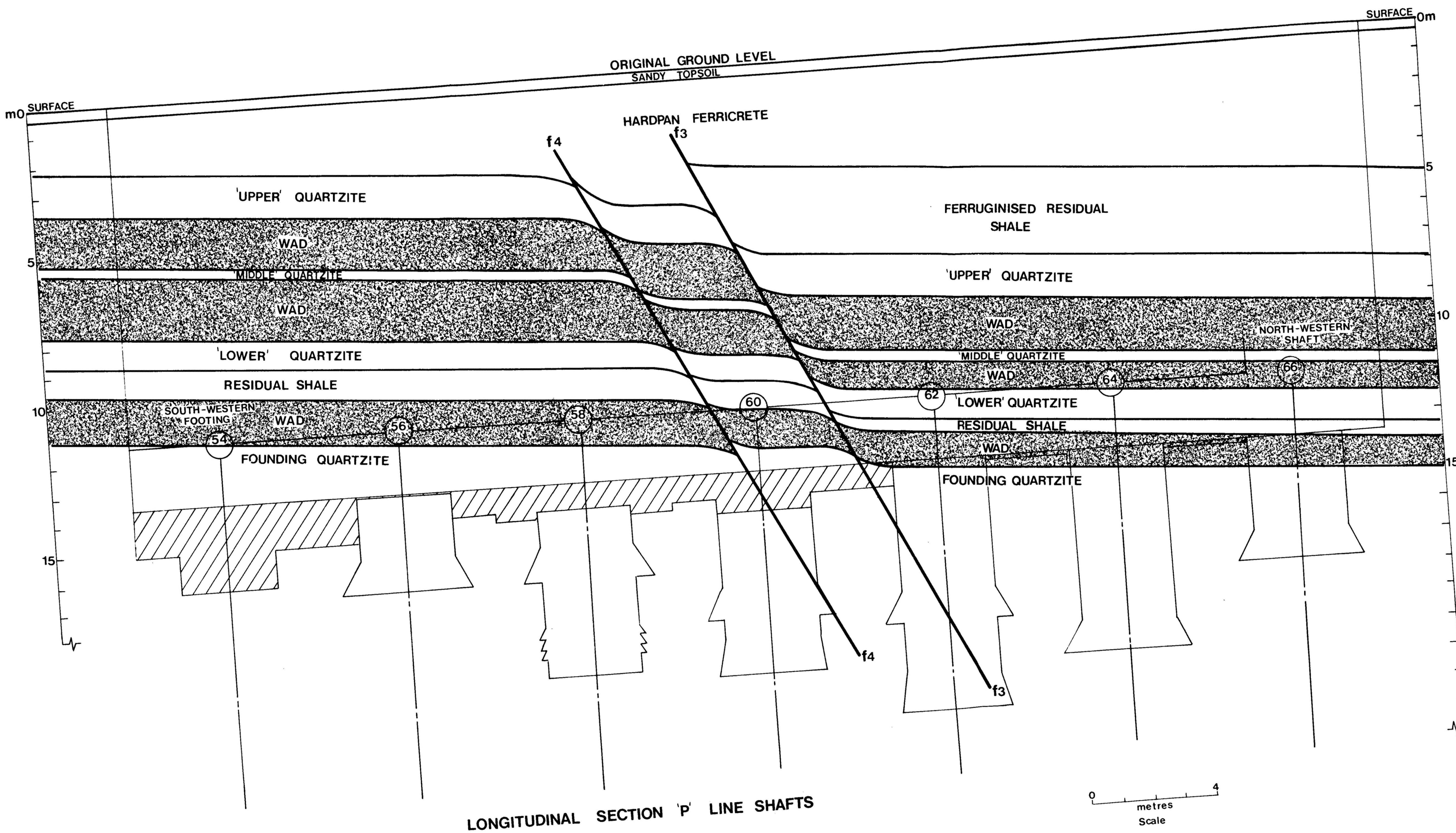


FIGURE 7/40 : Succession of Black Reef strata exposed on eastern face of bulk excavation for mill and silo complex, Randfontein



LONGITUDINAL SECTION 'P' LINE SHAFTS

FIGURE 7/41 : Succession of Black Reef strata exposed on western face of bulk excavation for mill and silo complex, Randfontein

CASE HISTORY I8

KAROO OUTLIER ON DOLOMITE SERIES

DIFFERENTIAL SETTLEMENT IN KILNS: DRIEFONTEIN BRICKWORKS

A unique opportunity for observing the effects on surface of the lowering of the water-table through thick bodies of wad was afforded during the early days of the dewatering of the Bank Compartment.

The Driefontein Brickworks is situated on a Karoo outlier within the Bank Compartment (Figure 7/9). Bricks are manufactured from the weathered Karoo shales. The basal strata of the Karoo outlier are underlain by dolomite. Below the Dolomite Series, as shown in Figure 7/10, is the intersection of the eastern and western limbs of the Bank 'Break' which dislocates the pre-Transvaal rocks.

The brick-making plant as originally built included six kilns, 122 m long, into which were fed the raw bricks on cars operated by hydraulic rams. In May, 1970, when dewatering of the Bank Compartment was approaching the maximum rate of pumping from the East and West Driefontein Mines, the brick cars jammed and it was found that the rails conveying the cars were out of alignment. The rails were re-levelled but the cars again started jamming and some came off the rails. At the same time structural cracks started developing in the kilns and in the batching plant at the southern end of the kiln complex (Figure 7/45). In June 1970 levelling surveys were commenced under the supervision of Dr O.L. Papendorf with a view to monitoring ground movements, and it soon became apparent that differential subsidence of a substantial order was taking place.

This programme of levelling continued till December 1970, at which stage precise levelling surveys were commenced by Professor I.B. Watt using a more sophisticated network of bench-marks.

In order to establish the cause and the nature of the movements a detailed gravity survey of the area was conducted by the Geological Survey and an intensive programme of exploratory drilling was instituted under the supervision of Professor J.E. Jennings. About two hundred boreholes were drilled during 1970 and 1971 and telescopic bench-marks were installed in 132 of them. Most of the boreholes were drilled with 'down-the-hole' jack-hammer type drills using compressed air to blow out the fragments of rock crushed by the percussion-bit; some were drilled with jumper-drills, and two were drilled with a special rotary soil sampling machine for the purpose of obtaining continuous undisturbed samples for laboratory testing. The 'chips' recovered from the boreholes were examined by geologists, and descriptive logs were prepared for each borehole.*

From the data obtained from the boreholes and from the gravity survey, a model of the subsurface conditions was constructed by Mrs Patricia Moon. Figure 7/42 shows an oblique photograph of the 'working model'. Figures 7/43 and 7/44 show oblique views of a further model, referred to as the 'demonstration model', which was based on the working model.

For the purpose of preparing the working model the borehole logs were simplified in terms of six categories of soil and rock materials which were colour-coded on the pegs used to construct the model. Descriptions of these materials follow, starting with the oldest and deepest formation and working up to the youngest and shallowest:

(i) <u>Dolomite</u> (shown in blue on the working model):

The dolomite rockhead was interpreted on the model as fresh rock constituting the upper part of the mass of bedrock of the Dolomite Series, and possibly including a few large 'floaters'. As will be seen from the photographs, the dolomite rockhead proved to be highly irregular and contained deep 'slots' or subsurface troughs separated by elongated pinnacles. Three deep, steep-sides slots were recognised during the construction of the working model, all of which were more than 150 metres

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* Boreholes 1 to 49 were logged by P. Roux of Geological Survey, and the other percussion and jumper-drill holes by H. Meaden, R. Daniel or D. Campbell of Gold Fields of South Africa Limited. Samples from BH 1 and BH 2, the two holes drilled with the special rotary machine, were described in the laboratory by Professor J.E. Jennings and Messrs J.A. Caldwell, M.R. Royal, J.H. de Beer and A.B.A. Brink.

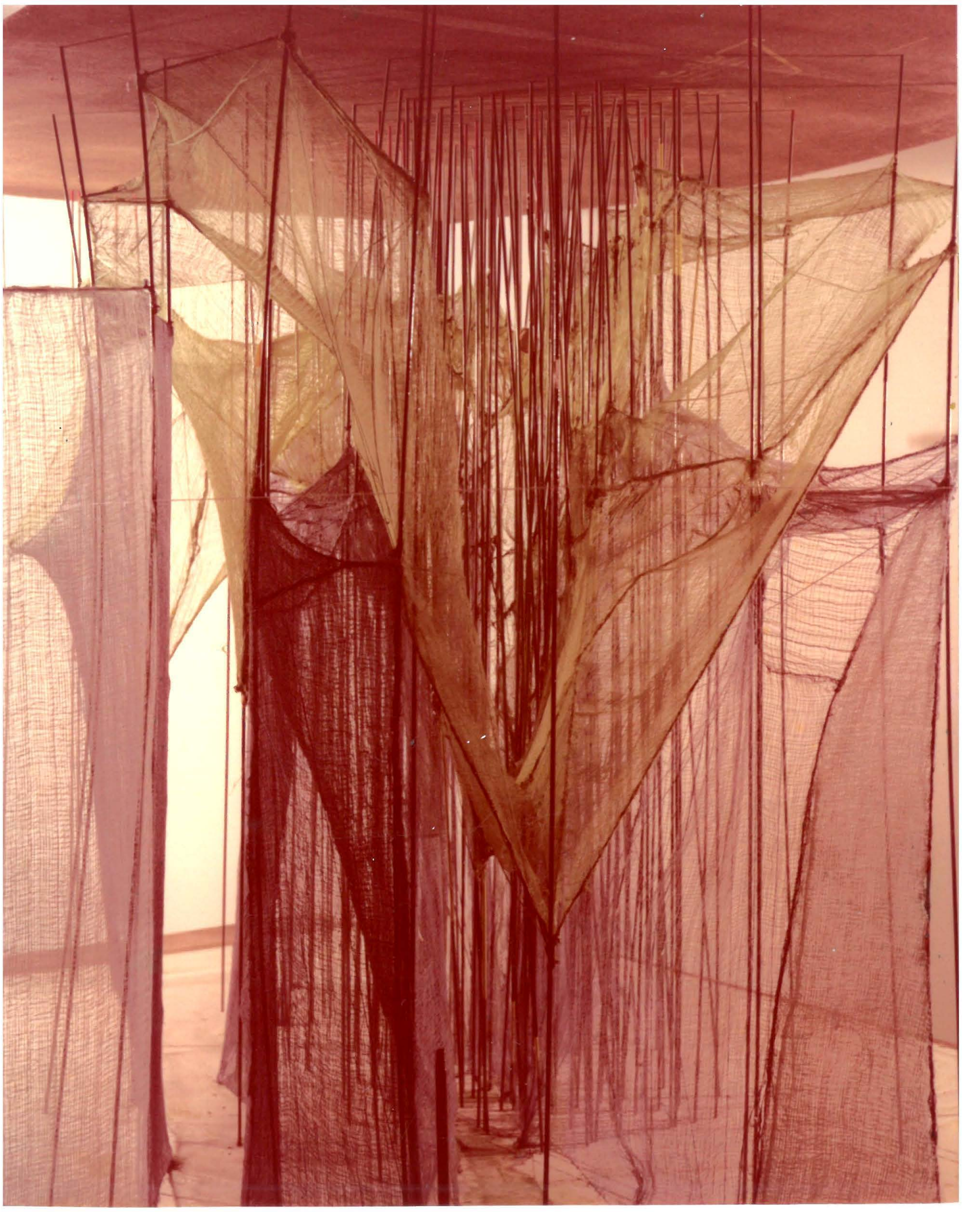


FIGURE 7/42 : *Oblique view of the working model of subsurface conditions at Driefontein Brickworks*

Base of the Karoo outlier shown in green and dolomite rockhead in blue
Vertical scale exaggerated

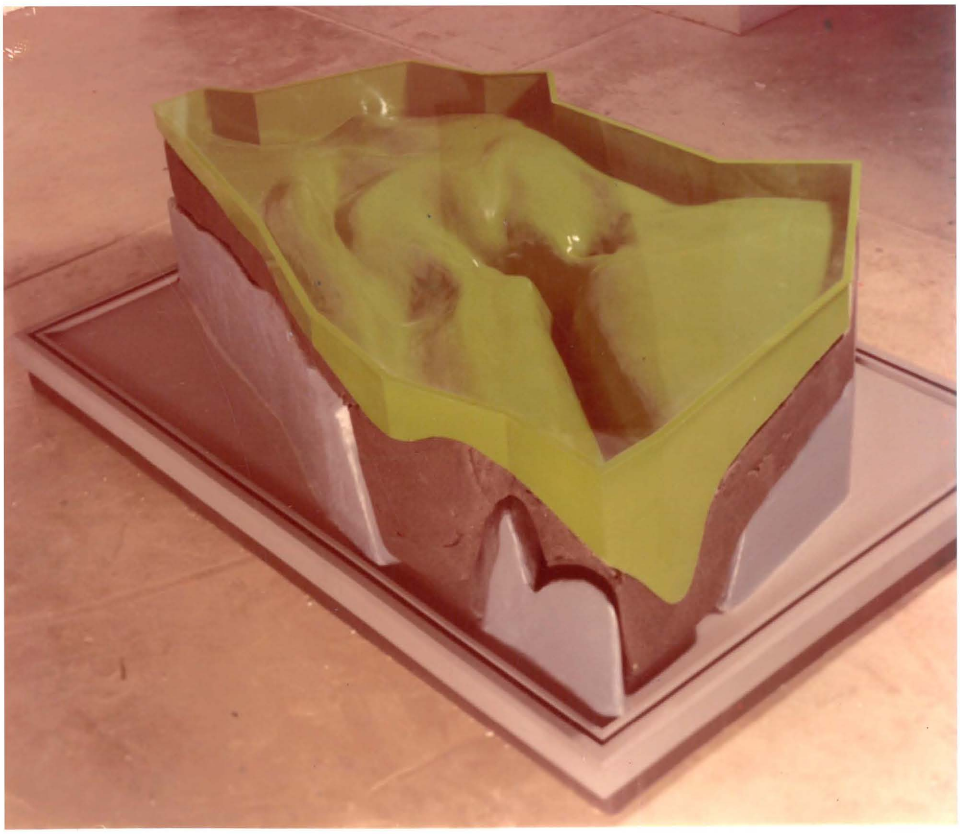


FIGURE 7/43 : Oblique view of the demonstration model of subsurface conditions at Driefontein Brickworks

Base of Karoo outlier shown in green Wad shown in black
Dolomite rockhead shown in blue

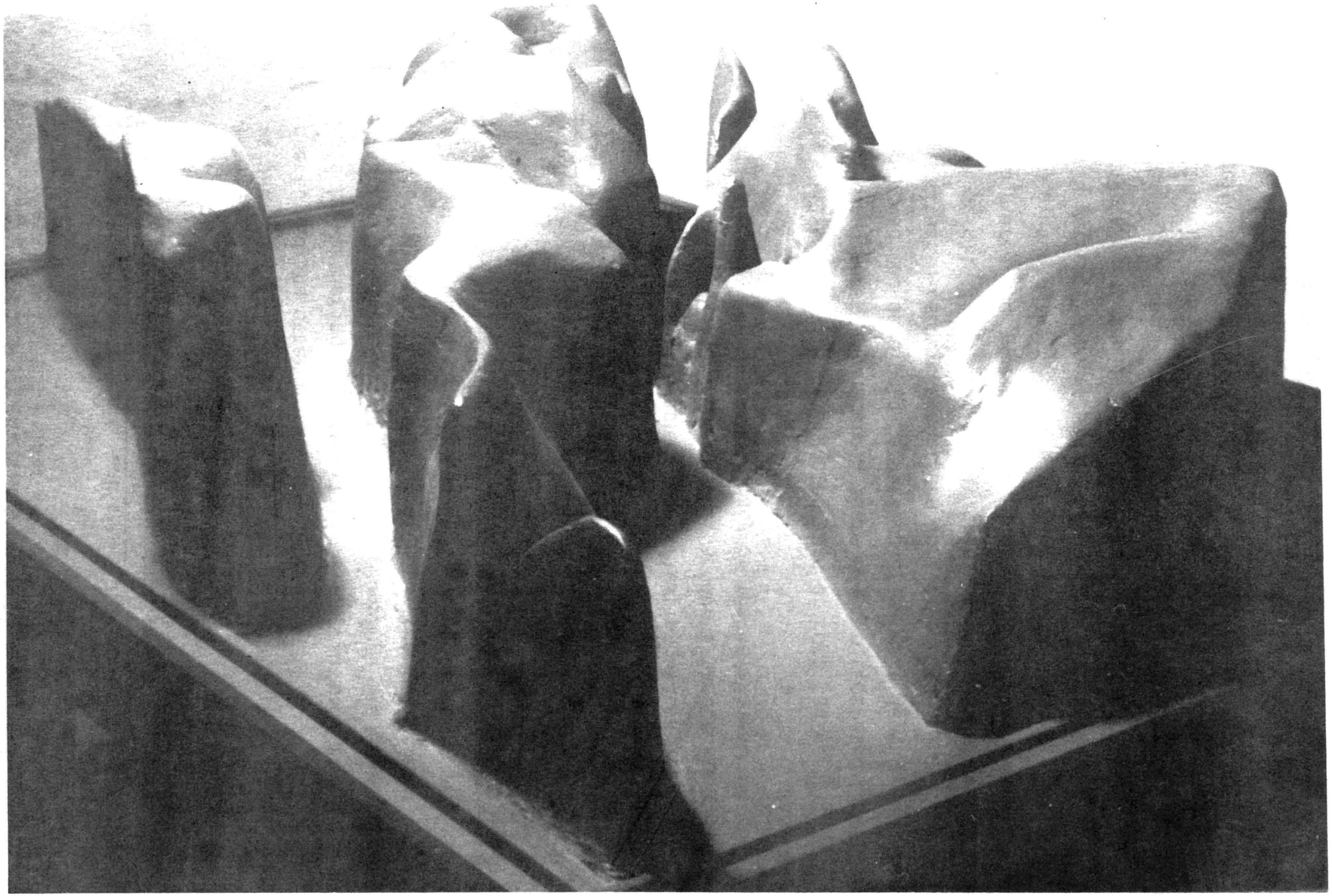


FIGURE 7/44 : Oblique view of the demonstration model of subsurface conditions at Driefontein Brickworks showing configuration of dolomite rockhead in depth. The two most prominent slots trend in a SSW-NNE direction: one to the east of the kilns, the other directly below the kilns. A somewhat smaller slot trends WSW-ENE is also present below the kilns.

(ii) Wad and weathered dolomite and chert (shown on the working model in yellow where no wad is present and in black where wad predominates):

As previously discussed, rocks of the Dolomite Series yield chert, wad and other minor insolubles as well as friable dolomite on weathering. While all these consitutents have a similar origin, a subdivision into two groupings was made according to whether wad was present or not. The zones shown in black on the working model and on the demonstration model (Figure 7/43) included voids, wad and chert.

Chemical analyses of four samples of chips of fresh dolomite selected at random from the boreholes revealed manganese contents greater than one per cent (mean 1,28 per cent) which confirmed the stratigraphic identification as that of the 'Upper Dark Dolomite Zone'. This accounts for the large quantities of wad encountered in many of the boreholes.

As will be seen from the photographs of both the working model and the demonstration model, there are two separated areas in which wad has developed to particularly great depth. These are situated below the northern portion of the kilns and southwest of kiln 1. Exceptionally thick occurrences of wad in the former area are present below kilns 4 and 6, to the east of kiln 6 and to the west of kiln 1; thicknesses are in excess of 30 metres. When relating wad thicknesses to the presence of slots it is found that the main SSW-NNE slot corresponds with most of the occurrences of thick wad but there is a broad belt where little or no wad has developed below the southern portion of the kiln site.

(iii) Coal, carbonaceous shales and tillite of the Dwyka Series (coal and carbonaceous shales are shown in green, grey was used to indicate tillite):

Sediments belonging to the lowest part of the Dwyka Series are thickest within the slots in the dolomite. The carbonaceous sediments consist of grey to olive silts with occasional subangular pebbles of quartzite and vein-quartz; they are considered to represent tillite or, more likely, fluvio-glacial sediments. These sediments represent deposition from the meltwaters of the glaciers which moved along the U-shaped slots in the dolomite. At the termination of glacial activity bodies of stagnant water in the slots would have favoured the development of carbonaceous sediments.

In many places the carbonaceous sediments immediately overlie either solid or weathered dolomite. Confirmatory tests were carried out on 38 samples to ensure that coal had not been mistakenly identified as wad. These tests were carried out by Mr T.S. McCarthy of the Department of Geology at the University of the Witwatersrand, using X-ray fluorescence spectrometry techniques; in all cases the logging was proved to be correct.

Bands and lenses of light grey silty shale were found interbedded with the carbonaceous sediments in many of the boreholes.

(iv) Non-Carbonaceous sediments of Dwyka and Ecca Series (shown in grey):

Overlying carbonaceous sediments above the slots, and solid or weathered dolomite above the pinnacles, are pale-coloured sediments of the Upper Stage of the Dwyka Series and of the Lower Stage of the Ecca Series. Sediments of this zone have a fairly constant thickness and are present over the full extent of the property. They consist of olive-grey shales and sandy shales with isolated grit bands and scattered subangular pebbles. Towards the top of this zone the sediments are somewhat oxidised and have a blotched dark and light reddish-orange colour.

(v) Pleistocene Sediments (shown in red on the working model):

Ferricrete, alluvium, hillwash and aeolian soils of Pleistocene age constitute the Upper horizons of the soil profile and extend to depths of between 4 and 20 metres.

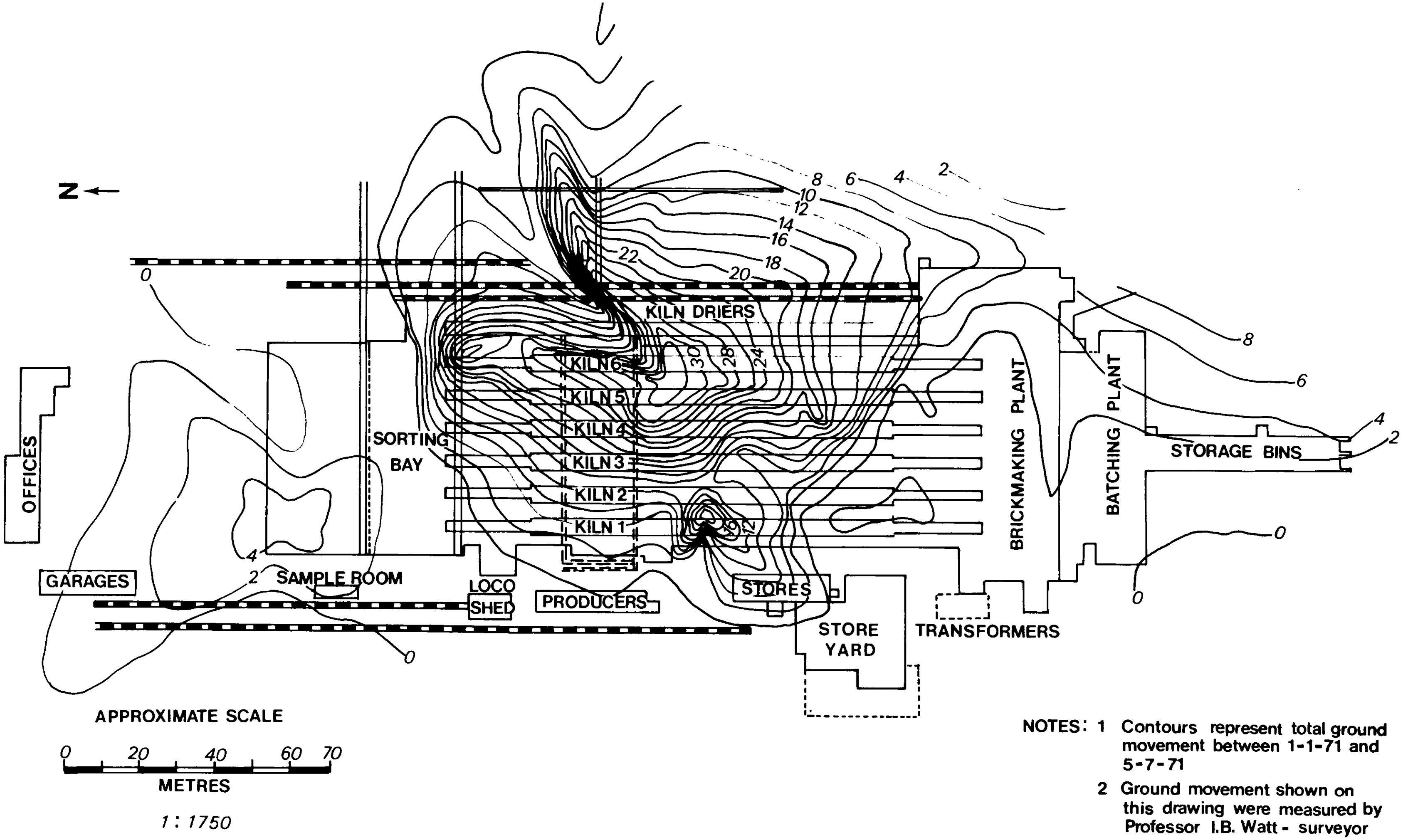
It may be seen from the photograph of the working model that these materials are particularly thickly developed where they are situated above the main SSW-NNE slot in the underlying bedrock.

As discussed earlier in this chapter, the level of the water-table in the Bank Compartment prior to artificial dewatering was related to the post-African period of minor incision of the Wonderfontein Spruit. At the time of this incision and the consequent natural lowering of the water-table to the 1 504 metre level, certain masses of previously saturated wad would have been drained and have come to lie in the vadose zone. Below the Driefontein Brickworks, however, the thick bodies of wad within the deep subsurface troughs below an elevation of 1 504 metres remained saturated until artificial lowering of the water-table commenced in the early 1970's.

During the progressive lowering of the water-table in 1970 and 1971 these bodies of wad became drained and underwent consolidation to a dramatic extent. Settlement due to this consolidation as shown by movements on the telescopic bench-marks, was transmitted upwards through the overlying Karoo strata and was reflected on the surface in a surprisingly short period of time.

Contours of subsidence of the surface during the six month period from 1st January 1971 to 5th July 1971 are plotted in Figure 7/45. These contours are based on the precise level observations made by Professor Watt during this period. The configuration of differential settlement of the ground surface as shown by these contours is an almost exact replica of the configuration of the dolomite rockhead as deduced from the working model.

So severe was the damage to the kilns as a result of differential settlement that they had to be demolished during 1972. New kilns were built farther to the east during that year, still within the confines of the



NOTES: 1 Contours represent total ground movement between 1-1-71 and 5-7-71

2 Ground movement shown on this drawing were measured by Professor I.B. Watt - surveyor

3 All contours shown in millmetres

FIGURE 7/45 : Layout of kilns and other plant at Driefontein Brickworks showing contours of subsidence of the ground surface during the period 1st January to 5th July, 1971

(Prepared by Professor I.B. Watt)

Karoo outlier. They were designed as small kilns of the downdraught type which could tolerate substantial differential settlement and cracking without being put out of operation. But by this time most of the consolidation of the deep-seated bodies of wad had already taken place, and the new kilns show no signs of distress.

8 TRANSVAAL SYSTEM

PRETORIA SERIES

SEDIMENTS

As a general observation it may be stated that residual soils formed from Pretoria shales and quartzites are shallow, often less than 2 metres thick, and that they commonly consist entirely of inert minerals. This is certainly the case with most of the quartzites, though one significant exception will be discussed later. It is true also of most occurrences of shale, particularly where these have given rise to residual soils which are red or yellow in colour. X-ray analyses of such residual and weathered shales show little variation in the mineral content, the main constituents being quartz, kaolinite, hydrous mica and mica with subordinate felspar and iron oxides, and sometimes chlorite (Loubser, 1967). High bearing capacities generally obtain at surface or at shallow depth, except where deep transported soils are present, and foundation problems are thus seldom encountered in the sedimentary beds of the Pretoria Series*.

Residual Magaliesberg shales with expansive characteristics

There are, however, exceptions to the above general observations, notably in parts of the Magaliesberg Stage. For example, a double-storey warehouse at Hermanstad, west of Pretoria, suffered severe damage as a result of heaving movements which started even before the roof had been placed. A trial-hole augered to a depth of 12 m revealed a superficial mantle of dense sand, 3 m thick - a hillwash deposit derived from the near-by Daspoort quartzite ridge - overlying soft, white residual Magaliesberg shale. Indicator tests which were carried out on three samples of the residual soil gave the following results**:

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* It must be appreciated that this is a very broad generalisation: cases are on record where residual Pretoria shales have a consistency no greater than 'firm' to depths in excess of 15 m.

** Personal communication from D.H. van der Merwe, 1966.

DEPTH	LL	PI	LS	% CLAY	% SILT
3,0 m	69	45	17	43	43
6,5 m	58	30	12	40	47
12,0 m	60	29	9	26	46

These figures, together with the results of X-ray analyses which showed a preponderance of montmorillonite and only small amounts of kaolinite, mica and quartz in all three samples, indicate the potentially highly expansive nature of the soil. Differential movement of the floors and the foundations was intensified as a result of the presence of tall bluegum trees desiccating the soil on one side of the warehouse, and rain-water accumulating in an open plumbing trench under the centre of the building. There is no record of the depth of the water-table, but it is thought that water was encountered at relatively shallow depth in the trial-hole.

The warehouse described above is situated on an extensive low-lying pediment at the foot of the Daspoort ridge. A similar occurrence of heaving has been reported from a school site in Queenswood in north-eastern Pretoria. The site here is again on a pediment developed on Magaliesberg shales, with a water-table at shallow depth, and the residual soil again being light in colour. Without further evidence, however, it is not possible to speculate as to whether this phenomenon is associated with a particular stratigraphic horizon near the base of the Magaliesberg shales, or rather with hydromorphic development produced by the presence of a water-table at shallow depth below a pediment. In support of the stratigraphic hypothesis it should be mentioned that expansive characteristics have also been observed in residual shales from near the base of the Magaliesberg Stage in Fochville.

Residual Magaliesberg quartzite with collapsible characteristics

Possibly also related to a stratigraphic control is the occurrence of a highly felspathic horizon in the Magaliesberg quartzite which, on decomposition in a favourable topographic situation, produces a highly leached residual soil with a collapsible grain structure. At present there is no indication of how widespread this phenomenon may be: it has been detected so far in only one locality, on a portion of the proposed township of Boschdal south of Rustenburg. First noticed in a road cutting in the area, the material was subsequently described and sampled in an augered hole (Figure 8/1). The residual quartzite at the sampling depth of 4 m has a consistency of very loose sand. The grading curve shown in Figure 8/2 is not dissimilar to that for residual granite (cf Figure 3/17). X-ray analysis showed the main minerals to be quartz and kaolinite. Laboratory testing produced the following results:

Liquid Limited	:	31%
Plasticity Index	:	12%
Linear Shrinkage	:	5%
Clay content	:	5%
Silt content	:	17%
Natural moisture content	:	7,6%
Degree of saturation	:	30%
Bulk density	:	1 709 kg/m^3
Dry density	:	1 585 kg/m^3

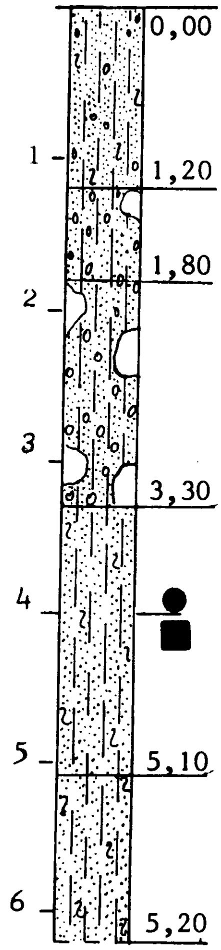
An undisturbed specimen loaded in the consolidometer to an effective stress of 170 kPa and then saturated produced the e log P curve shown in Figure 8/3, from which a substantial collapse potential of 3,5 per cent was determined.

The topographic situation in which this unusual soil developed was at the head of a pediment at the foot of a scarp face developed in hard quartzite. It may be seen from the soil profile in Figure 8/1 that the residual soil horizon exhibiting the collapsible grain structure was overlain by a stratum of very stiff residual soil containing large inclusions of unweathered quartzite. This suggests a stratigraphic control insofar as the material with the collapsible grain structure was confined to a stratum of highly felspathic quartzite.

French drains

In certain peri-urban and rural areas in Pretoria district difficulty is experienced in constructing effective french drains for the disposal of effluent from septic tanks, owing to the impermeability of the Pretoria shales. Long shallow trenches cut in the thin mantle of transported gravels overlying the dense shale have proved more satisfactory than the conventional drain two metres deep.

BOSCHDAL



Moist, reddish brown, in profile reddish brown, loose to medium dense, intact, silty, coarse, medium and fine sand with fine gravel and roots; hillwash.

Subangular to subrounded, boulders and coarse gravels in a matrix as above but very loose; pebble marker.

Moist, dusky red, in profile dusky red, very stiff, intact, silty, coarse, medium and fine sand with boulder-sized inclusion of hard rock quartzite; weathered (and partially ferruginised) Magaliesberg quartzite.

Moist, reddish brown, in profile yellow speckled red, brown and grey, very loose, intact, micaceous silty, medium and fine sand with roots; residual Magaliesberg quartzite.

As above but with consistency very dense; residual Magaliesberg quartzite.

NOTES

1. Hole augered to 6,3 m with 750 mm diameter auger-spiral of Hughes MF 60 Digger but not to refusal. Refusal depth nearly attained.
2. Water-table not encountered.
3. Disturbed and undisturbed samples taken at 4,0 m.

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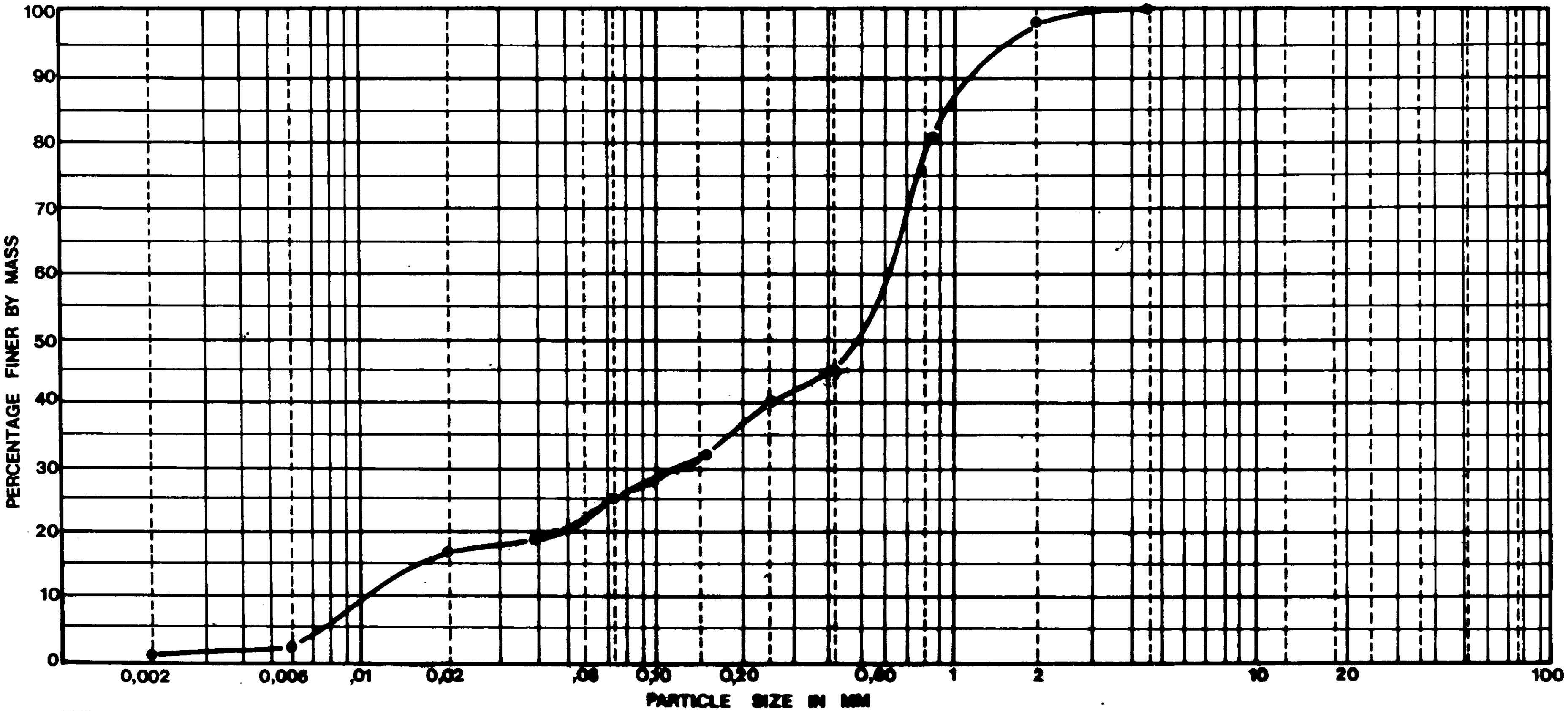
17/7/75

FIGURE 8/1 : Soil profile of residual Magaliesberg quartzite with collapsible grain structure : Boschdal, Rustenberg



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse
	SILT			SAND			GRAVEL		

FIGURE 8/2 : Grading curve for residual Magaliesberg quartzite with collapsible grain structure

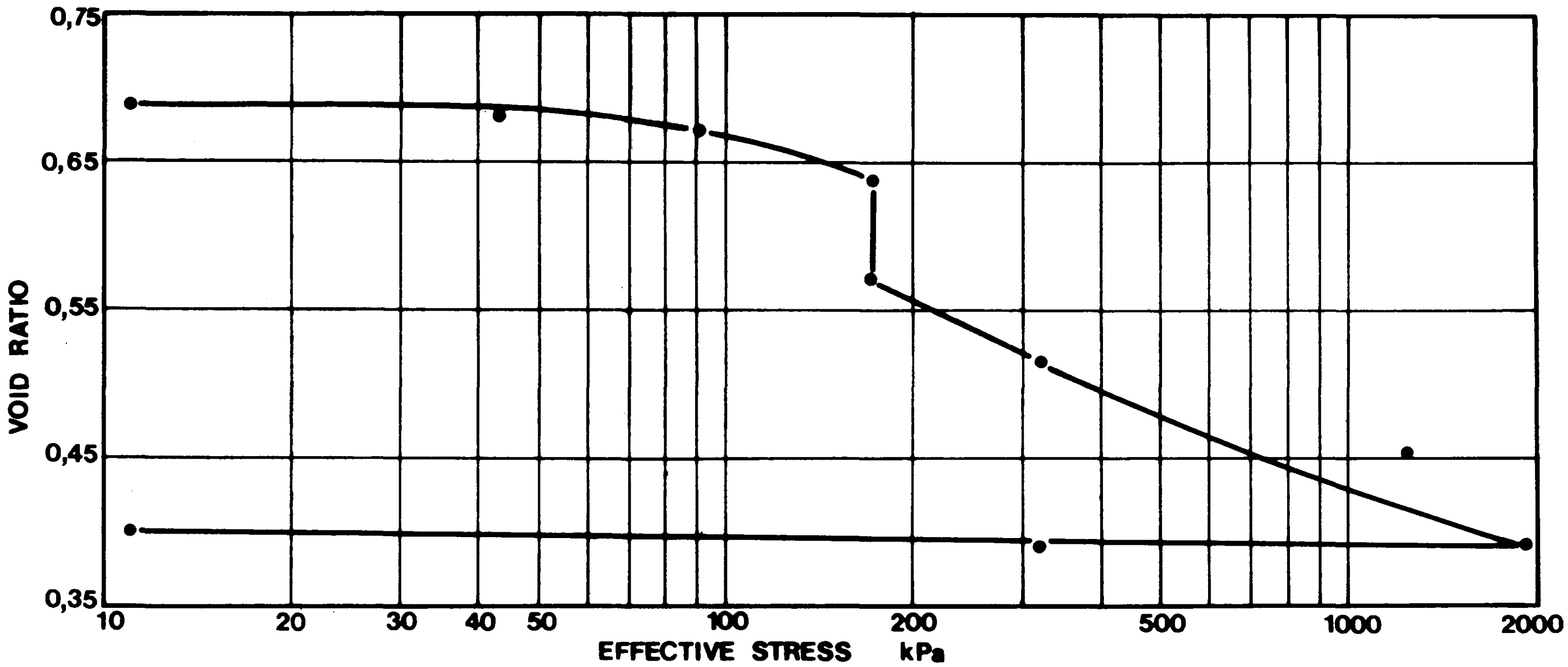


FIGURE 8/3 : Consolidation curve for residual Magaliesberg quartzite with collapsible grain structure

Sample saturated at a load of 170 kPa

Construction material

The quartzites, which contain a fairly high proportion of felspathic sandstone, generally provide an excellent source of crusher-run base-course for road construction. The shales, if carefully selected on the basis of the right degree of induration, may be used as sub-base and sometimes even as base-course material, but they require either an admixture of granular material to improve the grading, or stabilisation, preferably with lime rather than cement, to decrease plasticity (Loubser, 1967).

ONGELUK LAVAS

Residual soils developed on the Ongeluk andesite are expansive. As decomposition of the rock proceeds the ferromagnesian minerals change first to 2:1 lattice and then to 1:1 lattice clay-minerals in the following sequence: chlorite ---->vermiculite---->montmorillonite---->kaolinite (Heystek, 1954). All these stages are well represented in residual andesite soil profiles in Pretoria (Van der Merwe, 1964 a). It is only in either the most advanced or the most retarded stages of decomposition, represented by the uppermost and lowermost portions of the profile respectively, that the residual soils exhibit no expansive characteristics; the bulk of the soil profile, both red and yellow zones, contains active clays. A summary of available test data on residual andesite from building sites in Pretoria is given in Table 8.1. A comparison is made in this table between properties of soils from the red and yellow zones, and it will be seen that the red soils are more active than the yellow ones lower in the profile, in spite of the fact that the red soils represent a more advanced stage in the weathering sequence. This is clearly due to the progressive decrease in clay content with depth below the surface. Thus the colour change from red to yellow soil cannot be taken as the boundary between potentially expansive and non-expansive soils: it represents merely a transition from dehydrated to hydrated iron oxides.

An interesting feature about the Ongeluk lavas is the extreme variability in depth and in the degree of decomposition over relatively short distances. Within a few metres of an outcrop of solid rock a test-pit may disclose a substantial depth of decomposition. The greatest thickness

ENGINEERING PARAMETERS : INDICATOR TEST DATA		LIQUID LIMIT	LIQUID LIMIT RED SOIL	LIQUID LIMIT YELLOW SOIL	PLASTICITY INDEX	PLASTICITY INDEX RED SOIL	PLASTICITY INDEX YELLOW SOIL	LINEAR SHRINKAGE	LINEAR SHRINKAGE RED SOIL	LINEAR SHRINKAGE YELLOW SOIL	SHRINKAGE LIMIT
		LL	LL red	LL yellow	PI	PI red	PI yellow	LS	LS red	LS yellow	SL
MAXIMUM	x_M	73	73	64	44	44	44	18	18	16	32
MINIMUM	x_m	31	34	31	6	11	6	3	6	3	9
MEAN	$\bar{x}$	50	51	44	20	20	17	10	10	9	20
NUMBER OF TEST DATA	n	170	21	48	170	21	48	170	19	48	42
STANDARD DEVIATION	s	10,24	8,20	9,18	8,10	7,26	8,63	3,11	2,96	3,41	6,49
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,204	0,160	0,209	0,403	0,365	0,517	0,319	0,299	0,373	0,329

TABLE 8.1 : Engineering parameters of residual Ongeluk andesite in Pretoria

ENGINEERING PARAMETERS : CLAY, SILT AND WATER CONTENT		PERCENTAGE CLAY	PERCENTAGE CLAY : RED SOIL	PERCENTAGE CLAY : YELLOW SOIL	PERCENTAGE CLAY + SILT	MOISTURE CONTENT SCIENTIA (1951)	MOISTURE CONTENT SCIENTIA ABOVE 15 m (1951)	MOISTURE CONTENT SCIENTIA BELOW 15 m (1951)
		%<2μm	%<2μm red	%<2μm yellow	%<60μm	w (%)	w (%) above 15 m	w (%) below 15 m
MAXIMUM	x_M	53	50	45	76	40	20	40
MINIMUM	x_m	4	4	4	22	12	12	21
MEAN	$\bar{x}$	22	30	15	53	23	17	32
NUMBER OF TEST DATA	n	102	17	29	69	29	18	11
STANDARD DEVIATION	s	12,97	11,74	9,71	14,46	8,33	2,25	6,07
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,584	0,391	0,658	0,271	0,363	0,130	0,189

TABLE 8.1 continued

of residual soil from the records of over fifty soil profiles on this formation in Pretoria is 33 m, of which the red zone occupies the upper 20 m. Over most of the area, however, the soils are very much shallower. Of the trial-holes which had been augered to refusal, relatively few encountered solid Ongeluk andesite or indurated shale: in most cases the auger refused on weathering spheroids of andesite. (See Figures 8/4 and 8/5).

Effects of this extreme variability in the soil profile are well illustrated in the cracking patterns of houses in the Pretoria suburbs of Lynnwood, Menlo Park and Brooklyn. A badly cracked house will often be situated next door to one with few or no signs of cracking. Investigation will show the cracked house to be situated on deep residual soil in a zone of preferential decomposition, probably initially a zone of high joint intensity, while the adjacent house will be situated over a shallow suboutcrop of sound andesite, as illustrated diagrammatically in Figure 8/6.

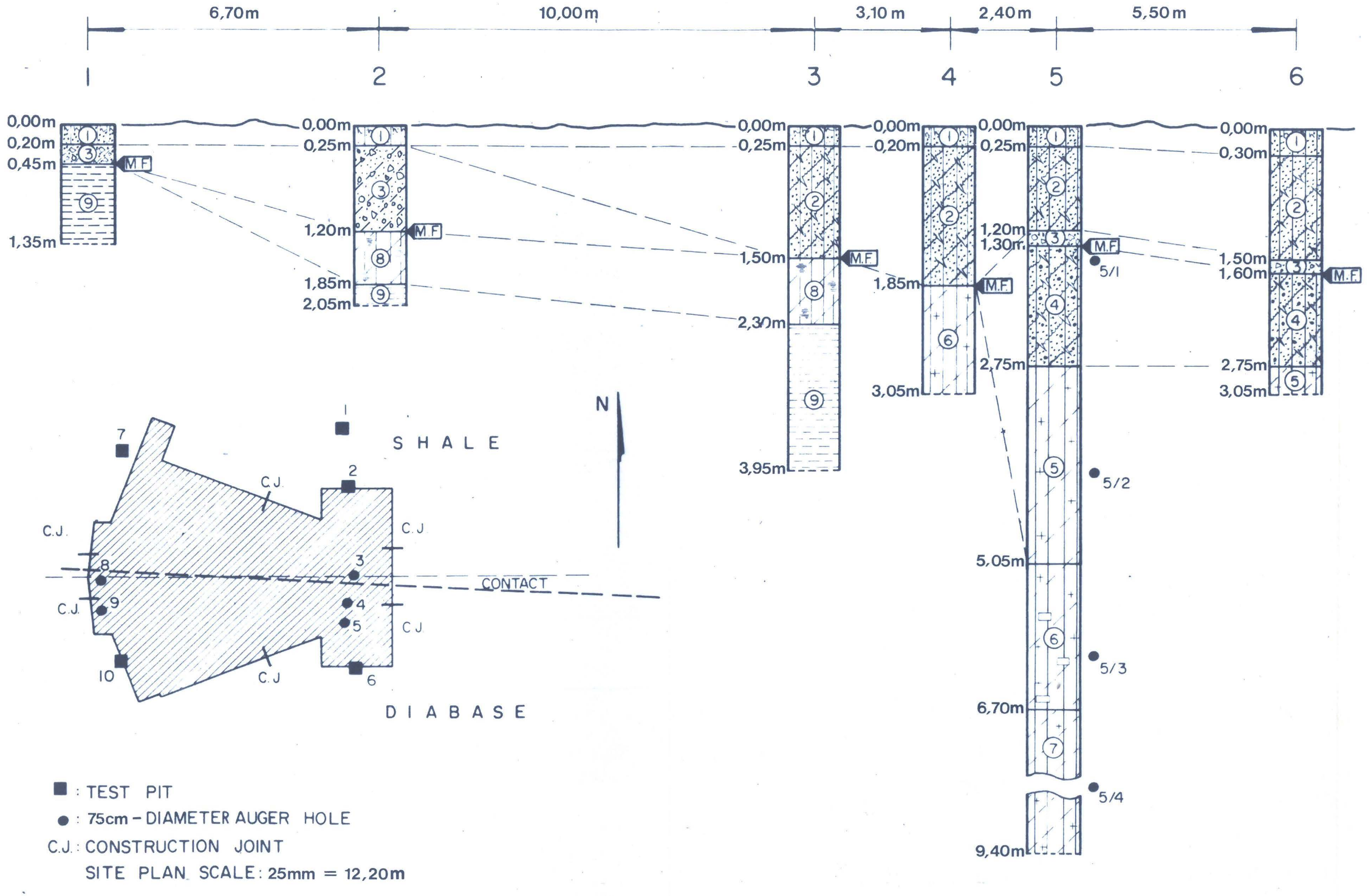
A further feature of the soils in this area is that they may not produce significant differential heave due to the natural increase in moisture content which takes place after a building has been erected, but may only do so when inundated locally. Thus a house which has stood for ten years without any noticeable cracks developing, may suddenly start to exhibit severe cracking as a result of a leaking drain.

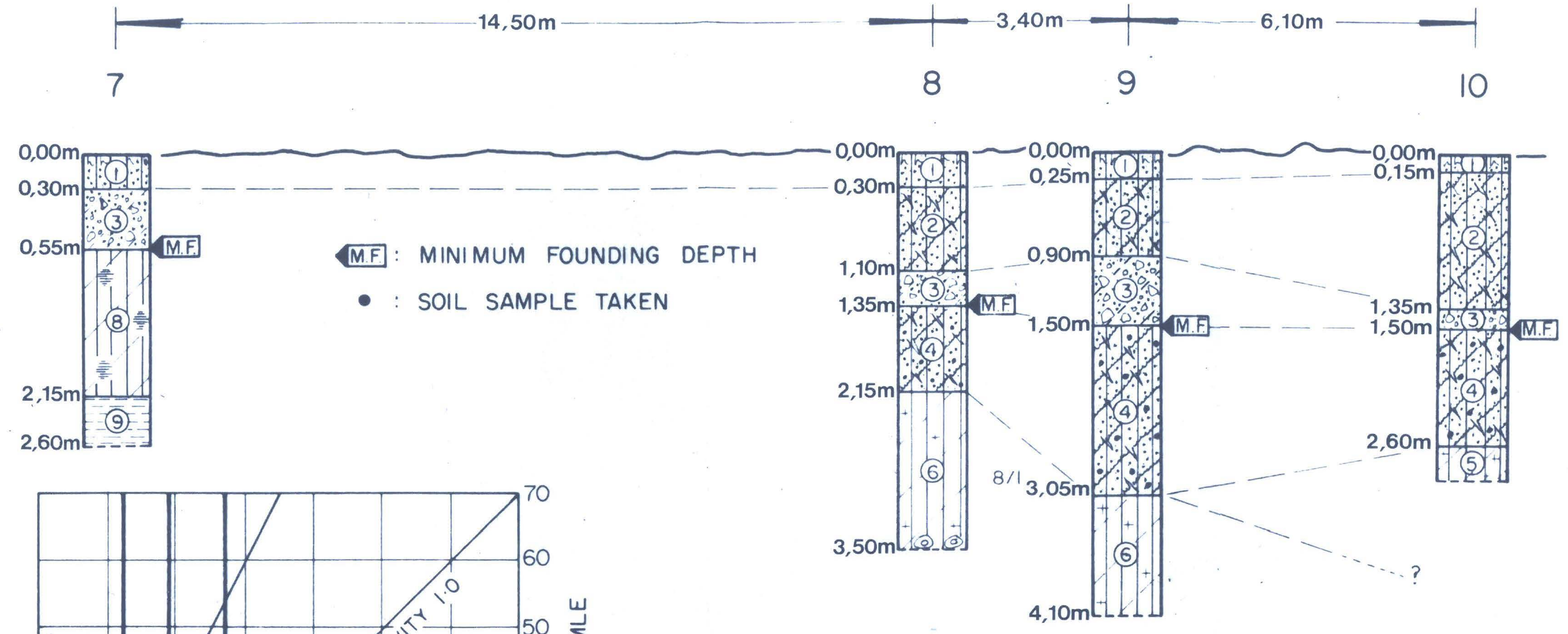
In view of the highly variable soil profile no building, however small, should be erected on these soils before an investigation has been carried out on the site. On a sufficiently large site the building, or possibly the individual foundations, may thus be positioned so as to avoid the need for costly construction. Where precautions against heaving do have to be taken, the nature of such precautions will be dictated by the depth and the degree of decomposition, and may involve anything from deep piling to ordinary footings or strip foundations carried down slightly deeper than normal. Cracking may be minimised in small buildings where deep foundations are not considered economically justifiable by the use of reinforced brickwork coupled with provision of flexible jointing in subsurface pipes and drains to prevent leakage, and by providing adequate drainage facilities to divert rain-water away from the foundations. It is important, too, that trees be kept well away from such structures, particularly varieties that have an excessive desiccating influence on the soil, such as jacarandas, willows, poplars and bluegums.

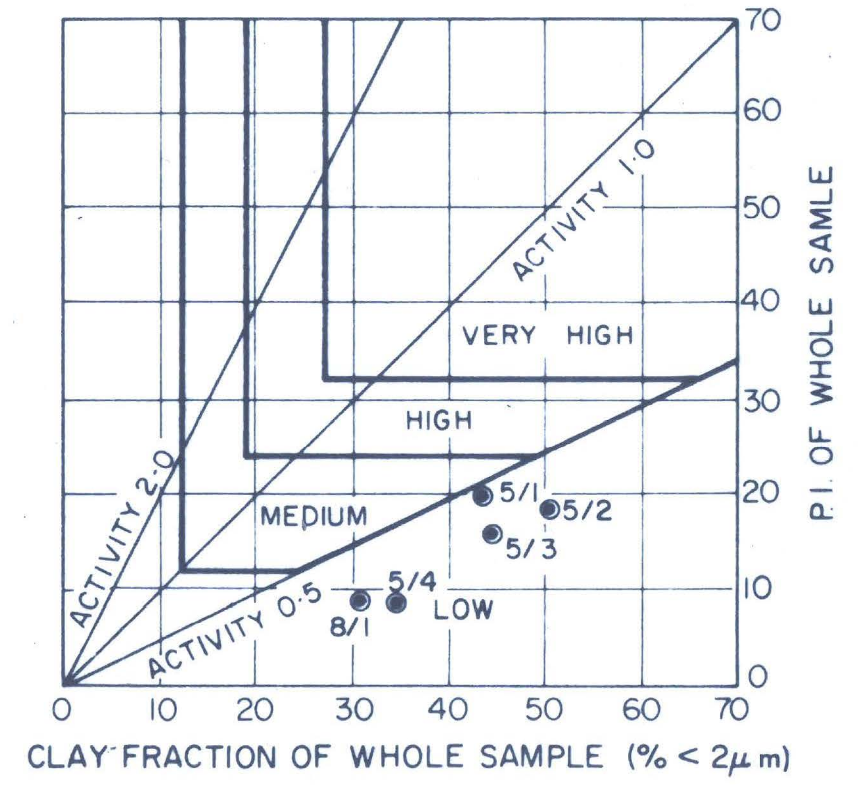
FIGURE 8/4 : Deep residual Ongeluk lava in road cutting north of Scientia, Pretoria

Note pebble marker and hillwash overlying the residual soil

FIGURE 8/5 : Variable depth of residual Ongeluk lava, with bedrock pinnacles and weathering spheroids, in road cutting directly opposite the spot where Figure 8/4 was photographed in Pretoria.







SAMPLE NO	DEPTH	LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	SHRINKAGE LIMIT	SHRINKAGE RATIO	% CLAY	% SILT
5/1	1,50m	44	20	9,7	18,5	1,78	27	33
5/2	4,00m	51	18	9,7	15,4	1,70	20	35
5/3	6,00m	45	16	6,0	29,6	1,47	8	39
5/4	8,00m	35	9	3,3	23,2	1,62	3	21
8/1	2,75m	31	9	5,0	16,7	1,83	5	27

1. Dry, dark reddish brown, firm, intact, sandy silt with roots: hillwash *topsoil*
2. Dry, dark reddish brown, stiff, shattered, clay-silt; *hillwash*
3. Fine and medium, subangular, quartzite and quartz gravels in matrix as above; *pebble marker*
4. Dry, dark red, firm, shattered, clay-silt with few Fe-concretions; residual *diabase*
5. Dry, yellowish red, firm, fissured, clayey silt with termite channels; residual *diabase*
6. Slightly moist, reddish yellow, soft to firm, fissured, clayey silt; residual *diabase*
7. Moist, in profile red speckled white, yellow and green, stiff, sandy silt; residual *diabase*
8. Shale fragments in matrix of dry, reddish brown, firm, intact, clay-silt; residual *shale*
9. Dry, olive-brown, stiff, laminated silt, with dip to north; weathered Magaliesberg *shale* (indurated in hole 3 by contact-metamorphism)

FIGURE 9/4 : Site plan of church building in Waverley, Pretoria, showing soil profiles developed on diabase sill and on Magaliesberg shales and indicator test data for residual soils

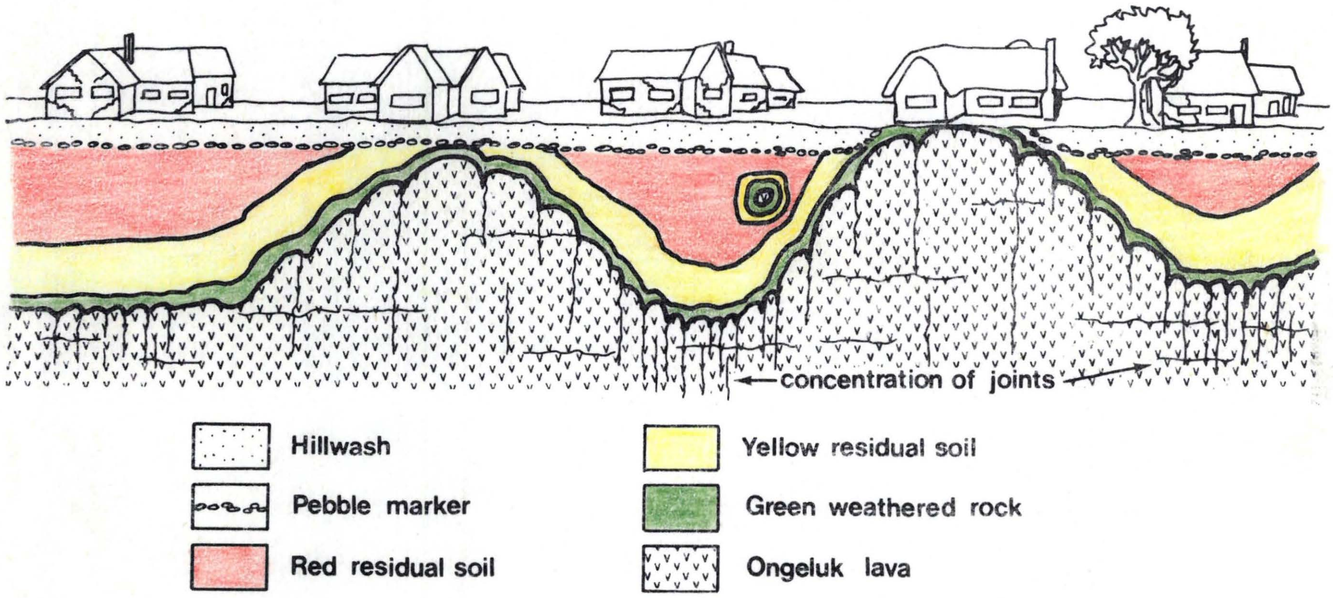


FIGURE 8/6 : Schematic illustration of variability in the depth of decomposition in Ongeluk lava controlled by joint spacing, and the effect of the consequent thickness of residual soil on the cracking of houses

9 DIABASE

INTRUSIVE PHASE OF BUSHVELD COMPLEX

There is a remarkably close similarity in the development of the soil profile and in engineering characteristics between the residual soils developed on diabasic bodies intruded during the early phases of Bushveld igneous activity, and those developed on the older Ongeluk lavas of the Pretoria Series. A summary of available test data for residual soils developed on diabase in Pretoria is given in Table 9.1. It will be seen that the mean values for available Indicator Test results are very similar to those for the residual Ongeluk adesite listed in Table 8.1. The sequence of development of secondary minerals has also been shown to be the same for soils derived from these two different parent-materials under similar climatic and topographic environments (Van der Merwe, 1964 a).

The clay-mineral and iron oxide contents of residual diabase from a dyke in Pretoria, as determined by De Bruijn are given in Table 9.2.

ENGINEERING PARAMETERS : INDICATOR TEST DATA		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	PERCENTAGE CLAY	PERCENTAGE CLAY & SILT	SHRINKAGE LIMIT
		LL	PI	LS	%<2μm	%<60μm	SL
MAXIMUM	x_M	66	41	15	49	68	30
MINIMUM	x_m	31	6	2	2	13	9
MEAN	$\bar{x}$	47	19	9	18	46	13
NUMBER OF TEST DATA	n	44	44	44	30	30	17
STANDARD DEVIATION	s	9,89	7,80	3,25	12,70	15,98	6,72
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,212	0,405	0,370	0,718	0,347	0,386

TABLE 9.1 : Engineering properties of residual diabase in Pretoria (excluding hydromorphic soils and decomposed sills immediately below the Magaliesberg quartzite range)

DEPTH IN METRES	1,8	4,5	7,8	13,2	16,5
% MONTMORILLONITE	0	14	15	33	43
% KAOLINITE	36	44	49	31	17
% IRON OXIDE	15	6	3	2	1

TABLE 9.2 : Clay mineral and iron oxide contents of minus 6,3 micrometre fraction of residual diabase from Lynnwood dyke, Pretoria, as determined by X-ray analysis

(After De Bruijn, 1959)

Three important observations may be made from these analyses:

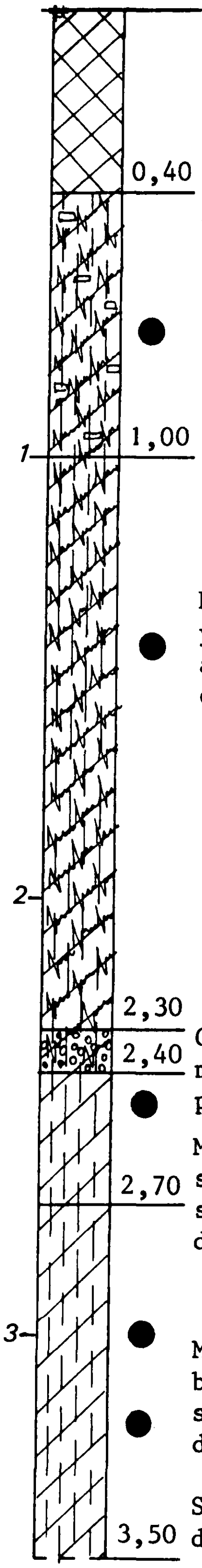
(i) the percentages of kaolinite and montmorillonite tend to be almost complementary at the different depths;

(ii) the percentage of montmorillonite increases with depth; and

(iii) the iron oxide content, which imparts colour to the soil, decreases with depth.

Owing to the high susceptibility of diabase to chemical decomposition, sills are often found to form troughs along strike within shales of the Pretoria Series, and these are commonly occupied by small watercourses. This is a common feature in the Pretoria Moot, the broad valley between the ridges of Daspoort and Magaliesberg quartzite. The usual soil profile along the watercourses consists of slickensided alluvial clay above the pebble marker and grey residual diabase below. These *hydromorphic* soils, both transported and residual, contain an excessively high proportion of montmorillonite, and only small amounts of kaolinite are detectable in the clay fraction. A typical example of such a soil profile, from a site one hundred metres north of a watercourse in Wonderboom South, Pretoria, is given in Figure 9/1, and it will be seen that the indicator test results are far higher than those for residual diabase on better drained slopes.

SURFACE



Description	Sample depth m	LL	PI	LS	% <2 µm	% <60 µm
Rubble, ash and soil; made ground.						
Moist, dark grey, soft, shattered and slickensided, silty clay with calcrete, nodules; alluvial.	0,75	74	55	15	50	81
Moist, grey mottled yellow, soft, shattered and slickensided, silty clay; alluvial.	1,5	86	60	25	51	83
Quartzite gravels in matrix as above; pebble marker	2,5	94	71	14	54	82
Moist, light grey, soft slickensided, silty clay; residual diabase.						
Moist, greenish to bluish grey, friable, silty clay; residual diabase.	3,0	64	29	14	30	70
	3,2	76	41	10	38	74
Soft rock, weathered diabase.						

(Water-table not encountered)

FIGURE 9/1 : Hydromorphic soil profile developed in alluvium overlying residual diabase : 12th Avenue, Wonderboom South, Pretoria

(After Van der Merwe, 1964)

Using the formula of Van der Merwe (1964 b) for estimating the heave from the Plasticity Index and the clay fraction, it has been calculated that potential heave at this site could, under conditions of desiccation, be about 120 mm. Owing to the relatively shallow water-table and the high natural moisture content of the soils, however, the practical problem in this situation is not one of heaving, but of settlement due to drying out of the soils during prolonged periods of drought. Houses in the vicinity of this particular site had been standing for about 15 years without any serious cracks developing but, after some years of drought, differential settlement resulted in severe cracking of the walls (Van der Merwe, 1964 a).

Special problems of differential settlement are always posed by a building site which straddles a contact between different rock or soil types. This situation is frequently encountered in those parts of the Pretoria Series which have been intruded by numerous diabase sills of varying thickness, such as in the Pretoria Moot. Where the building cannot be positioned so as to avoid straddling a contact between inert shale and expansive residual diabase, it becomes necessary to establish the line of contact with as much precision as possible in order that carefully placed construction joints may be incorporated in the design and special precautions taken on those portions of a building on the active soil, as illustrated in Case History 19.

Owing to the fact that diabase dykes and sills are often more deeply decomposed than the country-rock into which they are intrusive, these features commonly present problems in tunnelling or in open excavation. The hazards associated with 'mudrushes' in completely decomposed dykes, and ravelling from 'running dykes' where the material may be less decomposed but highly jointed, are well known to tunnelling engineers. A recent spectacular example is illustrated in Figures 9/2 and 9/3 on the Diepsloot Outfall Sewer in Witkoppen, Sandton. On 18th January, 1974, the tunnel encountered a narrow dyke of totally decomposed diabase at a point where it was only 15 m below ground level. The saturated residual diabase flowed into the tunnel as a mudrush and, by a process which could be described as headward piping, continued to flow until a 'sinkhole' had appeared at the surface.

Diabase sills are generally less notorious in this regard, unless the tunnelling is at shallow depth within the zone of decomposition. How-

FIGURE 9/2 : 'Sinkhole' which appeared at the surface of a narrow decomposed diabase dyke as a result of flowage into a tunnel situated 15 m below the surface : Diepsloot Outfall Sewer, Witkoppen, Sandton, Transvaal (Photo by Gordon Douglas and reproduced by permission of the City Engineer, Johannesburg)

FIGURE 9/3 : Blockage in tunnel caused by mudrush from decomposed diabase dyke referred to in Figure 9/2

(Photo by Douglas Gordon and reproduced by permission of the City Engineer, Johannesburg) ever, during the construction of the Daspoort Road Tunnel from Claremont to Danville in Pretoria during the early 1970's, tunnelling proceeded at a rate of 70 m per month through Daspoort quartzite, and problems only started when a water-bearing mud seam, 10 m wide, was intersected at the contact between the quartzite and a thick underlying diabase sheet, about 125 m in from the northern portal. Negotiation of this very tricky area took about six weeks. Once through the mud seam, driving in the diabase proved to be straightforward hard-rock tunnelling through half the sheet. Through the remainder of the sheet the material became blocky and tended to fall away in enormous pieces in spite of intensive rock-bolting. Nevertheless, the worst problems encountered on this unfortunate contract were not associated with the diabase, but with a small transverse fault which cut through, and displaced, the contact between the Upper Daspoort shale and the underlying Ongeluk lava at the south portal. The brecciated shale along the fault was water-bearing*. Tunnelling had eventually to be abandoned in the final section and an open cut resorted to (The Civil Engineer in South Africa, 1972).

Stability of open excavations through residual diabase is always problematical, and the problem becomes critical particularly in situations where the residual diabase is being squeezed out under the load of overlying sedimentary strata: a classical example is described in Case History 20.

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* Personal communication from Professor D.J.L. Visser, 1976.

CASE HISTORY I9

CONTACT BETWEEN SHALE AND DIABASE

A CHURCH BUILDING IN WAVERLEY, PRETORIA

Preliminary investigation by means of five test-pits, dug in the positions indicated on the site plan in Figure 9/4, revealed the presence of shales under the northern portion of the site and residual diabase under the southern portion. Both were overlain by a mantle of colluvial soil. Existing houses in the vicinity were inspected. Those underlain by shale were found to be undamaged. Those situated on the diabase sill were cracked: some only slightly, other very badly, but all exhibiting the characteristic doming pattern with apparent 'corners down' cracks typical of heaving conditions.

The contact between shale and diabase was established by drilling five further trial-holes with a truck-mounted auger, and the line of contact was found, fortunately, to coincide very nearly with the proposed centre-line of the building.

Indicator Tests performed on samples of the residual diabase showed this material, locally, to possess potentially expansive characteristics of a relatively low degree.

The following recommendations were put into effect in erecting the building:

(1) foundations were taken down to the base of the pebble marker;

(2) bearing pressures did not exceed 300 kPa;

(3) vertical construction joints were built in at the positions shown on the site plan and the photograph in Figures 9/4 and 9/5 respectively;

(4) brickwork on the southern half of the building (over the diabase) was reinforced horizontally with expanded metal strips between courses; and vertically at the corners and on either side of door and window openings and construction joints, with 12 mm steel rods;

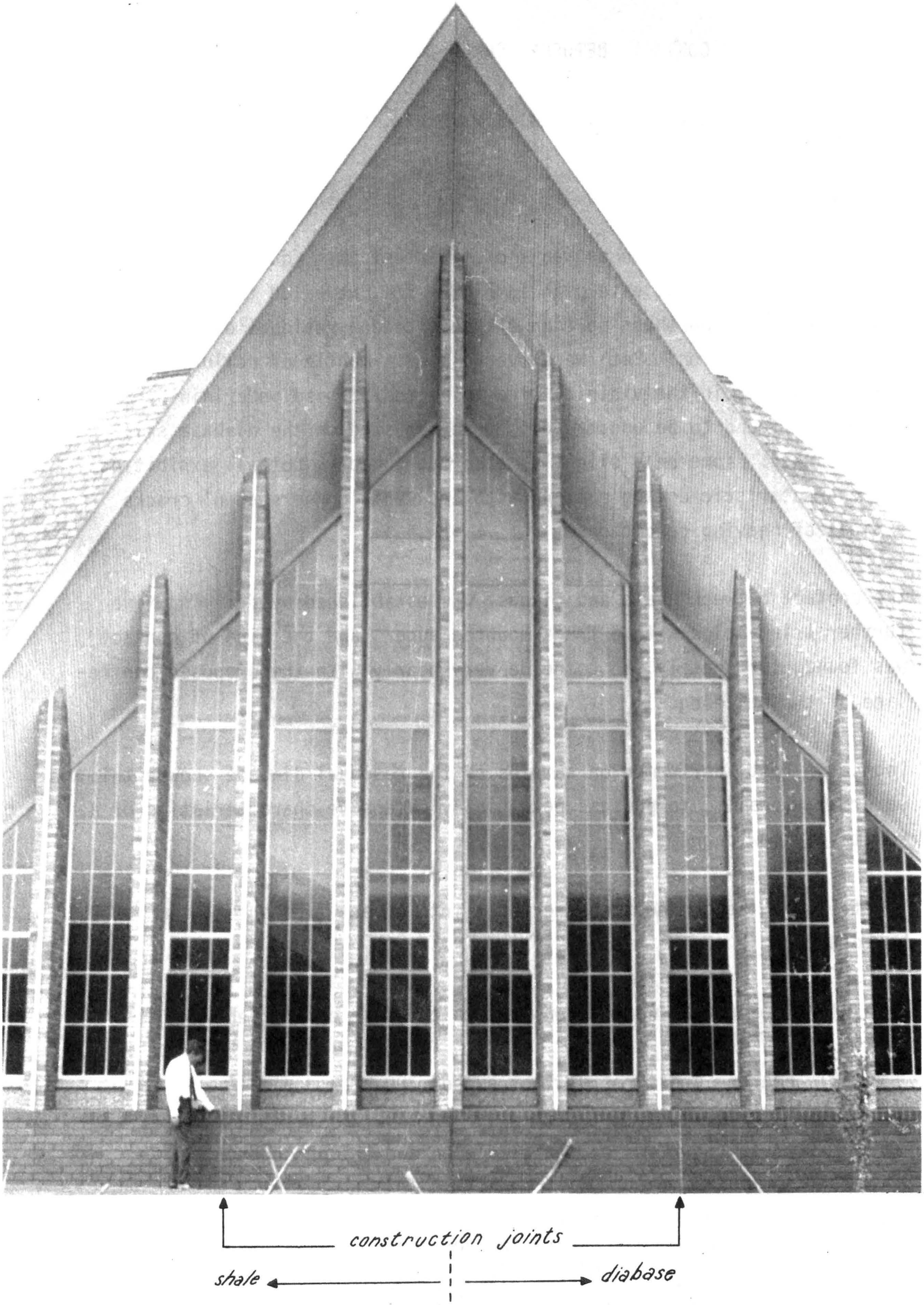

FIGURE 9/5 : Church in Waverley, Pretoria, sited symmetrically over contact between Magaliesberg shales and decomposed diabase sill

(5) the organ slab, entirely detached from the rest of the structure, was carried on four columns founded on rock at an average depth of 4,5 m, two of them on shale and two on diabase, in the vicinity of trial-hole 8;

(6) a concrete apron, consisting of slabs 1,2 m square and entirely separated from one another and from the building, was cast around the outside of the church, and other adequate provision made for draining rain-water away from the peripheral foundations;

(7) subsurface sewerage pipes were jointed with bitumen to provide flexibility;

(8) indigenous trees on the building site were dug out well in advance of the commencement of building operations, and new trees were planted at distances of at least twice their estimated full-grown heights away from the building.

The combination of these precautions has provied to be entirely effective. The church was completed in 1960. In spite of a period of several years of severe drought, followed by several seasons of soaking rain, a recent inspection of the building showed no signs of cracking (see Figure 9/5).

CASE HISTORY 20

EXCAVATION IN RESIDUAL DIABASE

WONDERBOOM RESERVOIR : PRETORIA MUNICIPALITY

The reservoir is situated immediately below the scarp slope of the Magaliesberg range, just to the east of Wonderboom neck at the top of Voortrekker Road, Pretoria. Excavation of the site was carried out during 1949 and the reservoir was completed the following year.

As shown in the sketch plan and the geological cross-section in Figure 9/6, the reservoir lies almost wholly on a deeply decomposed diabase sill which is intrusive into the Magaliesberg quartzites. Only the extreme south-eastern corner of the structure rests on the lower horizon of quartzite. The Magaliesberg ridge locally consists of massive and thickly bedded, white to pink, recrystallised quartzites dipping to the north at an average angle of 40^0. The subsidiary ridge immediately to the south of (and forming the floor of) the diabase sill, consists of thinly bedded whitish quartzites dipping to the north at much the same angle. The diabase sill, or sheet, is approximately 45 m thick, is concordant with the quartzites, and locally forms a shallow topographic trough which is partially filled with a thick deposit of talus derived from the overlying quartzites.*

Shortly after excavation for the floor of the reservoir had been completed and work had commenced on placing reinforcing steel for the northern retaining wall, the excavated cut in the hillside started exhibiting signs of collapse (Figures 9/7 and 9/8). Failure was observed to be taking place along well developed and highly polished slickensides (Figure 9/9). At this stage field and laboratory investigations were commenced by the National Building Research Institute, assisted by Dr D.J.L. Visser (at that time Senior Geologist with the Geological Survey).

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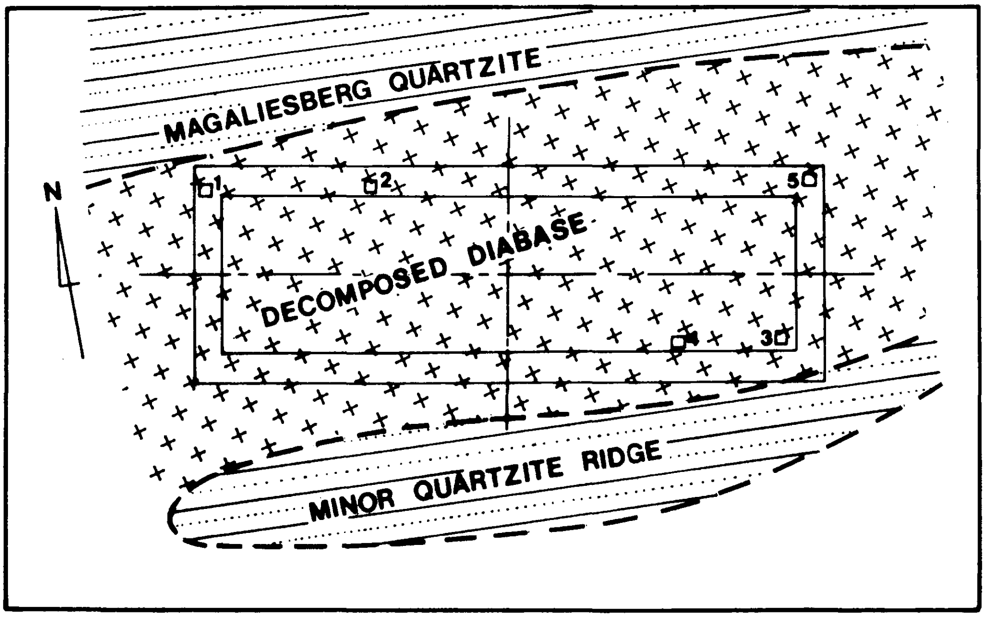
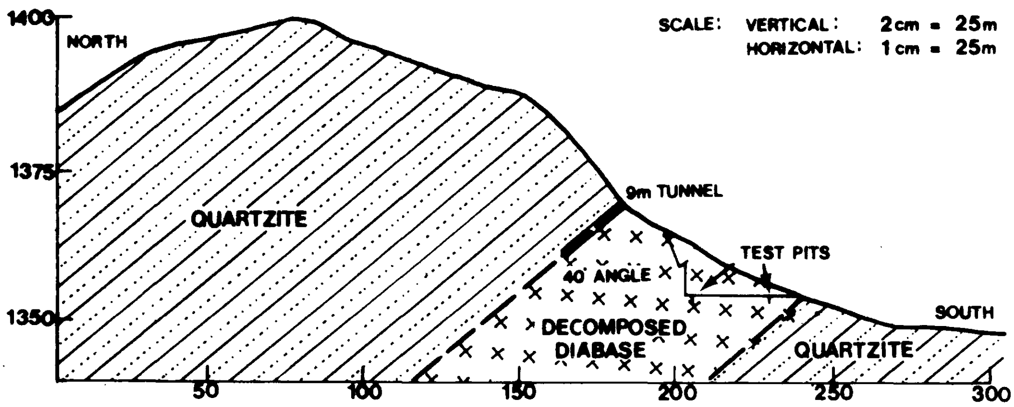
* The talus deposit locally contains a great abundance of Early Stone Age artefacts (Mason, 1962).



SITE PLAN SHOWING POSITIONS OF TEST-PITS



CROSS-SECTION

FIGURE 9/6 : Geological plan and cross-section at Wonderboom Reservoir site

(After Jennings, 1952)

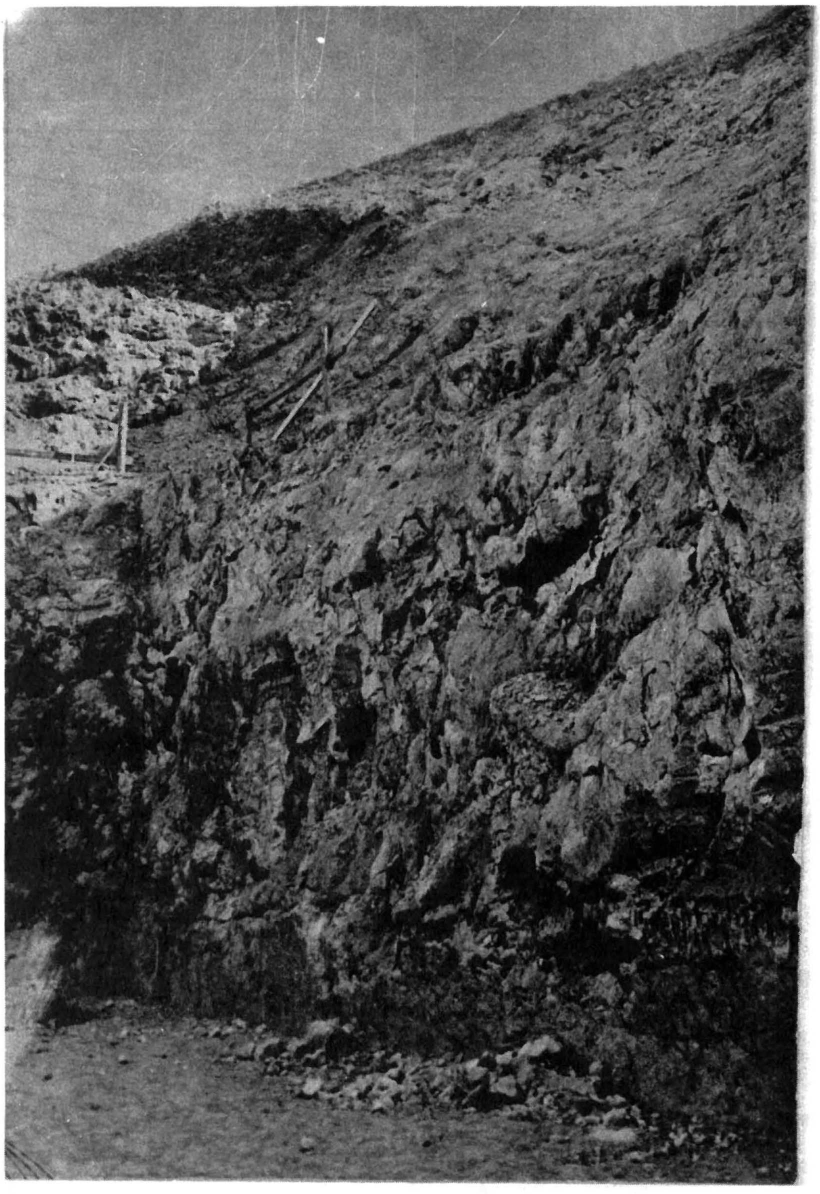


FIGURE 9/7 : Collapse of excavated cut in decomposed diabase sill at Wonderboom Reservoir, Pretoria (1949)

(Photo by NBRI—CSIR)

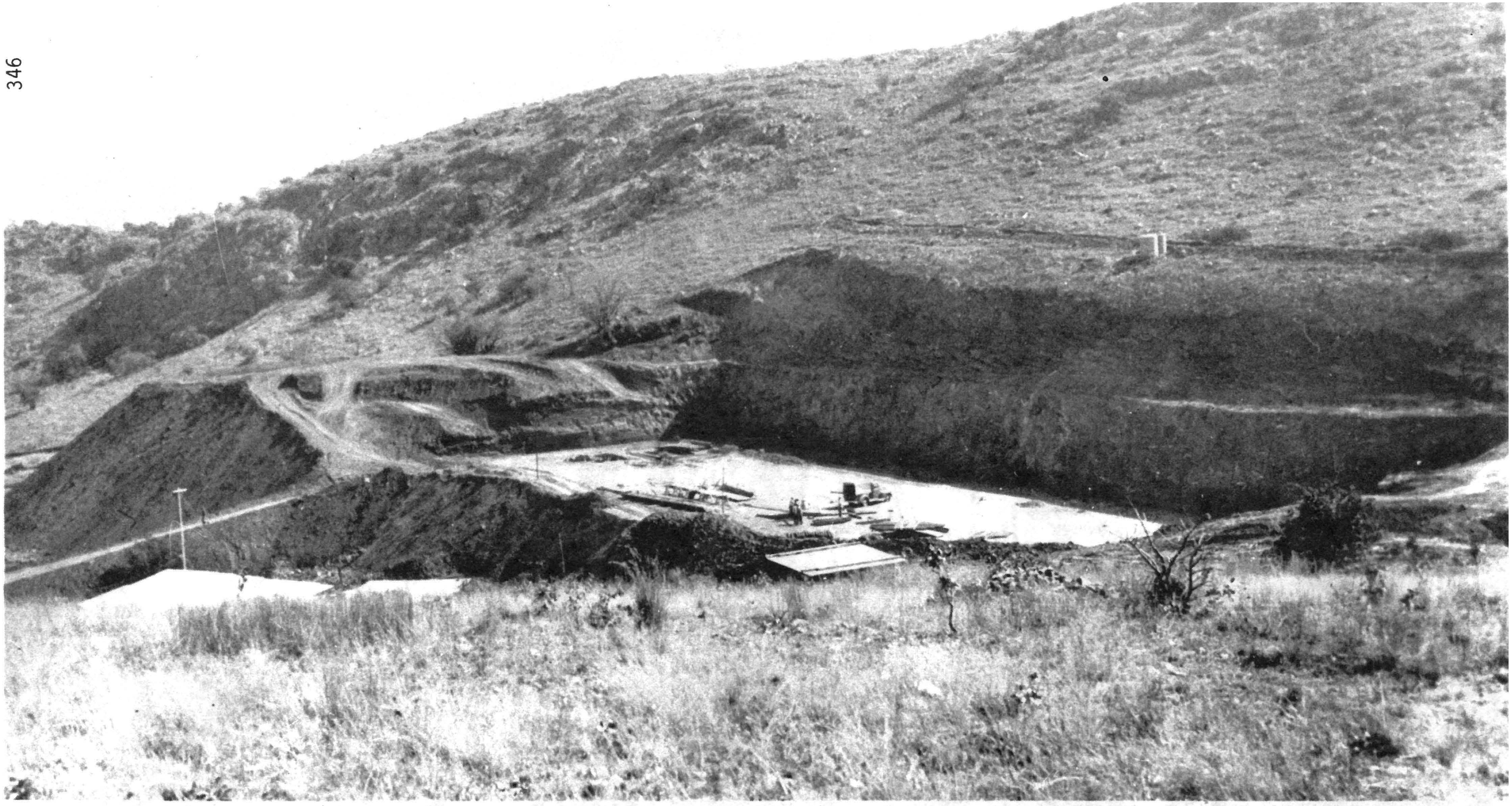


FIGURE 9/8 : Excavation for Wonderboom Reservoir in decomposed diabase sill beneath crest of Magaliesberg quartzite.

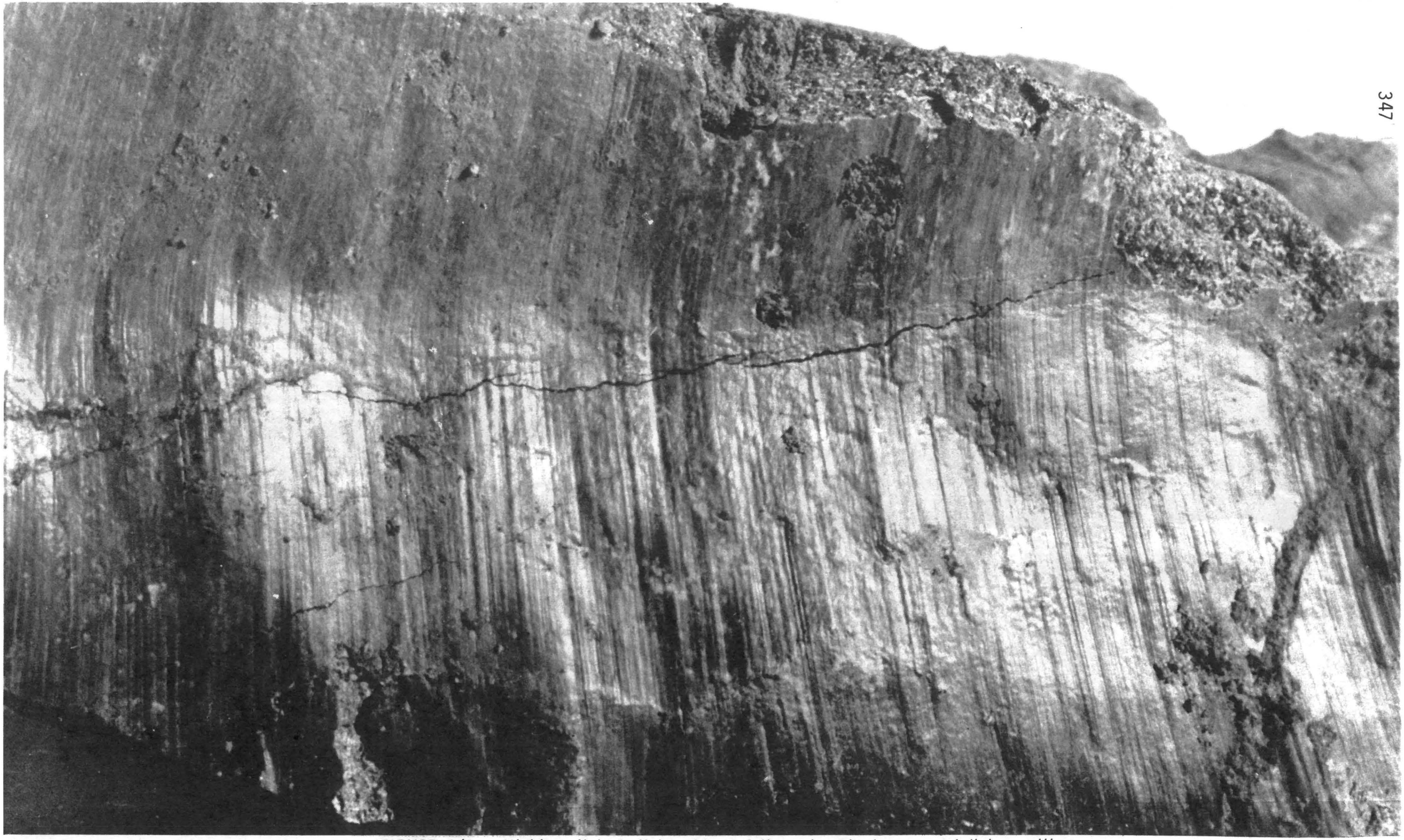


FIGURE 9/9 : Highly polished slickenside on failure plane in decomposed diabase sill at Wonderboom Reservoir, Pretoria

Note intact termite channel near bottom right-hand corner (Photo by NBRI—CSIR : slightly enlarged)

Field investigations

It soon became apparent that the slickensides along which failure was taking place in the north wall of the excavation were orientated in a limited number of reasonably well defined sets (Visser, 1949):

(i) parallel to the strike of the quartzites, i.e. east-west and at right angles to the bedding or at angles varying about 10^0 from this perpendicular: these slickensides thus strike E - W and dip at angles from 40^0S to 60^0S;

(ii) parallel to the dip of the quartzites, i.e. striking north-south, and also at right angles to the bedding planes or at angles varying 10^0 from this perpendicular: these slickensides thus strike N - S and dip at angles varying from 80^0 W to 80^0E;

(iii) two sets of diagonal joints striking on the average N 50^0 E - S 50^0 W and N 40^0 W - S 40^0 E and dipping steeply to the NW and NE respectively: these were regarded as planes of maximum shear, and there were signs of brecciation and faulting having taken place on a small scale along them.

Five test-pits were sunk from the floor of the excavation and a tunnel, 9 m long, was driven at the upper contact of the diabase and the overlying quartzite as shown in Figure 9/6. Slickensides were found to be present in the residual diabase in test-pits 1 and 2 though this material had been relatively unaffected by the excavation operation. The diabase was found to be in an advanced state of decomposition throughout the entire length of the tunnel, and again the slickensides were present. No slickensides were found in test-pit 4 which was very much shallower in relation to the original ground surface.

Detailed examination of structures on the slickensides indicated that shearing movements for individual slickensides had been in one direction only. Displacements in clearly identifiable textural horizons within the decomposed diabase led to the conclusion that movement along the slickensides had been substantial. The presence of termite channels consisting of loosely cemented grains and often lying in or across slickensides indicated that the rate of movement was probably slow.

It was concluded from this field evidence that considerable movements had taken place within the residual diabase in the form of shear failure, and that these movements were the results of the sill material being slowly squeezed out under the load of the overlying mountain.

Laboratory investigations

Undisturbed samples taken from various depths in test-pits 1 and 4 were subjected to laboratory testing, the results of which are summarised in Table 9.3.

Of the sixteen samples on which grading analyses were carried out, one was classified as a silty clay, six as clay-silts, eight as sandy silts and one as a clay-sand.

It will be seen that the Liquid Limit, Plasticity Index and Linear Shrinkage values were exceptionally high when compared with values given in Table 9.1. These values indicated that the material had potentially highly expansive characteristics. The favourable relationship between values for Shrinkage Limit and for natural moisture contents indicated, however, that only slight volumetric changes would accompany anticipated changes in moisture content.

Values for bulk density and void ratio indicated the exceptionally advanced degree of leaching of the material and, together with the high saturation moisture content, pointed to the likelihood that a considerable amount of additional water would probably be absorbed by the soil when the reservoir was in use.

Shear tests were carried out in quadruplicate on soaked samples of undisturbed material from a depth of 4 m in test-pit 1 using normal, constant rate of strain, shear boxes under consolidated quick shear conditions. The quadruplicate tests at each normal loading showed a wide divergence which tended to decrease with the high loads. This spread was probably due to failure taking place along planes of weakness in the samples and the results were consequently of no great value: it was interesting to note, however, that the shearing planes formed in the shear boxes in no way resembled the natural slickensides.

Oedometer tests were run in duplicate on soaked samples cut in three orientations from undisturbed material taken from a depth of 4 m in test-pit 1. Tests 1 and 2 were conducted on specimens taken from the horizontal plane; tests 3 and 4 from a vertical N - S plane perpendicular to the strike of the sill; tests 5 and 6 from a vertical E - W plane parallel to the strike. Coefficients of consolidation, compressibility and permeability were calculated for the load increment 110 kPa to 220 kPa: these are given in Table 9.4.

ENGINEERING PARAMETERS		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	SHRINKAGE LIMIT	SHRINKAGE RATIO	NATURAL MOISTURE CONTENT	CLAY CONTENT	SILT CONTENT	DRY DENSITY	BULK DENSITY	SATURATION MOISTURE CONTENT	DEGREE OF SATURATION	VOID RATIO	SPECIFIC GRAVITY
		LL	PI	LS	SL	SR	w %	<2μm %	>2μm <60μm %	kg/m^3	kg/m^3	smc %	Sr %	e	SG
MAXIMUM	x_M	91	61	19,7	43,7	1,75	45,0	26	76		1 590	65,5	75,2	1,878	2,88
MINIMUM	x_m	53	18	6,3	20,4	1,27	18,5	2	34		1 400	53,4	61,5	1,533	2,86
MEAN	$\bar{x}$	75	43	14,7	32,7	1,48	36,6	16	51	1 060	1 500	59,0	68,6	1,692	2,87
NUMBER OF TEST DATA	n	15	15	15	16	16	16	16	16	1	3	3	3	3	2
STANDARD DEVIATION	s	10,39	10,73	3,6	6,7	0,14	5,98	6,0	8,9						
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,138	0,252	0,242	0,204	0,094	0,164	0,377	0,176						

TABLE 9.3 : Some engineering properties of the residual diabase from below the floor of Wonderboom reservoir, Pretoria

OEDOMETER TEST NUMBER	COEFFICIENT of CONSOLIDATION Cv mm^2/min	COEFFICIENT of COMPRESSIBILITY a_v mm^2/kN	COEFFICIENT of PERMEABILITY k mm/min
1	12,8	0,458	$21,13 \times 10^{-6}$
2	47,9	0,265	$48,06 \times 10^{-6}$
3	50,6	0,387	$69,65 \times 10^{-6}$
4	16,4	0,484	$25,91 \times 10^{-6}$
5	26,4	0,367	$35,64 \times 10^{-6}$
6	14,1	0,262	$13,23 \times 10^{-6}$

TABLE 9.4 : Results of oedometer tests on residual diabase from 4 m below floor or Wonderboom Reservoir, calculated for increment of loading 110 kPa to 220 kPa

A summary of the swelling pressures and preconsolidation loadings for the six oedometer tests is given in Table 9.5.

OEDOMETER TEST NUMBER	ORIENTATION OF SAMPLE	SWELLING PRESSURE		PRECONSOLIDATION PRESSURE : pc	
		INDIVIDUAL TEST kPa	AVERAGE kPa	INDIVIDUAL TEST kPa	AVERAGE kPa
1 2	HORIZONTAL	14,5 -	14,5	524 448	486
3 4	VERTICAL PLANE N - S	14,5 15,2	14,8	290 290	290
5 6	VERTICAL PLANE E - W	21,4 27,6	24,5	483 -	483

TABLE 9.5 : Swelling and preconsolidation pressures calculated for residual diabase from 4 m below floor of Wonderboom Reservoir

The normal overburden pressure before excavation was 15 m of soil which, at a bulk density of 1 500 kg/m^3, would give a normal loading of 225 kPa.

Six conclusions concerning both the geological history of the site and practical considerations for construction of the reservoir were drawn from the test data:

(1) anistropic conditions existed with regard to preconsolidation pressure: that on the vertical plane parallel to strike was nearly the same as that on the horizontal plane and nearly double that on the vertical plane normal to strike;

(2) the preconsolidation pressures as determined confirmed the field evidence that the residual diabase was being squeezed out under the load of the overlying mountain;

(3) any restraint of this movement would result in very high pressures of the order of 550 kPa on the upslope retaining wall of the reservoir;

(4) substantial erosion of overburden had taken place producing a subsequent relief of overburden pressure;

(5) swelling pressures were insignificant and the empty load of the reservoir would be sufficient to resist heave;

(6) some settlement could probably be expected under full loading.

Recommendations

As the conclusions of the field investigation had been confirmed and quantified by the conclusions of the laboratory investigation, it was clear that the upslope retaining wall on the northern perimeter of the site would have to be built in such a way as to make provision for the relief of pressure resulting from stresses causing the residual diabase to be squeezed out under the load of the overlying mass of quartzite. It was suggested that this be done by cutting the material away completely at the back of the wall and backfilling the cut with large size crushed rock, with provision for adequate drainage of the rockfill. Further, it was recommended that some means of underdrainage be provided beneath the reservoir, and that the floor and roof of the reservoir should be made as flexible as possible to accommodate the anticipated settlements. Finally, it was recommended that level points be established on the retaining wall and elsewhere on the reservoir, and that regular monitoring be carried out for both vertical and lateral movements in relation to fixed points on the lower quartzite (Jennings, 1949).

These recommendations were put into effect and the fact that the reservoir has now been in use for twenty-five years without any major trouble testifies to the efficacy of the construction (Figure 9/10). It testifies, too, to the efficacy of the pioneering efforts of the research team under J.E. Jennings - particularly when laboratory investigations were conducted with what would now be regarded as outdated equipment.

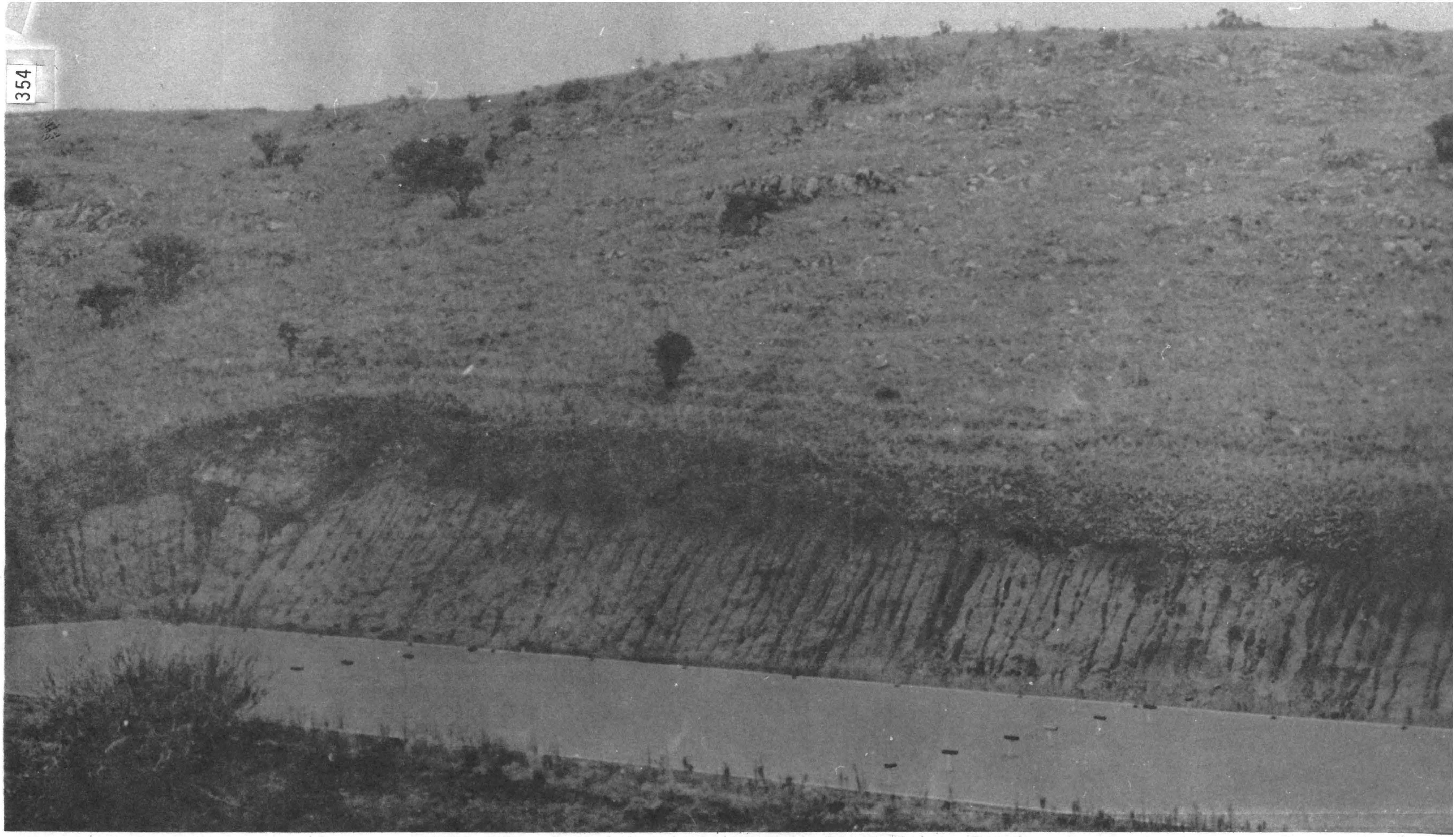


FIGURE 9/10 : General view of completed reservoir at Wonderboom, Pretoria.

Note thick talus gravels overlying decomposed diabase sill beneath Magaliesberg quartzite

(Photo by ABAB 1970)

10 BUSHVELD COMPLEX

MAFIC ROCKS & ASSOCIATED SOILS

BLACK CLAYS

Comparison with black clays elsewhere in Africa

The most highly expansive soils in South Africa are the black and grey subtropical clays developed as residual soils on the mafic rocks of the Bushveld Complex. These are known to the pedologist as 'lithomorphic vertisols' and to the layman as 'black turf'. Similar soils in other parts of the world are variously known as 'tropical black clay soils' and 'grumusols': in India they are called 'regur soils', in Indonesia 'margalitic soils', in Morocco 'tirs' and in Russia 'chernozems'. In many parts of the world where these soils occupy extensive areas there are no suitable natural gravels for use as materials for road construction or for concrete aggregates. Few roads constructed on them have proved satisfactory: many have failed completely.

Most tropical and subtropical black clays are residual, having been formed by *in situ* decomposition of basic rocks. Alluvial deposits with similar characteristics are of limited extent. The Lake Chad Basin contains the only extensive deposit of lacustrine black clay in Africa.

In the Transvaal, residual black clays similar to those developed on the Bushveld mafic rocks are also to be found on the basalts of the Springbok Flats and, in places, on dolerite sills. Throughout Africa the coverage of these soils has been estimated by Dudal (1965), who calls them 'dark clay soils', as 97 million hectares, of which about 2 million are in the Republic of South Africa.

The black colour was at first thought to be due to the organic content, but it was later established that the quantity of organic matter was negligible in many such soils (often no more than one per cent). Theron and Van Niekerk (1934) believed the colour to be due to the high percentage of silica in the clay fraction which would allow a small organic content to impose the black colour on the whole soil mass. Others have ascribed the dark colour to the activity of anaerobic bacteria which develop in the alkaline and poorly drained montmorillonitic clays during the wet season. Dudal (1965) maintains, however, that the colour results from the formation of a complex between the clay and the organic matter, even where the quantity of the latter is relatively low. The nature of such a complex may well be in the form of calcium humate. The evidence suggests that this is indeed the case, particularly in the absence of iron oxide as an agency of pigmentation.

The engineering significance of the black clays from the Bushveld and elsewhere is basically that they have a clay content usually higher than 30 per cent and sometimes as high as 60 per cent, in which montmorillonite is the predominant, and often the only, clay-mineral. Typical figures for particle size and Atterberg Limits from the published literature on black expansive clays from various countries in Africa are given in Table 10.1: these data are tabulated in order that the properties of the black clays developed on mafic rocks of the Bushveld Complex may be compared with those of similar soils from elsewhere in Africa.

From the comparative data given in Table 10.1 it will be seen that expansive black clay soils in Africa have the following characteristics in common:

(i) parent-material for the residual soils is usually mafic igneous rock, basalt being the most common;

(ii) all available pH values, with the exception of the quoted figure from Zambia, are high, i.e. the soils are alkaline, favouring the formation of montmorillonite as the principal clay-mineral, with calcium and magnesium as the principal exchangeable cations;

(iii) liquid limits are all exceptionally high, all of them within the range 50 to 110*.

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* Certain grey clays which underlie the Bushveld black clays in topographic depressions have liquid limits even in excess of these values (Van der Merwe, 1967). The mean value is 108 and the highest recorded value 158.

COUNTRY	PARENT MATERIAL	pH	% clay	% silt	% sand	LL	PI	REFERENCE
SOUTH AFRICA	Bushveld gabbro & norite	7,7	45	21	34	84	51	Van der Merwe (1967)
ANGOLA	Miocene limestone		61	12	27	66	23	Horta da Silva (1971)
CHAD BASIN	Lacustrine deposit	8,4	70	20	10	58	42	Morin (1971)
ETHIOPIA	basalt	7,2	56	38	4	109	81	Morin (1971)
GHANA	hornblende garnet-gneiss	7,9	50	40	10	99	70	Clare (1967)
KENYA	basalt		67	28	4	103	60	Williams (1961)
KENYA	basalt	8,8	62	8	22	104	70	Morin (1971)
LESOTHO	basalt, dolerite		91	2	1	57	34	Pollard (1964)
MOROCCO	alluvium derived from basalt		60-70	25-30	5-10	60	40	Moussaouri (1967)
MOROCCO			54	32	14	56	32	Morin (1971)

TABLE 10.1 : Particle size and Atterberg Limits of black expansive clays from various African countries (Note: Some of the values given, e.g. those for the Bushveld clays, are mean values: others are values for individual samples)

COUNTRY	PARENT MATERIAL	pH	% clay	% silt	% sand	LL	PI	REFERENCE
NIGERIA			76	23	1	82	44	Grainger (1951)
NIGERIA	calcareous rocks (?)		68	26	6	66	41	Evans (1967)
NIGERIA			57	30	11	66	46	Morin (1971)
RHODESIA			55	11	34	72	48	Grainger (1952)
SUDAN	alluvium derived from basalt	8,9				70	41	Morin (1971)
TANZANIA	alluvium	9,2	40	50	6	58	42	Morin (1971)
UGANDA	basalt	7,4	56	32	6	87	63	Morin (1971)
WEST CAMEROON	basalt		38	43	19	62	27	Morin (1971)
ZAMBIA	basalt	6,4	50	20	30	50	33	Morin (1971)

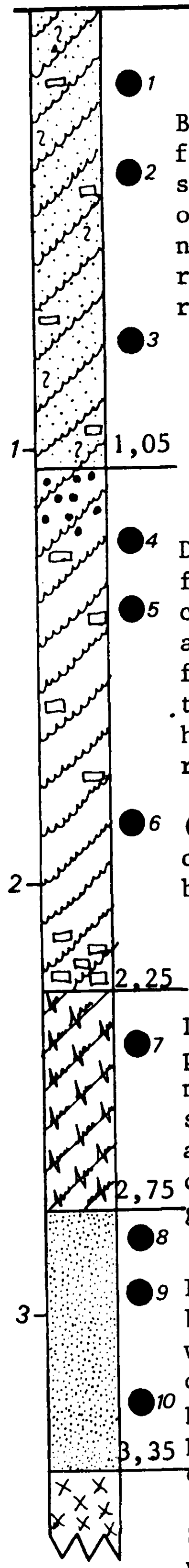
TABLE 10.1 : (Contd.)

(iv) with the exception of the examples quoted from Angola and West Cameroon, the clays are all potentially *very highly expansive* (Williams, 1958), with clay contents in excess of 38 per cent and plasticity indices in excess of 33 per cent.

Montmorillorite is the predominant clay-mineral in the black clays of the Bushveld, and small amounts of kaolinite are present in most occurrences (Van der Merwe, 1967). De Bruijn (1963) has also reported the presence of substantial quantities of an unidentified amorphous substance in these clays, which he considers may also contribute to the over-all expansiveness. Morin (1971) reports montmorillorite as the main clay-mineral in black clays from elsewhere in Africa with kaolinite commonly present in smaller amounts; the alluvial soils also contain illite, and soils derived from volcanic rocks often contain halloysite. The principal exchangeable cation is calcium in all cases; magnesium is also present in significant quantities, but potassium, sodium and manganese are present only in minor amounts.

Onderstepoort experimental site

A comprehensive research project involving both field and laboratory work was commenced by the CSIR in 1960 on a site near Onderstepoort, occupied by residual black clay derived from gabbro. The location of the test site is shown in Figure 10/10, a typical soil profile is given in Figure 10/1, and climatic data are listed in Table 10.2. The project was designed to study the distribution and changes in pore-water pressure (suction), strength and moisture content in the expansive clay under various 'boundary conditions', and the movements accompanying such changes. The different boundary conditions studied included those associated with surface covers of impermeable plastic or fibre-glass, and of a blanket of river sand 150 mm thick. The behaviour of the soil under these covers was compared with that for an area which was allowed to lie 'fallow' after the natural vegetation has been removed by a chemical weed-killer. Gypsum block field tensiometers were installed to measure moisture conditions at various depths below and beyond the covered and fallow areas. A series of level pegs were also established at various depths below and beyond the experimental covers in order to monitor the soil movements.



Black, stiff, fissured, slightly sandy clay, with occasional calcrete nodules and fine roots; reworked residual gabbro.

Dark grey, stiff, fissured clay with calcrete nodules and with black ferruginous spots towards top of horizon; residual gabbro.

(Concentration of calcite nodules at base of horizon).

Light grey, in profile grey mottled yellow, stiff, shattered and slickensided, clay; residual gabbro.

Light yellow, dense but friable sand with angular grains of weathered plagioclase and pyroxene; residual gabbro.

Soft rock weathered gabbro.

Sample No.	Liquid Limit	Plasticity Index	Linear Shrinkage	Clay % <2 µm	Montmorillonite	Specific Surface m²/gm		
						External	Internal	Total
1					28	160	329	489
2	75	48	19	49				
3					26	189	305	494
4	98	65	19	52				
5					29	210	300	510
6					51	235	361	596
7	108	67	17	46				
8					59	276	437	713
9	42	15	7	12				
10					58	160	424	584

FIGURE 10/1 : Typical soil profile for CSIR test site at Onderstepoort with relevant test data

Latitude	:	$25^{0}39'S$
Longitude	:	$28^{0}11'E$
Altitude	:	1 210 m
Mean annual precipitation	:	710 mm
Mean annual potential evapo-transpiration	:	830 mm
Annual moisture deficiency	:	120 mm
Thornthwaite classification	:	C_1B_2 'da'
Köppen-Geiger classification	:	Cwb
Weinert's climatic N-value	:	2,4

TABLE 10.2 : Climatic data for CSIR test site at Onderstepoort

(After De Bruijn, 1973)

Early on in the experiment it became apparent that large amounts of water were being conserved in the soil beneath the various surface covers. Of particular interest was the observation that moisture retention under the sand cover extended to a depth roughly twice that under the plastic cover (De Bruijn, 1965). This effect had not been anticipated: despite the fact that more rain-water entered the soil under the sand cover than under the plastic cover, it was expected that subsequent drying out would proceed at a faster rate in the former case because of the direct exposure here of the soil to the atmosphere. The observed phenomenon was explained by De Bruijn (1965) as probably due to the 'buffer action' of the sand cover in contrast with the 'insulating action' of the plastic cover. The sand layer was seen as providing an appreciable source of moisture that would retard the drying out of the underlying clay. A further factor which may be significant is associated with the seasonal opening and closing of fissures extending from the surface of the black clay to a depth of about 1,5 m (Blight and Williams, 1971); sand runnels pentrating these fissures during the dry season would remain as water conduits during the following wet season.

Figure 10/2 shows curves for heave versus time for the open field, the fallow area and the areas under sand and plastic/fibre-glass covers for the period 1963 to 1970. Monthly precipitation figures are also plotted for this period, together with figures for *storage change*. In order to determine storage change, monthly potential evapo-transpiration values are calculated from meteorological data, and these values are adjusted in relation to the records of monthly precipitation to indicate the amount of moisture storage or moisture deficiency for any month (De Bruijn, 1973). It will be seen from Figure 10/2 that practically the same amount of ultimate heave (130 mm) was measured at the fallow area and at the area under fibre-glass. This observation led De Bruijn (1973) to conclude that the ultimate heave for both these areas could be ascribed solely to the termination of evapo-transpiration, of which transpiration by the grassy vegetation constitutes the major part. During dry periods the uppermost layer of soil in the fallow area acts as a surface 'mulch' by drying out rapidly and providing an effective vapour barrier.

Under the sand cover the ultimate heave was found to be about 170 mm. De Bruijn (1973) points out that the mulch is provided here by the top part of the sand layer. He also observes that the lower part of the

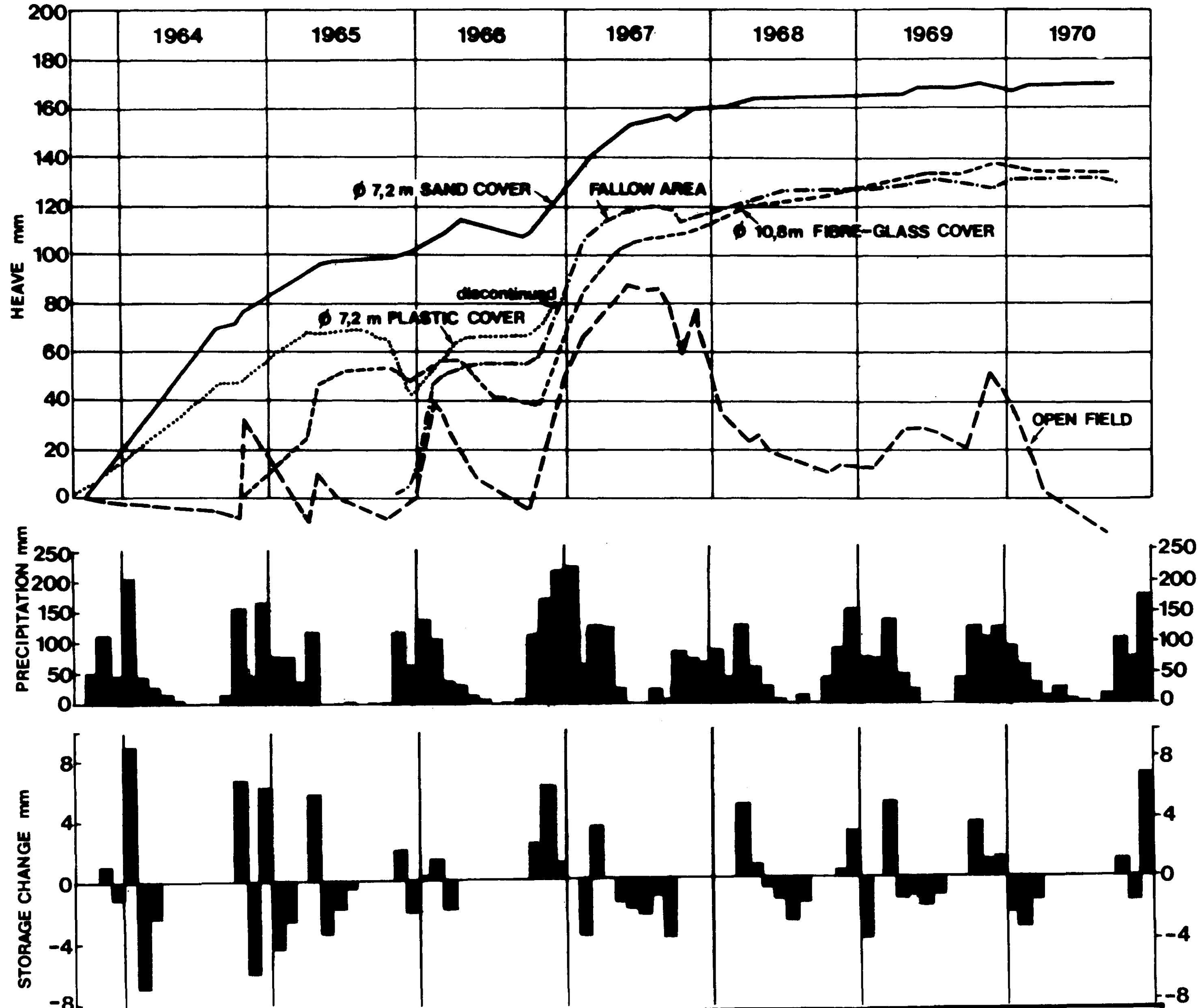


FIGURE 10/2 : Heave versus time curves for the surface of a residual gabbro soil profile under different boundary conditions at the CSIR test site, Onderstepoort (Based on De Bruijn, 1973) sand blanket remains water-saturated even after prolonged periods of drought (e.g. that from March to November 1966) and this acts as a permanent source of moisture for the underlying clay layer of low permeability.

Similar experiments have been conducted by the CSIR on expansive soils in very humid and very arid regions, and it has been found that the effect of surface covers on soil mositure conditions in such environments is negligible. The construction of roads or buildings in very wet or very dry regions should thus not affect the soil moisture regime to any significant extent.

The findings of this important series of experiments have obvious practical applications. If, in a dry subhumid climate, vegetation is removed from the surface to be later covered by a road, air-field or building, and the surface is allowed to lie fallow for as long as possible (preferably for three or four years), most of the potential heave will have been dissipated by the time construction commences. This effect will be even more marked if the area so stripped of vegetation is covered by a blanket of sand. It seems likely, too, that the use of a sand blanket will be most effective in areas of 'self-mulching' clays such as the black clays of the Bushveld. Such surface treatment, however, will not be effective in very humid or very arid regions.

Wheat crops on the Bushveld soils

It is of interest to note in passing that, whereas in the past the black clay soils of the Bushveld, with the exception of those in the Springbok Flats, have been traditionally regarded as suitable only for cattle pasture, the cultivation of wheat on these soils since the winter of 1974 has proved dramatically successful. Yields of more than 60 bags of top quality wheat per hectare in dry-land farming without the use of fertilisers have been achieved on these soils in the Bushveld of the north-western Transvaal (Van Schalkwyk, 1975). The secret of this success appears to lie in the capacity of these soils for retaining moisture on being allowed to lie fallow. The findings of the enigneering experiments conducted by the CSIR at Onderstepoort would thus seem to have a practical application also in the field of agriculture.

SOIL PROFILES DEVELOPED ON THE MAFIC ROCKS OF THE BUSHVELD COMPLEX

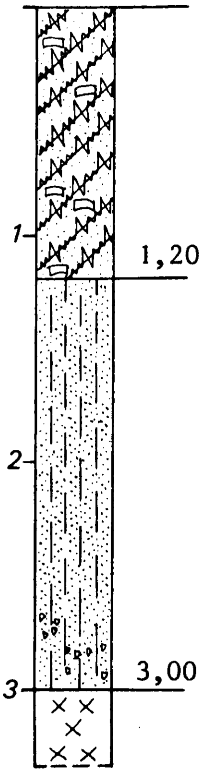
Mention has so far been made only of the black clays developed on the Bushveld gabbros and norites. However, a variety of red and reddish brown soils also develop in certain situations on these rocks. The factors controlling the development of a local soil profile include not only topographic situation, but also the lithology of the parent-material and the nature and thickness of the overlying transported soils.

Seven characteristic soil profile types have been recognised on the southern belt of mafic rocks stretching from Pienaarsrivierdam in the east to Pilanesberg in the north-west, and it is likely that similar soils, possibly with local modifications, extend throughout most or all of the area occupied by these rocks. The following descriptions of these characteristic soil profiles are based largely on the observations of Van der Merwe (1965), substantiated by the later observations of Partridge (1969) and extended and modified by the writer's observations.

A. Black clay

The typical soil profile for black clay, representing the reworked horizon on surface of residual gabbro or norite, is shown in Figure 10/3. The profile is characteristically developed on flat or gently sloping pediments. Reworking, or mixing *in situ*, is achieved by the self-mulching process: shrinkage cracks developed by the drying out of the smectite clay become partly choked by particles of the surface soil falling into them, and these become incorporated into the soil mass when the cracks close during rainy seasons. Seasonal repetition of this process produces a constant stirring of the soil mass. This action, coupled with the very highly expansive nature of the soil, results in the unique phenomena which so plague Bushveld farmers: pipe-lines buried in the soil work their way to the surface in a relatively short period of time, fencing posts planted in the soil tend to tilt over, and barn walls soon go out of alignment. Buildings founded in the clay suffer severe cracking: the potential heave that may take place in a light structure is 81 mm (Van der Merwe, 1965). As the black clay is seldom more

SOIL PROFILE A



Black, in profile dark grey to black, stiff, shattered and slickendsided (with open fissures up to 100 mm wide penetrating to base of horizon when dry), slightly sandy clay, sometimes with calcrete nodules; reworked residual gabbro or norite.

Yellowish brown, in profile yellow to olive speckled grey and white, very dense but friable, silty (often coarse) sand, with angular grains of weathered plagioclase and pyroxene; residual gabbro or norite. Weathering speriods sometimes present towards base of horizon.

Very soft rock, weathered, to very hard rock, fresh; gabbro or norite.

FIGURE 10/3 : Generalised soil profile for black clay on pediment of Bushveld gabbro or norite than about 1,5 m thick, however, a commonly applied practice is the excavation of the clay down to the light coloured underlying soil and replacement with compacted inert material after foundations have been built up to ground level. Stub piles underreamed in the soil below the expansive horizon have also been used successfully.

Indicator test data for the black clay and the underlying residual soil are given in Table 10.3, together with comparative data for other soil types developed on the mafic rocks.

B. Black and grey clays in depressions

A generalised soil profile for deep expansive soils developed in topographic depressions is given in Figure 10/4. It was within a soil profile of this type that the CSIR experiments at Onderstepoort were conducted (cf Figure 10/1). It will be seen from Table 10.3 that the Atterberg Limits and the clay content of the grey clay underlying the superficial black clay are excessively high and the potential expansiveness of this soil profile is consequently greater than for any other soil profile encountered in Southern Africa: as shown in Figure 10/2, the potential heave is 170 mm. The predominant clay-mineral is montmorillonite, with small quantities of hydrous mica in some localities. Kaolinite is absent.

C. Reddish brown sandy clay on concave sideslope or pediment crest

The characteristic soil profile for this group is shown in Figure 10/5. A thin mantle of sandy hillwash is commonly present at the surface, but seldom exceeds one or two metres in thickness. The predominant clay-mineral in the reddish brown sandy clay is kaolinite, but small amounts of montmorillonite are frequently also present (Van der Merwe, 1967). The soil profile is generally moderately expansive, with a potential heave of the order of 25 mm, but there is considerable variation from one occurrence to another, largely controlled by variations in the thickness of the reddish brown horizon.

D. Reddish brown clayey sand on pediment crest

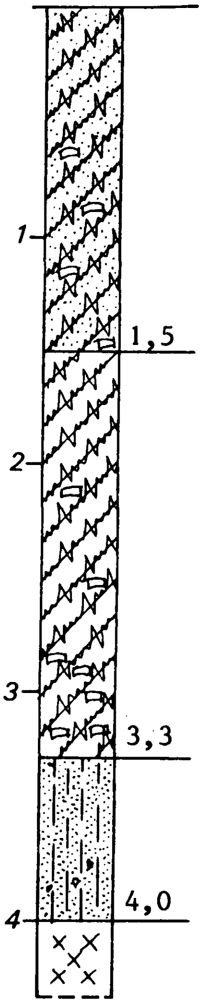
A thick blanket of sandy hillwash overlying the residual soils

Soil Horizon Number	Soil Profile Type	Soil Horizon Type	Origin	Number of Test Data	PH	LL	PI	LS	Percentage Particles smaller than			Main Clay Minerals M=Montmorillonite K=Kaolinite	Potential Expansiveness
									<2µm	<60µm	<417µm		
1	A,B	Black Clay	Reworked residual gabbro or norite	40	7,7 (7,2-8,4)	84 (59-128)	51 (36- 78)	19,5 (14,7-25,3)	45 (29-61)	66 (46-76)	94 (89-99)	M	Very High
2	B	Grey Clay	Residual gabbro or norite	7	7,9 (7,8-8,4)	108 (75-158)	69 (44-108)	23,4 (16,7-29,2)	54 (29-69)	70 (54-82)	94 (81-99)	M	Very High
3	All:A to G	Silty Sand	Residual gabbro or norite	54	8,1 (7,6-8,6)	39 (23- 54)	16 (1-29)	8,2 (1-15,3)	11 (2-28)	31 (12-64)	63 (36-90)	M > K	Low to medium
4	C	Reddish brown sandy clay	Reworked residual gabbro or norite	56	6,1 (5,7-6,5)	48 (20- 76)	24 (10-40)	12,4 (4-19,3)	36 (9-57)	53 (17-75)	88 (55-98)	K > M	Medium (low to very high)
5	E	Red sandy clay on ferrogabbro	Reworked residual gabbro or norite or ferrogabbro	10	6,1 (5,6-6,6)	38 (17- 51)	16 (6-22)	9,5 (2-13,3)	31 (18-42)	51 (23-60)	70 (61-94)	K	Low
6	D,E	Reddish brown clayey sand	Reworked residual gabbro or norite	15	6,1 (5,7-6,5)	38 (28- 52)	19 (5-32)	9,5 (5,3-13,3)	23 (9-37)	40 (22-54)	81 (55-98)	K	Low to Medium
7	G	Alluvial Clay	Transported Soil	18	?	56 (28-100)	37 (16- 70)	14,5 (7,5-32)	34 (21-47)	56 (45-78)	?	M > K	Medium to very high

TABLE 10.3 : Test data for soils developed on the mafic rocks of the Bushveld complex

(Mainly after Van der Merwe, 1967)

SOIL PROFILE B



Black, in profile dark grey to black, stiff, shattered and slickensided (with open fissures up to 100 mm wide), slightly sandy clay, sometimes with calcrete nodules; reworked residual gabbro or norite.

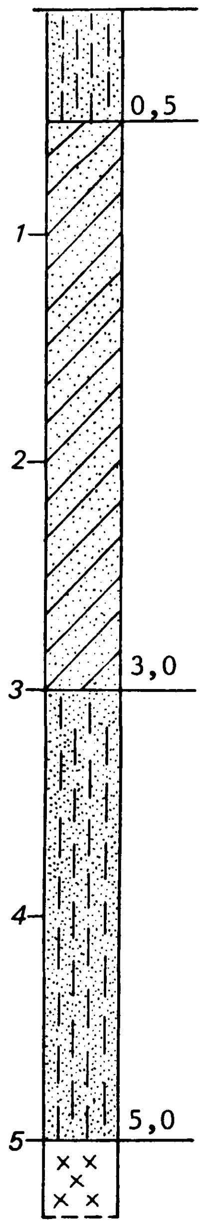
Light to dark grey, firm to stiff, fissured and slickensided, clay with calcrete nodules (usually concentrated towards base of horizon); residual gabbro or norite.

Light yellowish brown, in profile yellow to olive speckled grey and white, very dense but friable, silty (often coarse) sand, with angular grains of weathered plagioclase and pyroxene; residual gabbro or norite.

Very soft rock, weathered, to very hard rock, fresh, gabbro or norite.

FIGURE 10/4 : Generalised soil profile for deep black and grey clays in depressions on Bushveld gabbro or norite

SOIL PROFILE C



Reddish brown, soft when moist, to very stiff when dry, intact, silty sand; hillwash. (This horizon may be absent).

Dark brown, in profile reddish brown, stiff, intact to slightly shattered, sandy clay; reworked residual gabbro or norite.

Light yellowish brown, in profile yellow to olive speckled grey and white, very dense but friable, silty (often coarse) sand, with angular grains of weathered plagioclase and pyroxene; residual gabbro or norite.

Weathering spheroids or very soft rock, weathered, to very hard rock, fresh; gabbro or norite.

FIGURE 10/5 : Generalised soil profile for reddish brown sandy clay on concave side-slopes or pediment crest in Bushveld gabbro or norite, sometimes with thin cover of hillwash

produces a soil profile of the type illustrated in Figure 10/6. The topographic situation associated with this development is generally the pediment crest below an escarpment of marginal quartzite, either of the Magaliesberg dipslope or of a local outlier of Smelterskop quartzite within the mass of mafic rock. A similar profile development is associated with the hillwash soils derived from the Bushveld granite or granophyre or from Karoo grits near the contact of these rocks with the gabbro. Where the hillwash is derived from quartzite* there is often a well-developed pebble marker at its base but where the hillwash is of felsic derivation the pebble marker is either poorly developed or even absent.

That the upper horizon of the residual zone is thoroughly reworked is illustrated by its high content of sand, with a mean value of 60 per cent. Most of the sand is derived from the overlying hillwash, though Van der Merwe (1967) notes the fact that in one deep borehole within this soil profile type the parent-material was identified as a quartz-bearing norite.

The reddish brown residual soil contains kaolinite as the predominant, and often the only, clay-mineral, but small quantities of montmorillonite are sometimes present near the base of the horizon. Very small amounts of hydrous mica have been reported from some localities (Van der Merwe, 1967). It will be seen from the indicator properties listed in Table 10.3 that the material has a low to medium potential expansiveness.

This soil profile, and the climatic zone within which it develops, provide excellent conditions for citrus cultivation.

E. Red sandy clay on ferrogabbro

A soil profile very similar to that described in D above is found on the ferrogabbro where magnetite grains are present in the upper horizon (Van der Merwe, 1967). A typical description is given in

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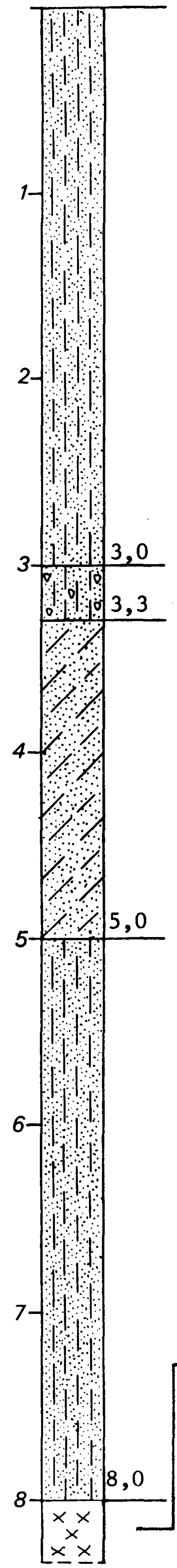
* Hillwash derived from quartzite in this area commonly has a dry density below 1 600 kg/m^3 and exhibits a collapsible grain structure.

SOIL PROFILE D



Reddish brown, soft (when moist) to very stiff (when dry), intact, silty sand (clay content often increasing with depth); hillwash.

Scattered, angular to subangular, vein quartz and/or quartzite gravels in matrix as above; pebble marker. (This horizon may be poorly developed or absent).

Dark brown, in profile reddish brown sometimes mottled yellow, firm to stiff, intact to slightly fissured, clayey sand; reworked residual gabbro or norite.

Yellowish or reddish brown, in profile yellowish brown to olive speckled white and grey, very dense but friable, silty (often coarse) sand with angular grains of weathered plagioclase and pyroxene; residual gabbro or norite.

Weathering spheriods or very soft rock, weathered, to hard rock, fresh; gabbro or norite.

FIGURE 10/6 : Generalised soil profile for reddish brown clayey sand on pediment crest below contact of marginal quartzite or near escarpment with Bushveld granite in Bushveld gabbro

Figure 10/7. This profile occupies extensive areas on the ferro-gabbro north of Rustenburg. Kaolinite is the predominant clay-mineral and montmorillonite may be present in small quantities. The engineering properties of the residual soil are almost identical to those of the reddish brown clayey sand developed beneath a thick cover of hillwash (see Table 10.3). The potential expansiveness is low.

With regard to the development of this 'intrazonal' soil type, and also of those described in C and D above, C.R. van der Merwe and H. Heystek (1955) make the following observations:

> "Reddish brown or reddish chocolate clays to sandy loams occur where the weathered product of norite intermixes with coarser material derived from sandstone, granite or magnetite bands in the vicinity of the norite formation. Notwithstanding the depth and heavy texture of the mixed weathered products constituting the soil material, the soils developed therefrom are always well aerated and leached free of soluble constitutents, possess good internal drainage, and have a red colour in sharp contrast to the black-coloured soils a few yards away on the same ridge. The presence of a relatively small quantity of foreign material in the norite weathered product presumably increases the porosity and allows access of air and water."

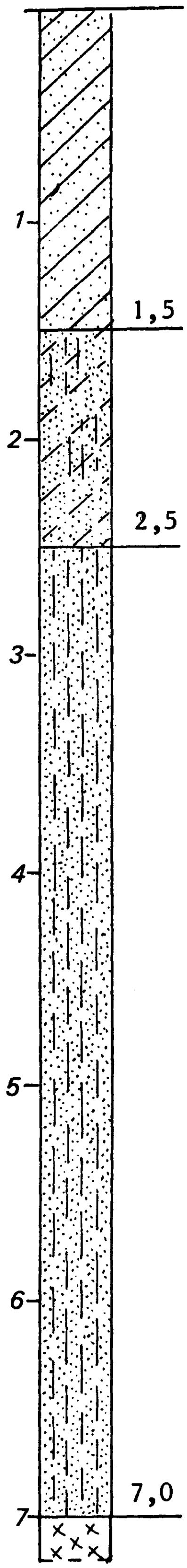
F. Soil profile with ferricrete in gulley-heads

A variant of soil profile D described above is illustrated in Figure 10/8. This soil profile, in which abundant nodular or hardpan ferricrete is present below a blanket of hillwash, is developed in gulley-heads on the pediment crest below escarpments of marginal quartzite or near the contact of the Bushveld mafic and felsic rocks. The ferricrete horizon is often developed within the pebble marker, and is underlain by residual soil in the form of silty sand with the granular fabric of the parent-rock.

G. Alluvium

Highly expansive alluvial clay derived from the weathered products of the mafic rocks occupy relatively narrow flood-plains flanking all the larger streams. The thickness of the alluvium varies within a wide range. The upper horizon of the alluvium is often a

SOIL PROFILE E



Red, stiff, intact, sandy clay with magnetite grains; reworked residual ferrogabbro.

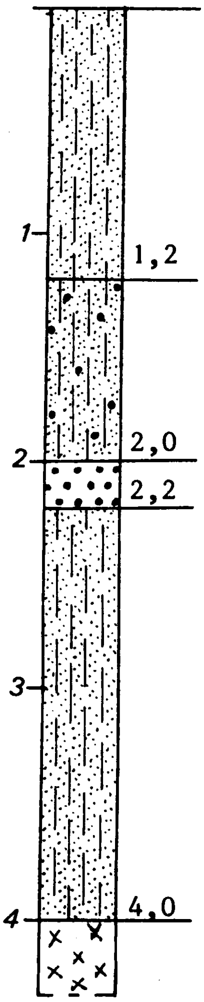
Red to reddish brown, firm to stiff, intact, clayey sand with patches of light brown silty sand; reworked residual ferrogabbro.

Light brown, in profile yellowish brown to olive speckled white and grey, dense to very dense but friable, silty (often coarse) sand with angular grains of plagioclase and pyroxene; residual ferrogabbro.

Very soft rock, weathered, to hard rock, fresh; ferrogabbro.

FIGURE 10/7 : Generalised soil profile for red sandy clay developed on Bushveld ferrogabbro

SOIL PROFILE F



Reddish brown, soft (when moist) to very stiff (when dry), intact, silty sand; hillwash.

Abundance of ferruginous concretions in matrix as above, sometimes reddish brown mottled yellow; ferruginised hillwash.

Hardpan ferricrete, or ferruginised pebble marker.

Yellowish brown to reddish brown, in profile speckled grey and white, very dense but friable, silty (often coarse) sand, with angular grains of weathered plagioclase and pyroxene; residual gabbro or norite.

Weathering spheriods or very soft rock, weathered, to hard rock, fresh, gabbro or norite.

FIGURE 10/8 : Generalised soil profile with ferricrete in gully-heads on pediment crest below excerpment of marginal quartzite or near contact with Bushveld granite in Bushveld gabbro or norite with hillwash cover

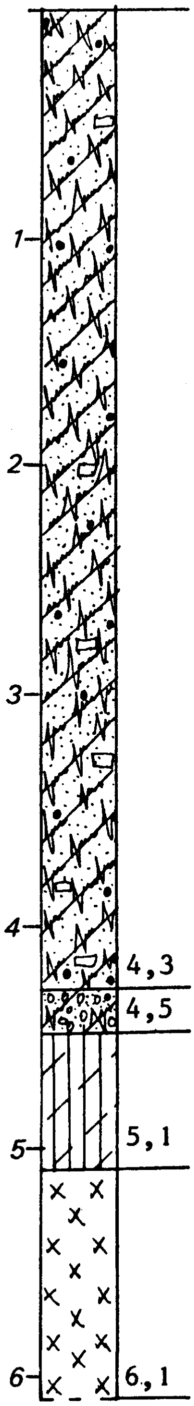
black clay which is difficult to distinguish from black residual soil: the distinguishing feature of the soil profile (Figure 10/9) is the presence of an alluvial pebble marker resting on the residual soil or weathered bedrock. A similar soil profile is found in vleis representing deranged drainage features. Indicator properties of the alluvial clay are listed in Table 10.4 for comparison with those of the residual soils from which they have been derived. It is of interest to note that the alluvial clays always become progressively more highly expansive with increasing depths.

GEOTECHNICAL MAPPING IN AREAS OCCUPIED BY MAFIC ROCKS OF THE BUSHVELD COMPLEX

The engineering significance of each of the seven soil profile types encountered on the mafic rocks of the Bushveld Complex is implicit in the description given above and in the properties listed in Table 10.3. It is thus clear that geotechnical mapping is a vital prerequisite for any development-planning in these areas. Fortunately the boundaries between the different soil profile types may be readily delineated by using the analytical techniques of airphoto interpretation. Airphoto interpretation carried out by D.H. van der Merwe (1963) for an area of more than 1 000 sq km north of Pretoria took only ten days or an average of 100 sq km per day, and intensive field checking was necessary only in a few relatively small areas, mainly where cultivation had obscured the photographic pattern. The most significant of the elements of the photographic pattern were found to be landform, grey tone and drainage pattern.

A geotechnical map of portion of the area studied by Van der Merwe (1965) is reproduced in Figure 10/10. The accuracy of Van der Merwe's soil boundaries has been verified by the writer during the course of several geotechnical investigations for proposed township development in recent years. Direct airphoto interpretation falls short only in the delineation of the ferricrete soil profile (Type F) where the gulley-heads in which it develops are choked by thick hillwash, and in the accurate delineation of the channel flanks occupied by alluvium (Type G). Some difficulty may also be experienced in detecting local occurrences of the deep soil profile of black and grey clay (Type B) where

SOIL PROFILE G



Buff, in profile grey mottled yellow and black, (upper 1 or 2 m often black or dark grey) stiff to very stiff, fissured, (when moist) to shattered (when dry), sandy clay with Fe-concretions and sometimes calcrete nodules; alluvium.

Subrounded gravels in matrix as above; alluvial pebble marker

Yellow, in profile yellow blotched grey, stiff, fissured, slightly clayey silt or sand; residual gabbro or norite.

Weathering spheriods or very soft rock, weathered, to hard rock, fresh, gabbro or norite.

FIGURE 10/9 : *Generalised soil profile for alluvium in stream flood-plains or vleis on Bushveld gabbro or norite*

ENGINEERING PARAMETERS : INDICATOR TEST DATA		LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	PERCENTAGE CLAY	PERCENTAGE SILT & CLAY
		LL	PI	LS	% <2µm	%<60µm
MAXIMUM	x_M	100	70	32,0	47	78
MINIMUM	x_m	28	16	7,5	21	45
MEAN	$\bar{x}$	56	37	14,5	33,7	55,7
NUMBER OF TEST DATA	n	18	18	18	18	18
STANDARD DEVIATION	s	25,8	20,3	7,9	7,8	9,6
COEFFICIENT OF VARIATION	$\frac{s}{\bar{x}}$	0,458	0,549	0,551	0,238	0,173

TABLE 10.4 : Indicator test data for alluvium derived from mafic rocks of Bushveld Complex

LEGEND OF SOIL PROFILES

(Soil boundaries and most point identifications of soil profile types taken from a map prepared by D.H. van der Merwe 1965)

- SOIL PROFILE TYPE A (black clay), WITH UNDIFFERENTIATED PATCHES OF TYPE B (black and grey clays in depressions) AND OF TYPE G (alluvium)
- SOIL PROFILE TYPE C (reddish brown, sandy clay under thin hillwash cover)
- SOIL PROFILE TYPE D (reddish brown, clayey sand under thick hillwash cover) WITH UNDIFFERENTIATED PATCHES OF TYPE F (ferricrete in gulley-heads)
- SOIL PROFILE TYPE E (red sandy clay on ferrogabbro) WITH UNDIFFERENTIATED PATCHES OF TYPE F (ferricrete in gulley-heads)
- OUTCROP OF MAFIC ROCK WITH LITTLE OR NO SOIL COVER

- Railway lines and stations
- Main roads
- Rivers
- •D Soil profile sites

LEGEND OF ROCKS

(Lithological boundaries taken from Geological survey map 2528 CA

SEDIMENTARY ROCKS

- Ecca Series sandstone and grit
- Smelterskop quartzite
- Magaliesberg quartzite

IGNEOUS ROCKS

- Diabase
- Granophyre
- Hartzburgite, dunite and serpentinite
- Ferrogabbro with magnetite bands
- Gabbro
- Norite
- Hybrid rocks

N

SCALE

1 0 1 2 3 4 5

kilometres

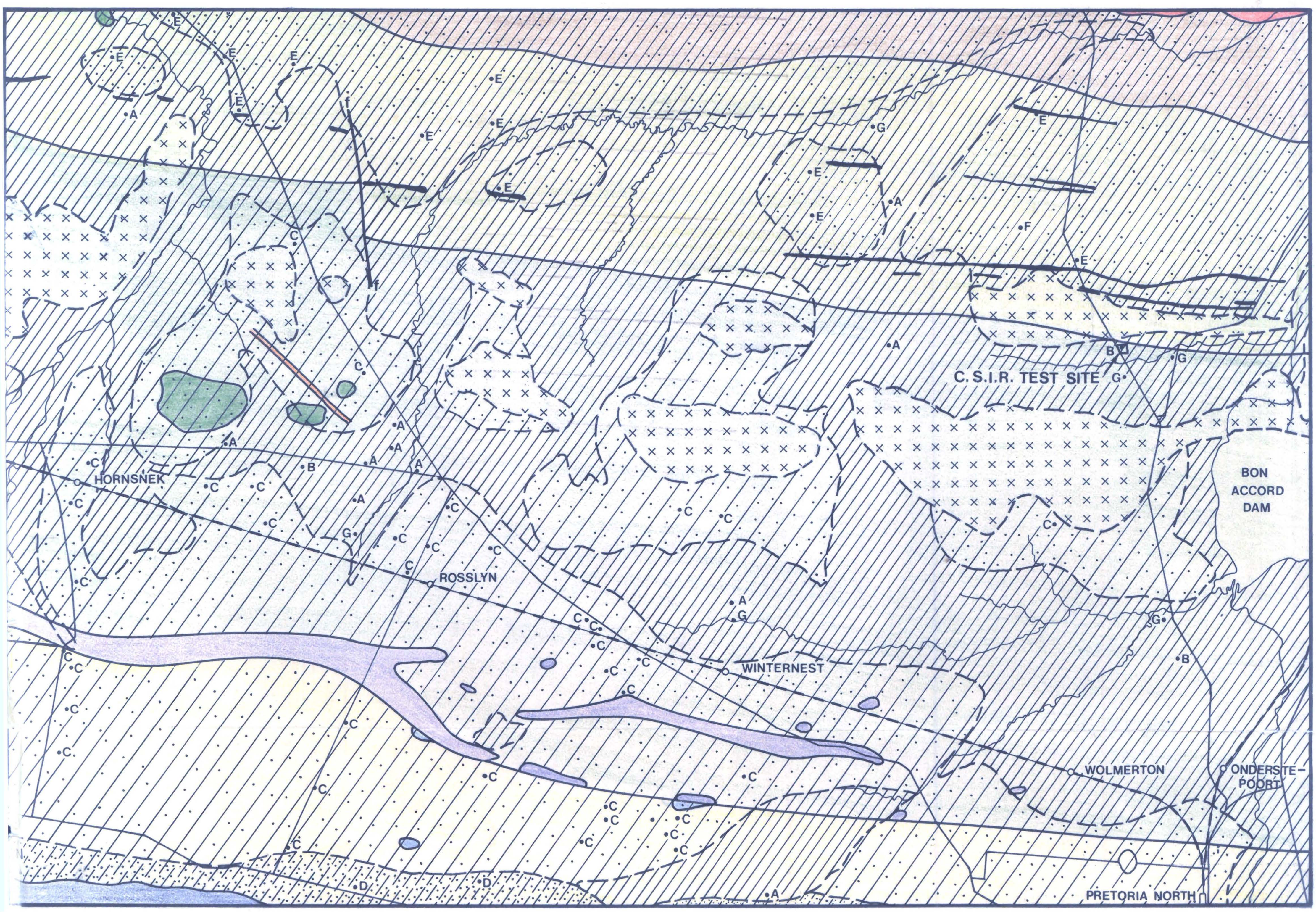


FIGURE 10/10 : Geotechnical map of an area of 200 sq km north of Pretoria occupied by mafic rocks of the Bushveld Complex these are developed within areas occupied predominantly by the shallow black clay profile (Type A).

It is of interest to draw attention to the following relationships between the development of a soil profile and the distribution of rock types as illustrated in the area covered by Figure 10/10:

* profile Type C, developed under a thin hillwash cover, occupies the colluvial aprons which surround outcrops of the mafic rocks and outliers of Smelterskop quartzite;

* profile Type E develops in the vicinity of magnetite bands within the ferrogabbro;

* profile Type D, developed under a thick cover of hillwash, occupies the colluvial apron marginal to the Magaliesberg quartzite range which strikes from west to east on the extreme southern boundary of the area;

* profile Type A occupies most of the remaining area, and is characterised by the development of a drainage pattern, with profile Type G flanking the streams.

PROPERTIES AND USES OF THE FRESH ROCK

In the fresh condition Bushveld norites and gabbros are hard and massive rocks which are widely used in the building industry, both as natural building stones and as coarse aggregate for concrete. When quarried for use as natural building stone or for monumental purposes these rocks are generally referred to in the industry as 'black granite' or 'blue granite'. Large quantities of the latter type are exported annually (NBRI, 1967). The crushed rock also furnishes an excellent material for use in road construction as base-course and as aggregate in bituminous carpets. Strength and deformation characteristics of fresh norite are given in Table 10.5, while Table 11.1 in the following chapter includes a figure for the percentage shrinkage of concrete made with fresh norite as the coarse aggregate.

Table 10.5 includes petrographic analyses and the grain size distribution of two samples of fresh norite from the Belfast District, but it should be mentioned that, within the belt of mafic rocks of the Bushveld Complex as a whole, there is a considerable variation in texture, from medium-grained to very coarse-grained, and also a variation in colour and mineralogical composition.

The quarries, however, are largely confined to two general areas within the Bushveld Complex, and they are almost entirely confined to a specific landform within these areas. Most of the quarries are in the belt between Pretoria North and Rustenburg, i.e. in the southern part of the western cone-sill of mafic rocks. A lesser number of quarries are in the belt between Wonderfontein (in the Belfast District) and Chuniespoort, i.e. along the length of the eastern cone-sill. All quarries are confined to pyramid-like hills which form the linear ranges in both of these areas. And it is of significance that these ranges are aligned along the 'strike' of the layered igneous rocks.

It is thus clear that we are dealing again with a stratigraphic, or rather *pseudostratigraphic*, control. The massive rocks which stand out as pyramid-like hills in these linear ranges are clearly more resistant to weathering, and thus to erosion, than are the other mafic and ultramafic rocks of the Bushveld Complex.

LOCALITY Stone quarry: Belfast District Depth: Sample (1): 7,6 m (tested 1966) Sample (2): (tested 1970)	Sample	σ_1/σ_3 ratio	STRENGTH MPa	Number of specimens tested	Standard Deviation
TRIAXIAL COMPRESSIVE STRENGTH	(1) (2)	38,20 36,60	479 507	3 3	1,63 2,30
	(1) (2)	13,34 20,10	771 684	3 3	1,71 0,90
	(1) (2)	9,19 12,90	993 912	3 3	1,53 2,10
UNIXIAL COMPRESSIVE STRENGTH	(1) (2)	∞	307 277	6 6	0,31 1,60
UNIAXIAL TENSILE STRENGTH	(1) (2)	$-\infty$	18,4 16,8	3 3	2,10 8,40

DENSITY: kg/m^3	(1) (2)	2 950 2 940	MODULUS OF ELASTICITY: GPa	(1) (2)	103 98
POROSITY	(1) (2)	0,3% 0,3%			
MOISTURE CONTENT	(1) (2)	0,07% 0,07%	POISSON'S RATIO	(1) (2)	0,23 0,25

PETROGRAPHIC ANALYSIS		
QUALITATIVE	COMPOSITION (VOL %)	GRAIN SIZE DISTRIBUTION
<u>Both samples</u> Felspar and pyroxene with few accessories. Ophitic texture: grains intersecting one another. Grains evenly distributed.	<u>Both samples</u> Labradorite (partly saussuritized): 60,0 Hypersthene : 36,4 Accessories (biotite, amphibole): 3,6	<u>Sample (1)</u> Evenly grained: grain size being 0,8 - 1,0 mm <u>Sample (2)</u> 0,3 - 3,5 mm Plagioclase: 0,4-3,1 mm Pyroxene : 0,3-3,5 mm

TABLE 10.5 : Strength and deformation characteristics of two samples of fresh Bushveld norite from Basal Zone

(NMERI-CSIR)

11 SEDIMENTARY ROCKS OF THE KAROO SYSTEM

The Karoo System occupies well over half a million square kilometres - nearly one half of the surface area of South Africa. It is therefore surprising that, with the exception of Bloemfontein, East London and Welkom, none of the major population centres is situated within this area. Although lacking in large towns however, the Karoo System carries a greater mileage of roads, railways, tunnels and canals, and a larger number of dams and bridges, than any other geological formation in the country. Undoubtedly, therefore, it must be regarded as the most important formation from the geotechnical viewpoint.

The system consists of a vast accumulation - nearly eight kilometres thick - of shales and sandstones, with tillite at the bottom, basalt at the top, and coal about halway up the sequence. For the most part the strata are horizontally disposed and have been intruded by dolerite sheets and dykes. The notable exception to this is the southern margin of the system, a strip less than a hundred kilometres wide flanking the Cape Folded Belt. Here, where the strata have been folded under compression, dolerite has not been intruded.

Apart from the fact that tillite is found only in the Dwyka Series, and basalt only in the Stormberg Series, the basis upon which stratigraphers have subdivided the System into four different series is palaeontological rather than lithological. The fossil content of strata is of little direct significance in engineering, however, and it is therefore proposed to deal here with certain of the rock types, particularly the sandstones, rather than dealing with each series in turn. Our main concern will be with the use of Karoo sandstones as concrete aggregate or natural building stone.

CONCRETE AGGREGATE

Use of Karoo sandstones as coarse aggregate, or of river sand derived from Karoo rocks as fine aggregate, can result in serious deterioration of concrete structures. This fact was first realised about 1950, when concrete-framed buildings were first introduced to the town of Graaff-Reinet: it was the occurrence of severe distortion of beams and cracking of brickwork supported on such beams in these buildings that first drew attention to this phenomenon (Stutterheim, 1954). Subsequent examination of these, and of other structures in which Karoo sandstones and locally derived sands had been used as concrete aggregate, revealed that deterioration of the concrete manifested itself in four different ways (Stutterheim, 1960):

1. Deflection of reinforced members

   Deflections of as much as 50 mm were measured at midspan on beams 9 m long. In buildings, such deflections resulted in the cracking of brickwork panels supported on the beams, buckling of ceilings and distortion of door and window frames. In multispan bridges, of which there are many examples built with these aggregates before the problem was known to exist, the deflections were recognised as a hazard to the unwary motorist attempting to cross these bridges at speed. In cantilevers the effect was particularly pronounced; e.g. the downward deflection of the unsupported end of a shop-front canopy, 4 m wide, in Graaff-Reinet was, in one case, as much as 130 mm.

2. Cracking of concrete coincident with reinforcing steel

   Where steel bars, stirrups, or even electrical conduits, were embedded near the surface of concrete members, coincident cracks had developed in the concrete.

3. Corrosion of reinforcing steel

   Where reinforced concrete was exposed to the atmosphere, cracks of the above type had widened owing to the oxidation and hydration of the steel which produced iron compounds of increased volume. This had often resulted in spalling of the concrete and, sometimes, complete destruction of the steel.

4. Surface crazing or pattern cracking

   The crazing normally associated with smooth concrete or mortar surfaces which are exposed to the atmosphere was found to be exhibited to an excessive degree in practically all concrete made of these aggregates (Figure 11/1).

That all of these features were evidence of substantial dimensional changes in the concrete was immediately clear, but the nature of these changes was not obvious at first. The possibility of expansion due to alkali-aggregate reaction as described by Stantón (1940) was considered, but had to be dismissed: South African Portland cements are generally low in alkali, and the aggregates proved to be free of amorphous or microcrystalline silica. Furthermore, while expansion of the concrete could have been the cause of excessive crazing, it could not have accounted for the types of deflections and other phenomena observed. The possibility of expansion due to soluble sulphates had likewise to be dismissed.

When consideration was finally given to the possibility that excessive *shrinkage* could have been the cause of the trouble, it became clear that all the observed phenomena could, in fact, be accounted for on this basis.

In order to understand how the observed deflections of beams and cantilevers could result from shrinkage of concrete, a basic principle of structural engineering must be appreciated, namely that steel is introduced into concrete to provide tensile strength where this is required. Thus, in a beam supported at both ends by columns, steel is placed near its base whereas, in a cantilevered slab, steel is placed in the upper fibres. When excessive shrinkage takes place in the concrete, the steel will effectively restrain shrinkage in its immediate proximity. There will, however, be no restraint to shrinkage in the unreinforced compression zone, and the member must deflect to accommodate the differential shortening in length. In the case of the simply supported beam such deflection will be reflected by a downward sagging at midspan. In a cantilevered slab such as a shop-front canopy the unsupported side will deflect downwards. Furthermore, cracks will tend to develop coincident with steel which has been placed near the surface of a concrete member, owing to the tensile stresses developed by shrinkage in the weakest section of the concrete cover. Penetration of water into these cracks will initiate corrosion of the steel which, in turn, will result in spalling of the concrete.

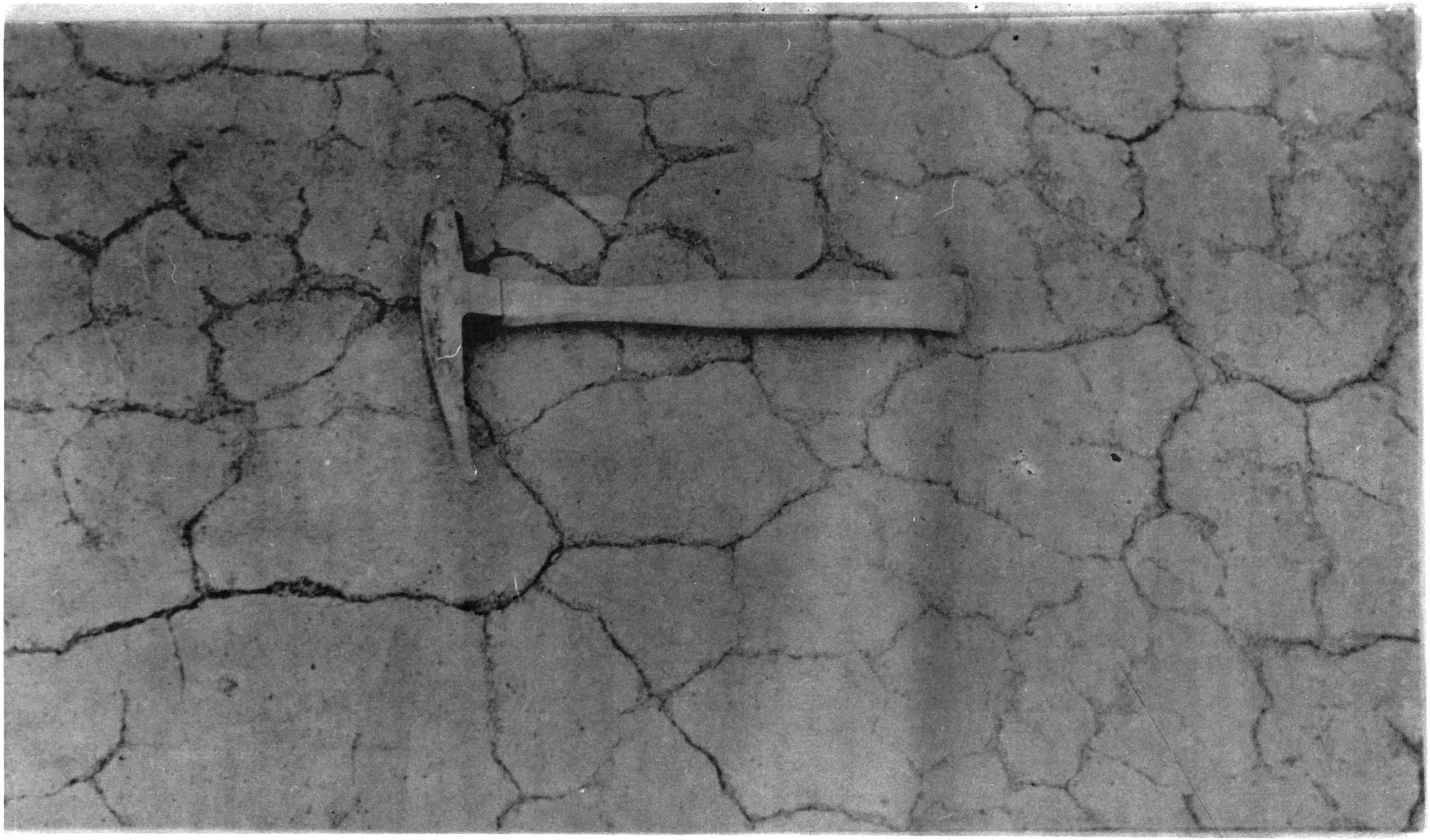


FIGURE 11/1 : Surface crazing of concrete landing, Town Hall, Graff-Reinet

That excessive shrinkage had, in fact, been responsible for all the features observed, was confirmed by laboratory experiments conducted by the National Building Research Institute (Stutterheim, 1954; Webb, 1959; Roper and Webb, 1961) and later by petrographic studies conducted by the Research and Development Laboratories of the Portland Cement Association, Illinois (Roper *et al*, 1965; Roper, 1969).

Shrinkage determinations were made on 76 mm x 76 mm x 300 mm concrete specimens of various aggregates mixed to 1:2:4 proportions. After moist curing for seven days the specimens were allowed to dry out completely and their drying shrinkage was determined. Some of the results are given in Table 11.1. The Municipal quarry sandstone, Adendorp quarry sandstone and Sundays River sand are all from the Karoo formation in the Graaff-Reinet area. A control specimen was made with Bon Accord norite and clean quartz sand from the Crocodile River, both from the Pretoria district, and both of which are known to behave normally in concrete.

COARSE AGGREGATE	FINE AGGREGATE	PERCENTAGE SHRINKAGE OF CONCRETE SPECIMENS
Municipal quarry sandstone	Sundays River sand	0,094
Adendorp quarry sandstone	Sundays River sand	0,115
Bon Accord norite	Crocodile River sand	0,012

TABLE 11.1 : Shrinkage determinations on concrete specimens

From these figures it will be seen that concrete in which certain Karoo sandstones and river sands derived from Karoo rocks have been used as aggregates may shrink as much as ten times more than concrete made with normal aggregates. The shrinkage was found to take place as the concrete dried out, and this explained why the deleterious effects manifested themselves in buildings and other structures within the first few months after completion.

Figure 11/2 shows the deflection in an experimental concrete beam made with aggregates of sandstone and river sand from the Graaff-Reinet area.

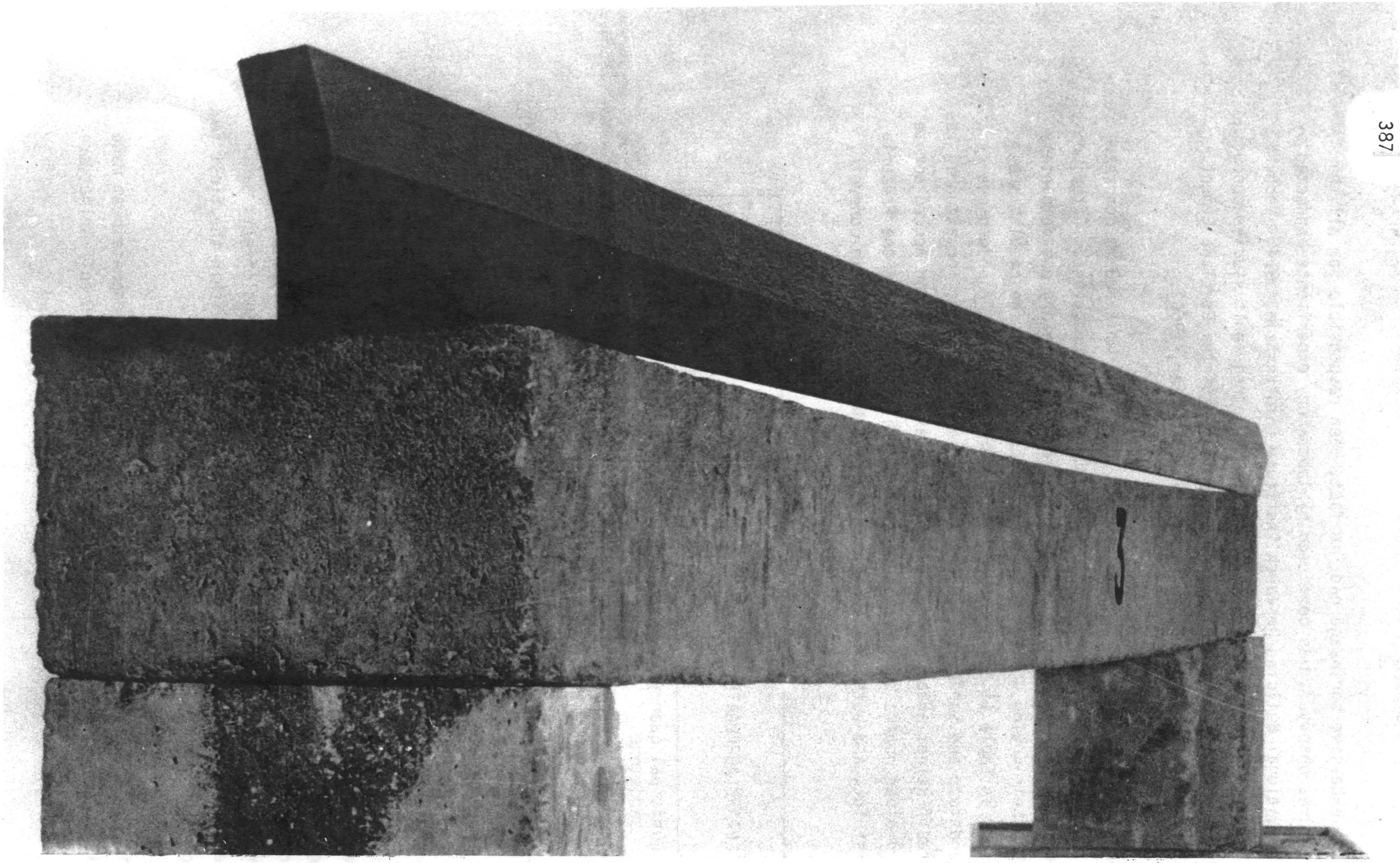


FIGURE 11/2 : Deflection in experimental concrete beam made with aggregates from Graaff-Reinet

The deflection is emphasised by the steel straight-edge resting on the beam (After Stutterheim, 1954)

That the aggregates themselves shrank as they lost moisture was shown by cutting prisms of the rock either parallel to or at right angles to the bedding planes, soaking them in water, and measuring their lengths both before and after allowing them to dry out. The results of some of these shrinkage determinations are given in Table 11.2.

Four significant facts emerge from these figures:

1. The drying shrinkage of norite has been measured as 0,001 per cent, and of fresh dolerite as 0,003 per cent. The drying shrinkage for sedimentary aggregates which behave normally in concrete is of the order of 0,005 per cent. Values given for Karoo sedimentary rocks in Table 11.2 are thus phenomally high.

2. The drying shrinkage is not isotropic. It is greater in the direction at right angles to the bedding than parallel to it: in the case of Adendorp sandstone, nearly four times greater.

3. The phenomenon is not confined to Karoo sandstone: other Karoo rocks, such as the Dwyka tillite specimen from Laingsburg, also exhibit high drying shrinkage.*

4. There is no apparent decrease in the values with increasing depth below the surface: the borehole core specimen from 311 m shrank as badly as that from 7 m. It would thus appear that the phenomenon is not confined to the zone of weathering.

Dimensional change of this type is directly related to change in moisture content. Expansion takes place again when the aggregates, or concretes made from them, are soaked in water after having dried out. Repeated cylces of wetting and drying result in a progressive over-all increase in length in the case of the aggregates themselves and in

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* The only formation in South Africa apart from the Karoo System which has yielded shrinking aggregate is the Table Mountain Series, and specifically from the Churchill Dam quarry near Port Elizabeth: the drying shrinkage of fresh concrete specimens made with this sandstone aggregate ranged from 0,017 to 0,050 per cent (Stutterheim *et al*, 1967)

ROCK TYPE	LOCALITY	REFERENCE	DEPTHS BELOW SURFACE	SHRINKAGE PER CENT	
				specimen cut parallel to bedding	specimen cut 90^0 to bedding
Beaufort sandstone	Graaff-Reinet Municipal quarry, C.P.	Stutterheim (1954)	quarry face near surface	0,038	0,058
Beaufort sandstone	Adendorp quarry (near Graaff-Reinet), C.P.	Stutterheim (1954)	quarry face near surface	0,23	0,84
Beaufort sandstone	Cores from borehole situated at: x = 324,300 y = 1235,350 approx. lat. 31^0 15' S approx. long. 25^0 30' E (cores supplied by Orange-Fish Tunnel Consultants; tests by NBRI-CSIR)	Pienaar (1966)	7 m 48 m 116 m 156 m 222 m 311 m		0,12 0,12 0,07 0,16 0,095 0,11
Beaufort sandstone	Aberdeen, C.P.	Roper (1959)	near surface	0,024	
Beaufort sandstone	Queenstown, C.P.	Roper (1959)	near surface	0,12	
Beaufort quartzite	Beaufort West, C.P.	Roper (1959)	near surface	0,04	
Dwyka tillite	Laingsburg C.P.	Roper (1959)	near surface	0,04	

TABLE 11.2 : Drying shrinkage results for some Karoo sedimentary rocks

concretes made from them, probably as a result of the development of cracks. Mortar specimens made with Karoo river sands, however, generally show a progressive decrease in over-all length after subjection to several repeated cycles (Webb, 1959). It should be mentioned that sands from many pans within the Karoo formation produce concrete which shows very little shrinkage.

Examination of thin sections of the sandstone under the petrographic microscope reveals that the rock is made up of subangular grains of quartz and felspar, with a small percentage of the heavy minerals ilmenite, magnetite and zircon, set in a fine-grained matrix in which biotite, chlorite, muscovite, calcite and zeolite can be identified. Clay-minerals are also seen to be present, but these cannot be identified under the microscope.

During the early stages of research into the cause of dimensional changes in these aggregates, some of the sandstones which had exhibited the highest drying shrinkage values were submitted to X-ray and differential thermal analyses in order to identify the clay-minerals present. The first specimens tested, including those with the highest recorded shrinkage, the Adendorp quarry sandstone, contained either montmorillonite or a mixed-layer montmorillonite-illite mineral, and it seemed clear that it was the presence of these minerals, which themselves show large volume changes with changes in moisture content, that was responsible for the trouble. This provisional theory was further supported by the results of experiments with synthetic aggregates made up of montmorillonite and aluminous cement: concrete test-specimens made with these synthetic aggregates behaved in the same way as those in which Karoo sandstone had been used as aggregate.

Further analyses disclosed, however, that a number of sandstone aggregates exhibiting high dimensional change contained only those clay-minerals with non-expanding lattices, such as kaolinite, illite and the altered felspar, sericite. It had, therefore, to be concluded that, while the presence of expanding-lattice clays could be a contributory, or even major cause of dimensional change in some shrinking aggregates, this was not necessarily the case for all such aggregates. Subsequent work has, indeed, indicated that the dominant clay-mineral present in sandstones of the Ecca Series and the Stormberg Series is kaolinite, together with lesser amounts of illite, yet these rocks swell and shrink excessively with changes in mositure content (Roper, 1969). It was thus clear that the basic reason for the unusual behaviour of these aggregates was to be sought in the forces outside the crystal lattices of their constitutent clay-minerals.

A more fruitful line of thought concerned the fact that colloidal substances are known to expand on absorption of water and to contract on drying out. As colloids possess a large surface-area * it was decided to determine the internal surface-area of various shrinking aggregates in order to establish whether any relationship existed between this and percentage shrinkage (Webb, 1959). That such a relationship was, in fact, found to exist is illustrated in Figure 11/3. The shrinkage aggregates which had been found to contain no expanding-lattice clay-minerals proved nevertheless to have high internal surface-areas and therefore a high colloidal content. It may thus be concluded that the dimensional change behaviour of such aggregates is dependent, at least to a large degree, on colloidal content, whether the colloids be of the expanding lattice type or not.

A further unusual property of the sandstone aggregates is their low modulus of elasticity. Whereas normal aggregates such as norite or diabase give a Young's modulus of the order of 100 GPa in the static test, and Witwatersrand quartzite an average of 80 GPa, the sandstone from Adendorp quarry gives a value of 35 GPa. Tests on sandstone specimens from two boreholes on the Orange-Fish tunnel, from depths ranging from 6 m to 375 m, gave values in the range 8 to 10 GPa (Pienaar, 1966). While there appears to be no direct relationship between percentage drying shrinkage and Young's modulus, it is possible that the phenomena observed in concretes made with the Karoo sedimentary aggregates should not be attributed entirely to their excessive propensity for shrinkage but also, at least in part, to their low moduli of elasticity.

Some strata of sandstone and other hard sedimentary rocks of the Karoo System, such as the Dwyka tillite from the Umgeni quarry in Durban,

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* A cube of colloidal material, of 10 mm side, would have a external surface area of only 600 mm but an internal surface area of nearly half a hectare.

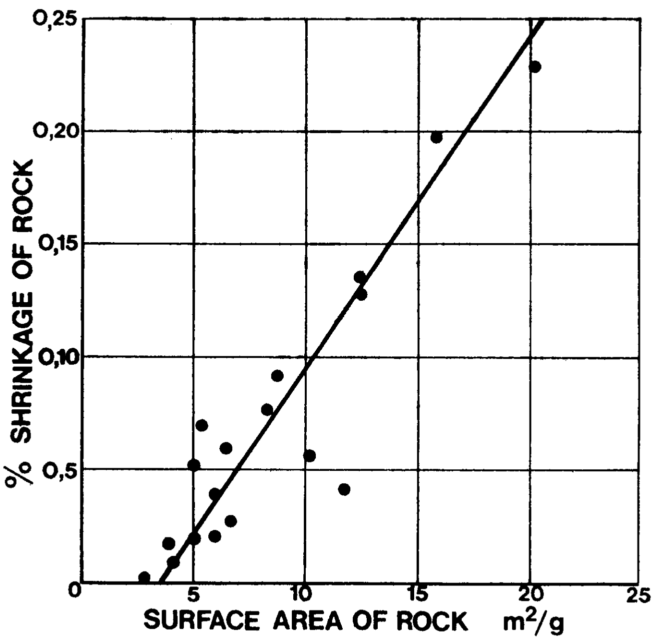

FIGURE 11/3 : Relationship between shrinkage of rock and surface area of rock for a variety of rock types including Karoo sandstones. (After Webb, 1959) provide coarse aggregates which do not exhibit the phenomena described above, and which therefore are perfectly acceptable for use in concrete. But this cannot be said of river sands derived from the Karoo rocks. Sand from any river traversing the Karoo System must be regarded as unsuitable for use as fine aggregate in concrete for most purposes. The reason for this is clear enough: not only do the sands usually contain fragments of shale and, in those rivers originating in the Lesotho and Drakensberg highlands, montmorillonite derived from the Stormberg basalts, but they are largely composed of material produced by the breakdown of those very sandstone strata which themselves exhibit excessive dimensional change characteristics. Furthermore, where the parent-sandstones contain illite, the sands derived from them have invariably suffered hydration to produce a mixed-layer montmorillonite-illite clay which will greatly increase the drying shrinkage.

Even the sands from those rivers flowing over other formations but originating in the Karoo System must be regarded as suspect. The consequences of using such sands are illustrated, for example, by the crazing and spalling of concrete curb-stones in the town of Willowmore in the southern Cape. Willowmore is situated on the Bokkeveld Series, but sand from the local Grootrivier, which rises on the Beaufort Series, had been used in the curb-stone concrete.

As it has been demonstrated that there is a wide range in the percentage drying shrinkage of these aggregates, both coarse and fine, they should not be used in concrete without prior testing. A higher shrinkage can be tolerated in mass concrete for heavy foundations than in concrete members such as cantilevers and long thin beams. Concrete exposed to the atmosphere and therefore subjected to changes in moisture content are particularly vulnerable. This is even the case in massive concrete dams. So, also, are mortars and plaster finishes. There are therefore no hard and fast rules as to permissible shrinkage values, and the decision as to whether aggregate from a particular source is acceptable or not must be based on the particular use to which it is to be put. Standard procedures for the determination of shrinkage have been evolved by the National Building Research Institute, and it is recommended that approved sources of potential aggregate be first tested according to these methods by the laboratories of the SA Bureau of Standards or some other suitably equipped laboratory, or by a field test conducted by a competent operator.

The obvious solution to the problem is the use of Karoo dolerite, as both coarse and fine aggregate. However, this is not always the most economical solution, particularly in the southern part of the Karoo System from which dolerites are absent.

CASE HISTORY 2I

THE USE OF RIVER SAND DERIVED FROM ROCKS OF THE KAROO SYSTEM AS FINE AGGREGATE IN CONCRETE

DEFLECTIONS IN UMZIMHLAVA BRIDGE

In 1945 a reinforced concrete bridge was erected across the Umzimhlava river near Mount Ayliff, on the road from Kokstad to Umtata. The bridge is supported on two buttresses and two piers, and thus consists of a central span and two outer spans, each 15 m long. The central span consists of a monolithic slab supported on the two piers and cantilevering out about 4,5 m beyond each pier, as shown schematically in Figure 11/4. Each of the outer spans is a simply supported deck slab, with one end resting on a buttress and the other on the cantilevered projection of the central slab. The main longitudinal reinforcement in the central span is thus situated in the upper fibres of the member while, in the outer spans, it is in the lower fibres.

Dolerite was used as coarse aggregate in the concrete and the local river sand as fine aggregate. From the geometry of the disposition of steel in the three deck slabs it is clear that excessive shrinkage of the concrete would result in a convex-up deflection in the central member, and in concave-down deflections in the outer members. These are, in fact, the deflections that were observed as can be seen in Figure 11/5, a photograph taken in 1954 (Stutterheim, 1960). The magnitude of these deflections, obtained by level observations along the upper surface of the bridge sidewalk, is shown in Figure 11/4. Deflections of this order clearly have an adverse, and possibly dangerous, effect on the riding qualities of the deck, but this is easily overcome by resurfacing after the deflections have reached their limiting values. Simple remedial measures of this sort have been successfully applied in the case of this bridge.

NATURAL BUILDING STONE

Karoo sandstone has been widely and successfully used as a natural building stone in many parts of South Africa, particularly in the

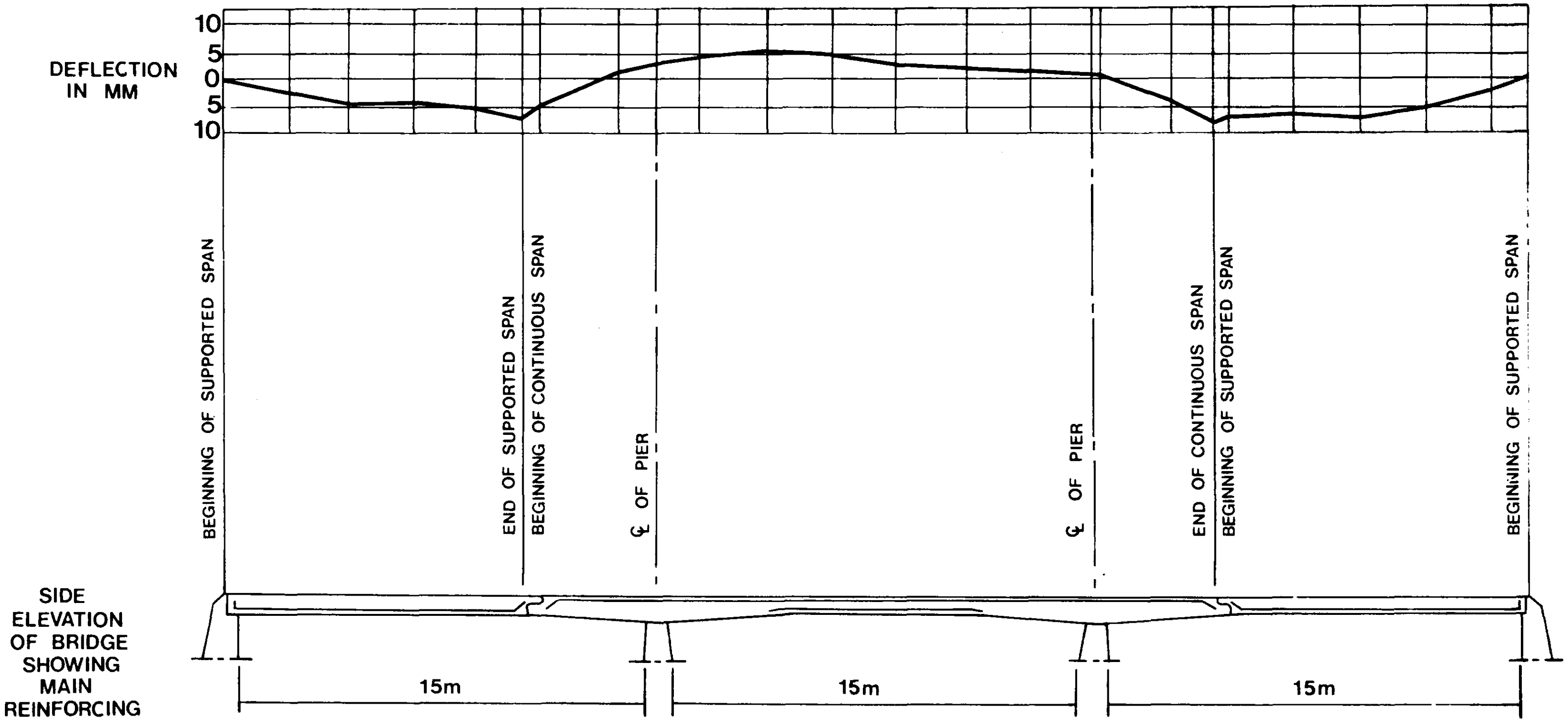

FIGURE 11/4 : Observed grade line of bridge deck as related to disposition of main longitudinal steel in bridge beams : Umzimhlava Bridge

(After Stutterheim, 1960)

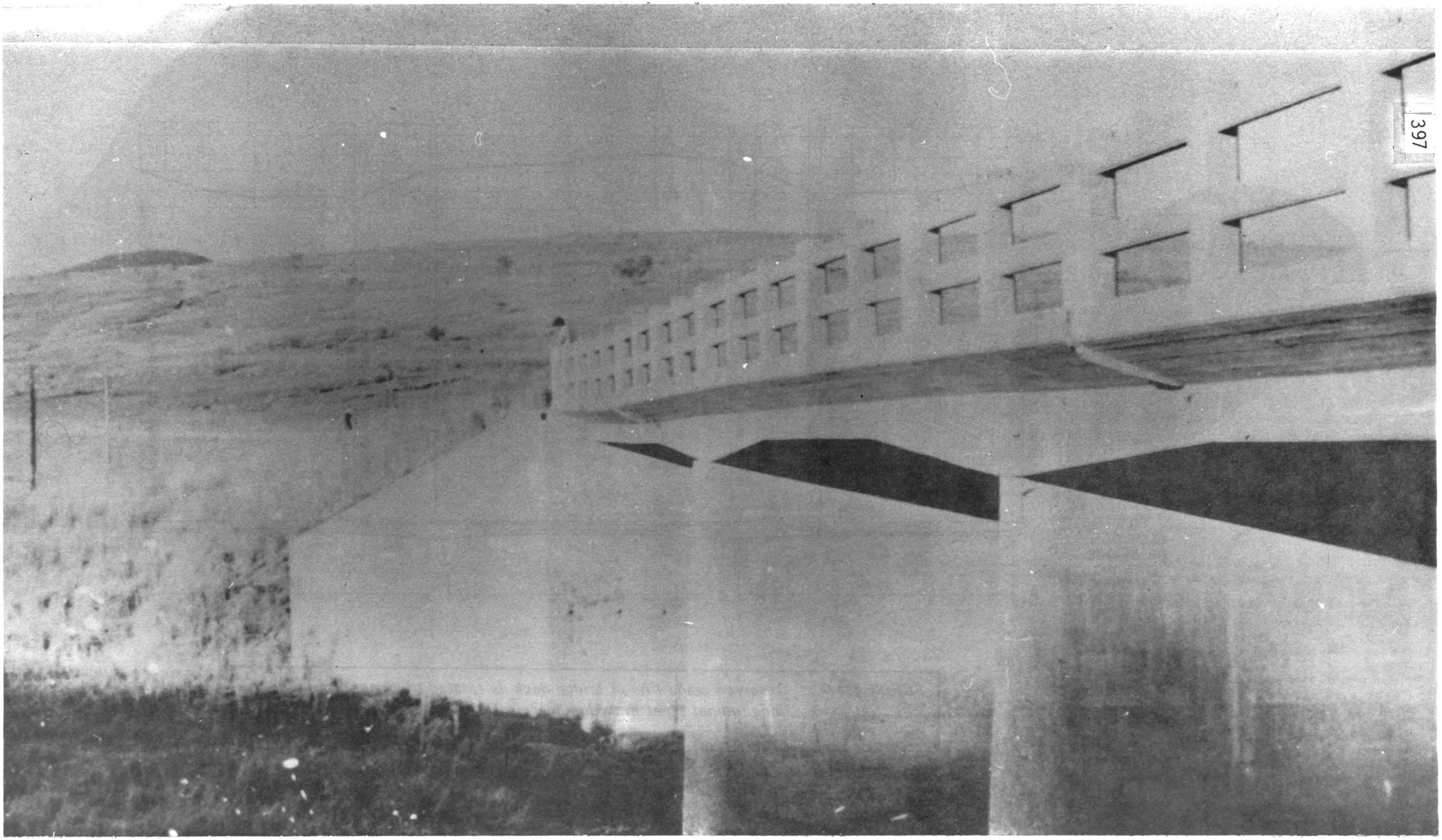

FIGURE 11/5 ; Bridge over Umzimhlava river near Mount Ayliff

Note deflections in central and far spans

Transvaal, the Eastern Cape and the eastern and northern parts of the Free State. The well-bedded varieties are particularly suited to successful quarrying, being for the most part horizontally disposed or with only slightly inclined bedding. While fine- to medium-grained strata are generally favoured, much use has even been made of the coarse sandstones of the Coal Measures in the Middle Stage of the Ecca Series in the eastern Transvaal Highveld and, to a limited extent, in Natal.

Beaufort sandstones in the Cape Province vary considerably in quality. While the harder quartzite varieties are generally durable, they are not easily worked, and their use as building stones is therefore limited (NBRI, 1967). Many of the softer varieties, though more easily worked, are susceptible to excessive dimensional changes when exposed to changes in moisture content and, as in the case of their use as concrete aggregates, this leads to rapid deterioration. As mentioned previously, these dimensional changes are anisotropic, being particularly pronounced at right angles to the bedding, and it follows that disintegration and scaling will tend to be reduced by placing the stones with their bedding-planes horizontal in a stone wall. Where such stones are not weighted down by the superstructure, however, as for example when they are used as window sills, steps, copings and ballustrades, unsightly disintegration takes place relatively rapidly. A common manifestation of such disintegration is known as the 'lifting of seams', where sandstones have a tendency to crack and open up along bedding planes. This tendency is most pronounced in sandstones which contain montmorillonite, such as the once popular "Flatpan Sandstone" which was quarried in the past from the Ecca beds at Wolwehoek in the Orange Free State. Stone from this quarry was extensively used in buildings in Cape Town and Bloemfontein, in spite of the early prediction that rapid disintegration would take place in exposed and lightly loaded blocks (Wybergh, 1932). Outstanding examples of total destruction of unweighted features such as cornices and the capitals of columns made of this stone are to be seen in the Archives building and the New Magistrate's Court in Cape Town (NBRI, 1967).

An even more common type of disintegration experienced with porous Karoo sandstones results from the crystallisation of salts within the pores. Bonnell and Nottage (1939) have shown that hydration of salts, particularly soldium sulphates, within the pores of building materials may generate stresses in excess of the tensile strength. This problem is most severe in highly porous sandstones. Whereas slow evaporation of moisture at the surface of a sandstone member of low porosity may result in nothing worse than unsightly efflorescence, the same process operating in a highly porous sandstone may lead eventually to complete disintegration. According to Wybergh (1932) even a few tenths of one per cent of soluble salts may cause distress, particularly in sandstone with a clayey matrix. Sources of soluble salt which may give rise to this problem are listed in the 'Handbook of South African Natural Building Stone' (NBRI, 1967) as follows:

1. Salt from the connate water in which the sandstone was deposited.

2. Salts in the soil-water, which is responsible for the extensive efflorescence and decay commonly seen at ground level where the stone comes in contact with the soil.

3. Salts from the jointing mortars.

4. Salts from the backing material, where stone facings are fixed to brickwork or concrete structures.

5. Sulphates formed by the reaction of carbonates within the sandstone with sulphurous acid derived from atmospheric sulphur dioxide.

6. Salts from detergents and other cleaning materials.

7. Sea salts absorbed by sandstones from the moist sea air.

8. Salts from polluted atmosphere.

Despite these problems, Karoo sandstones have traditionally been the most popular of all natural building stones in South Africa and, in spite of the fact that continuous maintainance and replacing of stones has to be undertaken in the Union Buildings, the good general state of preservation of this monumental edifice testifies to the basic soundness of this tradition.

To conclude this chapter, an indication is given in Table 11.3 of the petrographic composition of Karoo sandstone from the stone quarries at Warmbaths, and of the strength and deformation characteristics of the rock.

LOCALITY					
Warmbad Stone Quarry Transvaal Sample (1) tested 1966 : strong stratum Sample (2) tested 1970 : weak stratum	Sample	σ_1/σ_3 ratio	Strength MPa	Number of specimens tested	Standard deviation
TRIAXIAL COMPRESSIVE STRENGTH	(1) (2) (1) (2) (1) (2)	20,0 20,2 9,0 8,9 6,0 5,9	163 42,2 257 72,6 351 99,3	8 4 8 4 8 4	5,4 7,3 4,5 2,1 3,8 2,0
UNIAXIAL COMPRESSIVE STRENGTH	(1) (2)	∞ ∞	102 21,2	10 9	1,7 5,7
UNIAXIAL TENSILE STRENGTH	(1) (2)	$-\infty$ $-\infty$	7,2 0,9	5 3	1,3 6,6

Density : kg/m^3	(1) (2)	2 120 1 910	Modulus of Elasticity : GPa	(1) (2)	31 5,6
Porosity	(1) (2)	21% 22%	Poisson's Ratio	(1) (2)	0,17 -
Moisture Content	(1) (2)	0% 0,14%			

PETROGRAPHIC ANALYSES		
QUALITATIVE	COMPOSITION (VOL %)	GRAIN SIZE DISTRIBUTION
Sample 1 Fine-grained and even-textured. Grains cemented by secondary silica with some iron oxide and chlorite	Sample 1 Quartz : 80% Orthoclase : 20% with some iron oxide, muscovite, zircon and traces of kaolinite.	Sample 1 0,02 - 0,08 mm
Sample 2 Fine-grained, even-textured and porous. Quartz grains attached end to end and forming long chains around lumpy dickite particles.	Sample 2 Quartz : 67% Dickite : 33% Reddish colour imparted by small iron oxide content.	Sample 2 0,04 - 2,0 mm

TABLE 11.3 : Strength and deformation characteristics of two samples of Stormberg sandstone from Warmbaths (NMERI-CSIR)

12 SUMMARY AND CONCLUSIONS

Chapter 2 presents the characteristics of residual soils developed on ancient greenstones, mica schists, phyllites and metagabbros of the Basement-complex in different climatic zones throughout the subcontinent and in different topographic situations. For the most part these soils not only have expansive characteristics but are also highly compressible and exhibit a fast rate of consolidation. Explanations offered for these phenomena are based on the mineralogical composition and the metamorphic fabric of the parent-rocks.

In Chapter 3 the engineering characteristics of residual soils on the Archaean granite-gneisses are discussed. Mineralogical composition and selective decomposition of the primary minerals constituting the parent-rock are advanced to explain the development of a collapsible grain structure under certain geomorphological controls. The preservation of core-stones of fresh rock within the residual soil in certain situations is again attributed to mineralogical composition combined with geomorphological controls. The occurrence of pseudokarst phenomena within the highly leached residual soils is related to the fabric of the soils as inherited from the texture of the parent-rock.

Chapter 4 deals with the engineering properties of rock and residual soils of the Lower Division of the Witwatersrand System, and is almost exclusively devoted to the influence of these properties on foundation engineering in the city of Johannesburg. Reference is made to the influence of stratigraphic controls in determining the properties of the residual soils. Attention is also drawn to the constraints on deep excavation and on potential extensive underground tunnelling imposed by the juxtaposition of hard and soft strata within the local stratigraphic sequence.

Chapter 5, on mining subsidence, is again largely devoted to the Johannesburg area and specifically to the influence on surface of mined-out reefs within the Main-Bird Series. Attention is drawn to the significance of the variable engineering properties of the hanging-wall strata associated with the different auriferous conglomerates in this area, and to the influence of faults and associated intrusive igneous bodies in the propogation of earth-tremors associated with local mining activity.

In Chapter 6 emphasis is given to climatic controls in the development of residual soils from the Ventersdorp lavas.

Chapter 7 describes the unique engineering problems associated with the Black Reef and the Dolomite Series. Of particular significance within the succession is the presence of more highly manganiferous strata which give rise to the formation of wad.

In Chapter 8 attention is focussed on the variety of engineering problems associated with different strata within the Pretoria Series, and especially on the residual soils formed by decomposition of different horizons within the succession.

Chapter 9 deals with the engineering properties of residual diabase soils, mainly in the Pretoria area, and demonstrates the influence of topographic controls on the properties of the soils.

Chapter 10 discusses the development of a variety of different soil profiles on the mafic rocks of the Bushveld Complex. Even within this igneous 'succession' a significant part is played by different lithological horizons, e.g. magnetite bands, in determining soil profile development and hence engineering behaviour of the residual soils.

Chapter 11 discusses the engineering properties of different rock types within the Karoo succession, emphasis being placed on the shrinkage of concrete made from sandstones and sands derived from the rocks of the Karoo system.

The object throughout has been to demonstrate that specific engineering properties and problems are associated with the rocks comprising different stratigraphic units and with the residual soils derived from these rocks. The influence of climatic and geomorphological controls on the development of residual soils has also received attention.

It is concluded that a knowledge of the local stratigraphy, climate and geomorphological history at any site where an engineering structure is to be erected will indicate the specific nature of the engineering properties likely to be encountered, and that such knowledge may therefore be used to advantage in designing the most effective programme of site exploration. Where residual soils are concerned, a knowledge of the *origin* of the soil, in terms of its stratigraphic association, may lead to predictions regarding its engineering characteristics. On these grounds a case is made for the accurate identification and recording of the origin of soil horizons in the course of soil profiling, and also for the retention of stratigraphic identification of mapping units in the legends of soil engineering maps.

And finally, the reader must be reminded that the pioneer stratigraphers in both England and South Africa, William Smith (1769 - 1839) and Andrew Geddes Bain (1797 - 1864) respectively, were both Civil Engineers. It may be confidently concluded that they were among the first to recognise the significance of stratigraphy in engineering.

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