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SESSION 6 — GENERAL REPORT

FIELD EXPLORATION AND SAMPLING (GEOLOGICAL)

Reporter
D.H. BELL
Senior Lecturer in Engineering Geology, University of Canterbury, N.Z.

1 INTRODUCTION

This report reviews six of the papers presented in Session 10: four deal with lateritic soils associated with the Worsley Alumina Project (in south-western Western Australia), one is concerned with breakwater construction materials in Western Australia, and the other with seepage from a major settling pond at the Loy Yang Power Project in the Latrobe Valley, Victoria. The general theme for Session 10 is "Field Exploration and Sampling", with emphasis on geological aspects: in commenting on the material presented, I have considered also the extent to which the various authors have used an engineering geological approach to site investigation practice. The relevance of *engineering geology* to major geotechnical projects is, of course, widely accepted and documented (see, for example, Stapledon 1976; 1979): its relevance for all types and scales of engineering construction is less readily acknowledged although there have been many timely (?) reminders of the importance of "unforeseen" geological factors. It is my personal view that the engineering geologist occupies a key communication role in the design and construction of civil works: sound engineering geology is critical to the practice of rock and soil mechanics (Bell & Pettinga, in press).

2 ENGINEERING GEOLOGY DATA REQUIREMENTS

Site investigations for geotechnical projects are conventionally grouped into the following sequential stages (Clayton et al. 1982):

1) Pre-feasibility (or site reconnaissance).

2) Feasibility and site selection.

3) Design and specification of site works.

4) Construction, including any design (often "remedial") modifications.

5) Operation and long-term maintenance.

The conceptual approach to site investigation practice outlined by ISRM (1975), and elaborated by Stapledon (1979), stresses the correct formulation of project objectives, the development of an engineering geological site model, and its quantification by *geomechanics* testing methods (both field and laboratory). Specific *engineering geology data input* is required for hazard identification and evaluation, for the assessment of site foundations, and for the location of suitable construction materials: in particular, it is essential that major "geological constraints" to *site or project location* be identified by the conclusion of feasibility investigations, and that appropriate geotechnical decisions are therefore made prior to any commitment to construction.

Engineering geological data are obtained from existing construction records, air-photo interpretation, site-specific surface and subsurface investigations at various stages of design or construction, and in certain cases from longer-term monitoring or surveillance. *Presentation* of geological data usually involves some graphic means, such as plan or section compilation, in which mappable features or units are chosen to show relevant site information: this in turn requires the adoption of clearly defined and commonly accepted descriptive terminology for rocks and soils (Bell and Pettinga, in press). A further requirement is the clear separation of *"factual"* geological data (for example, drill core information presented in log format) from *"interpretations"* that involve a degree of geological inference concerning site conditions. Whether or not all such geological information is made available for contractual pruposes (see, for example, Stapledon 1983), engineering geological investigation methods nevertheless involve to varying degrees the *prediction* of the three-dimensional (or subsurface) distribution of both mass and material characteristics. The extent to which "unforeseen geological problems" (the so-called "latent conditions") arise during construction reflects in large measure the quality of engineering geology data input at the feasibility and/or design stages of investigation: although the *engineering geological site model* is at best semi-quantitative (Stapledon, 1979), it forms the critical basis for any *geomechanics evaluation* of the site.

3 RECENT NEW ZEALAND EXPERIENCES

3.1 Ruahihi and Wheao Canal Failures

Shortly before 2 pm on Sunday 20 September 1981 the eastern bank of the canal supplying water to the *Ruahihi Power Station*, which forms part of the Wairoa Hydro-Electric Scheme inland from Tauranga in the North Island of New Zealand, collapsed in an area just upstream from the penstock intake (Figs 1A & B). The escaping water rapidly eroded up to 600 m of the canal formation, and more than 1,000,000 m^3 of mud and rubble flooded down an adjacent valley and into the Wairoa River, causing damage to surrounding farmland and to State Highway 29. The *Ruahihi Canal* was constructed in Late Quaternary volcanic deposits that consist of ignimbrite materials (mostly non-welded pumiceous breccias and sandstones which have a pyroclastic flow origin) overlain by a variable thickness of weathered ash layers: many of these "engineering

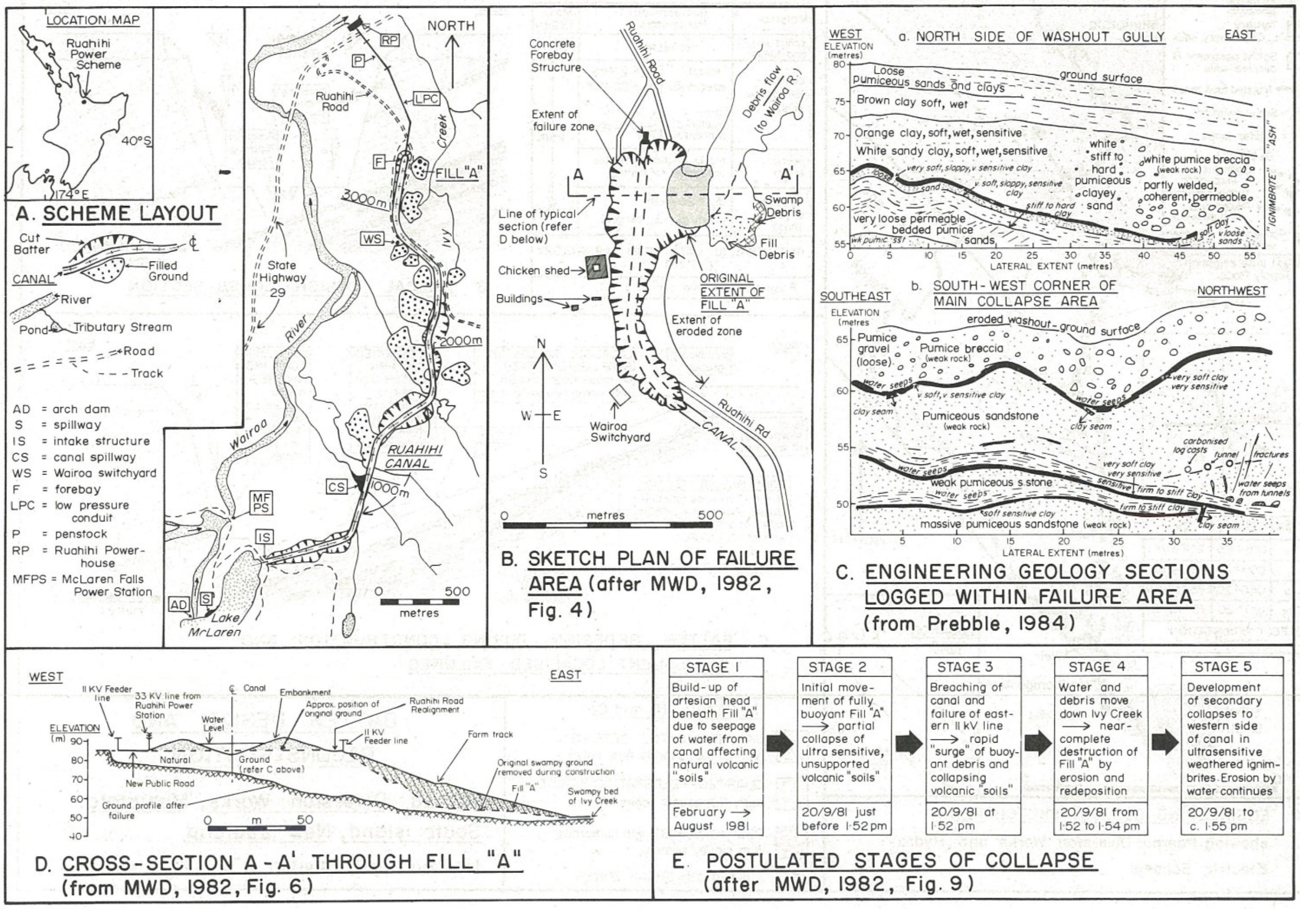

FIG 1: The Ruahihi Canal Failure of 20 September 1981

soils" are *highly sensitive, potentially erodible*, and the *site stratigraphy* is often complex (Fig 1C), whilst the ashes display unusual geotechnical properties because of their *allophane* content (Parton and Olsen, 1980). The effects of canal filling and operation were obviously significant in initiating failure by raising groundwater levels behind Fill "A" (Fig 1D), and by progressively weakening the in situ ignimbrite materials: the postulated stages of collapse (Fig 1E) indicate the very rapid nature of the "final" failure, and of its consequent downstream damage. The Committee of Inquiry recommended that prior to reinstatement of the Ruahihi Canal *"a thorough evaluation of the geological content of the work should be made ..... to define all major geological problems in nature and extent"* (Ministry of Works and Development, 1982 p10-12), as well as further detailed studies of the hydrogeology of the site and of any construction materials to be used.

At approximately 8 am on Thursday 30 December 1982 a breach occurred in the left bank of the Rangitaiki Canal adjacent to the penstock intake structure, which was located some 130 m above its powerhouse on the *Wheao River*. Most of the 220,000 m³ of water stored in the canal poured through the breach in less than 2 hours, carrying with it thousands of cubic metres of debris (large boulders and pumiceous materials) and causing extensive damage to the powerhouse superstructure and equipment, which was shortly to be commissioned: the escaping water and debris also caused a "flash" flood and high sediment loading in the Wheao River, which is one of many smaller rivers draining the Central Volcanic Plateau in the North Island of New Zealand. The *Wheao canal failure*, as it has come to be known, occurred in loose pumiceous sands and gravels (tephra deposits) overlying a welded ignimbrite rock mass that displays prominent vertical jointing: it is believed (Ministry of Works and Development, 1983) that failure resulted from initial seepage waters from the canal eroding a series of pipes (or tunnels) at the ignimbrite-tephra interface, and which in turn undermined the canal embankment until the water "supply" was such that the tunnel roof in the tephras burst open some 50 m from the canal, ejecting rock and soil debris into distributary channels. Whilst recognising some design deficiencies in the Wheao canal construction, the Committee of Inquiry also concluded that *"greater participation of specialist geotechnical expertise, especially in engineering geology, should have been enlisted by the principal consultants through both the preconstruction and construction phases"* (Ministry of Works and Development, 1983, p8-1).

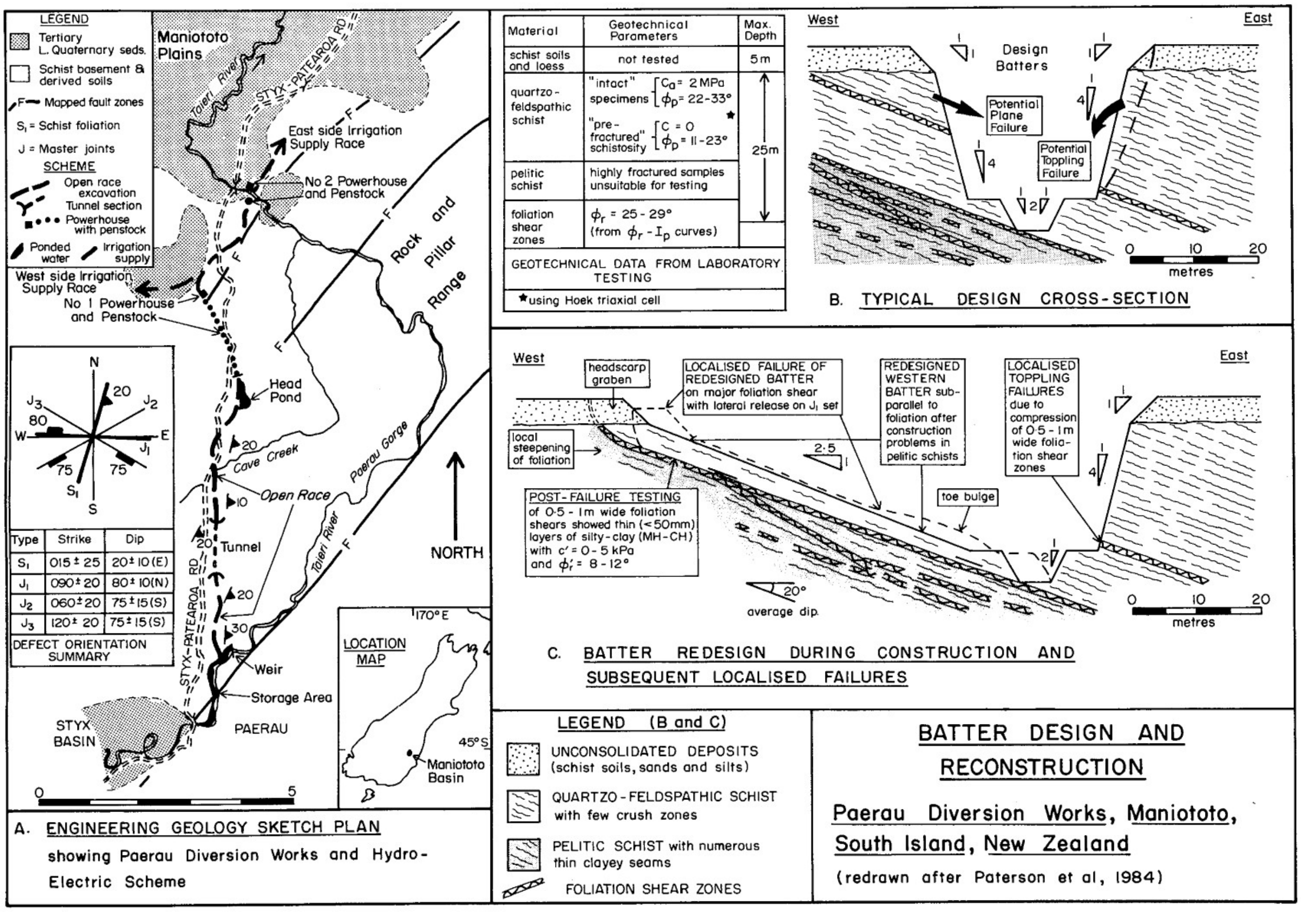

FIG 2: Batter Design and Reconstruction for the Paerau Diversion Works, Maniototo

Apart from the *economic cost* of the Ruahihi and Wheao canal failures (which amounted to some tens of millions of dollars for reinstatement), there are important *geotechnical considerations* that have been highlighted. In addition to the seismic and volcanic hazards that exist in an area of active (or ongoing) tectonism, the rock and soil materials display properties (such as sensitivity and erodibility) that demand considerable *geomechanics expertise* (Geological Society of New Zealand, 1983; Prebble, 1984). The inherent variability in material distribution and properties, coupled with complex groundwater conditions, requires a reassessment of conventional *site investigation methods:* for example, much greater use should be made of subsurface excavation techniques (shaft-sinking or trenching), and of engineering geological mapping and logging, to identify potential or existing geotechnical problems. The Ruahihi and Wheao canal failures suggest that geotechnical design practices, in what are clearly very difficult and complex materials, had exceeded the "state-of-the-art": much greater use of *geomechanics* (in its widest definition) is clearly needed in "active" volcanic areas such as the central North Island of New Zealand.

3.2 Maniototo Irrigation and Hydro Scheme

The Paerau Diversion Works are located in the schist terrain of Central Otago, and involve about 5 km of canals (to a maximum invert depth of 30 m) and a tunnel some 1.4 km in length: the scheme is designed primarily to provide irrigation water for the *Maniototo Plains*, but it also incorporates two small hydro-electric power stations (Fig 2A). Very significant cost increases have occurred during construction because the original *canal batter design* has been modified (Paterson et al., 1984): in essence, this has involved flattening of the western batter slope to correspond with the dip of the schist foliation along much of the canal length (Figs 2B & C). Batter stability problems have arisen principally because of the existence of *foliation shear zones* (subparallel to the schistosity) and the presence of low shear strength *pelitic schists* along the chosen alignment: it did not become apparent until an advanced stage of construction that the canal "corridor" essentially followed a major fault zone of low relief. In addition, the *geotechnical design parameters* adopted for the foliation shear materials proved inappropriate for the thicker (up to 1 m) zones: the presence of clayey silts with ϕ_r values of the order of 10°, even though often only a few millimetres wide at the margins of the shear zone, has resulted in renewed localised instability of even the redesigned batters.

The economic viability of the *Maniototo Irrigation Scheme* has been seriously affected by the escalation in construction costs. Whilst batter redesign has been a major factor in this, *geotechnical problems* (and consequential cost increases) have resulted from "squeezing ground" encountered during tunnelling, and also from the post-construction identification of potentially erodible loessial (wind-derived silty) soils beneath some fill areas. The adequacy of both the feasibility and the design investigations for the *Paerau Diversion Works* must therefore be questioned: whilst the "squeezing" problems associated with the thick foliation shear zones may not reasonably have been anticipated, it is also clear that limitations in materials testing for design purposes led to the adoption of inappropriate batter angles (Paterson et al., 1984). To simply dismiss the resultant *construction problems* as "unforeseen" geological conditions is to suggest that the documentation of widespread foliation-controlled landsliding in the schists of Central Otago (see, for example, Bell, 1976; 1982), and of the adverse geotechnical properties of foliation shear zones (see, for example, Cording and Mahar, 1974), is in fact not relevant to design investigation philosophy. Had the *alignment geology*, and its consequent *geotechnical* problems, been better appreciated at the conclusion of the feasibility investigations, then it is conceivable that the Scheme would not have proceeded at all: to reach that decision would, however, have required significant expenditure on *geomechanics* investigations, including possibly trial excavations along the canal alignment. With the benefits of *hindsight* it is of course much easier to critically analyse the investigation philosophy and the methods adopted, but equally the Paerau Diversion Works demonstrate very clearly the need for adequate *engineering geology data input* at all project stages from prefeasibility studies through to the completion of construction.

4. REVIEW OF CONFERENCE PAPERS

4.1 The Worsley Alumina Project

The Worsley Alumina Project is located in the Darling Ranges some 150-200 km south of Perth, Western Australia. A 51 km Cable Belt conveyor system links the open-pit mine at Boddington with the 5400 ha refinery site near Worsley, and the project area is characterised by deep *laterite weathering* of Precambrian igneous rocks, principally granites and dolerites. The *laterite profiles* differ as a function of both lithology and topography, and their engineering properties therefore vary laterally and vertically: the identification of site-specific laterite weathering profiles as a basis for geotechnical evaluation is stressed in each of the four papers (Gordon, 1984a & b; Gordon and Smith, 1984a & b) which deal with the Worsley project. The resultant data compilation is impressive, although there is a certain amount of repetition in the papers, presumably to make each "self-contained".

Gordon (1984a) discusses the laterite weathering profiles at the refinery site, and develops a number of important aspects of laterite genesis in the Worsley project area. He outlines the problems with *laterite terminology*, mentions some uncertainties regarding the *environment(s) of laterite formation*, and suggests a probable *origin* of the laterite profiles at Worsley during the warmer and wetter climates of the Late Tertiary. The paper is primarily concerned, however, with those aspects of *lateritisation* that have engineering relevance, specifically i) the recognition of *valley* and *plateau* weathering profiles; ii) the influence of parent rock *lithology*, with separate laterite sequences being identified for mica-rich granites, mica-poor granites, and dolerites; and iii) the variation in *engineering uses and properties* of the different parts of the laterite profiles. Gordon (1984a) also provides a very useful documentation of *mineralogical* and *textural* changes within the various laterite profiles: for example, in *granitic rocks* a general sequence can be identified from topsoil → ferruginous zone → mottled zone → pallid zone → zersatz (or decomposition) zone → weathered rock (MW/SW) → fresh rock. A final comment concerns the need for a term such as *"totally weathered"* to describe massive and pisolitic laterites: "traditional" weathering classifications (see, for example, Saunders and Fookes, 1970) assume a reduction in rock strength accompanying fabric alteration, but this is clearly inappropriate in *laterite terrain.*

Gordon (1984b) discusses the distribution and engineering influence of Precambrian igneous rocks at the Worsley refinery site, and this is in effect a companion paper to Gordon (1984a). It is clear that both the material and mass characteristics of the parent rocks will exert significant control over the laterite weathering processes, and this paper identifies those features (such as lithology and structure) which have been important at the refinery site. A new term *"scissure joints"* is introduced to describe fractures formed in kaolinitic clays beneath pisolitic laterite in forest-covered areas, where root penetration has influenced the fissure development. The paper concludes with a brief outline of some of the *engineering consequences* of the refinery site geology, and draws particular attention to the influence of differential weathering on both site investigation practices and reservoir watertightness: the Worsley project area no doubt abounds in geotechnical problems arising from complexities in bedrock geology coupled with the lateritisation process, and additional examples would certainly have been useful.

Gordon and Smith (1984a) develop some of the geotechnical characteristics of the lateritic soils, especially those of the *pallid zone.* They argue that the Worsley Plateau profiles differ from those of other lateritic soils in Western Australia because of greater degrees of mineralogical alteration: this they attribute to the area having the highest rainfalls and densest forest cover in Western Australia at the time of laterite formation. The dominant *pallid zone* is characterised by an open-voided structure formed of spherical aggregations of clay minerals (mostly kaolin) coated by iron and/or aluminium sesquioxides, which is invoked to explain its particular *engineering properties* that include i) Atterberg Limits plotting mostly below the A-Line (ML-MH) for both granite- and dolerite-derived soils, but with the latter displaying generally higher values for both plasticity index and liquid limit; ii) relatively low dry densities ($1.05 \rightarrow 1.75$ tm^{-3}) and moisture contents ($10 \rightarrow 60\%$); iii) a general (but not strong) trend toward lower peak shear strength and sensitivity with increasing depth, possibly accompanying a reduction in sesquioxide cementation; and iv) generally high shear strengths and low compressibilities. The paper concludes with a summary table relating engineering properties to geological structure, mineralogy and inferred origin: this is a most useful synthesis, but one that could have been better developed within the text itself.

Gordon and Smith (1984b) describe the investigation, design and excavation of a 30 m deep cutting on the Cable Belt conveyor route, where Plateau-type lateritic weathering of granitic rocks with associated dolerite dykes extended to depths up to 50 m. *Site investigation methods* included reconnaissance geological mapping, two seismic refraction traverses, and six boreholes in which thin-walled tube sampling and standard penetration testing were carried out. A *laboratory testing programme* determined Atterberg Limits, bulk density, pocket penetrometer values, triaxial strength characteristics, and falling head permeability: somewhat surprisingly, in view of its use at the refinery site, no Camkometer testing was carried out. For *batter design* purposes two soil models (termed A and B) were determined to describe, respectively, the *pallid zone silts and clays*, and the *lateritic "caprock"* overlying these materials along part of the proposed cutting: the adopted *design parameters* (ρ_d,c,ϕ) for the former model approximated the 35th percentile of measured (laboratory) soil strengths, whilst the "caprock" was assumed to have gravel properties. *Batter slopes* of 60° were adopted in the weathered granites and dolerites, corresponding to a factor of safety of about 1.5, whilst a maximum angle of 30° was used in the gravelly materials: 5 m wide benches at 8 m vertical separations were incorporated for drainage, access, and temporary debris collection. Soon after completion the cutting experienced the *100-year rainstorm* without significant damage, and the design of the excavation based on high undisturbed soil strengths clearly appears to be satisfactory: a detailed understanding of laterite weathering processes (such as sesquioxide cementation) gives much greater confidence to any geotchnical evaluation of this kind.

4.2 Loy Yang Settling Pond

The Loy Yang settling pond was constructed in highly plastic sandy and silty clays which overlie the Latrobe Valley Coal Measures, and borrow materials for embankment construction were mostly obtained from within the storage area. Dam surveillance monitoring of the completed structure indicated high uplift heads and significant seepage losses, which required further investigation and remedial measures. *Fletcher and Pedler (1984)* outline the history of construction of the $3.7M project, and detail subsequent remedial investigations which led to the installation of a relief well system.

From the authors' discussion it appears that *design investigations* identified the local presence of cohesionless fine sands at the unconformity between the coal measures and the overlying plastic clays: the settling pond itself was designed with a minimum 2 m cover of plastic clays to minimise seepage into the underlying sands. *Post-construction investigations* showed that two aquifer horizons were present, the upper one located about 2 m below natural ground surface and the lower immediately above the unconformity. The possibility of *embankment failure* either by uplift or by localised wedge movements was investigated, and *seepage control* by blanketing, slurry trenching or relief well construction was then considered. The subsequent installation of the cheapest option (a relief well system) has effectively controlled underseepage problems at the Loy Yang settling pond, but the original design investigations do not appear to have been adequate, at least in terms of *hydrogeology*.

4.3 Breakwater Construction

Mather (1984) discusses the availability of rock for breakwater construction in Western Australia, and particularly the requirements for *rubble-mound* structures. For *armouring* purposes blocks of high specific gravity igneous or metamorphic rocks up to 10 t are necessary: the bulk of the structure consists of lower classes of armour blocks, core and fill materials, which are often of low specific gravity (less than about 2.5). The author argues that extensive *geotechnical* investigations are usually necessary to locate suitable quarry sites containing essentially fresh rock meterials with widely-spaced (greater than 1 m) jointing: typically, engineering geological mapping, diamond drilling and trial blasting may be required to establish adequate quantities of rock *armour*. The Hopetoun Breakwater Project, in which suitable materials were not available within 40 km of the site, illustrates the interplay of geological and economic factors in design and construction. The paper in fact provides a very useful summary of *engineering geological requirements* for rubble-mound breakwater construction, not only in Western Australia: it does not, however, deal with possible cost-effective alternatives, such as *dolos* armouring.

5 SYNTHESIS

5.1 Laterite Geotechnology

The geotechnical data presented in the four papers dealing with the Worsley Alumina Project in south-western Western Australia adds significantly to the field of *"laterite geotechnology"*. The work of Gordon and Smith has demonstrated very clearly the need for sound geological site models when assessing *lateritic profiles*, and has also provided useful commentary on *testing methods* (both field and laboratory) that are appropriate for soils affected by remoulding or disturbance. Likewise, their site investigation practices and design philosophy, for example on the 30 m deep cutting on the Cable Belt route, should provide a sound basis for *future construction practices* in similar materials and terrain. Further research into the nature of *laterite weathering processes* is warranted to provide an even more thorough documentation of the mineralogical and textural changes that occur within these profiles: *electron microscopy* should prove to be a most useful technique in such studies, which must also incorporate a full assessment of engineering properties. A further extension of this work into the areas of *laterite genesis and classification* is also justified: there is certainly conflict in terminology and investigation techniques revealed by such authoritative sources as Gidigasu (1976).

5.2 Engineering Geology Data Input

Each of the six papers reviewed, as well as the three recent New Zealand "experiences" outlined, demonstrates the need for adequate engineering geology data input at appropriate stages of investigation. The Worsley Alumina Project highlights the relationship between *geological processes* and the resultant *engineering properties* of laterite soils: the problems at Wheao and Ruahihi also illustrate the geotechnical difficulties that may arise with complex volcanic soils in a tectonically active area. Both the Maniototo and Loy Yang projects suggest inadequate *engineering geological* data input at either the feasibility or design stages: in the latter case, however, it is not clear what *hydrogeological* investigations preceded design, and also the level of geotechnical (or engineering geological) control that was maintained during construction. The location of suitable rock for rubble-mound breakwaters is clearly an application of *engineering geological practice* using "conventional" investigation techniques.

5.3 Role of the Engineering Geologist

For most geotechnical projects it is preferable for the engineering geologist to form part of a *"geotechnical team"*: the interaction between the various areas of expertise is a necessary part of sound *geomechanics practice*. Likewise, it is important that the *engineering geologist* be in a position to influence *all* stages of an investigation programme, and that regular "review" meetings are held: as pointed out by Stapledon (1976; 1979), it is the clear definition of *objectives* that is critical to competent site investigation practices. The professional training and competence of the engineering geologist is also, of course, very important, but site and project experiences are in the longer-term an essential requirement.

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