STRUCTURAL FOUNDATIONS ON ROCK

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Structural Foundations on Rock

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Foreword

The papers in this volume were presented at the International Conference on Structural Foundations on Rock held in Sydney, Australia from the 7th to the 9th May 1980. The aim of the conference was to consolidate current experience and research regarding the design and construction of foundations on rock for structures such as large buildings, bridges and power stations, but excluding dam foundations. The papers come from twelve different countries and thus represent a reasonably broad expression of current practice regarding structural foundations on rock. This conference was arranged to coincide with the completion of two major research projects on rock socketed piles, conducted respectively at Monash University in Melbourne and at the University of Sydney. It was organised on behalf of the Australian Geomechanics Society under the sponsorship of the International Society for Rock Mechanics.

ACKNOWLEDGEMENTS

The Organising Committee wishes to acknowledge the considerable amount of work put into the organisation of the papers for this volume by Mrs M. Williams of the Institution of Engineers, Australia.

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1. Site investigation and rock properties

International Conference on Structural Foundations on Rock / Sydney / 7-9 May 1980

Verification of rock foundations for nuclear power plants

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1.0 INTRODUCTION

Extensive investigations have been performed to verify the suitability of rock foundations for a number of nuclear power plants in southern Europe. In order to confirm that the in situ rock over the whole of the foundation excavation corresponded to the conditions determined from the origianl subsurface investigations and used in the foundation designs, the following techniques were employed, in addition to visual observation:

- Geologic mapping,
- Shallow seismic refraction,
- Swiss Hammer measurements, and
- Shallow cross-hole seismic measurements.

To illustrate the application of these techniques in actual practice, a case history of the investigations performed and the results obtained at one of these sites is presented. It is concluded that visual inspection by experienced personnel is a most important factor in establishing the competency of a rock foundation. The various measurements provide quantitative documentation to substantiate the geologists's and engineer's opinion and serve to identify possible anomalies.

2.0 ROCK FOUNDATION VERIFICATION TECHNIQUES

The ability of a rock formation to support the load of a structure and the design of the structural foundation are often based on laboratory tests performed on rock cores obtained during the site subsurface investigations. The actual foundation excavation is then inspected by a geologist or an engineer to visually verify the competence of the rock. While this observation by experienced personnel (preferably, both a geologist and an engineer) is most important, the four methods listed above can be used in the following manner to provide quantitative evidence of the in situ conditions.

Detailed geological mapping of rock types and structural features such as faults, joints, bedding and folding is performed for all rock exposed in the excavation. The intentions of this detailed mapping are to show the physical continuity of the rock properties, measured by the three other techniques, over the whole foundation area and to ensure a thorough and systematic inspection of the entire excavation.

Seismic refraction measurements of the compression wave velocity (V_P) can be conducted using a hammer, or other impact device, as the source of the seismic waves. The time of arrival of the first impulse is received at varying distances from the source by a velocity transducer and the signal is recorded on a storage oscilloscope. The seismic lines are laid out in a radial or a rectangular pattern to cover the foundation area. The refraction measurements are used to ensure the complete removal of all weathered rock. The V_P's measured by the refraction survey may also be compared to the V_P's from the site investigation cross-hole or deep refraction surveys to ensure correlation with the foundation design parameters.

To provide further confirmation of the competence of the foundation rock, Swiss Hammer tests are performed on the exposed rock at numerous locations over the foundation area. The Swiss Hammer is a hand operated instrument used for the non-destructive determination of the unconfined compressive strength of concrete (USDIBR, 1966) or rock (Michalopoulos and Triandafilidis, 1976). The rebound, after impact on the surface being tested, of a spring-loaded steel plunger is recorded and

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is converted to compressive strength on the basis of an empirical relationship. At each test location, five Swiss Hammer readings are performed and, using the average value of the five readings, the compressive strength of the rock is determined from the conversion chart provided with the instrument. These Swiss Hammer measurements are compared to unconfined compression tests performed on rock specimens during the site investigation to verify the strength of the rock and the correspondence with the foundation design.

If the results of the three above investigative techniques indicate the existence of unusual conditions, such as a considerably harder or softer rock type or a large fault zone, over a significant portion of the foundation area, a shallow cross-hole seismic survey may be performed in the suspect area in order to evaluate the vertical extent of this condition. The procedure used for a shallow cross-hole survey (Stokoe and Woods, 1972 and Michalopoulos et al., 1979) consists of drilling a listening boring to the desired exploration depth and then advancing an adjacent impact boring as illustrated in Figure 1. At each depth where measurements are to be taken, the drill bit is advanced a short distance into the rock in the impact boring and a vertical velocity transducer is wedged against the wall of the adjacent listening boring at the same depth. The drilling rods attached to the drill bit are then struck with a hammer near the ground surface, thus triggering a storage oscilloscope and sending an impulse down through the drilling rods. This impact is transmitted to the subsurface material and body waves were generated in the rock. The arrivals of body waves are picked up by the velocity transducer in the adjacent listening boring and are displayed on the oscilloscope screen where a Polaroid photograph of the record is obtained. The measured shear and compression wave velocities can be used to evaluate the possible effects of the unanticipated conditions on the foundation design parameters.

If no unusual conditions are encountered, the first three techniques are generally sufficient to investigate the suitability of the foundation rock. The application of these methods at a soft rock site is discussed in the following section.

3.0 CASE HISTORY

The following case history for a nuclear

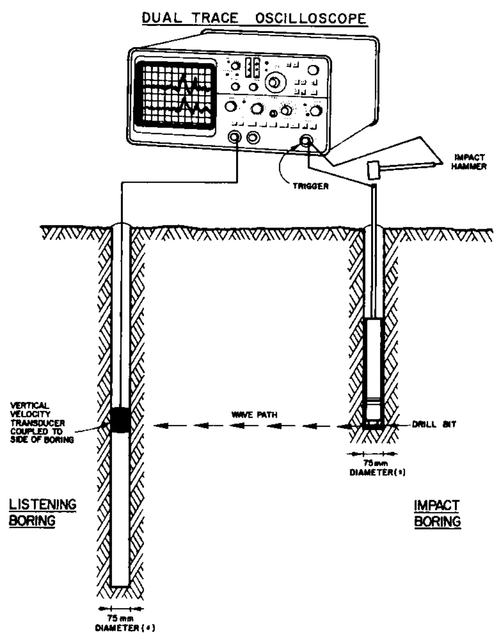

Figure 1: Schematic of Cross-Hole Seismic Survey

power plant founded on rock presents a summary of the site conditions as well as a description of the verification program performed at the site.

The site is located on a flat coastal plain within a structural basin filled with Tertiary-Quaternary sediments. It is located about 35 kilometers from a large extinct volcano which was last active about 50,000 years ago. The site exploratory borings and a 60 meter deep cross-hole identified four soil/rock formations:

- An approximately 12 meter thick layer of Upper Pleistocene sands, silts and clays with lenses and fragments of volcanic tuff, which indicate that volcanic activity occurred during deposition of these materials, overlying
- A seven to eight meter thick Middle to Upper Pleistocene, compact and highly fractured, volcanic tuff upon which the Reactor and Fuel Buildings will be founded. This tuff rests unconformably on
- An approximately three meter thick layer of Lower Pleistocene silty sands and sandy silts overlying
- A thick deposit of Lower Pleistocene silty clay. Deep borings in the site region indicate that this formation extends several hundreds of meters to bedrock.

The shear and compression wave velocities measured during the site investication cross-hole are presented in Figure 2.

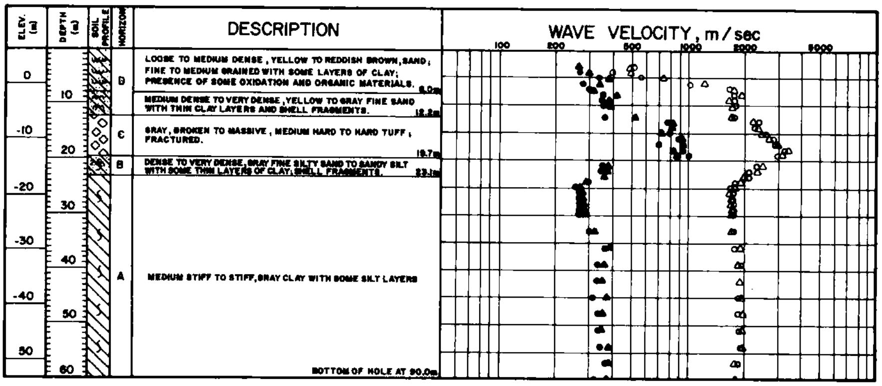

Figure 2: Measured Shear and Compression Wave Velocities from Site Investigation Cross-Hole

3.1 EXCAVATION

The site foundation conditions are somewhat unusual in that the main buildings will be supported on a relatively thin and soft layer of rock overlying deep soils. The excavation at the site was further complicated by a groundwater table located about five meters below the ground surface. Therefore, before any considerable excavation was performed, a reinforced concrete diaphragm wall extending into the tuff was installed around the excavation area. Five deep wells were constructed to remove water from the excavation and to control the piezometric pressures in the tuff and the underlying silty sands. As the excavation proceeded with conventional equipment, the diaphragm wall was anchored by tie-backs drilled and grouted into the tuff outside of the excavation.

The area of the excavation is about 3,600 square meters. In about one-half of the excavation only the soil and weathered tuff were removed while in the other half, about two additional meters of unweathered tuff was taken out due to structural considerations. The surface of the tuff was cleaned by mechanical and hydraulic action. This cleaning had to be repeated several times before the fill concrete could be placed, since rapid weathering of the exposed tuff produced a thin veneer of sandy clay on the tuff surface within a few days. The most important features observed in the detailed geological mapping of the excavation were a series of 40 extension fractures, all approximately parallel and dipping nearly vertically (Figure 3). Some of the fractures were open to a depth of at least one to two meters, while others contained various fillings such as loose silty sand,

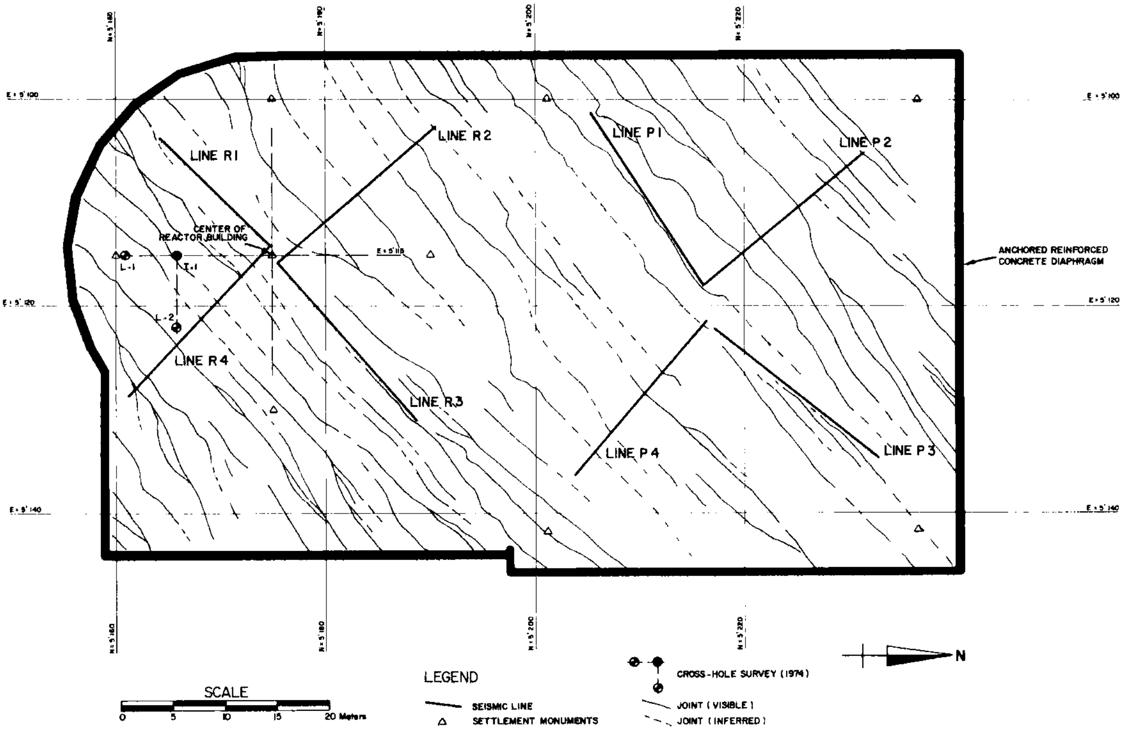

Figure 3: Geologic Map of Excavation

bentonite (from the construction of the diaphragm), grout (from injection of a grout curtain beneath the diaphragm), or filter sand (from installation of the dewatering wells). The fractures were spaced on two meters apart and were generally one to three centimeters in width, with the exception of one fracture near the center of the Reactor Building which was about ten centimeters wide.

The tuff was deposited as a volcanic mudflow and these fractures are joints formed by thermal contraction of the volcanic mudflow as it cooled. In a hot volcanic mudflow, cooling fractures commonly form perpendicular to the direction of the flow of the hot mud. This effect occurs because the flow tends to cool from the extremities of the flow inward toward the source of the flow. It is this differential cooling from the end of the flow toward the source area which produces extension joints perpendicular to the thermal gradient, i.e., perpendicular to the direction of the flow.

3.2 ROCK VERIFICATION

After apparent completion of the excavation and cleaning work, in situ compression wave velocity measurements of the rock were taken to confirm that all weathered rock had been removed. These seismic measurements were taken along eight 20-meter long lines as shown in Figure 3. Four of these lines were performed parallel to the fractures in the tuff while the other four lines were done normal to the direction of fracturing. The results of the measurements for the lines parallel to the fractures, indicate that the first arrival seismic waves were direct waves that were not refracted off a higher velocity layer, and that the exposed rock has a fairly uniform compression wave velocity of about 1,400 meters per second. For the seismic lines conducted perpendicular to the fractures which were either open or contained soil filling, the compression waves were often not recorded because of wave attenuation across the fractures and only higher amplitude waves, such as surface waves which yielded lower "apparent velocities", were recorded. The measured compression wave velocities are consistent with previous cross-hole seismic measurements performed in the Reactor Building area (Figure 2) taking into consideration the effects of the reduction in overburden stress on the wave velocity (Hardin and Drnevich, 1972).

To further confirm the competence of the foundation rock and to measure its in situ strength characteristics, Swiss Hammer testing was performed at 163 locations distributed relatively uniformly over the area of the tuff exposed in the excavation. The field strengths were somewhat higher than the unconfined compressive strengths measured in the laboratory using rock cores from exploratory borings. The average unconfined compressive strength determined from the Swiss Hammer readings is about 160 kilograms per square centimeter while the average strength of the rock cores is 105 kilograms per square centimeter. This difference may be due to weathering of the rock cores before testing.

Even though the vertical cracks in the tuff do represent an unusual condition, the performance of a cross-hole during the rock verification program was not warranted, since, in the original site investigation cross-hole (Figure 3), there were cracks between the impact boring and each of the two listening borings. Therefore, the foundation-structure interaction parameters (modeled as springs and dashpots and used to compute the dynamic response of the plant buildings under postulated earthquake loading conditions) determined from the cross-hole results, are representative of the actual subsurface conditions as exposed in the excavation. Geotechnical analyses for bearing capacity and settlement has conservatively considered the tuff as a gravel, i.e., completely cracked. Therefore, the rock verification program confirmed the adequacy of these analyses.

Since the tuff overlies a thick deposit of clay which could heave as a result of removal of 13 to 15 meters of overburden, settlement monuments (Figure 3) were installed on the tuff surface during the verification program and monitored by precise leveling. No horizontal or vertical movements were observed up to the time of placement of the fill concrete. Before placement of the fill concrete, the largest crack (ten centimeters in width) was injected with a cement grout.

3.3 SUMMARY OF CASE HISTORY

The conditions of the exposed tuff in the area of the Reactor and Fuel Buildings were assessed based on the performance of direct wave seismic measurements, Swiss Hammer testing and visual inspection. The exposed rock was competent and all loose or weathered rock was removed and replaced by fill concrete.

4.0 SUMMARY AND CONCLUSIONS

The case history of a rock foundation verification program for a nuclear power plant foundation has been presented. The various measurements and visual observations have verified that, despite the presence of vertical joints in the foundation rock, competent rock for building support was exposed in the excavation and that the foundation design parameters and analyses based on the original site investigation were appropriate for the actual foundation conditions.

Based on several such investigations, it is concluded that supervision of the final excavation and cleaning processes by an experienced team of an engineer and a geologist is the most important factor in correlating the in situ conditions with previous analytical assumptions. The geologic mapping and seismic and Swiss Hammer measurements provide substantiating documentation.

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The use of a high capacity pressuremeter for design of foundations in medium strength rock

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1. INTRODUCTION

Current design procedures for foundations to be constructed in rock rely heavily on the measurement of rock mass modulus and insitu shear strength. This is most logically carried out using insitu pressuremeter testing, in association with correlations with convenient laboratory tests. This paper outlines the development of the Coffey pressuremeter PMX-20, designed for insitu testing soft to medium strength rocks at pressures sufficiently high to initiate failure of these materials. The general operational characteristics of the PMX-20 pressuremeter are covered and discussion is presented regarding the estimation of modulus, insitu stresses and insitu shear strength. The relationship between corresponding laboratory and pressuremeter testing is also discussed, together with the application of pressuremeter testing for design of piled foundations.

2. THE COFFEY PMX-20 PRESSUREMETER

The high capacity pressuremeter developed by Coffey & Partners is similar in principle to the self boring pressuremeter developed by Hughes (1973) at Cambridge. It consists of a heavy steel cylinder onto which is clamped a flexible membrane which can be expanded using hydraulic fluid. The instrument construction is shown schematically in Fig. 1 and with associated surface equipment in Plate 1.

The instrument is designed for use in N sized boreholes (76.2 mm nominal diameter) and is capable of expansion to approximately 85 mm diameter. The expanding membrane is approximately 400 mm long, with measurement of the expansion being recorded electrically by four displacement transducers located around the centre of the

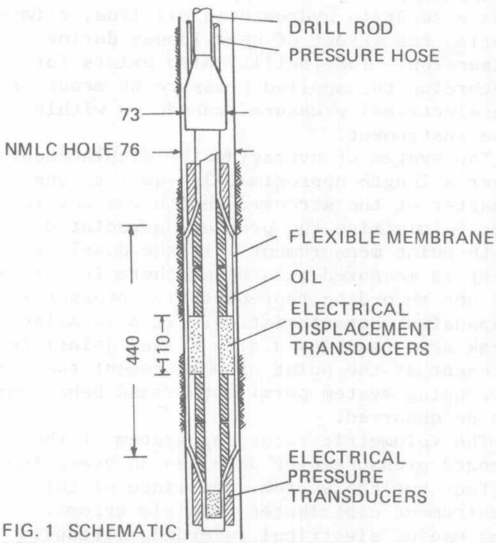

FIG. 1 SCHEMATIC DETAILS OF HIGH CAPACITY COFFEY PRESSUREMETER PMX–20

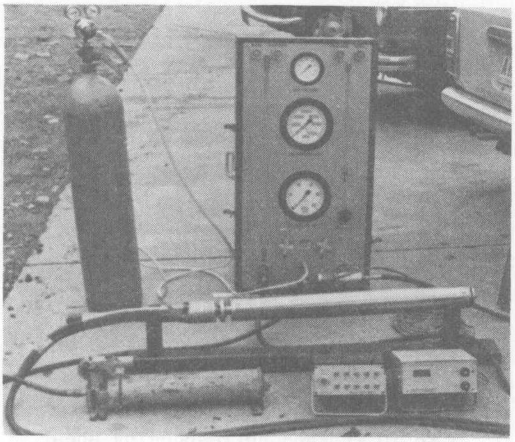

PLATE 1

instrument. Each transducer is recorded separately. The sensors are designed to measure the average displacement over a section 110 mm long and 10 mm wide at four positions 90° apart on the centreline. The separate recording facility allows a measure of the degree of anisotropy of the rock to be obtained as well as providing fourfold insurance against electronic malfunctioning of individual transducers during field operation.

Direct electronic measurement of radial displacement was adopted as being superior to the indirect measurement of volume changes of the pressure fluid as used in the Menard and other similar systems.

The applied pressure is provided by a combination of compressed gas and a fixed displacement hydraulic pump and is recorded via a separate hydrostatic oil line, eliminating the effect of head losses during expansion. The facility also exists for recording the applied pressure by means of an electrical pressure transducer within the instrument.

The system of averaging the displacement over a length approximately equal to one quarter of the stressed length was developed to minimise the problems associated with point measurement. If the displacement is measured at a point there is a risk of not recording representative pressure-expansion characteristics. If a localised weak area (such as a clay filled joint) is present at the point of measurement the averaging system permits the mass behaviour to be observed.

The volumetric recording system of the Menard pressuremeter also has an averaging effect but the system compliance of this instrument contributes possible errors. The use of electrical recording virtually eliminates any system compliance.

The PMX-20 pressuremeter has been specifically developed for high pressure operation in rock and its design provides particular advantages over other commercially available pressuremeters in respect of both ease and reliability of field operation and accuracy of interpretation.

* Direct electronic measurement of borehole displacements is without the errors and corrections inherent in indirect measuring systems which sense changes in volume of the pressure medium. Uncertainties related to undetected minor leakage of the pressure medium are completely overcome.
* High pressure capacity up to 20 MPa and with the capacity to measure shear strengths up to 8 MPa compared to pressure limits commonly in the range 5 to 10 MPa and corresponding shear strength limits of 2 to 4 MPa.
* Reliability due to the use of four separate displacement transducers.
* Virtually unlimited depth capacity in dry holes with a single purge system compared to problems encountered in operating liquid filled instruments in dry holes. The electronic readout system has been confirmed as suitable for depths of 200 m and could readily be extended beyond 1000 m with available technology.
* Use of relatively cheap reinforced duct-tubing as membrane. In over 800 tests carried out with two instruments over a 12 month period, membrane life has been in the order of 30-50 tests. Membrane replacement can be carried out in the field within 30 minutes.

3. OPERATION OF PMX-20 PRESSUREMETER

When performing a pressuremeter test with the PMX-20 it has been the practice of Coffey & Partners to apply small increments of pressure up to about 1 MPa, during which stage the membrane expands to the sides of the borehole and overcomes the effect of any borehole disturbance or stress relief. The subsequent increments of pressure are chosen by the operator to allow a well developed pressure-expansion curve to be established. Pressures as high as 20 MPa can be applied, but in some 800 or more tests carried out since the introduction of the instruments in 1978 the greatest applied pressure required to obtain sufficient information for interpretation has been 16.5 MPa. Frequently a reloading cycle is applied before significant plastic deformation has occurred and the final unloading cycle is also generally recorded.

The recording process adopted for each pressure increment is that the radial displacement (direct digital read-out) from each of "Channels" 1 to 4 is recorded, after which Channel 1 is again recorded when 30 seconds have elapsed from the first record of this channel at the given pressure. This allows a measure of the "creep" characteristics of the rock, which is useful for interpretation, both as a check on the test in progress and later for engineering analysis.

4. TYPICAL TEST RESULTS

As discussed above, the results of the individual displacement sensors are recorded and are then plotted for analysis. If the rock is perfectly homogeneous then the displacement-pressure curves in the "north-south" direction will be identical to those