SITE INVESTIGATION OF THE MAANSHAN NUCLEAR POWER PLANT IN TAIWAN

FRANKLIN C.C. FU, Taiwan Power Company

WIN-SHAN GUO, Moh and Associates

KUEN YU, Moh and Associates

Reprinted from

Proceedings, Sixth Southeast Asian Conference
on Soil Engineering, May 1980, Taipei

Vol. 2, pp. 355 - 369

Southeast Asian Society of Soil Engineering
Chinese Institute of Engineers
Chinese Institute of Civil & Hydraulic Engineering

6TH SOUTHEAST ASIAN CONFERENCE ON SOIL ENGINEERING
19-23 May, 1980, TAIPEI

SITE INVESTIGATION OF THE MAANSHAN NUCLEAR POWER PLANT IN TAIWAN

FRANK CHUNG-CHUN FU
Senior Geologist
Taiwan Power Company
R. O. C.

WIN-SHAN GUO
Senior Engineering Geologist
Moh and Associates
R. O. C.

KUEN YU
Associates
Moh and Associates
R. O. C.

SUMMARY The third Nuclear Power Plant of R.O.C. is located at Maanshan in the southern tip of the island of Taiwan. Previous geological investigation indicated the possible presence of a fault near the proposed location of the power plant. Since Taiwan is situated in the circum-pacific earthquake belt, site selection and design of foundations for the Nuclear Power Plant require extremely careful investigation.

The site investigation program for the Maanshan Nuclear Power Plant was performed in stages, which included drillings, sampling, exploratory trenches, test pits, geophysical survey, field instrumentation, and various types of laboratory tests. Results of the site investigation for power block area are presented in this paper.

INTRODUCTION

Site investigations for a Nuclear Power Plant are necessary to determine the geotechnical characteristics of a site that affect the design, performance and safety of the plant. The preliminary investigations of this project provide the information that is necessary for understanding of subsurface conditions and for identifying potential earthquake and other geologic hazards, especially the possible existence of faults in the proposed site. Detailed investigations are necessary to determine the local foundation and groundwater conditions as well as the geotechnical parameters needed for engineering analysis and design of foundations and earthworks. The detailed investigations for power block area was performed in two phases. Phase 1 was carried out during June and July of 1976 successfully identified an acceptable location for power block structures and yielded preliminary values for foundation's physical properties. Phase 2 was performed during the latter part of 1976, explored foundation conditions beneath the structures in detail and provided data from which specific foundation design criteria were developed.

SITE AND GEOLOGICAL CONDITIONS

The Maanshan site is located in the southern extremity of Taiwan, on Hengchun peninsula, it is just on shore along the northern side of Nan Wan (South Bay). It is situated in an area of sisal field and terraced rice paddies. The site is bounded on the east by highway 1 and on the south by the Nan Wan Bay. The site location is shown in Fig. 1

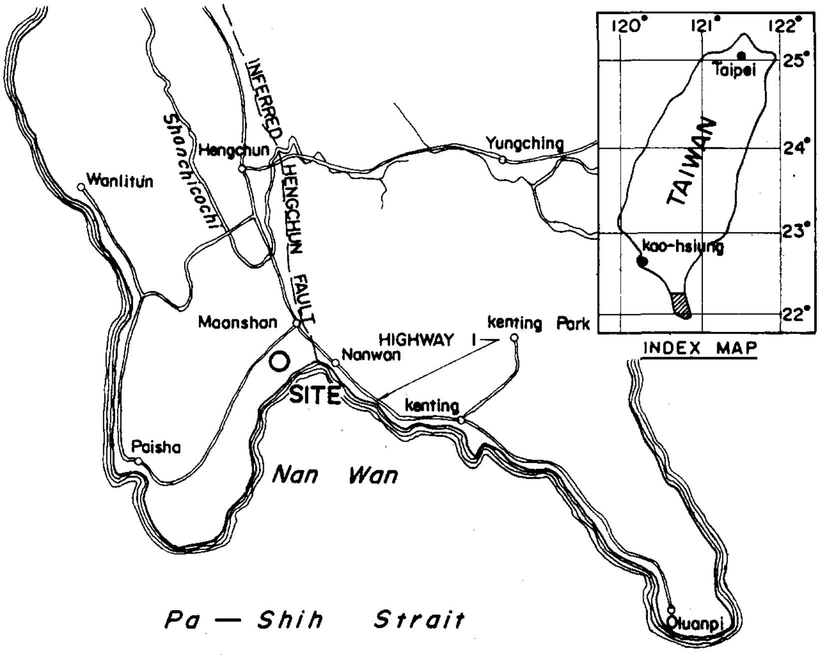

Fig. 1 Site Location of the Maanshan Nuclear Power Plant

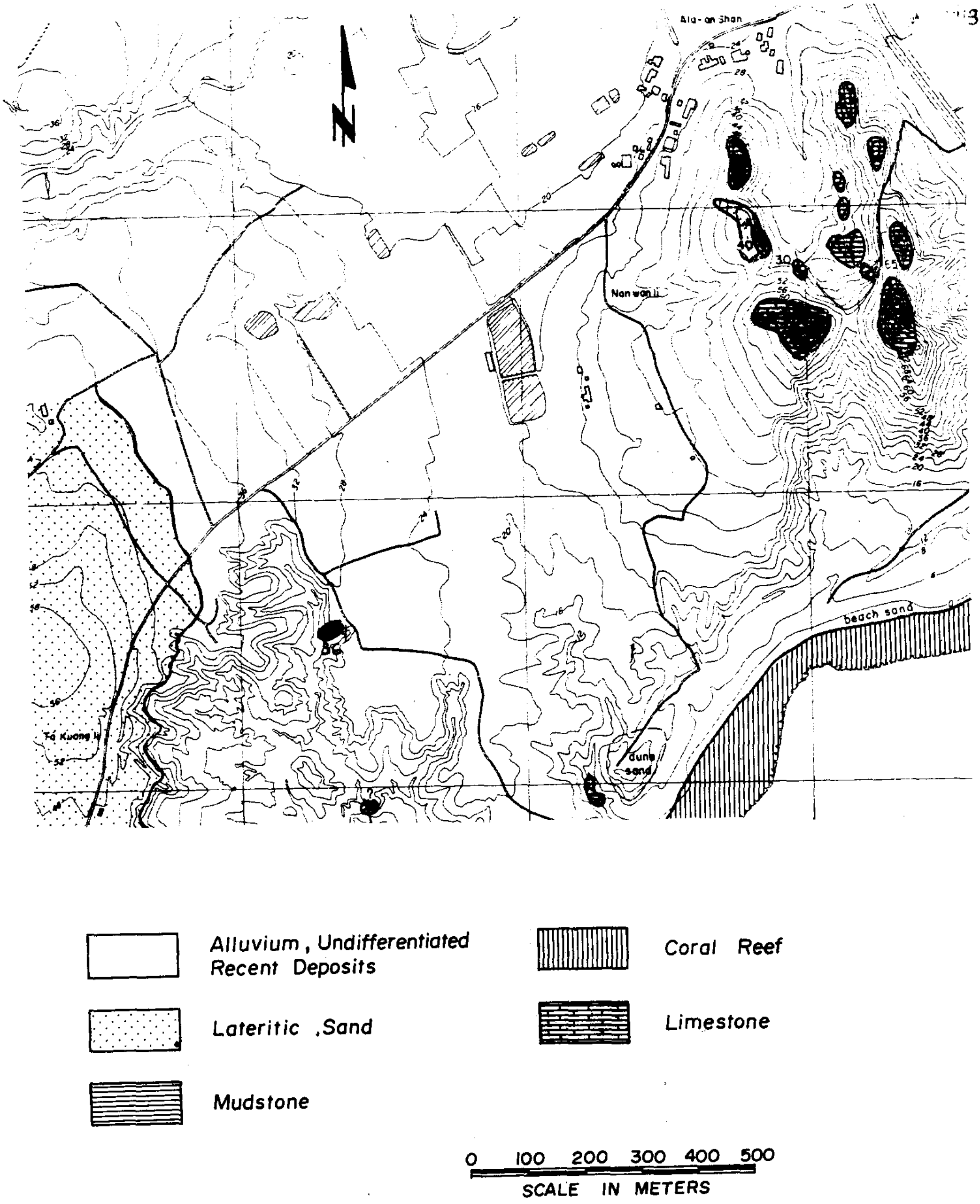

Fig. 2 Geological Map of Maanshan Nuclear Power Plant

The site covers an area of about 3 square kilometers. The power block area is centrally located in this site and occupies about 150,000 square meters.

The ground surface in this site varies from 5 to 70 meters above mean sea level. In the power block area, the ground surface varies slightly in elevation about from 20 to 30 meters above mean sea level.

The rock formation of the region surrounding the site is characterized by Miocene and Plio-Pleistocene marine sedimentary rocks.

The power block area is underlain by gray mudstone of Maanshan Formation covered by a layer of overburden (Fig. 2). The upper part of the mudstone is weathered. The overburden in the immediate power block area varies from 3 to 7 meters in thickness and consists of deposits of brown sandy clay with coral fragments. A layer of well rounded pebbles and cobbles is locally present at the base. Scattered large coral reef fragments are also present. The clay is generally well consolidated and stiff. Free groundwater is present in small amounts in the overburden.

All major power block structures will be founded in the mudstone beneath the weathered zone. The mudstone is a consolidated but uncemented sedimentary rock consisting primarily of clay and silt with small amounts of sand. The thickness of the mudstone has not been determined but the rock have been cored to a depth of over 200 meters in the power block area.

SITE INVESTIGATION

Site investigation for Maanshan Nuclear Power Station Units 1 and 2 was carried out in stages. Preliminary investigation was performed during the site selection process. After the Maanshan site was selected as the nuclear power station, detailed investigation works were performed in two phases.

Preliminary Site Investigation

Preliminary site investigation at Maanshan consisted of literature review, surface geological mapping, core hole drilling and seismic refraction survey. The purposes of this preliminary site investigation were to collect the existing informations on geology and seismicity as well as to identify the geological condition of the site.

Phase I Investigation

The purposes of the Phase I investigation were to find a location for the power block and to get preliminary physical properties of foundation. The Phase I investigation included the following items:

(a) Core Holes - A total of 29 vertical core holes were drilled during Phase I. Some of them were drilled for crosshole and uphole velocity measurements, downhole electrical and unclear logging and groundwater observation purposes. Only six of them were used for groundwater observation purposes.

The holes were NX diameter and were drilled to various depths up to about 61 meters, with deeper holes being used for crosshole velocity measurements.

(b) Exploratory Trenches — Three trenches were excavated in the power block area. The trenches varied in length from 50 meters to 215 meters, and were as deep as seven meters below ground surface. After completion of excavation, the trenches exposed the overburden and underlying mudstone.

(c) Downhole Electric and Nuclear Logging — Three types of geophysical logging were carried out; electrical resistivity, self-potential, and natural gamma radiation. The primary purpose of the electric and nuclear logging was to identify these natural electrical and gamma radiation properties of the mudstone, and also to show the absence of structural discontinuities within the rock.

(d) Crosshole and Downhole Velocity Measurements — Six of the drill holes were used for crosshole and downhole velocity measurements. The results were used to calculate dynamic elastic and shear modulus and poisson's ratio for mudstone.

(e) Groundwater Observation Holes — Six of the core holes were converted for groundwater observation purposes. Water level measurements were taken on a regular basis.

(f) Laboratory Testing — Selected core samples were shipped to laboratories for testing. The tests performed yielded preliminary data on the physical properties and behavior of the mudstone. X-Ray diffraction analysis were performed on selected samples.

Phase II Investigation

The field and laboratory works performed in the second phase provided the detailed information from which specific foundation design parameters were developed.

(a) Core Holes — A total of 28 core holes were drilled during Phase II. The holes were NX diameter and were either 40, 45, 65 or 80 meters deep depending upon locations. Samples were selected and waxed for determination of moisture content and degree of saturation.

(b) Test Pits and Block Sampling — Results of testing of core samples obtained during Phase I investigation indicated that drilling medium had a deleterious effect on the strength of the core samples. The Phase II investigation core drilling program was therefore supplemented with hand-excavated test pits. A total of four test pits were excavated, one in each of the containment areas (test pits No. 1 and No. 2), and one in each turbine pedestal area (test pits No. 3 and No. 4). The test pits, No. 1 and No. 2 were excavated down to elevation - 1 meter. The test pits, No. 3 and No. 4 were excavated to elevation - 5 meters. The upper portion of the excavation was carried out by the use of a backhoe. The dimensions of the pits were then reduced to approximately 2 m x 2 m and excavation was done manually. Wooden bracing was installed in all test pits and ventilation was provided by air compressors and hoses.

A total of 38 pairs of undisturbed block samples were taken from the four pits. At each sampling position, two block samples were taken. In pit No. 1 and pit No. 2, samples were taken from El. + 10 m to - 1 m at 1 meter intervals. In pit No. 3 and pit No. 4, the first set of samples was taken at El. + 10 m. From El. + 10 m to El. + 4 m, one set of samples was taken at every 2 meters intervals, whilst from El. + 4 m down to El. - 5 m, samples were taken at every 1 meter intervals. The size of each block sample was 20 cm wide x 45 cm long x 30 cm high. Since most samples contained fissure and fracture planes, extreme care had to be taken in order to minimize sample disturbance. The surrounding material of the block samples were carefully excavated and trimmed with an axe. Immediately after trimming, the block samples were waxed and wrapped with sponge on the four sides and supported with wooden planks. The wooden planks were held in position with an adjustable steel locking frame. The surrounding rock of the block samples were removed or trimmed with a sharp knife or thin-blade saw. In the meantime, geological logging of the four test pits was carried out as the test pit excavation proceeded. The orientation of beddings, joints and fractures were measured.

(c) Seismic Refraction Survey - Two seismic refraction profiles were established through the power block area. The two lines were paralled and measured 1200 meters in length each.

(d) Downhole Velocity Measurements - Eleven of the core holes were used for downhole velocity measurements to supplement those obtained in Phase I and provided detailed velocity data on the mudstone under structures. Another purpose was to explore in detail a velocity boundary at the east edge of the unit 1 containment area identified by the seismic refraction study.

(e) Field Testing - Testing of selected mudstone samples was performed in the field in order to reduce the effect of handling, transportation and moisture loss. These tests consisted of moisture content, unit weight, and unconfined compressive strength determination.

(f) Laboratory Testing - The laboratory testing program of the mudstone samples was designed to yield the basic physical properties and strength parameters required to develop the foundation design criteria for the various Seismic Category I and II structures.

Testing included index properties, unit weight, moisture content, and saturation determination. Static elastic properties and unconfined compressive strength measurements were made. Triaxial shear and consolidation tests were performed, and swelling and slaking tests were made. Both vertical and horizontal permeability tests were performed.

(g) Groundwater Observation Holes — Seven holes were drilled for groundwater observation purposes during Phase II. The holes were drilled to elevation - 20 meters and were not cored. Upon completion of casing and filter installation each hole was bailed dry. Water level readings were then started on a daily basis with periodic bailing of the holes and resumption of water level measurements after bailing.

(h) X-Ray Diffraction Studies — Selected mudstone samples were studied for X-Ray diffraction characteristics during Phase I and Phase II. The analyses were performed to determine the types and relative quantities of the various clay minerals present in the mudstone.

RESULTS AND DISCUSSIONS

Preliminary Site Investigation

From the result of preliminary site investigations of the proposed site, Hengchun feature was probably not a fault. Further, there was evidence to suggest that there is no major capable fault in the area. Observations of recovered cores indicated that the subsoils in Maanshan consisted of mainly mudstones or shaly mudstones. The seismic refraction surveys indicated that compression wave velocities in the mudstone vary generally between 1.00 km/s and 1.96 km/s.

Phase I Investigation

As in the preliminary site investigation, all core holes encountered the mudstone which underlies the site. The exploratory trenches also exposed the overburden and underlying mudstone. The mudstone in the trenches indicated that the rock generally improves in quality with distance from the shoreline. Groundwater seepage was evident along the contact of the overburden and mudstone in the exploratory trenches.

The downhole electric and nuclear logging did not reveal the presence of any discontinuities in the power block area. The crosshole and downhole velocity measurements indicated that the average P-wave velocity of the mudstone is 1.45 km/s and the average S-wave velocity is 0.56 km/s. Groundwater measurement in the observation holes indicated that all of the water entering these observation holes was coming from the overburden above the mudstone.

Phase II Investigation

As in the Phase I investigation, the core samples indicated that the mudstone underlies the site. The test pits also reveal the overburden and underlying mudstone in this site.

The seismic refraction survey results indicated the P-wave velocities varied from 0.85 km/s to 1.98 km/s. Velocities varied from 1.10 km/s to 1.40 km/s in the immediate power block area. The seismic refraction survey results also indicated the possible existence of a velocity boundary in the mudstone passing very close to the easterly edge of the unit 1 containment. At this boundary, P-wave velocities apparently increased abruptly from 1.34 km/s to 1.80 km/s. However, the downhole velocity measurement in angle hole indicated that no discrete velocity boundary exists in the mudstone. The apparent boundary in the refraction study probably is deposits of coral reef material at the base of the overburden. The results of downhole velocity measurements indicate a general P-wave velocity of 1.24 km/s under unit 1, and a velocity of 1.41 km/s under unit 2.

The field moisture contents of block or core samples of the mudstone varied within a rather narrow range from 12% to 17% and no apparent correlation between the moisture contents and depth of the samples. Since water was employed as drilling medium, it was found that the core samples obtained had been subjected to water contamination. A 2 to 3% higher moisture content was generally found as compared to that determined from the inner portion of a core immediately after sampling. Therefore, a precedure in obtaining moisture contents of core samples was adopted. First, the outside layer of a core samples, approximately 1 cm in thickness, was trimmed off and discarded. The trimmed samples was then divided into 2 portions, an inner core about 2 cm in diameter and an outer ring, about 2 cm thick. Moisture contents were determined for the inner core, outer ring as well as the whole specimen. Figure 3 shows the comparison of moisture contents of outer ring and inner core of core samples. Unit weight of core samples were about 2.2 t/m^3. Some of core samples measured had a degree of saturation near or above 95%. Unconfined compressive strength of samples which did not disrupted by fractures varied from 10 to over 16 kg/cm^2.

The general properties test results indicated that the block samples are quite uniform in their general properties, they are mainly silty clay or clayey silt in grain size and with low to medium plasticity. They are all located above the A-Line in the plasticity chart, as shown in Fig. 4 and belong to the CL group in the unified soil classification system.

The results of 38 unconfined compressive strength tests for block samples varied considerably, from less than 1 kg/cm^2 to over 25 kg/cm^2. In the field investigation, it was observed that majority of the mudstone samples contained fissures, fractures and weak planes. It was therefore decided that in order to obtain more representative samples, larger size samples, 10 cm diameters and 20 cm high, were used for the unconfined compression tests in this investigation. From the test results, it is apparent that many test specimens probably contain weak planes inside the specimens. A few of the specimens had rather high strength, more than 20 kg/cm^2, and they always failed at an axial strain more than 2% as compared to majority of the tests. The unconfined compressive strength of the mudstone is definitely controlled by the presence of fissures or other weak planes. The specimens would be quite differently if they were tested in confined condition such as that exist in situ. It is therefore concluded that unconfined compressive strength does not really represent the strength characteristics

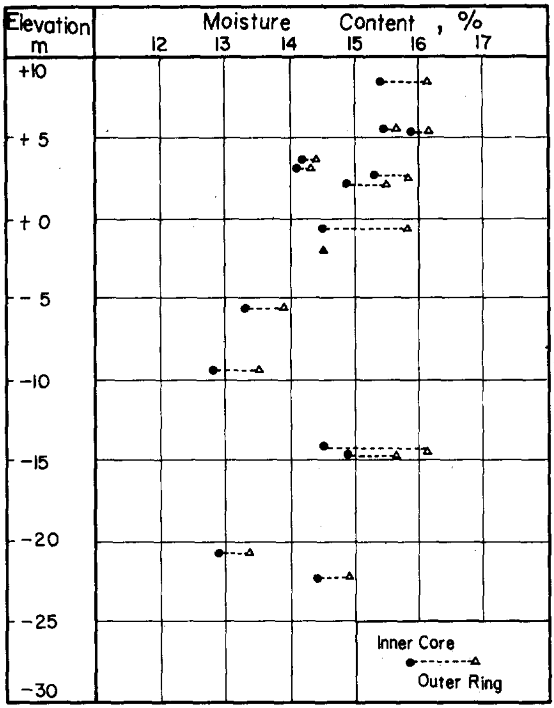

Fig. 3 Comparison of Field Moisture Contents of Outer Rings and Inner Cores of Core Samples (Core Hole UI - 10)

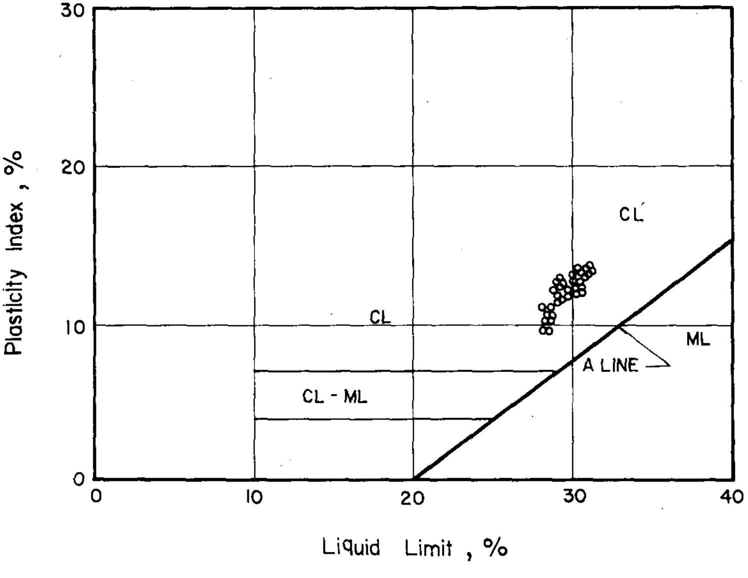

Fig. 4 Plasticity Chart

of the type of mudstone encountered at this site.

Twelve series triaxial compression tests were performed in this investigation. As explained in the previous section, majority of the mudstone samples contained fissures, cracks and other weak planes. It was rather difficult to draw a definite Mohr's envelope to enclose the four Mohr's circles in each test series. In order to get a better picture of the strength characteristics, the strength data were plotted in p-q diagrams shown in Figs. 5 and 6, where $p = \frac{1}{2}(\sigma_1 + \sigma_3)$ and $q = \frac{1}{2}(\sigma_1 - \sigma_3)$. The (p_f, q_f) Points in each of the plot fall within a rather narrow range. Best fitting lines by Least Square Method were draw through these points. The Mohr's strength parameters were then calculated from the a and α parameters from the following relationships:

$$a = c \cos\phi \text{ -------------------- } (1)$$

$$\tan\alpha = \sin\phi \text{ ---------------------- } (2)$$

The CU test gave a lower apparent cohesion intercept c and higher apparent angle of shearing resistance ϕ than the UU tests.

A total of 20 series of UU triaxial test was carried out to determine the elastic moduli and poisson's ratio. In each series, three tests were run on three individual samples. For each test, the elastic modulus was computed in three different ways, i.e. Secant modulus at 50% of the maximum strength, Tangent modulus at 50% of the maximum strength and average slope of the more or less straight line portion. Despite of the three different ways of calculating the elastic moduli, they did differ more than 10 - 15% in value. For all the samples tested, the poisson's ratio falls within a range from 0.24 to 0.38 and the average secant modulus varies from 437 to 942 kg/cm^2. In general, it would be expected that the larger the elastic modulus, the smaller should be the poisson's ratio. However, possibly due to the non-uniformity of the samples being tested, no consistent trend was found in these test results.

The consolidation tests results indicate that the mudstones at the site have quite low compressibility. Due to the low compressibility and also possibility that the pressure applied was not high enough, it was not possible to estimate the maximum past pressure according to normal procedure such as Casagrande's Method upon release of the consolidation pressure, several samples exhibited high swelling, the rebound curve even crossed over the compression curve.

One-dimensional swelling tests were performed under three conditions, i.e. swelling under foundation pressures (4 kg/cm^2 in the containment area and 1.5 kg/cm^2 for turbine area), free swelling and swelling pressure measurement. Majority of the swelling tests was concentrated on samples taken from location near the foundation grades. Table 1 presents a summary of the test results which indicate that the swelling pressure of the mudstones in both the containment and turbine areas are smaller than the anticipated foundation loads. However, swelling tests under the foundation loads did show a slight amount of swelling.

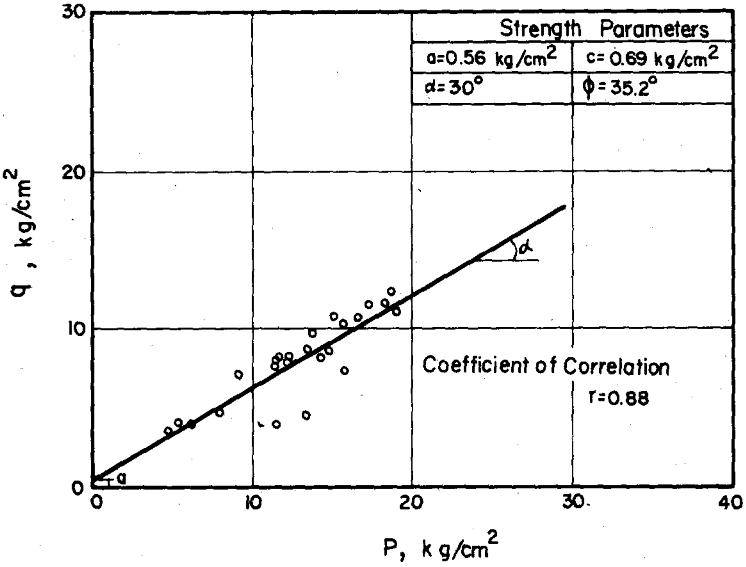

Fig. 5 Average Strength Envelope - CU Test

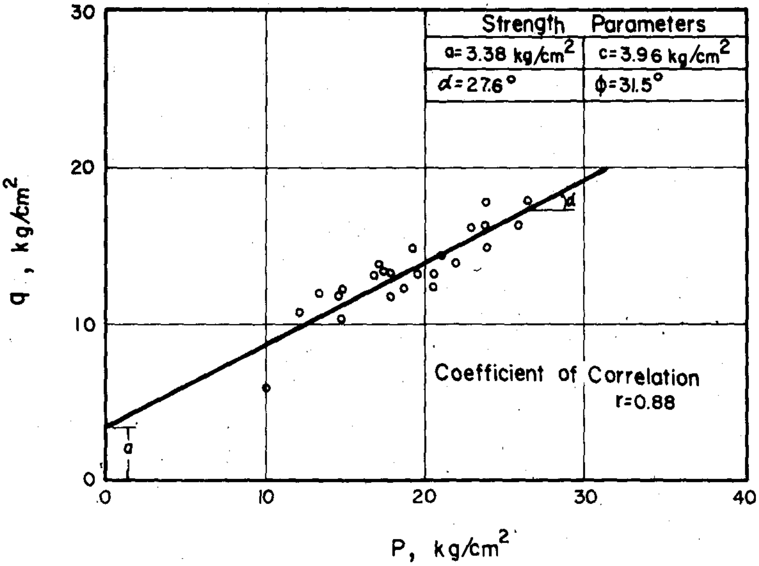

Fig. 6 Average Strength Envelope - UU Test

Two series of slaking tests were performed, the results are presented in Table 2. The tests were done in completely unconfined condition. It appears that surface cracking developed faster for thin specimen than thick specimens, upon contact with water. Complete slaking took more than 7 days. It is anticipated that surface cracking and softening would develop in excavated opening if water was allowed to accumulate.

Four series of permeability tests were conducted, each series consisted of one specimen cut vertically and a second cut horizontally. Tests results indicated that the horizontal permeability of the mudstone samples are generally higher than the vertical permeability. All permeability values are in the order of 10^{-8} to 10^{-9} cm/sec, the mudstones can be considered as impermeable for practical purpose. However, it should be noted that the permeability values along fissures, cracks, fractures and silt lenses of the mudstone are expected to be much higher.

Seven holes were drilled for groundwater observation purpose in this phase of investigation. The groundwater measurement results indicated that the water levels rose quite rapidly in all 7 holes. However, the hand excavated test pits presented the lack of seepage from mudstone. In general, it would be expected that the source of the water entering the holes is the overburden.

For X-Ray diffraction analysis, all samples had the same general composition, i.e. illite and chorite, quartz (greater than 20% each); feldspar (less than 10%); and a minor amount of an amorphous clay mineral (allophane) which is not detectable by X-Rays. Montmorillonite is present only in trace amounts.

CONCLUSIONS

(1) The site investigation at Maanshan was performed in stages. The investigation results indicated that the rock underlying the Maanshan site is competent to support a nuclear power plant.

(2) Previous studies revealed the possible existence of a fault near the Maanshan site. However, the later investigations confirmly indicated the absence of structural discontinuties in the area of Maanshan site.

(3) The subsurface of the Maanshan site is underlying by a layer of overburden. The upper part of the mudstone is weathered. The overburden in the immediate power block area varies from 3 to 7 meters in thickness. All the major power block structures are founded on the fresh mudstone beneath the weathered zone. Test pits did not encounter any free groundwater in the mudstone down to elevation - 5 meters. The rock can be readily excavated using conventional earthmoving machine or hand tools.

(4) Since water was employed as drilling medium, the core samples obtained had been subjected to water contamination. The core samples would have different physical properties and behavior such as they exist

TABLE 1 SUMMARY OF TEST RESULTS OF ONE-DIMENSIONAL SWELLING TESTS

Pit No.	Sample No.	Elevation, m	Swelling Condition	Average Swelling Amount, %	Average Swelling Pressure, kg/cm2
P-1	E+1A	+0.9 - +1.1	Swelling under Foundation Load	0.43	-
			Free Swelling	6.46	-
			Swelling Pressure Measurement	-	1.09
P-2	E+1A	+0.9 - +1.1	Swelling under Foundation Load	0.37	-
			Free Swelling	8.98	-
			Swelling Pressure Measurement	-	1.77
P-3	E-3A	-2.9 - -3.1	Swelling under Foundation Load	1.62	-
			Free Swelling	6.18	-
			Swelling Pressure Measurement	-	1.14
P-4	E-3A	-2.9 - -3.1	Swelling under Foundation Load	1.39	-
			Free Swelling	4.49	-
			Swelling Pressure Measurement	-	0.90

TABLE 2 SUMMARY OF TEST RESULTS OF SLAKING TESTS

Pit No.	Sample No.	Elevation, m	Specimen Size	Time Required for Initial Cracking, Hr.	Time Required for Slaking, Hr.
P-1	E+1A	+0.9 - +1.1	6.35 cm ϕ × 1.3 cm	2.5	over 7 days
			6.35 cm ϕ × 1.9 cm	2.5	over 7 days
			6.35 cm ϕ × 2.6 cm	3.5	over 7 days
			6.35 cm ϕ × 3.8 cm	2.5	over 7 days
P-3	E-3A	-2.9 - -3.1	6.35 cm ϕ × 2.0 cm	1.5	over 7 days
			6.35 cm ϕ × 2.0 cm	1.5	over 7 days
			6.35 cm ϕ × 3.9 cm	2.5	over 8 days
			6.35 cm ϕ × 3.9 cm	3.5	over 8 days

in situ.

(5) The mudstone has very low permeability and can be considered as impermeable for practical purpose. However, the permeability values along fissures, cracks and fractures of the mudstone are expected to be higher.

(6) It is anticipated that surface cracking and softening of the mudstone would develop in excavated openings if water was allowed to accumulate.

ACKNOWLEDGEMENT

The site investigation of the Maanshan Nuclear Power Plant in Taiwan was conducted by Taiwan Power Company, according to recommendations provided by Overseas Bechtel Inc. of U.S.A. Grateful appreciations are expressed to Dr. David S.L. Chu, the President of Taiwan Power Company, and Dr. Z.C. Moh, the President of Moh and Associates, for their encouragements and advices. Thanks are also to Taiwan Power Company, Overseas Bechtel Inc. of U.S.A., Sinotech Consulting Inc. and HCK Geophysical Company for making available informations presented in this paper.

REFERENCES

1. HCK Geophysical Company Ltd. (1974), Report on Seismic Refraction Survey Conducted in the Hengchun, Maanshan, and Kenting Area, Southern Taiwan, Republic of China, Submitted to Taiwan Power Company, Taiwan, R.O.C.

2. HCK Geophysical Company Ltd., (1976), Report on Seismic Survey Conducted in the Maanshan Area, Southern Taiwan, Report submitted to Taiwan Power Company, Taiwan, R.O.C.

3. HCK Geophysical Company Ltd., (1976), Report on Seismic Velocity Measurement Conducted at Maanshan Site, Southern Taiwan, Report submitted to Sinotech Engineering Consultants, Inc.

4. HCK Geophysical Company Ltd., (1976), Report on Seismic Survey, Refraction and Downhole Shooting, Report submitted to Sinotech Engineering Consultant, Inc.

5. Moh and Associates, (1976), Report on Maanshan Site Test Pits, Report submitted to Sinotech Engineering Consultants, Inc.

6. Moh and Associates, (1977), Report on Geological Logging of Exploration Shafts and Laboratory Testings, Report submitted to Sinotech Engineering Consultants, Inc.

7. Overseas Bechtel, Incorporated, (1976), Report of Hengchun Fault Investigation, Report submitted to Taiwan Power Company, Taiwan, R.O.C.

8. Sinotech Engineering Consultants, Inc., (1977), Report on Phase II Foundation Investigation, Report submitted to Taiwan Power Company, Taiwan, R.O.C.