

## PHOTOGRAPHIC FEATURE

Application of Mechanically-Stabilized Earth (MSE)  
by D.T. Bergado<sup>1</sup>

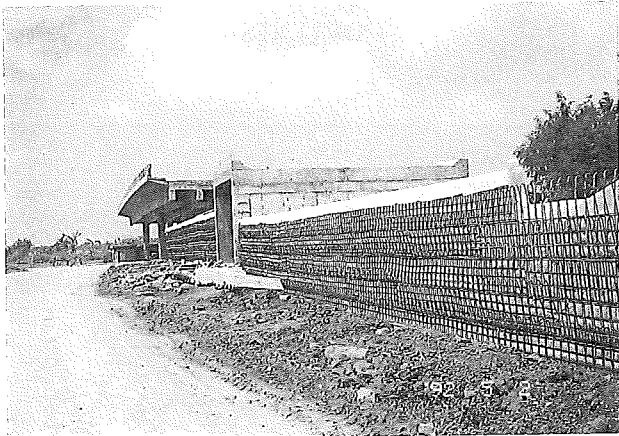


Fig. 1 MSE Construction at Bridge Approach

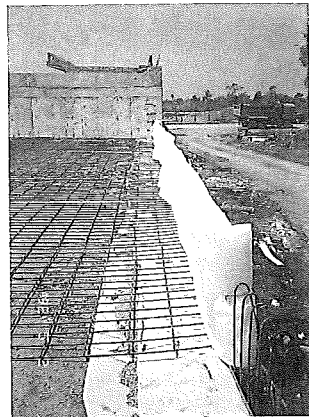


Fig. 2 Welded Steel Bar Reinforcements and Nonwoven Heat-Bonded Geotextile Filters

Mechanically-stabilized earth (MSE) consists of reinforcing the soil using polymer, steel or natural materials. The reinforcement which is strong in tension effectively combines with the soil which is strong in compression, forming a semi-rigid composite material. The reinforcing materials may consist of steel strips, steel grids, polymer grids, or geotextile sheets. The use of grid reinforcements has become necessary because grids have more pullout resistance than strips or sheets reinforcements. Consequently, cheaper, low-quality and cohesive-frictional backfill soils can be used. Welded steel bars were used initially in California, U.S.A. in 1970s. Figures 1 and 2 show an MSE construction at a bridge approach south of Manila, Philippines. In this construction, the reinforcements consist of welded 10 mm diameter steel bars forming a mesh with dimensions of 300 mm by 150 mm. Non-woven, heat-bonded geotextiles are utilized as filter materials to contain the backfill soil near the vertical face of the MSE wall. The vertical spacing of the reinforcements is 0.45 m.

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## SWELLING AND COMPRESSIBILITY CHARACTERISTICS OF COMPACTED SAND-BENTONITE MIXTURES

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### ABSTRACT

Compacted bentonite-sand soils are commonly used as lining, filling, and capping materials. Swell, compactability, and compressibility characteristics of two groups of compacted bentonite-sand mixtures are investigated. High and low quality bentonites are mixed with pure white silica sand. The percentages of the bentonites in the mixtures are: 10%, 15%, 20%, 25%, and 30%. Consistency limits of the mixtures increase linearly as the quantity of the bentonite increases; whereas, maximum dry density reaches an optimal value of 2 Mg/m<sup>3</sup> at 20% bentonite for the two groups. Maximum dry density decreases and optimal water content increases with further increase in bentonite percentage. Double oedometer swell tests are performed at 95% maximum dry density and at water content 3% less than the optimum water content. Results indicated that swell potential increases with bentonite percentage and decreases with valence of the interlayer cations. The study emphasises the important role of clay type and percentage, and soil fabric, on the swelling behavior of compacted bentonite-sand mixtures.

### INTRODUCTION

Compacted sand-bentonite mixtures are increasingly used in many geotechnical and geo-environmental applications such as landfills liners, covers, and vertical barrier walls. Bentonite and sand-bentonite mixtures are also used as barrier for disposal of radioactive waste (Pusch, 1998; and Pusch, 1995). The sand-bentonite supports the waste container, transmit waste heat to the surrounding environment, form an advective barrier to groundwater and a diffusive to movement of radionuclids (Tang, et al., 1998). In sand-bentonite, the bentonite is present in pores between the sand granules and coats the granules with a uniform, very thin layer of clay (Cancelli, et al., 1994; and Pusch, et al., 1990). Pusch et al. (1990) showed that, on exposure to water, the voids between the aggregates become filled by clay gels emanating from the expanded aggregates. The geometrical shape and variation in aperture of voids that create major passages are strongly dependent on the density and pore water chemistry. Borgesson (1990) indicated that, the hydraulic properties of the clays are strongly related to the compression and swelling properties. The high swelling up on wetting of bentonite enables the mixtures to self-heal and seal cracks developed when the bentonite dried and maintains low permeability (Shan and Daniel, 1991; and Boardnan and Daniel, 1996). A field hydraulic conductivity ( $k$ ) of  $1 \times 10^{-9}$  m/s is the maximum typically allowed by EPA regulations in the U.S.A. Control tests on numerous construction sites provides values of  $k \leq 10^{-10}$  m/s to  $10^{-11}$  m/s for sand-bentonite mixtures with 4% to 12% bentonite (Brand, 1992; Cancelli, et al., 1994; Haug and Wong, 1992; Reschke and Haug, 1991; and Evans and Quigley, 1992).

Several investigators studied swelling and engineering properties of compacted soils. Seed, et al. (1954) studied the role of dry density and water-content values on stability and swell of soils. Daniel and Benson (1990) investigated water-content-density criteria for compacted soil liners. Holtz and Gibbs (1956) investigated the volume change control through compaction. They showed that, placement moisture content and density and the method of compaction utilized may all have significant effect on the amount and rate of swelling that develop upon subsequent wetting. Day (1994) studied the swell-shrinkage behavior of compacted expansive clay subjected to cyclic wetting and drying. He found that ageing of curing cycles can reduce swelling and bond can be developed with time in compacted clay which reduces the amount of initial swell. Faure (1974) studied the evolution of compaction curves versus water content with variation of clay content, clay mineralogy, sand content, and compactive effort by using kaolinite and montmorillonite mixed separately with sand in different proportions. He showed that each curve had a compaction sensitivity threshold (CST) and he also reinforced the hypothesis that when the water content was lower than the CST, the clay still did not have its lubricating action. Daniel and Wu (1993) investigated compacted clayey sand liners and covers for arid sites. Soil samples from west Texas were compacted and tested for hydraulic conductivity,

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strength, and volume changes. An overall acceptable zone was given in the dry density and molding water content space. Stoicescu, et al. (1998) showed that a small addition of bentonite to a sandy soil caused a significant increase in the storage potential of the mixture. Faure and Mata (1994) investigated the penetration resistance along compaction curves and concluded that the maximum dry density of compaction curves had no effect on penetration resistance values.

The bentonite type and percentage, placing water content, and degree of compaction should be optimized to obtain the desired dry density, permeability, deformation modulus, adsorption capacity, swell potential and swell pressure. This study evaluates the effect of bentonite/sand contents and bentonite type on the compactability, swell pressure, swell potential, and compressibility of compacted sand-bentonite mixtures. Tests performed include consistency limits, compaction, and double oedometer tests.

## MATERIALS

### Bentonites

Two powdered bentonites, designated high quality bentonite (HQB) and low quality bentonite (LQB), were used in this investigation. The bentonites were obtained from a local supplier in eastern Saudi Arabia. Figure 1 shows the grain size distribution curves. Information on the engineering properties of the two bentonites used in this investigation is summarized in Table 1. X-ray diffraction patterns (XRD) of the clay fractions and chemical analysis of the two bentonites are shown by Alawaji and Alwail (1996). Properties of these bentonites indicate that they are very highly expansive materials.

### Sand

The sand used in this investigation was obtained from a quarry 80 km east of Riyadh, the capital of Saudi Arabia. It is a uniform, medium, white sand and classified as SP according to the Unified Soil Classification System (USCS). Grain-size data of the sand (Fig. 1) indicate a mean diameter  $D_{50} = 0.5$  mm, a coefficient of uniformity  $C_u = 3$ , a coefficient of curvature  $C_c = 1.73$  and an effective diameter  $D_{10} = 0.28$  mm.

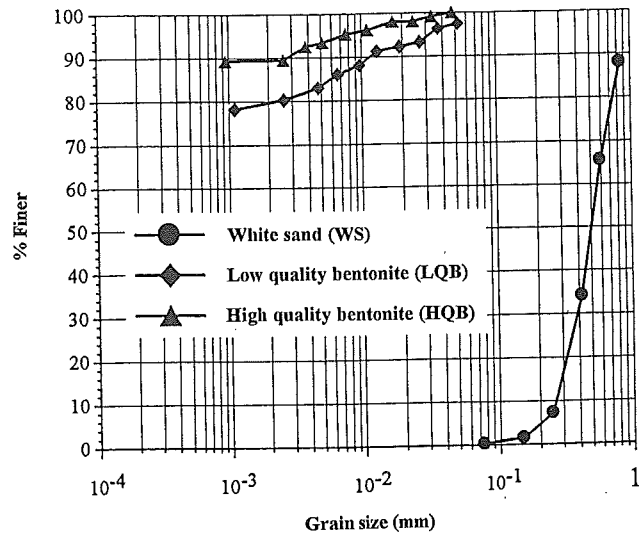


Fig. 1 Grain Size Distribution of the Sand and Bentonites

Table 1 Engineering Properties of Bentonites

Properties	Bentonite Type	
	LQB	HQB
Silt (%)	22	11
Clay (%)	78	89
Liquid limit, $LL$ (%)	316	505
Plasticity index, $PI$ (%)	254	459
Specific Gravity, $G_s$	2.61	2.68

## EXPERIMENTAL PROCEDURE

### Compaction Tests

The moisture density relationships of HQB and LQB-white sand mixture were determined using modified Proctor compaction test (ASTM D-1557). Compaction tests were carried out on both LQB and HQB mixtures with 10%, 15%, 20%, 25%, and 30% bentonite (by dry weight).

### Oedometer Test

Oedometer tests were performed using Bishop type, fixed ring, rear loading, lever arm bench model (Wykeham-Farrance Model WF 2400). Stainless steel consolidation rings with diameter of 70 mm and height of 19 mm were employed. Axial displacements were measured with dial gages of 0.002 mm precision. Loading procedure consisted of two stages. First, a seating pressure of 7 kPa was applied, then specimen was loaded dry up to 25 kPa pressure. At equilibrium, the specimen was submerged in distilled water under constant pressure. The specimen was allowed to swell. After the specimen had swelled and the swell deformation ceased, incremental loading of 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa, 1600 kPa was applied in the second stage. Each load increment was maintained for at least 24 hours until the deformation ceased.

### Sample Preparation

High and low quality bentonites were mixed with pure white silica sand. The percentages of the bentonites in the mixtures were: 10%, 15%, 20%, 25%, and 30%. To prepare oedometer test specimens, bentonite was added to the dry sand and initially mixed with a spoon. This was followed by adding distilled water in several stages using a spray bottle and mixing with a spoon until the desired water content was reached. Then the soils were mixed carefully by hands until reaching a uniform mixture. The mixtures were placed in plastic bags and sealed to avoid loss of water and allowed to cure in a dessicator at room temperature for 24 hours. Specimens were prepared by statically compacting the soil directly in the confining ring at 95% maximum dry density and at moisture content 3% less than optimum moisture content, according to the modified Proctor compaction tests. Table 2 shows the initial conditions for the double oedometer testing program. It should be emphasized that mixtures placed at these initial conditions were expected to have a very high swell potential according to recently published charts and empirical equations (e.g., El-Sohby, 1994; and Alawaji, 1997).

## RESULTS AND DISCUSSION

### Atterberg Limits

The variation of liquid limit (LL) and plasticity index (PI) of HQB and LQB mixtures with different percentages of bentonite are shown in Fig. 2. The results indicate that LL and PI values decrease with increase in sand content. As shown in Fig. 2, when the HQB percentage exceeds 13% and the LQB exceeds 16%, the LL values are greater than 50% and the PI values are greater than 20%, which accounts for potential of high swell.

Table 2 Dry Density and Initial Water Content of Bentonite-Sand Specimens

Bentonite (%)	Low Quality Bentonite		High Quality Bentonite	
	$\gamma_d$ (Mg/m <sup>3</sup> )	w (%)	$\gamma_d$ (Mg/m <sup>3</sup> )	w (%)
10	1.829	7.25	1.834	7.3
15	1.872	7.25	1.897	7.1
20	1.888	7.85	1.929	7.4
25	1.869	8.5	1.925	7.5
30	1.842	9.25	1.885	7.9

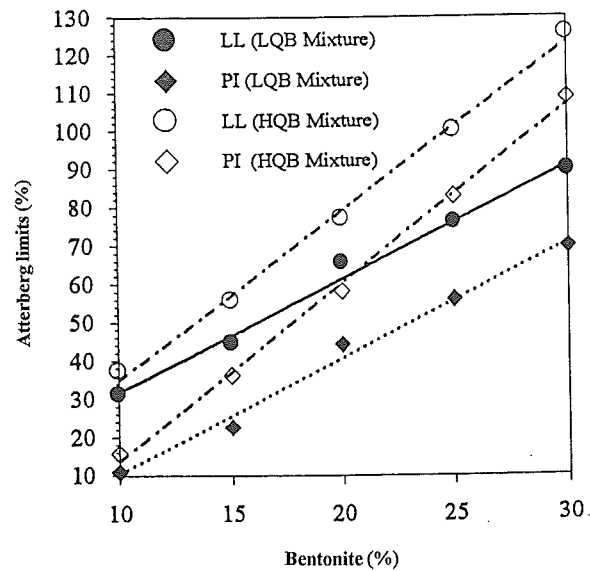


Fig. 2 Variation of Consistency Limits with Bentonite Percentage for HQB and LQB Mixtures

### Compactability

Sand was mixed thoroughly with both high and low quality bentonites at the following percentages: 10%, 15%, 20%, 25%, and 30%. Modified Proctor compaction test was performed in order to determine the compaction characteristics of soil mixtures. Compaction curves of the low and high quality bentonite mixtures are depicted in Fig. 3 and Fig. 4, respectively. Table 3 shows the maximum dry density (MDD) and the optimum moisture content (OMC) for various mixtures. The following observations can be inferred from the compaction curves:

1. For the 10%, 15%, and 20% bentonite mixtures, both MDD and OMC increase with percentage of bentonite.
2. For 20%, 25%, and 30% bentonite mixtures, MDD decreases and OMC increases with percentage of bentonite, which is consistent with the general trend shown by Henderson and Lisle (1988), Faure and Mata (1994), and Alawaji (1997).
3. The optimal MDD is for 20% bentonites, and the compaction curves shifts to the right as the quantity of bentonite increases.
4. For the same percentage of bentonite, high quality bentonite possesses higher MDD and lower optimum water contents than those for low quality bentonite. Also, slopes of the compaction curves of high quality mixtures are steeper than that of low quality bentonite mixtures.

The 20% bentonite can be considered at a threshold percentage at which there is an alteration in the clay structure present in the mixtures. Clays in mixtures with 10%, 15%, and 20% bentonites have no definite structure and mainly act as lubricant and binding agent between sand particles leading to the increase in MDD. When the percentage of the bentonite becomes larger (25% and 30%), the swelling of the clays becomes more dominant. The clays begin to fill the voids between sand grains leading to the elimination of sand particle contacts in a given volume and further reduction in pore air space, and results in decrease MDD. The steeper compaction curves of high quality mixtures may be attributed to the greater adhesion and swelling of high quality bentonite.

Table 3 MDD and OMC of Various Sand-Bentonite Mixtures

Bentonite (%)	High Quality Bentonite		Low Quality Bentonite	
	MDD (Mg/m <sup>3</sup> )	OMC (%)	MDD (Mg/m <sup>3</sup> )	OMC (%)
10	1.930	9.95	1.925	10.25
15	1.998	10.1	1.970	10.25
20	2.030	9.95	1.988	10.85
25	2.026	10.5	1.968	11.50
30	1.985	11.00	1.939	12.25

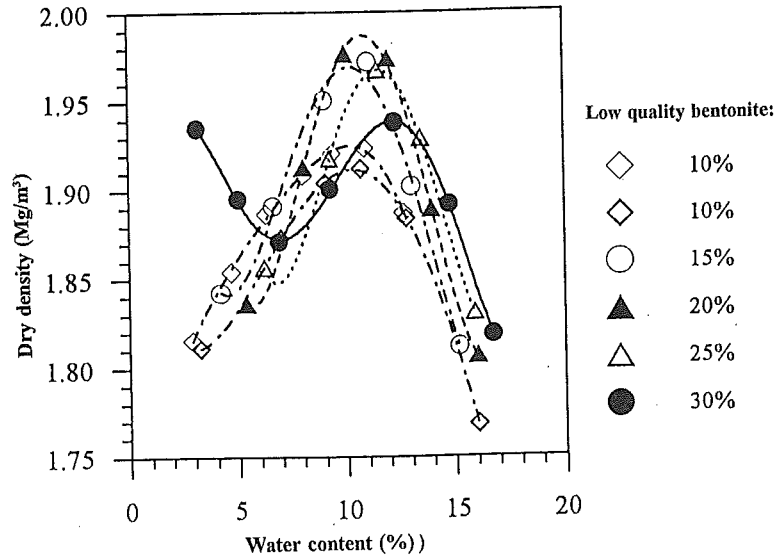


Fig. 3 Variation of Dry Density with the Water Content for LQB Mixtures

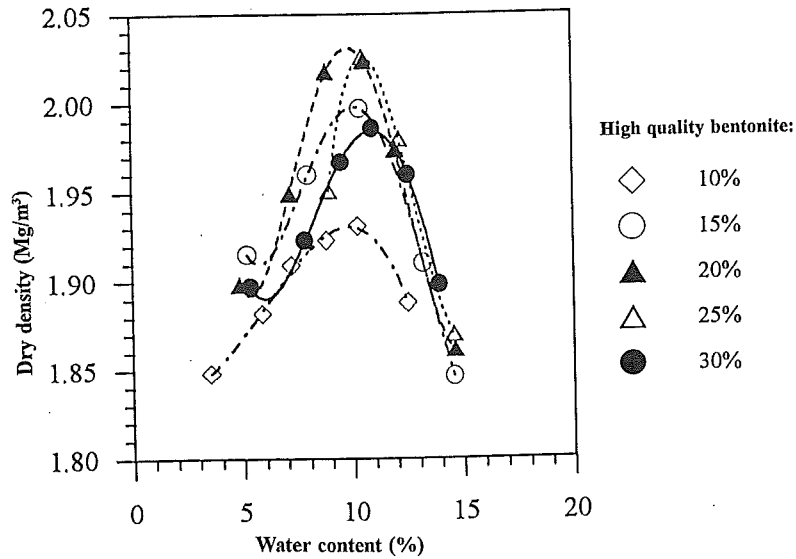


Fig. 4 Variation of Dry Density with the Water Content for HQB Mixtures

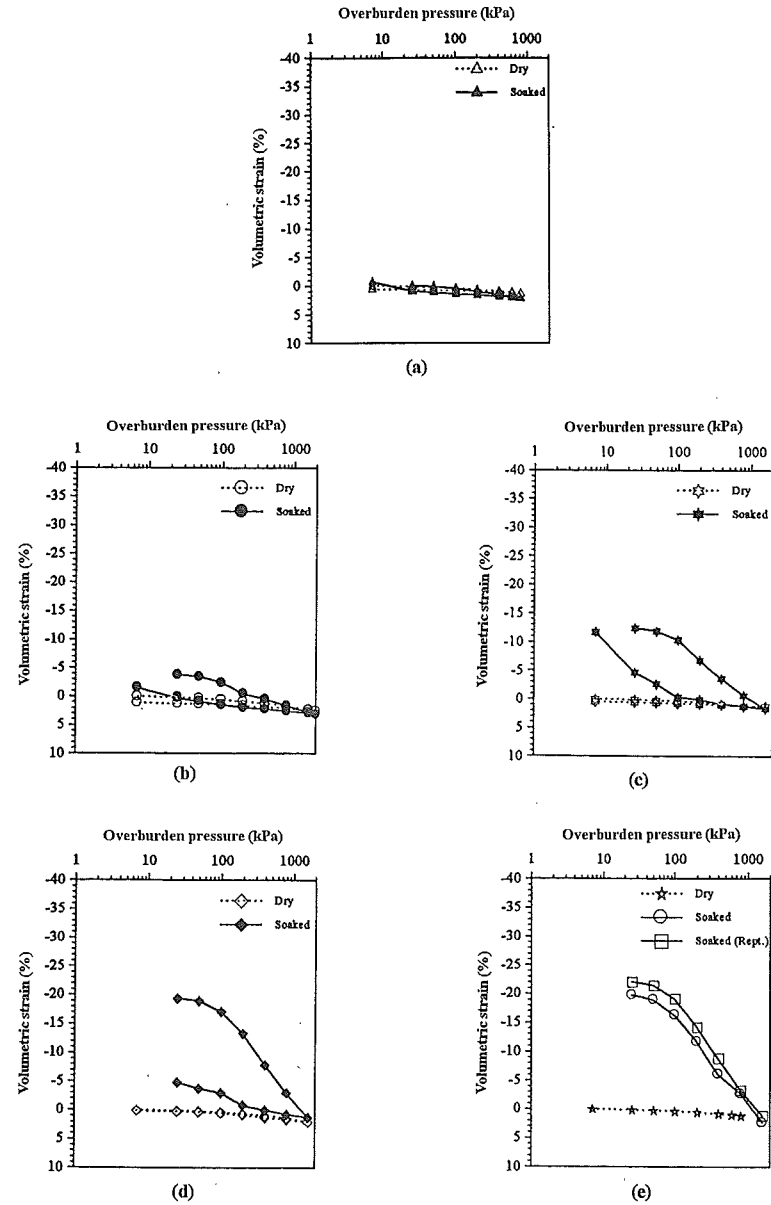


Fig. 5 Double Oedometer Test Results for Sand-Low-Quality-Bentonite Soils Compacted at 95% MDD and Moisture Content of 3% less than OMC: (a) 10% Bentonite, (b) 15% Bentonite, (c) 20% Bentonite, (d) 25% Bentonite, and (e) 30% Bentonite

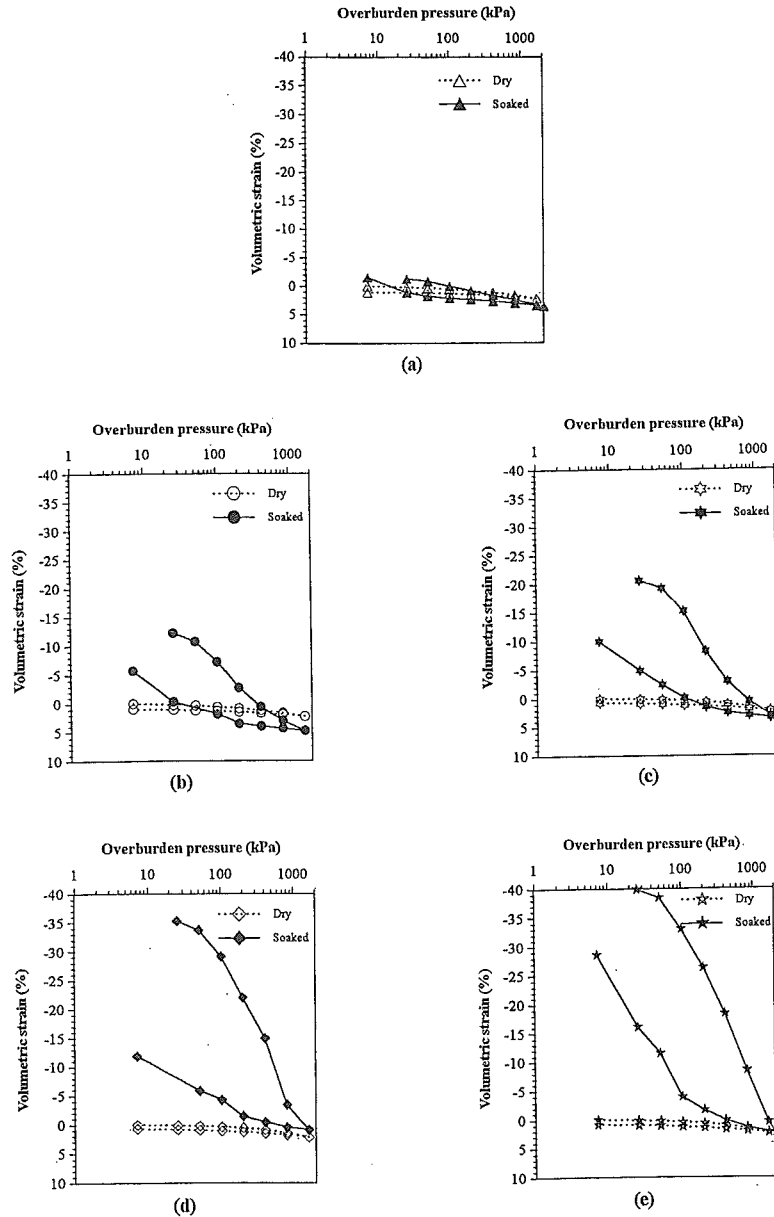


Fig. 6 Double Oedometer Test Results for Sand-High-Quality-Bentonite Soils Compacted at 95% MDD and Moisture Content of 3% less than OMC: (a) 10% Bentonite, (b) 15% Bentonite, (c) 20% Bentonite, (d) 25% Bentonite, and (e) 30% Bentonite

### Swell Potential

Figure 5 and Fig. 6 show the volumetric strain vs.  $\ln(p)$  curves for the LQB and HQB mixtures, respectively, under dry and soaked conditions. The swell potential is defined as the difference in volumetric strain between soaked and dry specimens for a given pressure. As commonly known, swell potential decreases with pressure. Variations of swell potential, at 25 kPa pressure, are given in Fig. 7 for the HQB and LQB mixtures. These results give rise to the following general observations: (1) HQB mixtures swelled more than LQB mixtures, which reflects the effects of interlayer cations; (2) swell potential increases with bentonite percentage; (3) swell potential decreases with molding moisture content; (4) swell potential decreases with increasing the maximum dry density of the mixed soil. The continuous increase in swell potential of HQB mixtures with bentonite is a result of the extremely high swell capacity of the HQB, that compensates for the reduction in density and increase in moisture content as shown in Fig. 6 (c, d, e). Whereas the reduction in the rate of swell potential with bentonite at high bentonite percentage (25% to 30%), noticed in Fig. 5(c), is due to the reduction in dry density and increase in moisture content that govern the response despite the further increase in LQB percentage, which has a lower swell capacity than the HQB. In practice, in-situ compaction control criteria are derived from laboratory compaction test. Therefore, bentonite type and percentage should be considered when selecting compacted bentonite-sand mixture to suit desired field characteristics.

### Compressibility

From the previous volumetric strain ( $\epsilon_v$ ) vs.  $\ln(p)$  curves, for two types of bentonite-sand mixtures (Fig. 5 and Fig. 6), it is clear that virgin compressibility characteristics are not evident up to a vertical pressure of 25 kPa and 50 kPa for LQB and HQB mixtures, respectively. Mathew and Rao (1998) indicated that by increasing the valence of exchangeable cations in the homoionized clay, the overall compression in the system is reduced and the pre-consolidation pressure ( $p_p$ ) is increased. The coefficient of volume compressibility ( $C_{\epsilon_v}$ ) is defined as the slope of the linearized portion of the virgin compression curve,  $C_{\epsilon_v} = \Delta\epsilon_v / \Delta \ln(p)$ . Figure 8 shows the variations of  $C_{\epsilon_v}$  with bentonite percentages for the LQB and HQB mixtures. The  $\epsilon_v - \ln(p)$  relations show that bentonite mixture with higher-valence cations in the adsorbed complex (LQB) has appreciably lower swelling potential (lower void ratio) at any given pressure. The equilibrium void ratio at any applied pressure is a direct function of the repulsive forces arising from the interaction of adjacent diffuse double layers and pore fluid. As the valence of exchangeable cations in the clay is increased, there is a reduction in the diffuse double-layer thickness and in the magnitude of the repulsive forces. These finally result in a lower equilibrium void ratio at any given pressure until higher pressures are reached. All of these factors contribute to the variations in  $C_{\epsilon_v}$  seen in Fig. 8.

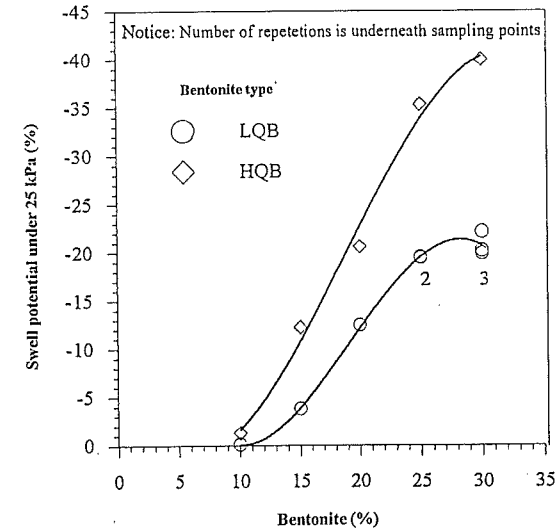


Fig. 7 Variation of Swell Potential Under 25 kPa with Bentonite Percentages for LQB and HQB Mixtures

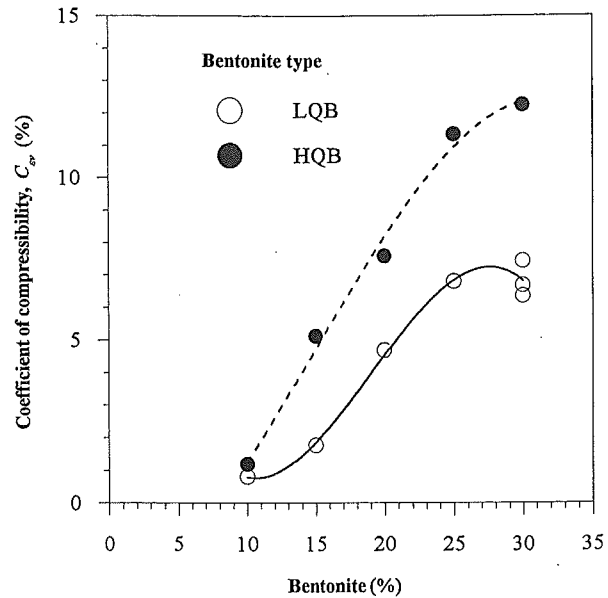


Fig. 8 Variation of Volume Compressibility with Bentonite Percentages for LQB and HQB Mixtures

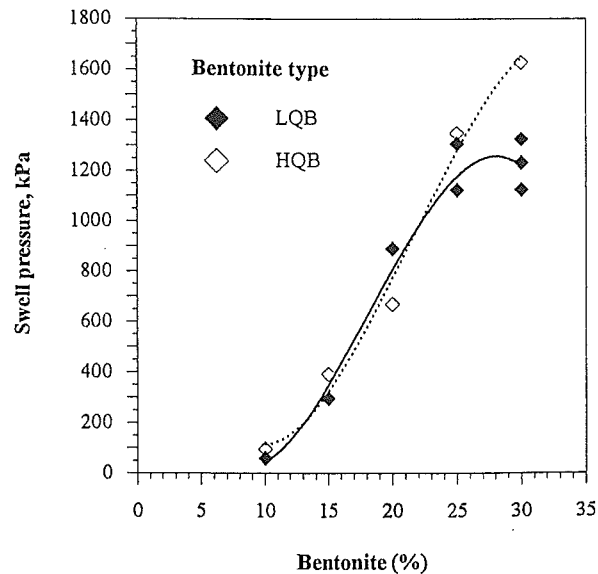


Fig. 9 Variation of Swell Pressure with Bentonite Percentages for LQB and HQB Mixtures

### Swell Pressure

The swell pressure can be measured either at the original density (constant volume tests) or after swelling or compression at a different density (swelling or compression tests). Borgesson (1989) showed that, for Na-smectite clay at high density of 18 kN/m<sup>3</sup> to 20 kN/m<sup>3</sup>, the three methods yield very close results. For simplicity, swell pressure in this study is defined as the pressure required to compress the specimen, that has been soaked and completed the swell under 25 kPa pressure, back into its original configuration (before swell). The variations of swell pressure with bentonite percentages are given in Fig. 9 for the LQB and HQB mixtures. The swell pressure increases with bentonite percentage, increases with increased swell potential under 25 kPa pressure, and decreased with increased volume compressibility.

### CONCLUSIONS

Based on the results of laboratory testing program on two types of sand-bentonite mixtures, the following conclusions can be drawn:

1. The addition of sand caused a linear reduction in the liquid limit and plasticity index of the bentonite-sand mixtures.
2. Complete compaction curves of every mixed soil had the same general pattern with a well-known MDD, OMC, and wet of optimum linear part slope both linked to bentonite mineralogical nature, bentonite content, and also compactive efforts. As the water content increased from the dry side, the dry densities increased to the maximum values at different water contents.
3. MDD increases with bentonite up to a threshold percentage, then MDD decreases with further bentonite increases.
4. OMC continuously increases with bentonite percentage.
5. Swell potential under 25 kPa pressure depends on bentonite type and percentage, dry density, and water content; and their relative magnitudes to optimum values.

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## PILE BEARING CAPACITY DETERMINATION BY HIGH-STRAIN STRESS WAVE METHODS

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### ABSTRACT

The development of the methods for pile static bearing capacity determination from dynamic pile tests is reviewed in this paper. Main equations of the two methods for the determination of pile static bearing capacity are then presented. The two methods are a closed-form traveling wave equation method (CASE method) and a back-analysis traveling wave equation method. Main features of the two methods are discussed. The state-of-the-art RSM instrument and software developed at the Institute of Rock and Soil Mechanics, the Chinese Academy of Sciences for dynamic pile testing is briefly described. The original data from high-strain dynamic pile tests at three sites in China obtained using the RSM instrument and another instrument are presented, compared and discussed. The advantages and limitations of the testing methods are pointed out. Issues for further research in this subject area are identified.

### INTRODUCTION

In foundation engineering, pile bearing capacity determination from dynamic load testing is an important topic with practical applications (Yuan, 1988a,b). Static load testing for determining the pile bearing capacity is generally recognized to be reliable. The static load testing requires great effort and is time consuming, taking from a few hours to a number of days. The overall cost is high, depending on the length and section size of piles. In recent years, dynamic testing technique for pile bearing capacity determination, known as high-strain dynamic pile testing method, has been adopted and put into extensive applications in many countries. Main merits of the dynamic testing technique are in its being economical and taking less time and less disturbance of construction site and schedule compared to the static pile testing. The whole driving process of a driven pile can be monitored using a dynamic testing instrument. Moreover, the high-strain dynamic testing technique can be used to examine pile defects such as cracks and pile necking.

Smith (1955, 1960) studied the one-dimensional pile-driving problem based on stress wave theory. A mathematical model was developed (Smith, 1955, 1960). Generally speaking, the one-dimensional wave equation method for the analysis of dynamic pile driving is considered to be a reasonable approach and may offer advantages over static and other approaches if it can be successfully applied. Successful applications of the wave equation method require knowledge of static and dynamic soil properties, the dimensions and properties of piles, the physical properties of the pile driver and associated equipment used.

With the development in electronics and computers in the past twenty years, Rausche, et al. (1972), and Goble and Rausche (1976) developed mathematical models (methods), computer programs and hardware for pile bearing analysis based measured data of acceleration and force. The CASE method (Goble and Rausche, 1976) is one of the methods, conventionally used for the determination of pile static bearing capacity from dynamic pile testing. In the CASE method (Goble and Rausche, 1976), a closed-form solution of the one-dimensional wave propagation theory was developed using empirical correlation to static pile test results. Another method, the CAPWAP (Case Pile Wave Analysis Program) (Rausche, et al., 1972; Goble and Rausche, 1976; and Rausche, et al., 1985), is an improved version of the CASE method. The two curves of velocity vs. time and force vs. time are measured. The velocity-time curve is used as initial input for the calculation of the force-time curve. Iterations are often needed to make the calculated curve to be in good agreement with the measured results by adjusting soil parameters. This CAPWAP method can be classified into the category of back-analysis wave equation methods.

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