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Classification of expansive soils using void ratio index: An experimental study of bentonite-sand mixtures

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Abstract

This study investigates the one-dimensional consolidation and swelling behavior of compacted expansive soils upon hydration. Four distinct expansive soil mixtures were prepared by blending fine sand with calcium bentonite at varying weight percentages (40%, 55%, 70%, and 85% bentonite) to model a wide spectrum of swelling potential. Oedometer consolidation tests were performed using two different loading pathways following the free swelling phase: Method 1 involved mechanically restoring the specimen to its original pre-swell height prior to consolidation testing, whereas Method 2 completed the consolidation process directly from the equilibrium state reached after full swelling. To characterize these structural volumetric transitions, a Void Ratio Index (VRI) was derived based on the initial void ratios (e_0) and those measured after free swelling (e_{fs}). The laboratory results revealed a clear distinction in compression behavior between the two methods; while the swelling index (C_s) remained practically constant across all mixtures, the compression index (C_c) exhibited a sharp increase under Method 2 due to the structural collapse of newly formed macro-pores. The proposed index (VRI) successfully classified the swelling potential into specific structural categories (ranging from low at $VRI < 25\%$ to very high at $VRI > 60\%$), demonstrating a strong correlation with swelling pressure ($22 - 113$ kPa) and final water content. This newly developed index provides a practical, physics-based framework for predicting the compressibility of expansive soils without requiring lengthy laboratory testing cycles.

1. Introduction

Expansive soils are clayey soils also termed swollen soils. This soil is also residual soil, which is a result of destruction of rocks as a result of weathering in some areas, up to a depth of 6 meters, and in other places for example China, up to 100 feet. Expansive soil size increases when water is added to it and it is more dangerous and then cracks when it dries up and clearly shows cracks at the surface of the soil. This soil has a very high plasticity index; its color tends from dark gray to black and can be simply distinguished at the location during the dry period. Cracks can be observed easily on the surface and may reach over 20 mm [18].

The swelling of clay soils arises primarily from the physicochemical interaction between clay minerals and water, resulting in an increase in soil volume through mineral hydration and changes in the electrochemical forces acting between clay particles and within their interlayer spaces. The degree of swelling varies considerably with the type and proportion of clay minerals present. Montmorillonite, a member of the smectite group, exhibits particularly high swelling potential because of its layered structure and ability to accommodate water within its interlayer spaces, whereas illite and kaolinite generally exhibit lower swelling potential because of differences in

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their mineral structures and interlayer bonding. However, swelling is not governed by mineral composition alone; it also depends on the initial soil structure, moisture condition, void ratio, dry density, degree of saturation, pore-water chemistry, and stress conditions. As water enters the soil, hydration of clay surfaces and changes in interparticle and electrochemical forces can modify the arrangement of clay particles and produce volumetric expansion. Thus, swelling is a coupled physicochemical and mechanical response resulting from the interaction of mineralogy, soil structure, hydrological conditions, and mechanical stresses**,** rather than from a single controlling factor. A great variety of factors might influence the swell and swell pressure characteristics of the expansive soils. These factors may be summarized as below:

- Effect of non-swelling coarser fractions
- Initial moisture content.
- Dry density
- Type and amount of clay mineral.
- Soil structure
- Time
- Degree of saturation
- Load condition
- Thickness of clay layer
- Soil profile
- Depth of seasonal moisture variation
- Climate
- Temperature
- Types of exchangeable ions [17]

1.1 Research gap and Objective

Despite the widespread use of current classification frameworks, they exhibit significant limitations. Most conventional systems rely exclusively on index parameters (LL, PL, PI) measured on fully remolded samples in a slurry state. Consequently, they fail to reflect the actual influence of soil fabric, initial compacted density, or the structural evolution of voids during vertical swelling and consolidation pathways. Hence, there is a critical need for a classification criterion that directly reflects the physical state transition and the reorganization of macro-pores under actual confinement.

To address this deficiency, this study introduces a physical state parameter called the Void Ratio Index (VRI). This innovative index is derived directly from the boundary void ratios during one-dimensional ($1D$) free swelling and subsequent consolidation testing in the oedometer, where the VRI provides a quantitative framework based on physical foundations for classifying the aggressiveness of swelling soils and interpreting the variation in compressibility resulting from different loading paths in the consolidation test.

The effects of expansive soils extend beyond volumetric changes to include the economic aspects of structural design, construction, and maintenance. Swelling and shrinking movements resulting from changes in water content can damage foundations, floors, walls, roads, and utility structures, leading to increased remediation, repair, and maintenance costs. In some cases, special measures may be required to mitigate swelling risks before or during construction. This problem is particularly significant in areas where expansive soils are widespread, as damage can recur with cycles of wetting and drying. Therefore, accurate assessment and classification of swelling potential are not only geotechnically important but can also contribute to improved design decisions and reduced risks and costs associated with structures constructed on expansive soils.

2. Literature Review

One of the important points in engineering that are taken into consideration is the properties of some clay soils whose volume changes periodically according to the season and the percentage of moisture and regardless of the loads imposed on them. Expansive soils periodically shrink and swell depending on climate change. The change in water content mainly affects the volume change under the facilities and this causes an increase in the soil above the ground surface and results in the destruction of facilities or damages that depend on the amount of soil movement. In general, the percentage of moisture content, voids ratio and type of minerals are the main factors in soil volume change [12]. Moreover, recent experimental models on stabilized expansive foundations [1] have emphasized that surface layer boundary conditions and physical density adjustments strongly control macroscopic swell-pressure mitigation.

[11] Used three types of swelling soils (low, medium and high swelling) and studied the effect of the initial load (1, 6 and 10) kilonewtons on the properties of expansive soils. The results showed that with increasing the initial load of the soil, the ratio between swelling and shrinkage decreases. The change in soil density was also studied in the cases of shrinkage and swelling under the influence of the initial load, and the ratio between the two densities decreases with increasing the initial load. As for the percentage of voids and moisture in the soil, there was a conformity between the cases of shrinkage and swelling after the fifth cycle of the soil. [22] They studied the effect of the initial surcharge load on the soil density and the change during repeated immersion and drying for several cycles on the swelling properties of the soil. The results showed that increasing the initial surcharge load from (6) to (100) kPa reduce shrinkage by up to 50%, in addition to the fact that the soil swelling will be much less. As for increasing the soil density, it had no effect on the soil shrinkage, but that increase increased the soil swelling from 25% to 50%.

[14] They studied the effect of initial moisture content in expansive soil on swelling pressure, internal friction angle, cohesion, shrinkage and swelling ratio. The results showed that with increasing initial moisture content from 25% to 40%, swelling pressure decreased from 300 kPa to 25 kPa and internal friction angle, cohesion and swelling ratio also decreased (13°-6°), (115-20) kPa and (10%-1%) respectively, while shrinkage ratio increased from (8%-15%). [15] They studied the effect of sand drain columns on the time and rate of swelling in expansive soils, where they used three models containing (0, 5 and 9) sand drain. The results showed that the presence of these columns increases the rate of swelling and reduces the time required for soil swelling. [16] They also studied the effect of the initial load on the swelling and shrinkage of the soil and used initial loads between 30 and 60 kPa. The results were the same as before. No change was observed in the swelling percentage for the same model by repeating the cycles. With increasing the initial load, the swelling percentage decreased.

Several classification systems have been advanced to identify and evaluate the swelling potential of expansive soils. Early approaches relied primarily on conventional index properties and clay characteristics, including plasticity, clay content, and activity. [7] incorporated the liquid limit and standard penetration resistance, while [8] established classification schemes based on initial moisture conditions [19]. Established an important approach for estimating swelling potential based on the relationship between clay content, clay activity, and measured swell. Subsequently, various classification and identification methods have been developed using index properties, mineralogical characteristics, and swelling parameters. [17] summarized these approaches and emphasized the importance of combining soil identification, classification, and swelling characteristics in evaluating expansive soils.

Regarding the time factor for pressure development, a Hungarian study by [2] on highly plastic clays indicated that swelling pressure and consolidation behavior are critically time-dependent. The onset of full mechanical pressure is delayed due to poor soil permeability and slow movement of the moisture front within the micropores.

2.1 Critical Synthesis and Identification of Research Gap

A critical analysis of the aforementioned literature reveals that, although void ratio changes have been investigated in relation to swelling and consolidation behavior, they have generally been treated as a response variable associated with changes in soil state and mechanical loading rather than as a direct classification metric for swelling potential. Moreover, conventional approaches do not clearly distinguish the structural volume change associated with free hydration from that occurring under mechanical loading. The difference in structural compressibility between a swollen specimen being reconsolidated toward its initial state and a specimen consolidated directly from its expanded state also remains insufficiently incorporated into conventional swelling assessment. The Void Ratio Index (VRI) introduced in this study addresses this gap by providing a normalized framework that relates changes in void ratio to the structural compression behavior of expansive soil.

3. Experimental Work

3.1 Material Properties

The materials used in the experimental work were bentonite (Type Ca). [6] classified two main types of Bentonites the Na-Bentonite and the Ca Bentonite, the first one has the greatest ability to swell. The sand was from Karbala governorate, which is fine and poorly graded.

Table 1. Some types of Bentonite clay with free swell [6]

Name	Free swell %
Sodium montmorillonite	1400-2000
Calcium montmorillonite	45-150
Halloysites	70-100
Indian bentonite	300-1000

Ca-based Bentonite was adopted in the experimental work, it was obtained from Anbar Governorate, Bushayrah Valley, from a depth of 3 meters. The second soil is sand; it was obtained from Karbala Governorate. Four percentages of Bentonite were mixed with four percentages of sand to prepare four groups of soil samples (A, B, C and D), (40, 55, 70 and 85) Bentonite and (60, 45, 30 and 15) % sand respectively. Each of them was divided into four groups to conduct four experiments on them.

These percentages were used to get highly swelling soil. The sand was mixed with Bentonite to increase the permeability and reach the state of saturation in the shortest time.

3.2 Chemical Properties

The chemical tests were carried out for Bentonite and sand in the laboratories of the building research directorate to find out the chemical elements and compounds that contribute to the classification of soil as shown in the Table 2.

Table 2. Chemical analysis of Bentonite and sand

Property	Bentonite	Sand
T.S.S %	4.61	0.9
SO_3 %	1.46	0.23
Gypsum %	1.2	0.51
pH	8.25	7.74
K_2O %	0.51	1.42
MgO %	3.31	1.36
CaO %	14.8	33.32
SiO_2 %	53.26	59.1
Al_2O_3 %	13.8	2.71
Fe_2O_3 %	5.08	0.96
Na_2O_3 %	1.04	0.14

3.3 Soil Classification and Compaction Tests

Soil classification tests were carried out on soil samples used to determine their physical properties and be able to classify them properly. Atterberg limits tests were conducted on the prepared soil (bentonite +sand). ASTM procedure designated [4] was followed in the determination of the liquid limit and plastic limit. The specific gravity of specimens was determined in accordance with the ASTM D854. The compaction characteristics of the soil were conducted in accordance to [3] and obtained using standard compaction hammer. The manual compactor was used for this purpose. The Moisture-Dry unit weight relationship for the soil is shown in Figure 1. The results of the physical tests for samples are given in Table 3 and Figure 2. Distilled water is used throughout this study in all tests and specimens' preparation. On the other hand, the study [20] focused on evaluating the effect of the mineral content of 'Calcium-Bentonite' when mixed in varying proportions with sand on the geometric behavior and pore size distribution of the unsaturated mixture.

Table 3. Physical properties

Bentonite %	Sand %	Optimum moisture content (%)	Maximum unit weight (kN/m^3)	Specific gravity	Liquid limit %	Plastic Limit %	Plasticity Index IP	Initial void ratio
40	60	17.0	17.6	2.673	65	31	34	0.49
55	45	22.0	15.9	2.77	74	36	38	0.71
70	30	27.9	14.52	2.87	79	43	36	0.94
85	15	28.22	13.66	2.93	94	51	43	1.1

Their results showed that increasing the proportion of calcium clay leads to a complete restructuring of the internal pores and increases the soil's ability to retain water, which is exactly in line with what our laboratory study recorded when moving from model A (lower bentonite percentage) to model D (higher bentonite percentage) in Table 3, where a sharp increase in the optimum moisture content was observed.

3.4 Sample Preparation

To prepare samples for oedometer, the natural soil was oven dried in the laboratory for 24 hours and mixed with the predetermined amount of Bentonite, then the required amount of water was added and mixed thoroughly using a trowel.

The standard compaction method was followed in the preparation of soil samples used in this study. The compaction mold of known dimensions, (105 mm) in diameter and (115 mm) in height, was lubricated with a thin oil film in order to minimize the effect of mold side friction on the compaction process. Then the soil was placed into three successive equal layers and each layer was compacted by a specified number of blows. After compaction is finished the upper face was trimmed flat. Different percentages of Bentonite were used with different percentages of sand to obtain different samples. For each sample group four identical samples were prepared.

Fig. 1. Sample preparation

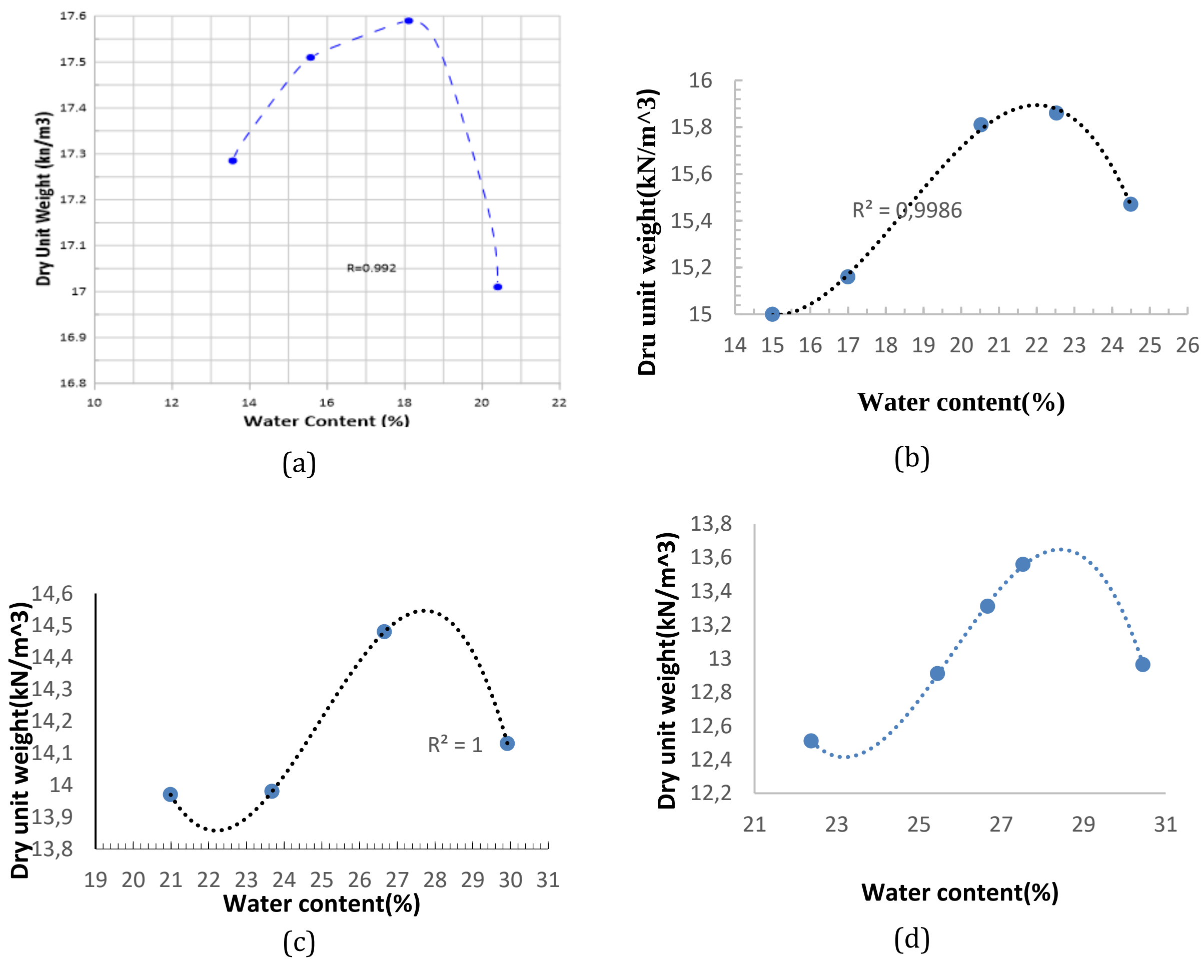

Fig. 2. Compaction curves for different percentages of Bentonite: (a) 40% Bentonite 60% Sand, (b) 55% Bentonite 45% Sand, (c) 70% Bentonite 30% Sand, (d) 85% Bentonite 15% Sand

3.5 Consolidation Tests

For each ratio of Bentonite to sand four test samples were prepared, the specimen height is made to be 7mm less than the height of the consolidation ring to ensure that the specimen will remain laterally confined during swelling. The inner surface of the ring is oiled to minimize the frictional effect. Then, sample is inundated with water and left to swell freely. The increase in sample thickness is recorded using dial gauge of 0.001 mm/division. This test is conducted according to [5]. namely: the first test is a test to find the density, the second test is a free swelling test, the third test is a swelling pressure test and the last test is a traditional consolidation test on dry samples. In the first three models the samples are immersed. Distilled water is used throughout this study in all tests and the change in the dial gages reading is observed with time until it is observed that there is no change in their readings. After that, in the free swell sample the consolidation test is completed on it. As for the sample for measuring the swelling pressure loads are applied to return the sample to its initial state before swelling take place. where the total load amount is equal to the swelling pressure. After that, the consolidation test is performed.

3.6 Free Swell and Consolidation Test

The free swell percent is calculated as:

$$FS\,\% = \left(\frac{H - H_0}{H_0}\right) \times 100 \tag{1}$$

Where: H: The final thickness of the soil sample after the completion of the swelling stage mm, H_o: The initial thickness of the soil sample before wetting mm.

From a geotechnical perspective, since the cross-sectional area of the oedometer sample (A) is constant and rigid throughout the examination, the change in height is directly related to the change in the void ratio via the well-known geometric relationship [9].

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0} \tag{2}$$

Where; ΔH: The change in the soil sample thickness during the swelling or consolidation stage, Δe: The change in the void ratio of the soil mass corresponding to the structural deformation, e0: The initial void ratio of the soil specimen at its initial compacted state.

$$FS\ \% = \left(\frac{e_{fs} - e_0}{1 + e_0}\right) \times 100 \tag{3}$$

Where; e_{fs} : The final (equilibrium) void ratio of the soil sample reached at the completion of the hydration and free swelling phase. Then the test is continued by increasing the stresses on samples to study the consolidation characteristics of samples according to [13].

3.7 Swelling Pressure and Consolidation Tests

After samples inundation with water, samples were prevented from swell by increasing applied stresses on the top of samples gradually to satisfy that no swelling will take place. It takes time to reach the equilibrium state. The maximum stress required to prevent the sample from swelling is recorded. This stress is defined as swelling pressure according to [5].

Then the test is continued by increasing the stresses on samples to study the consolidation characteristics of samples according to [13]. The test was conducted according to the procedure proposed by [13]. This procedure could be summarized as follows:

- The specimen was prepared in the ring as described in section (3-3). Then it was placed in the oedometer cell and fitted in the loading frame. The loading yoke was set and the loading beam was leveled.
- The dial gauge above the cell, with sensitivity up to (0.025 mm/ div), was set to zero reading and a seating pressure of 7 kPa was applied to the specimen and the dial gauge reading was corrected back to zero.
- The cell was filled with water and at the same instant the clock was started and the dial gauge monitored.
- As needed, weights were added thus increasing the pressure on the specimen up to a level that would maintain the dial gauge back to zero reading.
- After that the specimen was test in consolidation.

4. Results

The expansion soil adhesion tests were conducted for the four models (A,B,C and D) according to the [5] and each model had four tests as mentioned previously for the purpose of knowing the swelling pressure and the adhesion behavior of the expansion soil after returning it to its previous state before swelling and its behavior after swelling and knowing the moisture content and density as well as calculating the swelling percentage and swelling pressure and classifying the soil on this basis. The results showed as in Table 4 that with increasing the percentage of bentonite or clay particles, the swelling percentage increases and the swelling percentage ranged between (1% to 18%) approximately, i.e. from low swelling to very high, and the swelling pressure ranged between (22 to 113 kPa). In a related vein, [24] investigated the effect of initial dry density and roller energy on the long-term swelling pressure of highly compressible bentonite. The study concluded that the maximum pressure is directly and sensitively proportional to the dry density and the confining structure surrounding the sample. This scientifically explains the results obtained in model D (high density and higher void ratio), where the swelling pressure jumped to its highest value of 113 kPa due to the higher dry density and stronger mechanical confinement within the odometer.

Table 4. Expansion potential and swelling pressure

Soil type	A	B	C	D
Expansion potential %	1.34	5	7	17.8
Swelling pressure kPa	22	31	75	113

In addition to this, the results clearly showed a significant increase in the percentage of voids after swelling with increasing the swelling percentage as in Figure 3.

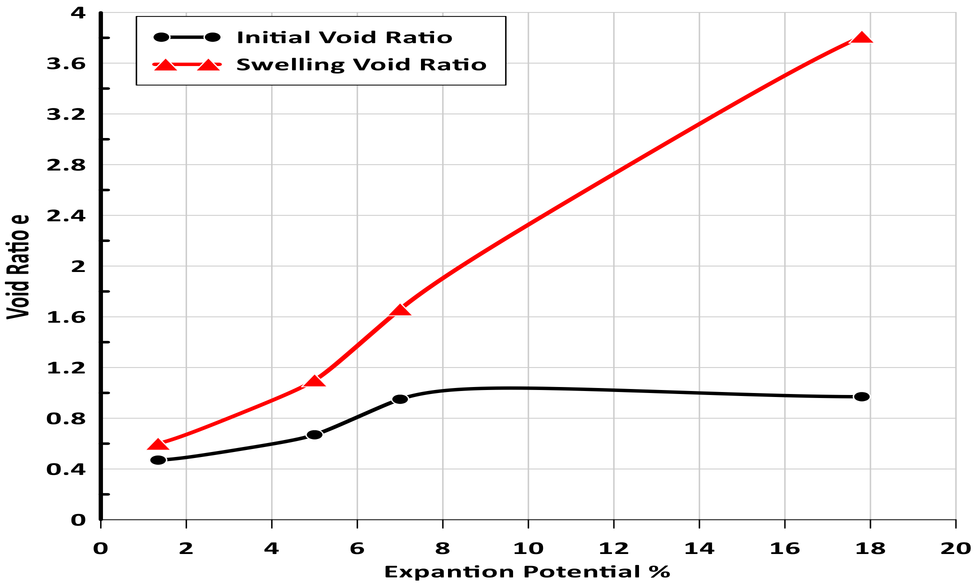

Fig. 3. Relation between void ratio and expansion potential

The void ratio index was calculated as in the equation:

$$Void\ Ratio\ Index\ (e_i) = \frac{swelling\ void\ ratio - initial\ void\ ratio}{swelling\ void\ ratio} * 100 \tag{4}$$

This index gives a clear idea of the increase in the percentage of voids after swelling and can be considered as an indicator of the extent of soil swelling in addition to the swelling percentage as shown in Figure 3. The Table 4 shows the relationship between the void ratio index and the expansion potential. Through this comparison, the expansive soil can be classified based on the void ratio index. In a related context, previous investigations on soil mechanics and bentonite mixtures [9] and [10] confirmed that void ratio evolution during hydration is governed by both structural packing and clay mineral expansion.

Table 5. Relation between expansion potential and void ratio index

Expansion potential	Very Low	Low	Medium	High	Very High
Expansion Index [17]	0-2	2-5	5-9	9-13	>13
Void Ratio Index %	0-25	25-40	40-50	50-60	>60

Figure 5(a) shows the relation between water content (ω) and dry density(γ_d) and the ω increased with decrease the γ_d (18%,15gm/cm^2) to 27%,10.6gm/cm^2) respectively. Figure 5(b) shows decreasing void ratio (e) with increase γ_d and decreasing was sharped initially in swelling case and then decrease slightly while other hand (e) decreased gradually with increase γ_d in dry case.

Figure 6 shows the consolidation test for the four models. The behavior of the models has been studied and it is clear to us that there is a similarity in the behavior of the models in the process of unloading, as the swelling index is close between the four models, while there is a large difference in the compression index in the models.

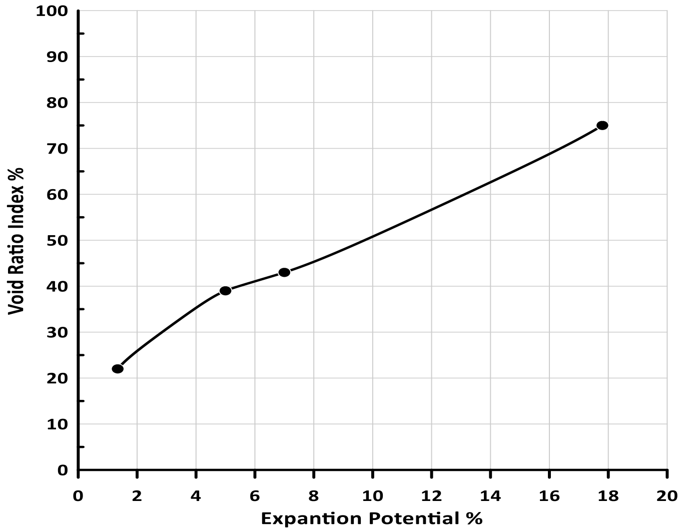

Fig. 4. Relation between void ratio index and expansion potential

In a comprehensive review by [21] on the volumetric change and consolidation characteristics of expansive soils, it was confirmed that the microstructure changes occurring in the soil during free swelling completely alter the subsequent compressibility behavior (C_S-C_C). This reference conclusion is consistent with what our study demonstrated empirically: a significant variation in the compressive strength (C_S-C_C) values between the two testing methods (returning the sample to its original height versus completing the test directly), since allowing the sample to swell without reshaping it gives it a new porous structure that is more compressible under loads.

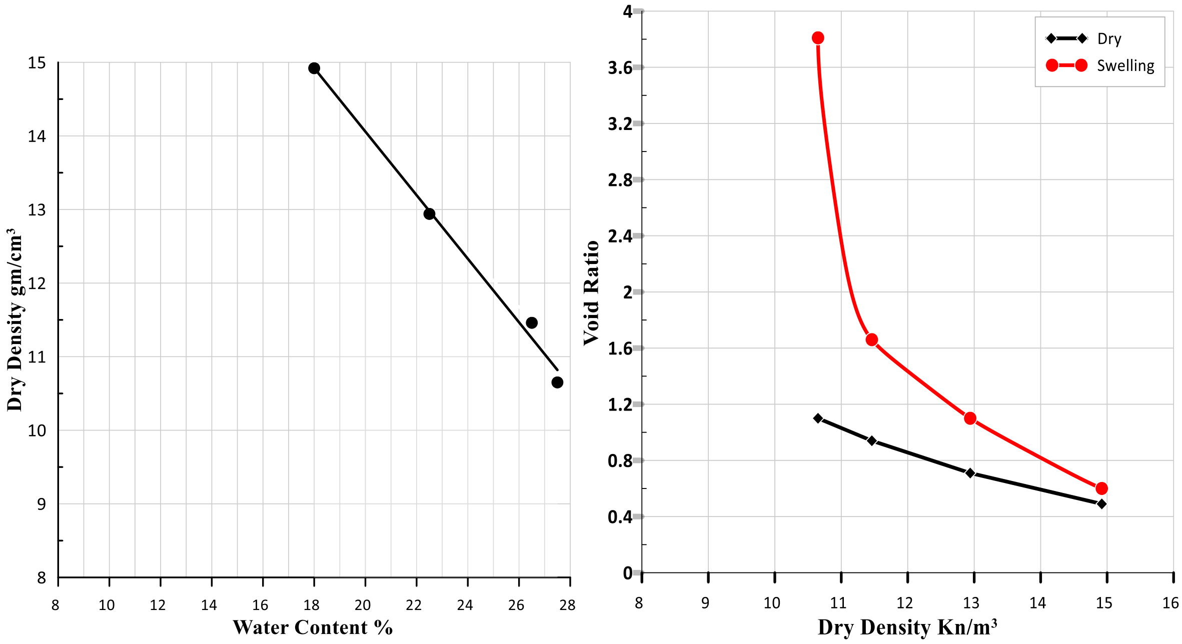

Fig. 5. Relation between (a) dry density and water content (b) void ratio and dry density

The results also showed a difference in the percentage of voids for the same model between the beginning and end of the test, and the divergence of the values in the free swelling test is clearly evident, as the difference between them is very large, unlike the normal test and for all models.

The results also showed that the compression index varied for all cases of the same model in the three cases and also differed according to the expansion potential, while the swelling index was almost close to all models for all cases.

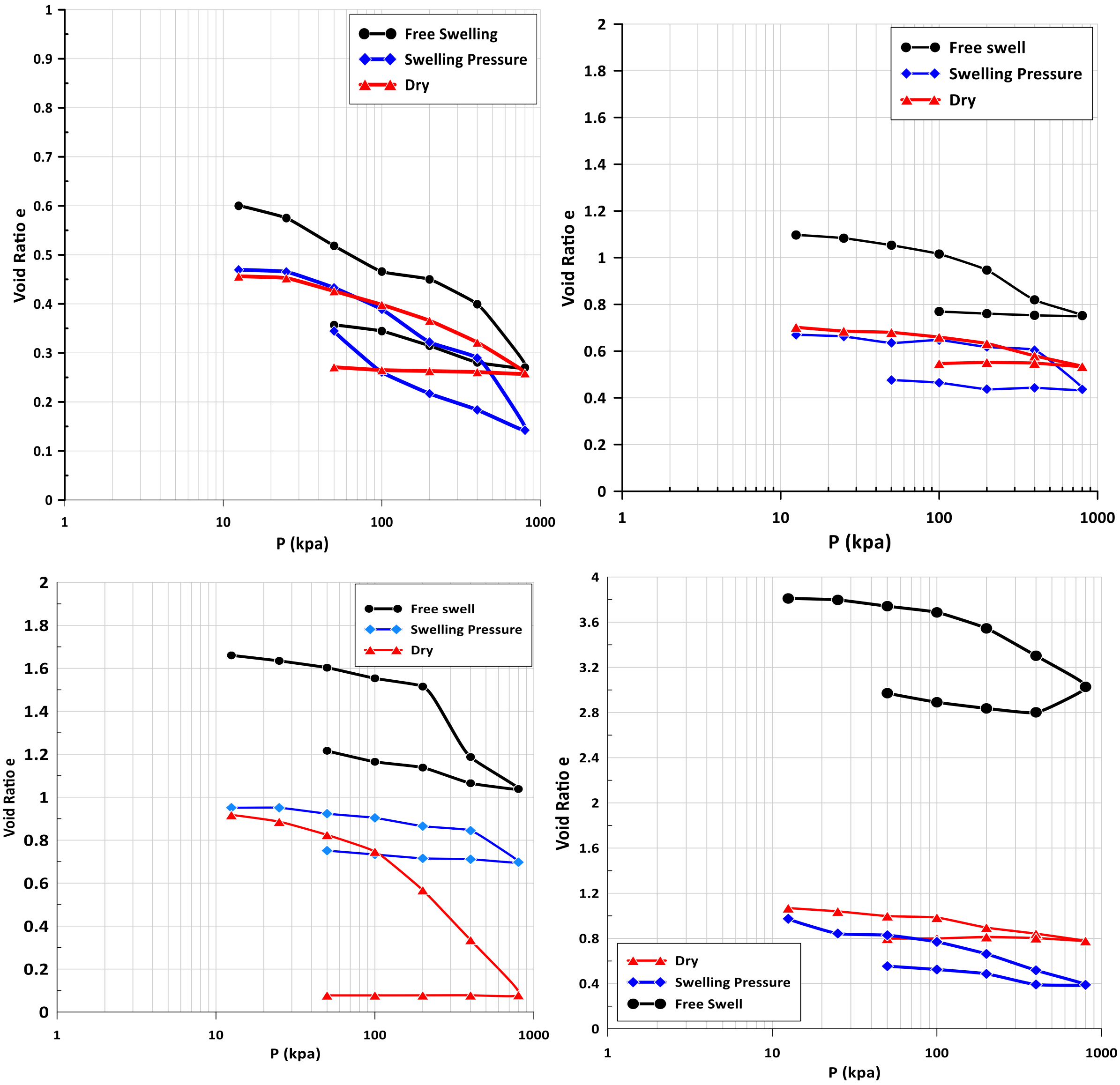

Fig. 6. Relation between stress and void ratio

5. Discussion

5.1 Comparative Evaluation with Literature

The observed swelling percentages $(1.34\% - 17.8\%)$ and swelling pressures $(22 - 113 \text{ kPa})$ align closely with published data for compacted bentonite-sand mixtures [1] and [17]. The non-linear rise in swelling pressure with clay content reflects the findings of [24] confirming that rigid oedometer lateral confinement amplifies swelling stress as initial dry unit weight decreases and clay fraction increases.

5.2 Physico-Chemical Mechanism of Bentonite Hydration

The systematic jump in swelling performance from Model A to Model D is rooted in the mineralogy of calcium montmorillonite. Hydration forces and osmotic potential draw water molecules into the $2:1$ interlayer spaces, swelling the diffuse double layer (DDL). As bentonite concentration increases, the soil fabric transitions from a sand-dominated matrix to a continuous clay matrix, maximizing interlayer repulsive forces and macro-structural volumetric expansion.

5.3 Rationalization of VRI as a Classification Parameter

Unlike traditional classification systems [19] and [23] based on remolded index properties (LL, PL, PI), the proposed VRI accounts for initial dry density, molding moisture, and actual compacted fabric. It directly connects microstructural pore expansion during free hydration to macroscopic field heave under effective confinement.

5.4 Microstructural Collapse and Compressibility Disparities (C_c vs C_s)

The marked difference in C_c between Method 1 and Method 2 provides critical insight into soil fabric evolution. In Method 1, squeezing the specimen back to H_0 mechanically expels macro-pore water and resets the soil fabric to its initial dense state, yielding lower C_c values. In Method 2, consolidating directly from e_{fs} preserves the open macro-pore structure, which undergoes irreversible collapse under vertical load. Conversely, the constancy of C_s confirms that elastic particle rebound is an intrinsic physico-chemical property independent of loading pathway.

5.5 Practical Engineering Implications & Study Limitations

In foundation engineering, estimating post-hydration settlement using standard C_c values from Method 1 severely underestimates field deformation. Utilizing VRI enables design engineers to predict post-swell void ratios (e_{fs}) and adopt realistic C_c values. Study limitations include: (1) testing was restricted to calcium bentonite and silica sand; (2) testing was limited to $1D$ oedometer conditions without $3D$ field boundary effects; and (3) pore-water salinity was not varied.

6. Conclusions

- This study successfully establishes the Void Ratio Index (VRI) as a physically grounded, state-dependent parameter for classifying expansive soils. Derived directly from boundary void ratios during $1D$ free swelling (e_0 and e_{fs}), the VRI effectively captures microstructural macro-pore evolution under lateral confinement. A strong linear correlation was observed between VRI and expansion potential, enabling a refined five-tier classification system: Very Low ($VRI < 25\%$), Low ($25\% - 40\%$), Medium ($40\% - 50\%$), High ($50\% - 60\%$), and Very High ($VRI > 60\%$). This index provides a more reliable assessment of soil aggressiveness compared to traditional index-property systems (LL, PL, PI) because it incorporates initial dry density and compaction fabric.

- The experimental investigation revealed a major hydro-mechanical disparity between the two post-swell consolidation pathways. Under Method 1 (mechanically restoring the fully swollen specimen to its initial height H_0 prior to loading), mechanical compression forcefully expels macro-pore water and resets the soil skeleton, resulting in stiffer consolidation behavior and lower compression index (C_c) values. Conversely, under Method 2 (consolidating directly from the swollen state e_{fs}), the open, highly porous soil fabric undergoes structural micro-pore collapse under vertical stress exceeding the swelling pressure, manifesting as a sharp increase in C_c.

- The swelling index (C_s) remained remarkably uniform across all four mixtures (Models A through D), demonstrating that the elastic rebound of clay particles is an intrinsic physico-chemical property governed by montmorillonite double-layer interaction. In contrast, the compression index (C_c) is highly path-dependent and governed by the volumetric macro-pore structure quantified by VRI.

- In terms of physical properties, increasing the calcium-bentonite proportion from 40% to 85% resulted in a proportional increase in post-swell water content (up to 28.2%) and a corresponding sharp reduction in dry unit weight. Swelling pressure increased non-linearly from 22 kPa to 113 kPa due to enhanced double-layer repulsions and rigid oedometer confinement.

- From a practical engineering standpoint, evaluating post-hydration settlement using C_c values obtained from conventional pre-swell testing (Method 1) severely underestimates field deformation. Utilizing the proposed VRI classification allows geotechnical engineers to account for structural void collapse, ensuring accurate settlement prediction and foundation design over expansive soil formations.

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