Evaluation of the relationship between swelling pressures determined by consolidation-swell test and constant volume test

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ABSTRACT

Foundations on expansive soils pose a unique challenge to geotechnical engineers. Oftentimes they cost more to design and construct than foundations on ordinary soils. Free-field heave is the fundamental parameter on which foundation heave is calculated. A method for prediction of free-field heave using oedometer test data was outlined in Nelson and Miller (1992). A refinement of that method is presented in Nelson et al. (2006). To determine free-field heave, it is necessary to have a measured value of percent swell from a consolidation-swell (CS) test and the swelling pressure, $\sigma_{cs}$, measured in a constant volume (CV) test. Only the CS test is commonly conducted in geotechnical engineering practice. Therefore, it is convenient to have a relationship between the swelling pressure, $\sigma_{cs}$, measured in the CS test and $\sigma_{cv}$, so that heave prediction can be determined from only a single test. Several investigators have proposed relationships between $\sigma_{cv}$ and $\sigma_{cs}$ (Edil and Alanazy, 1992; Reichler, 1997; Bonner, 1998; Thompson et al., 2006; Nelson et al., 2006 and 2012). Nelson and Chao (2014) proposed a relationship between $\sigma_{cv}$ and $\sigma_{cs}$ based on the experimental results of expansive soil behavior and facilitated the heave prediction from a single oedometer test for each soil. Their method used the parameter “m” which relies on specific soil property. This paper further investigated the change of parameter “m” for various soil samples. The soil samples used in this study were obtained from two different locations in Myanmar and Thailand. Bentonite and sand mixture was also tested in this study for comparison purposes. Furthermore, the relationship between the “m” parameter and the degree of saturation of the soil samples were evaluated in this study. It was concluded that the linear trendline determined by Nelson and Chao (2014) provides the most simple and accurate equation for determination of the m value and the m value decreases as the degree of saturation of the soil increases.

Keywords: expansive soil, swelling pressure, percent swell, consolidation-swell test, constant volume test, m value, free-field heave, heave index

1 INTRODUCTION

Expansive soils exhibit significant volume change in the form of swelling and shrinking due to the fluctuations in moisture content. The volume change behavior of these soils causes extensive damage to structures that rest on it. Therefore, it is essential to know beforehand the presence of expansive soils before design and construction of a structure to minimize the risk of heaving of the expansive soils.

The swelling pressure of an expansive soil is commonly determined using the consolidation-swell (CS) oedometer test and the constant volume (CV) oedometer test. The swelling pressure determined from the CV test is needed for an evaluation of heave potential for the expansive soil. However, it is common that only the swelling pressure determined from the CS test is obtained in the geotechnical engineering practice. Therefore, it is necessary to obtain a relationship between the swelling pressures determined in the CV and CS tests for calculations of heave potential.

A number of investigators have proposed relationships between the swelling pressures determined in the CV and CS tests (Edil and Alanazy, 1992; Reichler, 1997; Bonner, 1998; Thompson et al., 2006; Nelson et al., 2006 and 2012). Some forms of the equation that utilize a simple ratio the swelling pressures determined in the CV and CS tests were proposed by those investigators. Nelson and Chao (2014) proposed an equation that requires determination of a parameter “m” to obtain the relationship between the swelling pressures determined in the CV and CS tests. A brief discussion of the m method is provided in Section 2.2 of this paper.

The objectives of this study are to further evaluate the “m” method with various types of expansive soils and to evaluate the relationship between the m value and degree of saturation. In this study, 20 oedometer tests were performed on three types of soils with
different inundation stresses and water contents. The results of the oedometer tests are provided in the following sections. All symbols used in this paper are defined in Table 1.

| Symbol | Definition |
|--------|------------|
| CV     | Constant volume swelling pressure |
| M      | Parameter defining relationship between \( \sigma'_{cs} \) and \( \sigma''_{cv} \) |
| \( \varepsilon_{sv} \) | Percent swell |
| \( \sigma'_{cs} \) | Consolidation-swell swelling pressure |
| \( \sigma''_{cv} \) | Constant volume swelling pressure |
| \( \sigma''_i \) | Inundation stress |
| MDY    | Mandalay clay |
| LP     | Lampang clay |
| BS     | Bentonite-sand mixture |

2 DETERMINATION OF SWELLING BEHAVIOUR BY OEDOMETER TESTING

2.1 Oedometer Testing

Various oedometer tests have been investigated to study the swelling behavior of expansive soils. Some prominent testing methods include free swell, loaded swell, restricted swelling, zero swell, consolidation-swell (CS) and constant volume (CV) oedometer tests. Among these tests, the CS oedometer test is the most widely used methods for measuring swelling properties of expansive soils. The results from the CS test is used to design foundations on expansive soil sites.

For the CS test, the sample is initially subjected to applied stress specified in the oedometer and inundated under that vertical stress. The percent swell, \( \varepsilon_{sv} \), is measured as the vertical strain occurs due to wetting of the sample. The sample is subjected to additional vertical loads. The consolidation-swell swelling pressure, \( \sigma''_{cs} \), is measured as the stress that would be required to restore the sample to its original height. The loading path of the CS test is shown in Figure 1 as Line HGE.

\[
\rho = C_H \Delta z_i \log \left( \frac{\sigma''_i}{\sigma''_{cv}} \right)
\]

where:
- \( \rho \) = free-field heave,
- \( C_H \) = heave index,
- \( \sigma''_i \) = in-situ effective stress state,
- \( \sigma''_{cv} \) = swelling pressure from the constant-volume oedometer test; and
- \( \Delta z_i \) = layer thickness.

The heave equation shown in Eq. (1) consists of a parameter called heave index, \( C_H \) which is defined as a linear relationship between percent swell and applied stress, as shown in Figure 1. The heave index, \( C_H \), can be calculated from the following equation:

\[
C_H = \frac{\varepsilon_{sv}}{\log \sigma''_{cv} - \log \sigma''_i}
\]

The line BDE in Figure 1 and the heave index, \( C_H \), can be determined by conducting consolidation-swell and constant volume oedometer tests on samples of the same soil and connecting point B obtained from the consolidation-swell test with point E obtained from the constant volume test. However, to do so is generally not practical, mainly because it is almost impossible to obtain two identical samples from the field. Therefore, a relationship between \( \sigma''_{cv} \) and \( \sigma''_{cs} \) needs to be determined so that the value of the heave index can be determined from a single consolidation-swell test.

2.3 Relationship between swelling pressures

As described in the previous section, two oedometer tests, namely the consolidation-swell and constant volume tests would be required to determine the heave index. In order to determine the heave index from just the CS test, it is therefore convenient to derive a
relationship between \(\sigma''_c\) and \(\sigma''_v\). Various researchers proposed the relationship between these swelling pressures, generally in the form of an equation that utilizes a ratio of \(\sigma''_c\) and \(\sigma''_v\) (Edil and Alanazy, 1992; Reichler, 1997; Bonner, 1998; Thompson et al., 2006; Nelson et al., 2006 and 2012).

Nelson and Chao (2014) proposed an equation that requires determination of a parameter "m" to obtain the relationship between the swelling pressures determined in the CV and CS tests. Nelson and Chao (2014) indicated that as the \(\sigma''_i\) increases, the swelling pressure decreases for the oedometer tests performed on a number of samples at different inundation stresses, as shown in Figure 2. In the case where the soil sample does not swell when it is inundated, the inundation stress, \(\sigma''_i\), would be equal to the constant volume swelling pressure, \(\sigma''_v\). Therefore, the value of \(\sigma''_c\) will converge to \(\sigma''_v\) at the point where \(\sigma''_i\) is equal to \(\sigma''_v\). This is shown in Figure 3 where values of varying swelling pressures of \(\sigma''_c1\), \(\sigma''_c2\), and \(\sigma''_c3\) are plotted with corresponding inundation stresses of \(\sigma''_i1\), \(\sigma''_i2\), and \(\sigma''_i3\). The point M in Figure 3 corresponds to the same point M in Figure 2. This point M represents a point where \(\sigma''_im = \sigma''_v\). This point can be plotted on a line with a 1:1 slope.

![Fig. 2. Oedometer test results for different values of \(\sigma''\)_i (Nelson and Chao, 2014)](image)

The equation of the line can be written for any value of \(\sigma''_cm\) and its corresponding \(\sigma''_im\) as follows:

\[
\frac{\log \sigma''_cm - \log \sigma''_cv}{\log \sigma''_cv - \log \sigma''_im} = m
\]

Equation (3) can be rewritten to obtain the relationship between \(\sigma''_c\) and \(\sigma''_v\) as follows:

\[
\log \sigma''_v = \log \sigma''_cm + m \log \sigma''_im
\]

As long as the m value for a particular soil is known, the \(\sigma''_v\) value can be obtained using Eq. (4), and thus, the free-field heave of the soil can be calculated using Eqs. (1) and (2).

3 EXPERIMENTAL WORK

3.1 Selected materials and index properties

In this study, three types of soils including Mandalay clay, Lampang clay, and bentonite-sand mixture were tested. The Mandalay clay was collected from Pathein Gyi Township, east of Mandalay city, Myanmar. The clay was very dry and hard at its original state, however became soft and sticky with the addition of a small amount of water. The sample was excavated at a depth of approximately 1 m below the ground surface. Cracks up to one to two cm wide were observed at the ground surface close to the sampling point. The Lampang clay was collected from Mae Moh mine in Lampang province, Thailand. The Mae Moh mine is the largest open-pit lignite mine in Southeast Asia. The sampling point was located at a golf course near the Mae Moh electricity plant. The artificial soil (bentonite-sand mixture) was made by using 50% sodium bentonite and 50% fine sand. Basic index properties of the soils were determined according to the ASTM standards and are shown in Table 2. In order to study the changes in the m values with various values of the degree of saturation, the Mandalay clay was tested using three different water contents. These water contents were selected using the results of the standard Proctor compaction test. The selected water contents were, -2% of the optimum water content, the optimum water content, and +2% of the optimum water content. The Lampang clay and the artificial soil were tested at only the optimum water content.

The particle size distribution curves for the soils were plotted in accordance with ASTM C136 and D422 Standards and are presented in Figure 4. The standard

![Fig. 3. Convergence of \(\sigma''_c\) and \(\sigma''_i\) to \(\sigma''_v\) (Nelson and Chao, 2014)](image)
Proctor tests were conducted in accordance with the ASTM D698 Standard for all three soils. The values of maximum dry density and optimum water content for the soils are presented in Table 3. The compaction curves of the soils are shown in Figure 5.

| Physical properties | MDY | LP | BS |
|---------------------|-----|----|----|
| Specific Gravity, G_s | 2.69 | - | 2.61 |
| Natural water content (%) | 5.49 | 10.94 | - |
| Liquid limit, LL | 60 | 69 | 275 |
| Plastic limit, PL | 18 | 19 | 19 |
| Plasticity index, PI | 42 | 50 | 256 |
| Gravel (%) | 1 | 1 | - |
| Sand (%) | 44 | 37 | 49 |
| Silt and clay (%) | 55 | 62 | 51 |
| Unified soil classification | CH | CH | CH |
| Color | Gray | Yellow | Brown |

Table 2. Physical properties of all soils tested.

| Soil | Maximum dry density (kN/m³) | Optimum water content (%) |
|------|-----------------------------|---------------------------|
| MDY  | 16.37                       | 19.8                      |
| LP   | 15.37                       | 21.6                      |
| BS   | 15.51                       | 19.5                      |

Table 3. Standard Proctor test results.

![Graph showing particle size distributions of all soils tested.](image1)

Fig. 4. Particle size distributions of all soils tested.

3.2 Oedometer tests

Twenty (20) mm thick remolded specimens were prepared in 67 mm diameter floating rings. The mass of the soil that was required was weighed and separated into three equal portions to be compacted into three equal layers inside the ring. A small steel rod was used to compact the soil layer by layer in the confining ring. The CV test was performed in accordance with ASTM D4546 Standard for five samples to determine the constant volume swelling pressure. In this test, the specimen was confined in order to prevent the soil from swelling, and the stress required to prevent any swelling is termed the constant volume swelling pressure, σ'_{cv}.

![Graph showing compaction curves of the soils tested.](image2)

Fig. 5. Compaction curves of the soils tested

The CS tests were carried out on remolded expansive clay samples under successive incremental vertical loads to determine the percent swell and the CS swelling pressure. In this study, fifteen remolded samples were tested with various water contents and inundations stresses of 10, 20, and 30 kPa. For Mandalay clay, the oedometer tests were performed with three different water contents while maintaining the same dry density. The Lampang clay and bentonite-sand mixture were tested using only the optimum water contents.

4  RESULTS AND DISCUSSION

4.1 Effect of degree of saturation on the m value

The samples of Mandalay clay were used to evaluate the effect of degree of saturation on the m value. Furthermore, the study also compiled swelling data from Thompson et al. (2006) to derive a trend. Figure 6 presents the relationship between the m value and the degree of saturation for the Mandalay clay samples in this study and the Colorado expansive claystone from Thompson et al. (2006).

The highest value of the m value for the Mandalay clay shown in Figure 6 was 0.95 when the degree of saturation was 56%. However, when the degree of saturation was increased to be 71%, the m value was decreased to be 0.7. It was observed that the m value decreases with increasing water content of the soil.

Further evaluation of changes in the m value for soils with various degree of saturation was performed using swelling pressure data obtained from Thompson et al. (2006). The m values were calculated using constant volume and consolidation-swell swelling pressures obtained from Thompson et al. (2006) on Colorado claystone. The results of the Thompson et al.
(2006) data are also shown in Figure 6. The plot presented in Figure 6 shows a similar trend to the results obtained in this study.

The plot presented in Figure 6 shows a similar trend to the results obtained in this study. Nelson and Chao (2014) assumed a linear trendline for determination of the m value. The results of this study affirm the use of the linear function to determine the m value by evaluating the regression R\(^2\) values for different trendline equations. It was concluded that the linear trendline provides the most convenient way to determine the m value.

The effect of degree of saturation on the m value was evaluated for the Mandalay clay and Colorado claystone using experimental test results and data compiled from Thompson et al. (2006), respectively. The results showed that the water content indeed effects the m value of the soil. The m value decreases as the degree of saturation of the soil increases. This finding suggests that the degree of saturation should be considered when determining the m value for prediction of free-field heave.

4.2 Determination of the m values for soils tested

As shown in Figure 3, Nelson and Chao (2014) assumed a linear function for determination of the m value. The linear function was verified in this study by using soil samples of the Mandalay clay, Lampang clay, and artificial soil. Figure 7 shows the m values using the linear function for all three soils tested. A summary of the m values using the linear function with the regression R\(^2\) results for all three soils tested is shown in Table 4. Table 4 indicates that the linear function provides a reasonable match among the observed data.

Different trendline functions other than the linear function were used to evaluate the observed data for determination of the best fit of the observed data. Table 5 shows a summary of the regression R\(^2\) values for different trendline equations. Among these equations, the polynomial trendline shows the best R\(^2\) value. However, the R\(^2\) values obtained from the polynomial trendline are not much higher than those obtained from the linear trendline. Furthermore, the equation of the polynomial trendline is much more complicated to use compared to that for the linear equation. Consequently, it is concluded that the linear trendline provides the most simple and accurate equation for determination of the m value.

5 CONCLUSIONS

In this study, three types of soils including Mandalay clay, Lampang clay, and bentonite-sand mixture were subjected to oedometer tests with various inundation stresses and water contents. The study intended to evaluate the m values for these soils and to evaluate the relationship between the m value and degree of saturation. Chief conclusions and observations of this study are summarized below:

- Nelson and Chao (2014) assumed a linear trendline for determination of the m value. The results of this study affirm the use of the linear function to determine the m value by evaluating the regression R\(^2\) values for different trendline equations. It was concluded that the linear trendline provides the most convenient way to determine the m value.
- The effect of degree of saturation on the m value was evaluated for the Mandalay clay and Colorado claystone using experimental test results and data compiled from Thompson et al. (2006), respectively. The results showed that the water content indeed effects the m value of the soil. The m value decreases as the degree of saturation of the soil increases. This finding suggests that the degree of saturation should be considered when determining the m value for prediction of free-field heave.

![Fig. 6. Relationship between m value and degree of saturation for Mandalay clay and Colorado claystone](image)

![Fig. 7. M values using the linear function for all three soils](image)

| Soil   | Water content (%) | m value | Regression value (R\(^2\)) |
|--------|-------------------|---------|---------------------------|
| MDY    | 17.75             | 0.95    | 0.9443                    |
| MDY    | 19.75             | 0.8     | 0.9877                    |
| MDY    | 21.75             | 0.7     | 0.9867                    |
| LP     | 21.6              | 0.75    | 0.9676                    |
| B+S    | 19.5              | 0.7     | 0.9343                    |
Table 5. Summary of regression values based on trendline equation results for all three soils

| Soil               | R²       | Equation                        | Trendline     |
|--------------------|----------|---------------------------------|---------------|
| Bentonite-sand     | 0.9343   | y = -0.7018 x + 4.0067          | Linear        |
|                    | 0.9278   | y = 4.3137 e⁻⁰.²₅⁸x            | Logarithmic   |
|                    | 1.0000   | y = -0.5215x²+1.0785x+2.6302   | Exponential   |
|                    | 0.8499   | y = 3.3783 x⁻⁰.₉⁹⁷             | Polynomial    |
| Lampang clay       | 0.9676   | y = -0.7455 x + 3.6019          | Linear        |
|                    | 0.9602   | y = -1.108 ln(x)+2.8943        | Logarithmic   |
|                    | 0.9724   | y = 3.9076 e⁻⁰.₃₀⁸x           | Exponential   |
|                    | 0.9689   | y =0.0827x²-1.039x+3.7904      | Polynomial    |
|                    | 0.9553   | y = -2.917 x⁻⁰.₄⁵⁴             | Power         |
| MDY (-2%) optimum  | 0.9443   | y = -0.9563 x + 3.811          | Linear        |
| water content)     | 0.9643   | y = -1.41 ln(x)+2.9052        | Logarithmic   |
|                    | 0.9910   | y = 0.6904x²-3.0118x+5.2735   | Exponential   |
|                    | 0.8857   | y = 2.9266 x⁻⁰.₅₈³             | Polynomial    |
| MDY optimum water  | 0.9877   | y = -0.7884 x + 3.4717         | Linear        |
| content)           | 0.9937   | y = -1.134 ln(x)+2.7172        | Logarithmic   |
|                    | 0.9939   | y = 3.8042 e⁻⁰.₃₄³x           | Exponential   |
|                    | 0.9888   | y = 0.2244x²-1.4589x+3.9449   | Polynomial    |
| MDY (+2%) optimum   | 0.9867   | y = -0.684 x + 3.029           | Linear        |
| water content)     | 0.9568   | y = -0.91 ln(x)+2.3581        | Logarithmic   |
|                    | 0.9759   | y = 3.2938 e⁻⁰.₃₃⁵x           | Exponential   |
|                    | 1.0000   | y = -0.3044x²+0.1753x+2.4553  | Polynomial    |
|                    | 0.9389   | y = 2.371 x⁻⁰.₆₆₆             | Power         |

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