ANISOTROPIC STRENGTH CHARACTERIZATION OF COMPACTED DHAKA CLAY

By

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DECLARATION

I hereby certify that the research work reported in this thesis has been performed by me in the department of civil engineering, BUET and that this work has not been submitted elsewhere for any other purpose at any other places, except for publications.

December 2003

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ABSTRACT

A series of direct shear testing program was undertaken in the laboratory conditions upon specially treated and prepared (i.e., reconstituted) soil samples collected from different locations/ depths of Dhaka clay (i.e., one from 2m depth at Mohammedpur and the other from 6 m depth at Bijoynagar, Dhaka) to study the anisotropic strength characterization of compacted Dhaka clay. The collected soil samples were air dried, powdered by grinding and tested for the determination of index properties, optimum moisture content and maximum dry density under different compaction energy. Soil samples were reconstituted by mixing the powdered soil with predetermined amount of water. It was then thoroughly mixed and compacted in a mold by using various energies. Test samples, thus prepared as compacted soil cakes were extruded from the compaction mold. The test specimens were then retrieved at different vertical angular orientations relative to its compaction direction.

The major principal stress direction along with the application of compaction energy during preparation of the compacted soil sample was considered as the 0° (i.e., a horizontally oriented specimen). Each set of soil specimens comprising 0°, 30°, 45°, 60° and 90° orientations were prepared at different moisture contents such as optimum minus two percent moisture content (OMC-2) %, optimum plus two percent moisture content (OMC+2) % and optimum moisture content OMC %. Again each set of soil specimens was reconstituted at different compaction energy such as Standard Proctor, Modified Proctor and Intermediate compaction. Where, Intermediate compaction is defined as the energy in between Standard Proctor and Modified Proctor compaction. Direct Shear Test was conducted for each set of specimen at normal pressure of 50 kPa, 100 kPa and 150 kPa.

The existence of strength anisotropy was observed for both the soils (Dhaka Clay) collected from two locations of Dhaka city. The coefficient of anisotropy (i.e. $I = S_\theta/S_0$), defined as the ratio of strength of specimens in inclined orientation (i.e., θ is other than 0^0) to that of a horizontally oriented specimen (i.e., $\theta = 0^0$) at OMC%, varied in the range of 0.95~1.04 for Standard Proctor compaction, 1.00~1.11 for Modified Proctor and 0.96~1.13 for Intermediate compaction for Dhaka clay type-1 under normal stress of $\sigma_v = 50$ kPa. The same was varied in the range of 0.90~1.00 for Standard Proctor, 0.99~1.11 for Modified Proctor and 0.93~1.03 for Intermediate compaction for Dhaka clay type-2 under otherwise similar conditions.

The coefficient of anisotropy I varied in the range of 0.95~1.04, 0.92~1.03 and 1.00~1.05 at OMC%, (OMC+2)% and (OMC-2)%, respectively, for Dhaka clay type-1 tested under normal stress of σ_v = 50 kPa and reconstituted under Standard Proctor compaction.

The value of I varied in the range of 0.95~1.04, 0.93~1.00 and 0.98~1.03 for specimens tested under normal stress σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, at OMC% of Dhaka clay type-1 reconstituted under Standard Proctor compaction. The same was varied in the range of 0.91~1.00, 0.90~1.00 and 0.85~1.00 for Dhaka clay type-2 under otherwise similar conditions.

In short, strength of reconstituted compacted Dhaka clay is anisotropic. However, the degree of anisotropy is not substantial. Anisotropy is observed to increase slightly with the increased compacted energy. Moisture content and σ_v (i.e., normal stress) have little influence in the anisotropic response.

Keywords: anisotropy, clay, strength, compaction, reconstitution, moisture content.

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LIST OF NOTATION

θ	Orientation angle in a vertical plane
ϕ	Angle of internal friction
τ	Shear stress
$(\sigma_a - \sigma_c)$	Deviatoric stress
ε_a	Axial strain
γ_d	Dry density
τ_m	Maximum shear stress
τ_{45}	Shear stress of 45^0 orientated specimen
σ_v	Effective vertical stress
c	Cohesion
c_h	Coefficient of consolidation in horizontal direction
c_u	Undrained shear strength
G_s	Specific gravity
k_o	Earth pressure at rest
LL	Liquid limit
OCR	Over consolidation ratio
P_c	Preconsolidation pressure
PI	Plasticity index
q	Compressive stress
q_u	Compressive strength
S_0	Shear Strength of horizontally oriented specimen ($\theta = 0^0$)
S_θ	Shear strength of a specimen having θ orientation angle
SS	Simple shear
UU_{DS}	Unconsolidated undrained direct shear test
w	Moisture content
w_n	Natural moisture content
I	Coefficient of anisotropy

Chapter 1

Introduction

1.1 General

Clay, silt and sand found in nature varied widely in respect of all behaviors with their particular proportion of mixing ratio, nature of deposition, age of deposition, over consolidation, moisture content, foreign particle content, historical movements of the earth, heating and freezing. It is impossible to conduct the in-situ tests for each type of soil to characterize their parameters. Laboratory testing under similar conditions of practical field are well popular to the scientist, because development of a set of constitutive equations through this process are very useful. With the advancement of geotechnical knowledge and technology, complex geotechnical structures are being analyzed and designed more accurately, safely and economically in recent times over the world. This has been made possible due to achievement of modern soil testing equipment to meet the site conditions and enough research work on the soil, data acquisition system and powerful computing technology.

In nature, most soils are anisotropic because of their mode of deposition, the stress metamorphosis after deposition, or both and other complex geologic movements experienced by the soil in the past. In most conventional analysis of foundation problems, the influence of anisotropy is neglected claiming conservatism, which is not always true. Stress states in field under loading condition are not unidirectional, rather the magnitude of stress differs from one element to other. Besides, the principal stress direction, in soil mass under loaded conditions rotates along the potential failure surface. As a result of imposed stressed conditions, the response of the soil may vary along the potential failure surface under an embankment, for example, due to anisotropy. Their response to imposed load depends, among other factors, on the orientation of the direction of the principal stresses. Different laboratory tests will, therefore, be required to simulate the soil samples located at different portions of the failure surface.

The geotechnical problems are, therefore, three-dimensional (3-D), for which elastoplastic analyses are becoming popular. For 3-D characterization of elastoplastic deformation of geomaterials, their possible anisotropy should be properly accounted for in the following three categories:

a) inherent anisotropy, produced when deposited in air or water, or when compacted;
b) stress state-induced anisotropy, developed as the stress state becomes anisotropic; and
c) strain history-induced anisotropy, produced by dominant shear (monotonic or cyclic) strain in a certain direction.

The stability analysis of slopes stands out as the most important design aspect for embankments built of clayey soils and for natural slopes in hilly terrain. There are several methods available for stability analysis of slopes. Whatever may be the method, the analysis mostly depends on the assumption of a circular arc surface and consideration of uniform soil properties to prevail all along the length of the circular surface. Such an assumption of isotropic strength and deformation characteristics of soil in slope may be reasonable only in case of drained normally consolidated condition. But in other cases and in compacted clay mass, such an isotropic assumption may not hold rational.

While compacting an embankment layer-by-layer, the clay particles, which are platy, are brought to a dispersed structure. Dispersed structure is appreciably weaker than flocculated structures (Seed and Chan., 1961). That is, the compaction effort brings about great changes in micro-level of clay structures, which may cause anisotropy in clay in macro- level. But during the analysis of side slopes for stability, the failure arc is normally assumed to be circular. In other words, the critical slip surface shears perpendicular to the particle array in the higher reaches where the maximum resistance is offered and slides parallel to the lower reaches encountering the least resistance. This factor has not been incorporated in the design criteria for stability. It is reasonable, logical and pertinent, therefore, to study this aspect of anisotropy for compacted fill soils, as all the civil engineering works involving earthwork fall within this purview.

Substantial works on the anisotropy of soils are reported in literature, such as Oda (1972), Graham and Houlsby (1983), Ampadu (1991) and Jardine (1994). Lightly over consolidated natural clays are commonly anisotropic because of their mode of deposition. They exhibit substantial ranges of approximately linear reversible behavior at stress levels, which do not produce yielding of the particle structure of the clay (Graham and Houlsby, 1983; Jardine, 1994). Most post-glacial clays are deposited in condition, which produce varved, laminated or banded structures (Quigley, 1980). Even if they appear massively bedded, electron microscopy reveals floccculent, pedal microstructures with preferred particle orientations, which makes soil mass behave anisotropically (Oda, 1972; Baracos, 1977). The undrained strengths of such deposits vary with the orientation of the failure surface (Mitchell, 1972; Freeman & Sutherland, 1974; Graham, 1979) and are anisotropic. Because of periodic deposition of finer and coarser particle sizes, their permeability's are also anisotropic. Graham (1983) showed that the clay is approximately 1.8 times stiffer in the horizontal than in the vertical direction. Undrained strength anisotropy in natural clays has been reported by Lo (1965) and Delory and Lai (1971) on Welland clay, by Ward, W.H., Marsland, A. and Samuels, S.G. (1965) and Bishop (1966) on London clay and by Duncan and Seed (1966a) on San Francisco Bay Mud. Clays consolidated under anisotropic stress state exhibit anisotropic behavior under undrained conditions though the response of the same clay may be isotropic under drained condition.

Among many other research works done with the soils of Bangladesh are; Structure and engineering behavior of compacted soils, (Ali, M.H., 1980), Geotechnical characteristics of Dhaka clay (Ameen, S.F., 1985), Permeability and consolidation characteristics of normally consolidated clays (Siddique, 1986), Analysis and design of plain and jacketed stone column in clays (Alamgir, 1989), Compressibility and shear strength of remolded Dhaka clay (Uddin, 1990) and Strength deformation anisotropy of clays (Islam, 1999). Of them, the last one is related to the anisotropy of clay.

Islam (1999) investigated the presence of anisotropy in clay (natural clay as well as two types of reconstituted clays – compacted clay and Ko-consolidated clay). It was found that the co-efficient of anisotropy S_l/Si (strength of horizontally oriented specimen/ strength of vertically oriented specimen) varied from 0.6 to 1.25 for compacted clay, and the results were not similar for unconfined compression (UC) and unconsolidated undrained direct shear (UU_{ds}) tests. The co-efficient varied from 1.58 to 2.2 in case of triaxial test and 0.88 to 1.08 from direct shear test for the constituted Ko-consolidated clay. For natural clay it was 0.75-1.25 in UC tests, 1.0-1.55 in UU_{ds} tests and 1.0-1.16 in consolidated undrained direct shear (CU_{ds}) tests. For that research program, emphasis was given for tests on the natural and the constituted Ko-consolidated samples, However, limited tests were performed to investigate anisotropy in compacted soils, although this type of soil could exhibit more anisotropy behavior than the other naturally deposited or reconstituted Ko-consolidated samples because of the preferred unidirectional energy application during reconstitution. Moreover sufficient data were not available to establish a correlation between the compaction effort, normal load and the co-efficient of anisotropy of the compacted samples. Deformation characteristics and coefficient of permeability obtained by one dimensional consolidation tests on reconstituted clay were directionally independent in a vertical plane. Natural clay was, however, anisotropic both in deformation and hydraulic characteristics.

Therefore, research was necessary to characterize the strength anisotropy of compacted soils (as such soils are frequently used for land reclamation) found in different parts of Bangladesh and to apply the results to the analysis of various geotechnical problems (subjected to principal stress and strain rotations), such as bearing capacity of footing and raft, slope stability, retaining wall, embankment, etc.

1.2 Objectives of the Research

The research was carried out with the following objectives;

a) Study the shear strength characteristics of compacted Dhaka clay collected from two different locations and depths of Dhaka city.

b) Study the directional dependency of strength of compacted Dhaka clay of Dhaka city.

c) Study the effect of different molding moisture content on directional variation of the strength of Dhaka clay.

d) Study the effect of different compaction efforts on strength anisotropy of Dhaka clay.

e) Study the effect of different overburden pressures on strength anisotropy of Dhaka clay.

1.3 The Research work

To carry out the research work, the disturbed soil Dhaka clay type-1 samples were collected from 2 m depth at "Panthoneer" (an apartment project of Neer Ltd.) at plot No. 7/3 Aurangojeb Road, Mohammedpur, Dhaka and the Dhaka clay type-2 soil samples were collected from 6 m depth at "Mahtab Center" (a 20 storied commercial development project of Shajahan & Group) at Bijoy Nagar, Dhaka.

The disturbed soil samples were transported from the site to the laboratory. It was then dried in air, powdered by grinding and sieved through #200 sieve to remove any foreign particles before testing. Index properties were determined for each type of soil. Optimum moisture contents were determined for each type of compaction effort (Standard Proctor, Modified Proctor and Intermediate Compaction) for each type of soil using different size of mold. Each type of soil was mixed thoroughly adding water at optimum moisture content determined earlier as well as at optimum plus two percent and optimum minus two percent moisture content.

The soil samples having different moisture content were separately compacted in a compaction mold at Standard Proctor, Modified Proctor and Intermediate Compaction (varying hammer weight, blows and height of fall to obtain different compaction effort). The orientation of a sample extracted from the soil with keeping sampler axis vertical (normal to the compaction direction) is defined as $\theta = 0^0$, while that collected with the sampler axis along in the horizontal direction is defined as $\theta = 90^0$. A change in the value of θ means a change in the sample orientation in the vertical plane. Test specimens were extracted from the samples dissembled from the mold in different orientation like $\theta = 0^0$, 30^0, 45^0, 60^0, 90^0 for determination of shear strength under different normal loading condition (50 kPa, 100 kPa and 150 kPa). Different normal load (50 kPa, 100 kPa and

150 kPa) for each type of soil was applied during Direct Shear testing for considering overburden pressure effect on the soil.

Undrained direct shear test was performed on each specimen to evaluate the shear strength and deformations. If a clay mass is isotropic, strength and deformation characteristics will be independent of the values of orientation.

1.4 Organization of Thesis

The research works conducted for the achievement of the objectives aforesaid are presented in dissertation in a number of chapters. Brief description of the contents of each chapter is as follows:

Related literatures are discussed in Chapter 2, which covers the anisotropy, origin and types of anisotropy, previous works on anisotropy of both sand and clay, anisotropy in compacted clay and effect of compaction in micro-level soil structure etc.

Chapter 3 presents the sources of samples, their physical properties and classification. The experimental techniques and details of the test procedures and program used in this investigation are also described in this chapter.

Description of soil samples, procedure for preparation of soil samples and test results of Dhaka clay (type-1 and type-2) on strength anisotropy and deformation anisotropy are described in Chapter 4. Effect of compaction efforts and summary of test results are also described in this chapter.

Chapter 5 outlines the conclusions of the present investigation and recommendations for future research in this field.

Chapter' 2
Literature Review

2.1 Introduction

The objective of this Chapter is to compile the excerpts from review of the available literatures critically related to strength anisotropy. It was tried to cover the available literature related to clay, sand and compacted clay. Origins of anisotropy and types of anisotropy, clay structures and mineralogy, effects of compaction on micro level soil structure etc. are also described in this chapter.

2.2 Anisotropy

The term anisotropy is used in this research to describe the directional variations of shear strength of clay soils and other soil properties such as compressibility, permeability etc. Anisotropy of clays with respect to compressibility and strength is related to the orientation of plate-shaped clay particles. Direct shear tests were performed to find out the directional variations of strength, strain and other properties of compacted Dhaka clay collected from two different locations and depths of Dhaka city under different normal loadings, moisture contents and compaction efforts.

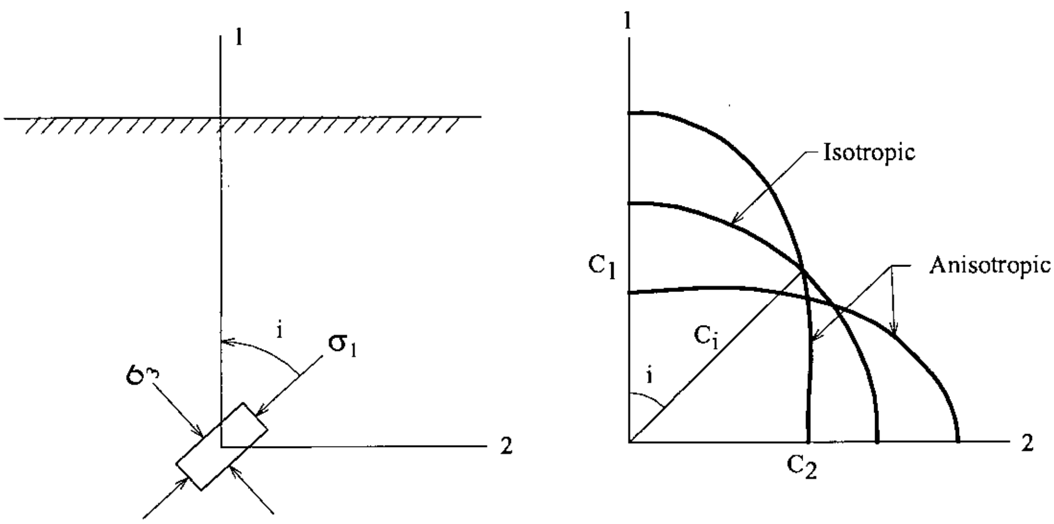


Fig. 2.1 Definition of strength variation with direction

The study showed that in some cases, the vertical strength was higher than the horizontal strength and in other cases the behavior was reverse, The samples with intermediate orientations showed that the strength could be lesser or higher than the similar horizontally oriented specimens or vertically oriented specimens.

The definition of shear strengths for different orientations is schematically shown on Fig. 2.1. The physically vertical and horizontal directions (these usually coincide, respectively, with the lines perpendicular and parallel to the bedding planes of a soil deposit) are the principle directions. If samples are tested separately with the directions of the principle stresses coinciding with the physically vertical and horizontal directions and the strengths thus determined, respectively, are termed together as 'principal strengths'.

In the Fig. 2.1, if the strength $S_i = S_1 = S_2$ (i.e. isotropic soil) is traced in a vertical plane will be a circle. For anisotropic clay, the curves of S_i can assume any other form than the circle. The ratio of the principal strength (S_1/S_2) be termed the degree of anisotropy and may be less than one, greater than one or of equal to 1.0.

2.3 Origin and Types of Anisotropy

The fundamental causes of anisotropy are yet to be understood clearly. In nature, most soils are anisotropic because of their mode of deposition, the stress metamorphosis after deposition and of the both reasons and other complex geologic movements experienced by the soil in the past.

Analytical studies by Hansen and Gibsen (1949) showed that reorientation of the principal stress directions during loading would result in variation of undrained strength along the failure plane even if the clay were isotropic with respect to all physical properties. This variation of results from the fact that the changes in pore water pressure, and thus the values of undrained strength, depends on the degree of reorientation of principal stresses, which, in turn, depends on the failure plane.

Microscopic evidence indicates that one-dimensional consolidation produces an alignment of preferred orientation in plate-like clay particles (Lambe, 1958). Preferred particle orientation has been observed in some natural clays (Mitchell, 1956, Martin 1962) but found absent from others (Quigley and Thompson, 1966) and would appear to depend upon numerous depositional environment. And probably the deviation from isotropy in the strain increment response to isotropic consolidation of vertical and horizontal specimens results from preferred particle orientation (Barden, L., 1963).

Comprehensive microscopic studies by Smart (1967) suggested that permanent shear deformations in Kaolin involve randomly oriented shear zones between packets of oriented particles. This mechanism may explain the absence of any large deviation from isotropy in the permanent strain increment and thus the shear strength.

The anisotropy of clays is intimately connected with their structure, which depends on the environmental conditions prevailing at the time of deposition and of the soil as well as the stress changes occurring in the soil subsequent to its deposition. Rosentqvist (1959) has given an excellent review of the concept of structures. He demonstrated that clays laid down in salt water acquire an open card house structure with the particles randomly oriented. In a fresh water deposit, the structure is somewhat dispersed and a certain degree of parallelism is achieved between the clay particles. It is conceivable, therefore, that in the former case the clay is more or less isotropic in a macroscopic scale, while in the latter case, the clay will possesses some inherent anisotropy because of the directional biases of particles orientations. However, it is known that consolidation under deviatoric pressures tend to align the clay particles. Under heavy overburden pressure, therefore, clay, which is initially isotropic, may become anisotropic.

Mitchell (1972) described that parallel orientation of clay particle could cause both the strength and compressibility of the clay to vary with direction. That is, it could cause the clay to be anisotropic with respect to shear strength. He concluded the following in the case of anisotropy:

i) Directional variations in the recoverable strain increment (i.e., elastic strain increment) response may occur in clay specimens, which are formed under the anisotropic stress system (also Mukabi, 1995). These variations contributed to directional variations in the undrained strength.

ii) For lightly over consolidated clays, the directional variations in undrained strength may be a function of the over consolidation ratio as well as the magnitude and direction of the applied stress path. These considerations are of particular importance when applying the results of laboratory tests to field situations.

iii) Different responses to stress reorientation (or stress rotation) have been observed in soils of different origin suggesting that there are several possible causes of deviation from isotropic behavior.

iv) Variations in the undrained strength of clay soil may also result from other factors not associated with the deviations from isotropy, such as irrecoverable pore water pressures which result, particularly in sensitive clays, from cyclic loading (Sangrey, D.A., Henkel, D.J. & Estrig, M.J., 1969).

v) In addition to the well-known stress history effect, the behavior of soils may also exhibit a dependence on the direction of previous applied stresses. Similar dependence in metals, known as the Bauschinger effect, is attributed to residual stresses mainly due to the different states of stress existing, on the microscopic scale, in oriented crystals (Hill, 1950). If residual stresses exist in an anisotropically consolidated specimen, it is expected that the magnitudes of subsequent applied stresses necessary to cause yield will be a function of the directions of these applied stresses.

In some natural clay, which appears randomly oriented under microscopic examination, it is possible that residual stresses will be present after external stress relief because of the cementation of the particles under the in-situ anisotropic stress system. Mitchell (1970) has reported deviations from isotropy in the results of fully drained triaxial compression tests on cemented Leda clay from Ottawa. Such deviations would appear to originate from causes other than preferred particle orientation, which is cementation.

Baracos (1977) studying the Winnipeg clays by optical and x-ray analysis concluded that varving, layering, marbling, inclusions and differential shrinkage are the causes of strength variation among specimens of different orientations.

2.4 Review of Previous Works

The objective of this section is to examine the results of previous investigations that are related to the influence of anisotropy and reorientation of principal stresses on undrained strength. The studies made by various investigators consist of experimental and analytical investigations of different types. A large amount of literature is available on directional variation in strength devoted to in-situ soils and to laboratory samples under controlled mineralogy. Anisotropy of sand is briefly described, which is followed by the behavior of clay in detail.

2.4.1 Anisotropy in Sand

Ko and Scott (1967) and Siddiquee (1994) showed that when subjected to hydrostatic compression, sand specimens produced by air-pluviation exhibit strains in the lateral/radial direction that are larger than those in the vertical direction. Many researchers have showed inherent strength anisotropy in sand specimens. For air-pluviated specimens of sands tested in drained triaxial or plane strain compression, the strength of a specimen with $\theta = 0°$ (i.e., where the pluviation direction and the major principal stress direction are coinciding) was always stronger than that of a specimen with $\theta = 90°$, while specimens with $\theta = 0^0$ had the largest pre-peak stiffness (Arthur and Menzies, 1972; Oda, 1972a, 1972b and 1981; Symes et al., 1984; Tatsuoka and Shibuya, 1992; Park and Tatsuoka, 1994). In parallel to the above, Oda (1972a, 1972b) and Symes (1983) showed that there was a strong horizontal bias to the long axis directions of sand particles and the directions of inter particle contact planes, which was considered to make an air or water-pluviated specimen stiffer and stronger when compressed in the vertical direction.

Inherent anisotropy in elastic deformation characteristics, however, is not quite well understood. Based on wave velocity measurements, Stokoe et al. (1991), Lo Presti and O'Neill (1991) and Bellotti et al. (1994) showed that elastic deformation characteristics of granular material produced by pluviation in air are inherently anisotropic with the elastic Young's modulus in the horizontal direction being always larger than that in the vertical direction. These results are apparently not consistent with the anisotropy in pre-peak deformation characteristics and peak strength as described above. Recently, by using a sophisticated triaxial testing system Hoque et al. (1996, 1997) and Hoque and Tatsuoka (1998) showed rather consistent results on the anisotropic behavior of granular soils.

2.4.2 Anisotropy in Clay

Lightly over consolidated natural clays are commonly anisotropic because of their mode of deposition. They exhibit substantial ranges of approximately linear, reversible (elastic) behavior at stress levels, which do not produce yielding of the particle structure of the clay. The undrained strengths of such deposits may vary with the orientation of the failure surface.

Wolf (1935) is usually accepted to be the pioneer to consider anisotropy of the soil. Bishop (1948) extracted samples of London clay from an inclined borehole. The axes of the samples were presumably inclined at about $30°$ to the horizontal. The samples were reconsolidated in an oedometer and tested in unconfined compression. The strength of the inclined samples was 28% less than the strength of vertical samples.

Analytical studies by Hansen and Gibsen (1949) showed that reorientation of the principal stress directions during loading would result in variation of undrained strength orientation of the failure plane even if the clay was isotropic with respect to all physical properties. This variation results from the fact that the changes in pore water pressure, and thus the values of undrained strength, depend on the degree of reorientation of principal stresses which, in turn, depends on the failure plane. At the same value of consolidation pressure the strength of the horizontal plane strain sample was only about 75% of the strength of the vertical plane strain sample.

Jacobson (1955) cut samples of Swedish post-glacial marine clay from approximately 3m below the top of a slope. Vertical (θ =0°), inclined (45°), and horizontal (θ =90^0) samples were taken alternately in a line; altogether 12 vertical, 12 inclined, 10 horizontal samples were tested. The greatest average difference between the strengths of two types of samples was 14%.

Ward (1956) showed that the undrained shear strength of one-dimensionally consolidated clay might vary with orientation of the failure plane by performing experimental studies.

Ward et al. (1959) performed unconsolidated undrained triaxial tests on horizontal and vertical samples of London clay from different locations. More than 130 samples were cut from blocks trimmed out of the walls. The horizontal samples were from 15% to 39% stronger than the vertical samples. The only inclined (45°) samples tested were 1% weaker than the vertical samples.

Hvorslev (1960) conducted tests on samples trimmed vertically, horizontally and at an angle of 45° from Vienna and Little Belt clays consolidated one-dimensionally. In Vienna clay, the strength of the inclined and horizontal samples was found to be lower by 8 to 13 % respectively, than the strength of the vertical samples. For the other clay, the strength of the inclined and horizontal samples was higher by 8 to 20 % respectively, than that of the vertical sample. It is surprising that two clays with similar stress histories, tested by the same investigator, had different trends:

Skempton (1961) showed that the horizontal strengths were higher than the vertical strengths and that the ratio of the horizontal/vertical strength was 1.3±0.1. For the purpose of investigating strength variation with direction, tests were performed with the major principal stresses inclined to the physical vertical axes at 0°, 15°, 30°, 45°, 60°, 75°, and 90°. The decrease in strength with the angle of rotation of the major principal stresses was evident. The ratios of the undrained shear strengths (horizontal/vertical) varied from 0.80 to 0.64 for the entire block samples tested. The clay deposit was lightly over consolidated with an over consolidation ratio of 2.0.

Tan (1961) reported that in China a great number of natural clay deposits are strongly anisotropic and in general the degree of anisotropy was equal to or greater than 3.0.

Lo (1965) performed unconfined compression tests on undisturbed samples of a clay from Wellard, Ontario, Canada. Block and tube samples taken from the wall of an accessible boring, were trimmed horizontally, vertically and inclined, while the inclination values of samples ranged from 0° to 90° in increments of 15°. The horizontal samples were found to be 20% to 36% weaker than vertical samples from the same location.

Bishop (1966) summarized the laboratory and field tests of several investigations on anisotropy of shear strength and stressed the importance of anisotropy and variation in undrained strength with depth, in the solution of practical problems. The effect of over- consolidated clays and difficulties in interpretation of laboratory data were brought out.

Duncan and Seed (1966) performed unconsolidated-undrained triaxial tests trimming samples with their axis vertical, inclined to 45°, and horizontal using laboratory prepared Kaolinite clay. The ratio of the peak deviatoric stresses i.e., inclined/vertical was 0.75 and that of horizontal/vertical was 0.87.

Mitchell (1970) reported deviations from isotropy in the results of fully drained triaxial compression tests on cemented Leda clay from Ottwawa. Such deviations would appear to originate from causes other than preferred particle orientation, which was cementation.

D'Appolania (1972) found that the coefficient of anisotropy, i.e. the ratio of the undrained shear strength on horizontal samples (S_{uh}) to that of the vertical samples (S_{uv}) of one-dimensionally consolidated Boston Blue Clay and Gunridate were varying from 0.6 to 1.1. The ratio depended on over-consolidation ratio (OCR.) and increased with increase in OCR.

Mitchell (1972), defining a vertical specimen as one in which the major principle stress was applied in the same direction during the shear test, arrived at the following general conclusions:

i) The undrained strength, or total stress difference at failure in undrained tests, was highest for a vertical specimen and the lowest for a horizontal specimen (trimmed at 90° to the vertical specimen) with the difference as much as 30%. Specimens trimmed at other angles had undrained strengths between the limits defined by the vertical and horizontal specimens and higher the over consolidation ratio, higher the degree of anisotropy.

ii) Failure envelops in terms of effective stress (expressed, say, as a function of c' and ø') did not vary significantly between specimens trimmed in different directions.

iii) The difference in undrained strength was the results of the differences in the pore water pressure developed during the shear tests and related uniquely to the angle of rotation of the principal stress directions for a given sample of clay and were in some way related to the preferred particle orientation.

Bhaskaran (1974) showed a coefficient of strength anisotropy varying from 0.89 to 1.14 on laboratory-prepared Kaolinite clay samples and also showed the variation is dependent upon maximum past pressure and O.C.R. The macro and micro fabric features also influenced the ratio S_{uh}/S_{uv} to a certain degree while testing under in-situ stress isotropy.

Loh and Holt (1974) performed unconfined compression tests on lacustrine clay from Winnipeg, Manitoba and showed that both the undrained shear strength and the normalized secant modulus of undisturbed clay were anisotropic. The fabric determined by X-ray diffraction analysis was also found to be anisotropic in the undisturbed samples. The same material in the remolded condition was isotropic with respect to both undrained shear strength and fabric.

Krizek et al. (1977) consolidated high water content slurries of Kaolin clay with two different pore-fluid chemistries (flocculated and dispersed) under isotropic and anisotropic stress-state to produce bulk samples (blocks with dimensions on the order of 150 to 200 mm) of clay soils with widely differing micro-fabrics. Then, cylindrical specimens with dimensions of 30 or 40 mm were trimmed in orthogonal directions from these block samples and tested for undrained creep at different stress levels. The macro-behavior of these specimens was interpreted in terms of rate process theory (rate process theory employs the concept of statistical mechanics to describe time-dependent soil behavior), and the parameters in the formulation were correlated with quantitative measures of fabric. For a given stress intensity, specimens trimmed in the vertical direction were found to exhibit lower strain rates and larger creep deformations than specimens trimmed in the horizontal direction.

Toh and Donald (1979) reviewed briefly the methods for measuring anisotropy of undrained shear strength in soft clays and attempted to explain the apparent anomalies. Using the test data on Launceston clay, the influence of anisotropy on stability calculations was exemplified by a finite elasto-plastic analysis of load-deformation behavior of a flexible strip footing. It showed that the bearing capacity increased from a range of 0.66 to 1.23 kg/cm2 if the anisotropy in strength was considered.

Ranganatham and Sreenivasulu (1979) developed an analytical solution for the stability of cut slopes for $ø_u = 0$ condition. The variations of undrained strength with depth and direction were incorporated in the analysis. Strength increase with depth was shown to have greater effect on improving the stability than a corresponding increase in the value of anisotropy. Consideration of average strength resulted in conservative estimate of the slope height to the tune of about 10 %.

Memon (1980) presented the results of laboratory triaxial and in-situ vane shear tests with various rectangular and triangular vane shapes and showed that the undrained strength of Nong Ngoo Hao clay varied widely depending on the direction of shearing of failure. The ratio of S_{uh}/S_{uv} was found to be lower than 1.0 throughout the depth tested and the undrained shear along 30°, 45° and 60° sample orientations corresponded approximately to an elliptical variation in a polar diagram.

Graham and Houlsby (1983) performed triaxial tests on Lake Agassiz clay from Winnipeg, Canada and examined five elastic parameters needed to describe transverse isotropy, sometime called cross anisotropy. They have showed that the clay was approximately 1.8 times stiffer in the horizontal than in the vertical direction.

Nanda and Kuppusamy (1992) developed an Elasto-plastic model with isotropic hardening for initially anisotropic undrained clays. The influence of anisotropy on the deformations and yielding was investigated. It was observed that the anisotropy of the clay had a significant influence on the deformation and ultimate load of the footing. For the range of anisotropy analyzed, the anisotropic ultimate load for the strip footing can vary over 70% of the isotropic ultimate load. The anisotropic ultimate load for the rectangular footing can vary over 62% of the isotropic ultimate load, and for the square footing, the anisotropic ultimate load can vary over 55% of the isotropic ultimate load. The anisotropic elastic displacement can vary over 50% of the isotropic elastic displacements for all footings. The influence of anisotropy on the surface profile was not significant. However, the hardening parameter had a large effect on the determination or the footing. The orthotropic ultimate load could vary over 40% of the cross-anisotropic ultimate load.

W.S.Freeman and Hugh B. Sutherland (1973) investigated four types of clay and carried out stability analysis. Factor of safety was found for the conventional method of analysis assuming circular arc failure surface and isotropic shear strength properties. These factors of safety were found to be up to 0.5 greater than those obtained from non circular arc failure surfaces which partially passed along the layers and so mobilized the low shear strength properties of these layers.

A Baracos (1976) examined samples of Winnipeg tan silt, brown clay, grey clay, and grey plastic clay using X-ray diffraction and the scanning electron microscope. Freeman had shown all these soils are anisotropic in shear strength and permeability.

Baracis found the presence of non-clay minerals in varves, veins, and inclusions in otherwise clayey material was a cause of anisotropy. Clay particle horizontal orientation and the presence of horizontal planes of uniformly graded silt are further reasons for anisotropy.

J. Graham (1978) has shown in a circular arc total stress stability solution for anisotropic soils that embankment stability on soft clays is strongly influenced by the properties of the weathered crust, and particularly by the shear strength anisotropy.

Islam and Hoque (1999) performed laboratory tests on specially prepared specimen of clay soil in undisturbed, reconstituted and compacted conditions to study the stress and deformation anisotropy and concluded the following;

Compacted clay:

1) The existence of strength anisotropy was clearly noticeable in reconstituted compacted clays irrespective of molding moisture and compaction effort. The observed anisotropy was however stress induced type.
2) The coefficient of anisotropy, defined as the ratio of strength of vertical specimen to that of a horizontal specimen, varied from 0.75 to 1.25 in unconfined compression and unconsolidated direct shear tests, while it was from 0.6 to 1.08 in unconsolidated direct shear tests on soaked samples.
3) Effect of compaction effort on anisotropy was not similar to UC, UU_{DS} and UU_{DS} tests on soaked samples. In UC tests, both coefficients of anisotropy, S1/S2 and S1/S3 (where S3 is the strength of $\theta = 45^{\circ}$ oriented specimen), were increased with the increase of input energy during compaction. In UU_{DS} tests, the value of S1/S2 increased, but S/S3 was decreased with the increase of input energy. But in UU_{DS} tests on soaked samples, both the coefficients were decreased with the increase of input energy.
4) Test specimens in UC tests were observed to fail at axial strains less than 5% with a sharp decrease in stress in the stress-strain curve. Failure mode was shear type with developing distinct cracks passing diagonally though test specimens.

Reconstituted clay:

1) Under similar stress conditions and histories, the undrained strength of the specimens trimmed at different orientations from horizontal plane was similar, as they had been reconstituted uniformly and homogeneously. That means, the strength of reconstituted clay mass was isotropic in a horizontal plane. For K_o-consolidated (vertically) clay mass, the undrained strength was not, however, similar for specimens trimmed from a vertical plane at different orientations.
2) Due to stress induced anisotropy, triaxial specimens trimmed (at consolidated undrained condition) from vertical plane exhibited the coefficient of anisotropy as high as 1.58 (in terms of total stress) and 2.2 (in terms of effective stress). Under similar conditions in direct shear tests, the coefficients of anisotropy were $c_v/c_h = 1.08$ and $c_v/c_{45} = 0.88$

Field/ Natural clay:

1) Field clay was reasonably isotropic in undrained strength in a horizontal plane.
2) Undrained strength became anisotropic as the major principal stress direction changed gradually from the depositional direction to the normal to the depositional direction.
3) The coefficient of vertical anisotropy was more pronounced in filed clay.

Consolidation behaviour:

1) In reconstituted Ko-consolidated specimens, the deformation indices (c_c & Cr) were directionally independent in a vertical plane. Preconsolidation pressure evaluated by empirical method was also isotropic for different θ values although the clay had been consolidated vertically during reconstitution. It was due to the fact that the clay had been behaved like fluid material during reconstitution under vertical pressure as the moisture content of the wet clay had been above the liquid limit of clay material. The coefficient of permeability (k) was, however, more or less isotropic for a given load increment.

2) Natural clay was anisotropic both in deformation and hydraulic characteristics. The indices Cc and Cr were maximum in the vertical direction. But the value of k in horizontal direction was higher than that in vertical direction.

2.5 Anisotropy in Compacted Clays

Although a large amount of literature is available, as mentioned earlier, on directional variation in strength, it is solely devoted to insitu soils and to laboratory samples under controlled mineralogy. Literature on compacted soils in relation to anisotropy, inspite of wide application of mechanical and chemical stabilization methods and hectic activity of earthwork projects all around, is scarce. Although the concept of anisotropy has been a subject of intensive research for decades with regard to the multi-directional strength variation and the influence of mineralogy and stress history on anisotropy, studies on its influence in compacted clay fills have not been the focus of systematic and scientific attention, barring a few desultory attempts. The basic difference between the anisotropy in naturally occurring soils and that in man-made earthwork projects is that the former is predominantly a anisotropy while the latter is a stress-induced one.

Rao and John (1982) investigated the role of anisotropy in compacted soils and quantified the directional variation in shear strength based on comprehensive laboratory testing programme. Twelve samples of different plasticity indices were used for preparation of specimens of $\theta = 0^0$, 30^0, 45^0 and 90^0 orientations and were subjected to direct shear testing. Compactive effort and moulding water content were varied to study their influence on anisotropy qualitatively. They concluded that maximum and minimum strengths for vertical and horizontal specimens, respectively, varied from 15 - 40 %. The strength of other inclined samples lie, by and large, between those vertical and horizontal specimens. The coefficient of anisotropy increases with the decrease in Plasticity Index value, and modulus of elasticity decreased with the increase in coefficient of anisotropy.

2.6 Clay Structures and Mineralogy

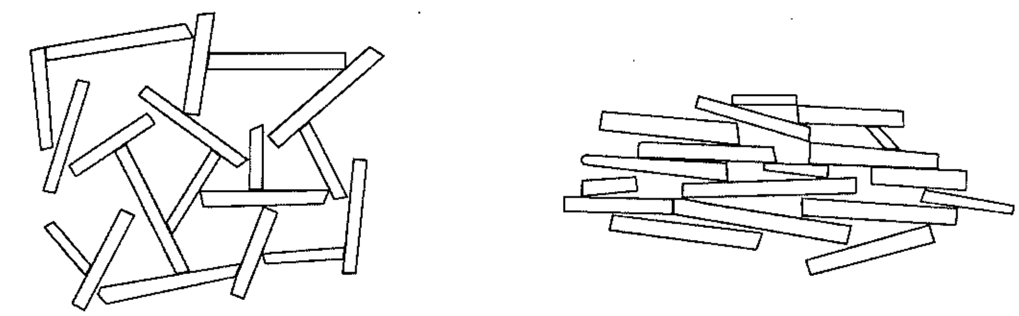
For many years, soils structure has been recognized as having a fundamental influence on the engineering properties of clay soils. Structure is generally regarded as consisting of two components;

a) Fabric, which is the geometrical arrangement of the particles.

b) The nature and strength of the bonds acting between these particles. Clay structures are mainly of two types, which are,

1) Flocculated Structure

Flocculated structure occurs in clays. The particles have large surface area and, therefore, the electrical forces are important in such soils. The clay particles have a negative charge on the surface and a positive charge on the edges. Interparticle contact develops between the positively charged edges and the negatively charged faces. This results in a flocculated structure, Fig. 2.2a



(a) Flocculated structure (b) Dispersed structure

Fig. 2.2: Soil structure in clay

Flocculent structure is formed where there is a net attractive force between particles. When clay particles settle in water, deposits formed have a flocculated structure, The degree of flocculation of a clay deposit depends upon the type and concentration of clay particles, and the presence of salts in water, clays settling out in a salt-water solution have a more flocculent structure than clays settling out in a fresh water solution. Salt water acts as an electrolyte and reduces the repulsive forces between the particles

Soils with a flocculent structure are light in weight and have a high void ratio and water content. However, these soils are quite strong and can resist external forces because of a strong bond due to attraction between particles. The soils are sensitive to vibration. In general, the soils in a flocculated structure have a low compressibility, a high permeability and high shear strength.

2) Dispersed Structure

Dispersed structure develops in clays that have been reworked or remoulded. The particle develops more or less a parallel orientation (Fig 2.2b). Clay deposits with a flocculent structure when transported to other places by nature or man get remoulded. Remoulding converts the edge-to-face orientation to face-to-face orientation. The dispersed structure is formed in nature when there is a net repulsive force between particles.

The soil in dispersed structure generally has a low shear strength, high compressibility and low permeability. Remoulding causes a loss of strength in a cohesive soil. With the passage of time, however, the soil may regain some of its lost strength. Due to remoulding, the chemical equilibrium of the particles and associated adsorbed ions and water molecules within the double layer is disturbed. The soil regains strength as a result of re-establishing a degree of chemical equilibrium. This phenomenon of regain of strength with the passage of time, with no change in water content, is known as thixotropy.

Quigley and Thomson (1966) described the relationship between the anisotropic consolidation characteristics of sensitive marine clay by using x-ray diffraction techniques. Leda clay was used because it was thought to have a "card-house" type of fabric (Lambe, 1953) which would undergo marked particle orientation when consolidated. They showed the fabric of marine clay at different stage of anisotropic consolidation (Fig. 2.3)

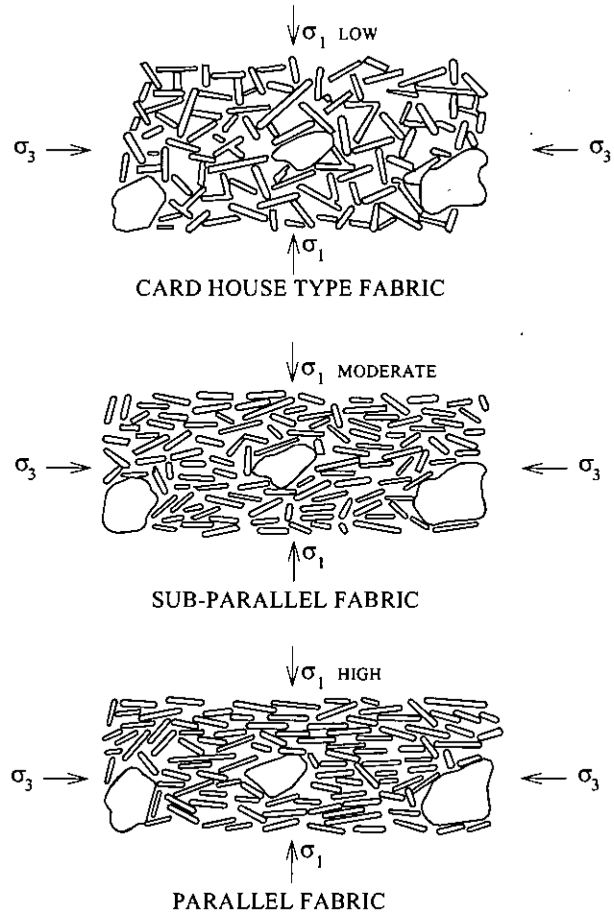


Fig.2.3: Fabric of Marine Clay at different stages of anistropic consolidation
(after, Quigley and Thompson, 1965)

2.7 Methods of determining Soil Fabrics

Clay fabric has much influence on anisotropic behavior of clay (Quigley, 1971; Barden, 1972; Kirkpatric and Rennie, 1972). During past few years, several semi-quantitative techniques for measuring soil fabric have been developed and studies into the correlation between soil fabric and engineering properties have met with considerable success. For example, petrographic techniques have been used and reported by many researchers including Rosenqvist (1955), Mitchell (1956), Seed and Chan (1961), and others. Buessem and Nagy (1953), Silverman and Bates (1960), Quigley (1961), Martin (1962), Olsen (1962), and others have reported X-ray diffraction techniques. Other ingenious techniques for observing or speculating on fabric have also been reported. For example, Rosenqvist (1959) studied the clay fabric of sensitive marine clays by means of an electron microscope and Penner (1963) used relative thermal conductivity techniques checked by photographic methods.

2.8 Effect of Compaction on Micro level of Soil Structure

The structure of a compacted soil depends upon the type of soil, molding water content, and the type and amount of compaction. Since this research is devoted to clay soils, the effect of compaction on clay soil is discussed here.

The arrangement of the clay particles in a compacted soil, as originally conceived by Lambe (1960a, b) is illustrated in Fig. 2.4 and further modified by Seed and Chan (1961). The changes in arrangement of particles at different stages of the density-water content relationship are explained as follows;

- At point A, the small amount of water present results in a high concentration of electrolyte which prevents the diffuse double layer of ions surrounding each clay particle from developing fully. The double-layer depression leads to low inter-particle repulsion, resulting in a tendency towards flocculation of the colloids and a consequently low degree of clay-particle orientation in the compacted soil. This type of structure has been termed as flocculated arrangement (Rutledge and Osterberg, 1948) of soil particles having lower density.

- If the water content is increased to point B, the electrolyte concentration is reduced, resulting in an expansion of the double layer, increased repulsion between particles and a low degree of flocculation, i.e. an increased degree of particle orientation and give higher density.
- Further increase in water content at point C increases this effect and results in a still greater increase in particle orientation. A system of parallel particles, which is approached at point C, has been termed as dispersed system (Rutledge and Osterberg, 1948). However, the density will be decreased because the added water will dilute the concentration of soils percent volume.

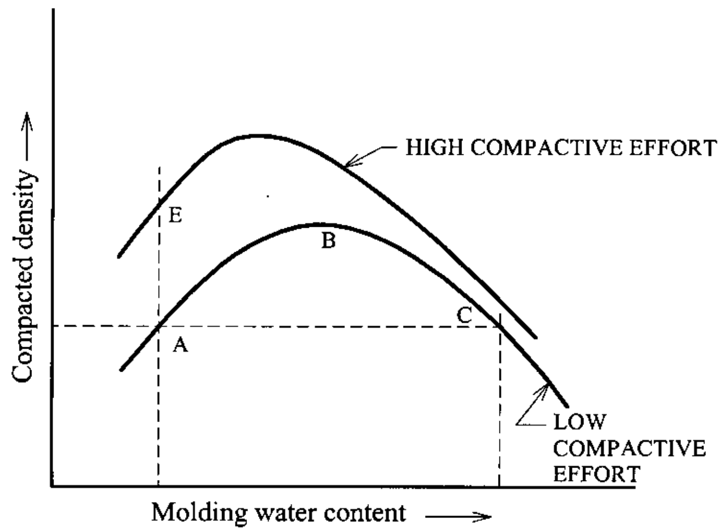


Fig. 2.4 Effect of Compaction on Soil Structure (Lambe, 1960)

Thus, in general it may be stated that compaction of a clay soil dry of optimum tends to produce a flocculated arrangement of particles, while compaction of the same soil wet of optimum tends to produce a dispersed arrangement of particles.

At a given moisture content, higher compactive effort tends to give a more parallel orientation to the clay particles, thereby giving a more dispersed structure, the particles are closer together and the soil becomes denser; in this connection, point A can be compared with point B in Fig. 2.4.

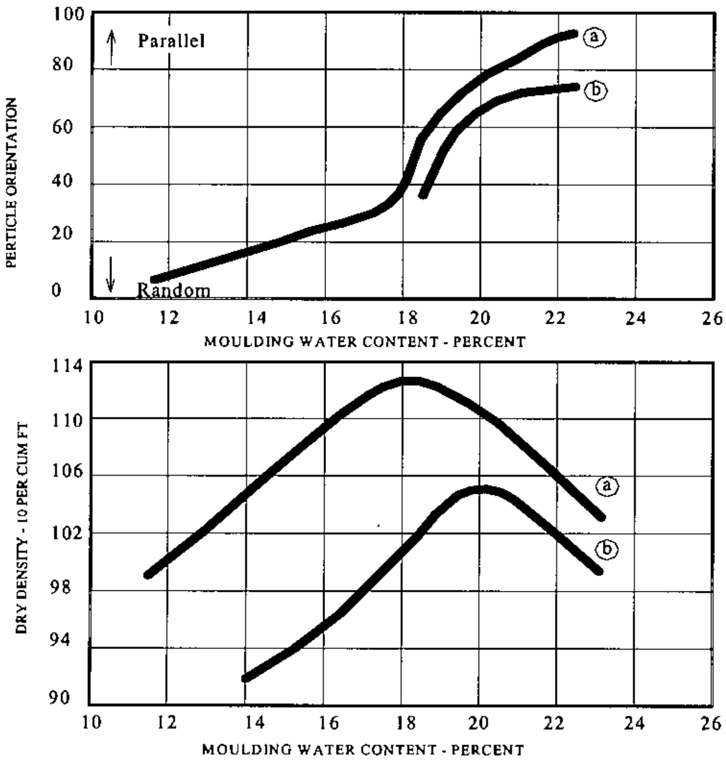


Fig.2.5: Orientation vs. water content for Boston Blue Clay
(After T.W Lambe, 1958)

Figure 2.5 shows the variation of degree of particle orientation with molding water content on compacted Boston Blue Clay (Pacey, 1956). Works of Seed and Chan (1961) have shown similar results.

Differences in structure resulting from compaction can have a pronounced effect on the engineering properties of a soil such as permeability, compressibility and strength. Macrae (1952) showed the theoretical soil structure for wet side and dry side for the three groups as shown in Figure 2.6. This picture could be modified as degree of dispersion to account for variations in the amount of shear strain when using various methods of compaction. The soil structure of group A as shown in figure 2.6 is not affected by initial water content or method of compaction. The structures of other two groups, named group B and C are affected by initial water content and method of compaction.

Dry side

Wet side

(a) Group A (GW, GP, SW, SP)

Dry side

Wet side

(b) Group B (GM, GC, SM, SC)

Dry side

Wet side

(b) Group B (ML, CL, OL)

Dry side

Wet side

Group C (MH, CH, OH)

Fig. 2.6: Compacted soil structure (after, Mcrae, 1952)

Seed and Chan (1961) showed that the more flocculent structures produce an initially steep stress-strain curve with only a slight increase in stress being developed after approximately 5% strain, however, the more dispersed structures result much flatter curve with a consistent increase in resistance to deformation upto strains as high as 25% (Fig. 2.7).

2.9 Effect of structure on Stress-Deformation Characteristics of Compacted Clays

Difference in structure resulting from compaction can have a pronounced effect on the engineering properties of soil (Lambe, 1960a) i.e. the structure of compacted clay can have a significant effect on the stress versus deformation characteristics of a soil determined from undrained tests.

Evidence of this can be shown from the study of Seed and Chan (1961) for samples tested in compacted condition as shown in Figs 2.7 and 2.8 which present test data for samples of Kaolinite compacted at various water contents using the same compactive effort. It is seen (Fig 2.5) that as the water contents increases, there is a progressive increase in the degree of particle orientation. Associated with the changes in structure there is also a progressive change on the form of the stress- strain relationships for the sample determined by unconsolidated – undrained triaxial compression tests (Fig. 2.7). The more flocculated samples have much steeper stress-strain curves (i.e., brittle behavior) and develop their maximum strengths at low strains. The more dispersed samples have much flatter stress-strain curves and continue to increase in strength even at high strains (ductile behavior). Similar results of silty clay were obtained by Seed and Chan (1961).

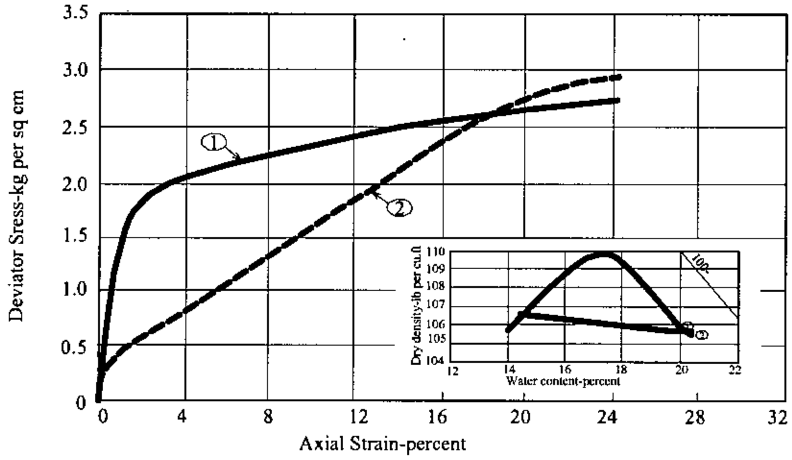


Fig.2.7 Influence of Molding water content on structure for compacted Sample of Kaolinite (after Seed and Chan, 1961)

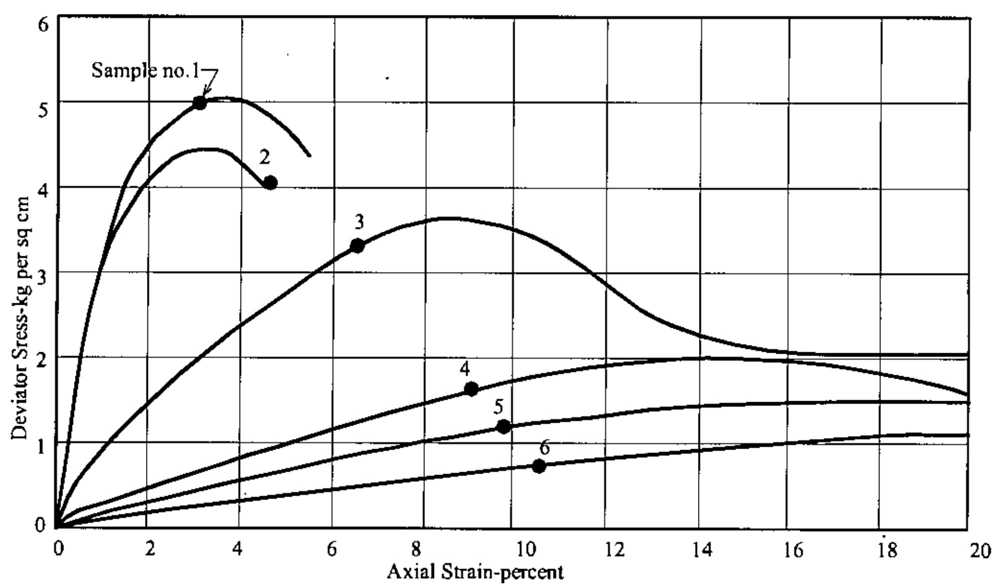


Fig.2.8: Stress versus strain relationship for sample of Kaolinite (after Seed and Chan, 1961)

2.10 Influence of Structure on Undrained Strength of Compacted Clay

Clay structures have pronounced effect on undrained strength of compacted clays. Evidence of this was made by Seed and Chan (1961). They performed unconsolidated undrained triaxial tests on two specimens of silty clay having the same density but different water content as shown in Fig. 2.7. Both specimens, although having quite different structures and stress-strain relationships, reached about the same ultimate strength at approximately 25% strain. Thus it might be claimed that structure has practically no influence on soil strength in undrained tests. On the other hand, this conclusion would be quite erroneous if the strength of the samples were compared at some limiting strain, perhaps 10% at which stage the more dispersed structures is appreciably weaker than the flocculated structure.

The influence of structure on the undrained strength of soils thus depends on the deformation criterion adopted a basis for strength determination. In compression tests on specimens having plastic stress-strain characteristics the strength is often taken as the stress causing approximately 20% strain. On the other hand, in pavement design tests the strength index of a soil is usually determined at low strains of the order of 5%. Thus, engineers concerned with pavement design problems are likely to conclude, from the data presented in Fig. 2.7 or similar data obtained in pavement design tests, that structure has a pronounced influence on the strength of compacted soils, with dispersed arrangements producing much lower strength than flocculated arrangements. On the other hand, engineers concerned with testing soil for foundation studies or earth dam design would like to determine strengths at large strains (approximately 20%). If the specimen has not yet reached its maximum resistance at that point, they are likely to conclude that structure has little or no influence on soil strength. These apparently conflicting conclusions can readily be reconciled by a consideration of the influence of structure on the entire stress-strain relationship for compacted clay.

2.11 Undrained Strength of Compacted Clays after Soaking

For clays, which become saturated after compaction, interpretation of the relationship between composition and strength is somewhat simplified by the fact that the density and water content are related and knowledge of one determines the corresponding value of the other. Thus the important compositional factors determining pore-water pressures and soil strength (dry density, water content and structure) are, in effect, reduced to the final water content and the soil structure.

On the other hand, the problem of determining these two factors becomes more complex than for soils in the as-compacted condition. The final water content of compacted clay after a period of soaking depends on the initial structure (which in turn, depends on the method of compaction, the compacted density, and the water content at compaction and influence the swelling characteristics of the soil) and on the surcharge pressure. The stress-strain relationship of the soil in undrained tests depends on this final water content (and the corresponding dry density), on the structure after swelling or soaking has ceased (which influences the pore water pressures developed during loading), and on the change in structure as the sample is deformed during loading. Finally, the strength depends on the strain criterion selected as a basis for determining failure and on the form of the stress-strain relationship. In view of the number of variables involved, careful consideration is required in attempting to predict the influence of initial composition on the undrained strength of compacted clays under these conditions.

2.12 Dhaka clay and its Geology

As mentioned earlier that only Dhaka clays collected from two different locations and different depths of Dhaka City, was used in this research work.

Sediments washed down from adjacent highlands such as Himalayas, where the slopes are steep have formed the Bengal Basin. The greater part of this land building process has been due to the sediments carried by the Ganges and the Brahmmaputra rivers. Morgan et al. (1959) classified the geological age of Bangladesh as recent alluvium and older Pleistocene sediment. The Pleistocene sediments are flood plain deposits of earlier Ganges and Brahmmaputra. They are also indications of differential movement of these Pleistocene deposits.

Dhaka stands on the southern part of Modhupurgar, which is formed by older Pleistocene sediments. It is at an elevation of 7m to 9m above the mean sea level. In general, top layer of the formation up to a depth of about 7m to 8m is a mixture of silt and clay. Deposits of sand and gravels occur at relatively deeper horizons with a sequence of finer material at top and coarser materials downward. The consistency of the top layer is medium to stiff and the soil is overconsolidated. A description of soil profile over Dhaka is provided by Eusufzai et al. (1970).

2.13 Practical Application of Anisotropy in the Stability Analysis of Slopes

Stability analysis of slopes depends on the properties of the soils, which are often complex. Factors affecting the stability of slopes are including the increase of strength with the depth, OCR, pore water pressure response and anisotropy.

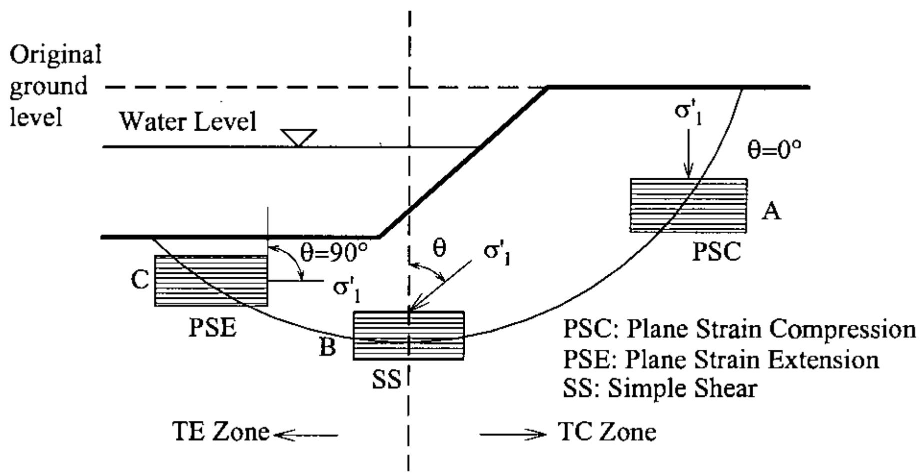


Fig.2.9: Modes of deformation along a potential failure surface

For anisotropic example, failure surface under a slope is shown in fig. 2.9. Due to imposed stress conditions, response of soil may vary along the potential failure surface because of stress reorientation. That is, the major principal stress compared to its physical vertical direction varied from $\theta = 0^0$ (vertical specimen) at tip to $\theta = 90°$ (horizontal specimen) at the toe of the slope. As a result, different laboratory tests will, therefore, be required to simulate the different stress conditions. Necessity of proper simulation of field stress condition increases if the strength characteristics vary with the reorientation of major principal stress directions (i.e., if the soil is anisotropic in strength). Specimens at location A and C are to be simulated, respectively by shear tests to simulate the specimen at B.

The anisotropy in strength is not considered in the conventional methods of stability analysis of slopes. As a result, under-estimation of strength leads to uneconomical/ over designed structures. In order to highlight this aspect, a field problem of design of a slope was worked out by Shanmukha et al. (1982) and the improvement in the factor of safety taking anisotropic effect was illustrated. The strength actually available was not mobilized at the design stage. The shear strength of soil samples varied between vertical and horizontal orientations.

A 15m high embankment made up of compacted clay having $c' = 0.25$ kg/m^2. The shear strength was presumed to be 15% higher in the orientation of 90^0 to 45^0. 10% higher from 45^0 to 30^0orientation and no increase beyond the slip surface tends to be horizontal. The assumptions were very conservative, in spite of many test specimens having strength variation of 30-40%.

They calculated the factor of safety in both dry and saturated conditions considering both anisotropy and without anisotropy. Factor of safety in dry condition without considering anisotropy was 1.427 and considering anisotropy was 1.507 and in saturated condition those were 1.063 and 1.121, respectively. Thus the factor of safety improved by 6-8% in each of the cases. It is, therefore, possible to get 10-15% higher values of factor of safety by incorporating actual available strength. Lo (1965) also presented similar results quantitatively.

In the above analysis, the strength anisotropy S_1/S_2 was assumed to be greater than 1.0. The assumption may not be true in many cases, where S_1/S_2 could be less than unity. In such a case, the factor of safety will decrease instead of improving.

Besides Lo (1965) computed stability index of slope for a practical range of degree of anisotropy and slope angle. These numerical results show that for steep slopes, the effect of anisotropy is small. However, for flatter slopes, the influence of anisotropy on the stability condition is significant.

Chapter 3

Testing Program and Procedure

3.1 Introduction

The two types of Dhaka clay collected from two prime locations in Dhaka City are described in this chapter. The routine tests performed for determining the index properties of the two types of Dhaka clay are described in brief. The desired laboratory-testing program of these two types of Dhaka clay for finding strength anisotropy and procedures are also described on detail. All tests were performed following the procedures specified in ASTM D854 (1985), BS1377 (1975) and ASTM D422 (1972).

3.2 Dhaka Clay Type-1

It is known from the geology of Dhaka that there is a clay layer at the top surface up to a certain depth called 'Dhaka clay' and the immediate bottom layer is medium dense to dense sand, which is following a layer of dense sand. One sample of Dhaka clay (type-1) was collected from a depth of 2 meter at "Panthoneer" (an apartment project of Neer Ltd.), plot No. 2/1, Aurangojeb Road, Mohammedpur, Dhaka. The soil was collected as disturbed, dried in air, powdered by grinding and sieved through #200 opening for obtaining the test soil. Routine tests were performed and the test results are tabulated in Table 3.1. Grain size curve is shown on Fig: 3.1. Compaction test was performed to determine the optimum moisture content and maximum dry density of the soil in each case of compaction effort (Standard Proctor, Modified Proctor and Intermediate energy). The soil was then compacted in a compaction mold using different moisture content.

3.3 Dhaka Clay Type-2

The second sample of Dhaka clay (type-2) was collected from a depth of 6 meter at "Mahtab Center" (20 storied commercial development project of Shajahan and Group), Bijoy Nagar, Dhaka. The test specimen was prepared following the same procedures of Dhaka clay type-1. Optimum moisture content was maintained in all the test specimens of Dhaka clay type-2. The index properties of this soil are tabulated in Table 3.1 and Grain size curve is shown on Fig: 3.1

Table 3.1 Index Properties and Soil Classification

Index properties/ Classification	Dhaka Clay Type-1, Low plastic (CL)	Dhaka Clay Type-2, Low plastic (CL)
Specific Gravity, Gs	2.65	2.65
Liquid Limit (%), LL	35.80	32.30
Plastic Limit (%), PL	16.30	14.70
Plasticity Index, PI	19.50	17.60

3.4 Testing Apparatus

A number of major laboratory instruments and apparatus were used for compaction of soil, index tests, preparation of test specimen, strength tests and associated works, such as:

- i) Compaction Machine,
- ii) Rotary Mixer,
- iii) Direct Shear Test Apparatus,
- iv) Sample Extruder

Brief descriptions of these apparatuses were given in Appendix-A

3.5 Testing Program

Soil sample was collected as disturbed after excavation up to the desired depth in bulk volume. The soil was then dried in air, powdered by grinding and sieved through #200 opening to obtain the test soil. The soil was compacted by Standard Proctor, Modified Proctor and Intermediate Energy having different moisture content. The test specimens were extruded from the compaction mold at different angular orientation with respect to the direction of compaction. The specimens were tested in Direct Shear test machine under different normal loading condition. The shape of the test specimens was circular having dimensions of 63.5 mm in diameter and 25.4 mm in thickness.

Direct Shear test was conducted in case of all the specimens following the procedure as follows;

The shear box was assembled with the frames aligned and locked in position. A light coating of grease between the frames ensured water tightness during consolidation and reduced friction during shear. The test specimen was carefully inserted. The loading devices were connected and the displacement gauge was positioned for measuring shear and vertical displacement of the test specimen. Initial thickness of the specimen was determined by using the first reading of vertical displacement gauge after positioning the upper plate of the shear box on the specimen. Predetermined vertical load was applied so that it can be transmitted to the test specimen through the yoke. Then the locks, holding two parts of the shear box together, were unscrewed. Shear rate controlling knob was brought in the desired position based on strain rate to be imposed. After checking all the gauges (two displacement gauges and a dial gauge for proving ring) test was started by switch on the motor. For performing a undrained test (UU_{ds}) with the shear box, after applying the normal load, shearing is started immediately so as not to give any time for consolidation. It took 2 to 3 minutes to complete each UU_{ds} test (i.e. quick test).

After the completion of each test, the activator (i.e. the motor) was stopped, measuring instruments were dissembled, shear box was unlocked so as to separate the two parts, and the specimen was retrieved. To determine the moisture content, the specimen was kept into the oven. A number of UU_{ds} tests were performed on each type of soil.

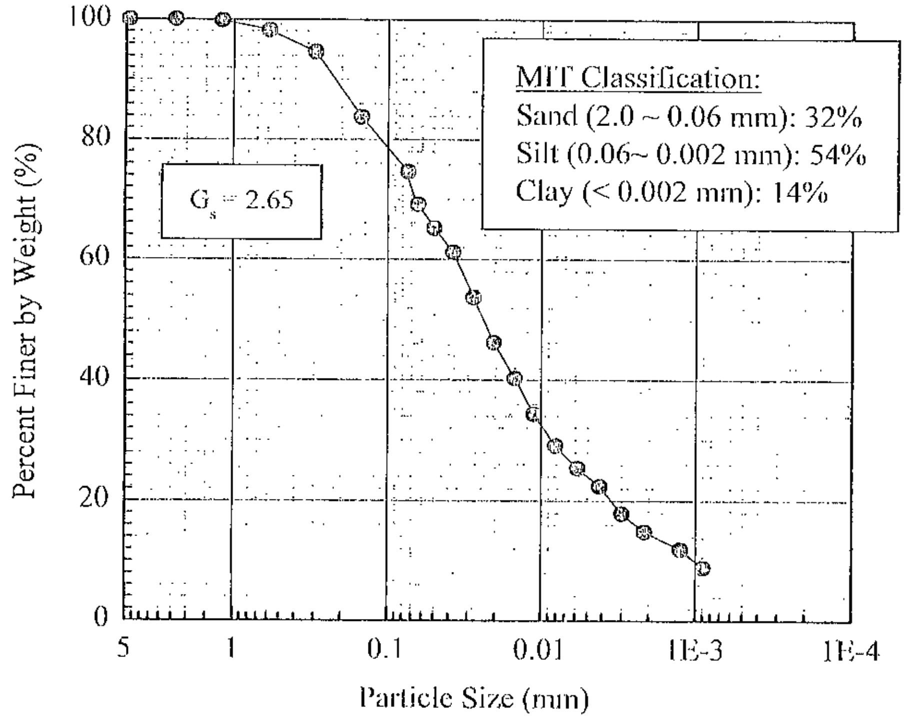


Fig. 3.1a: Hydrometer analysis of Dhaka Clay Type-1

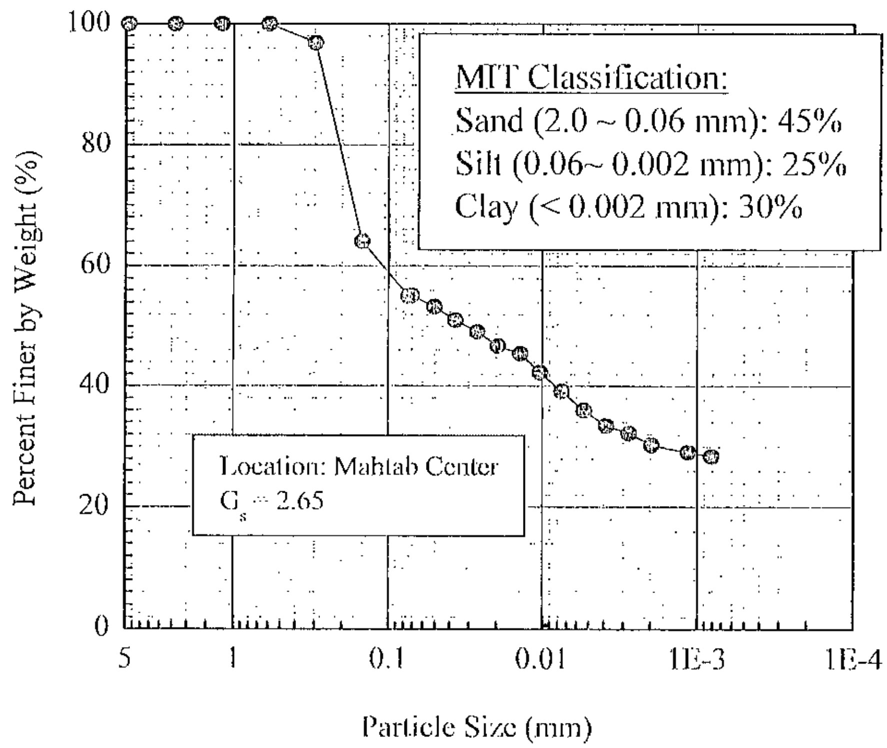


Fig. 3.1b: Hydrometer analysis of Dhaka Clay Type-2

Detail testing program is briefly outlined in the flow chart presented as follows;

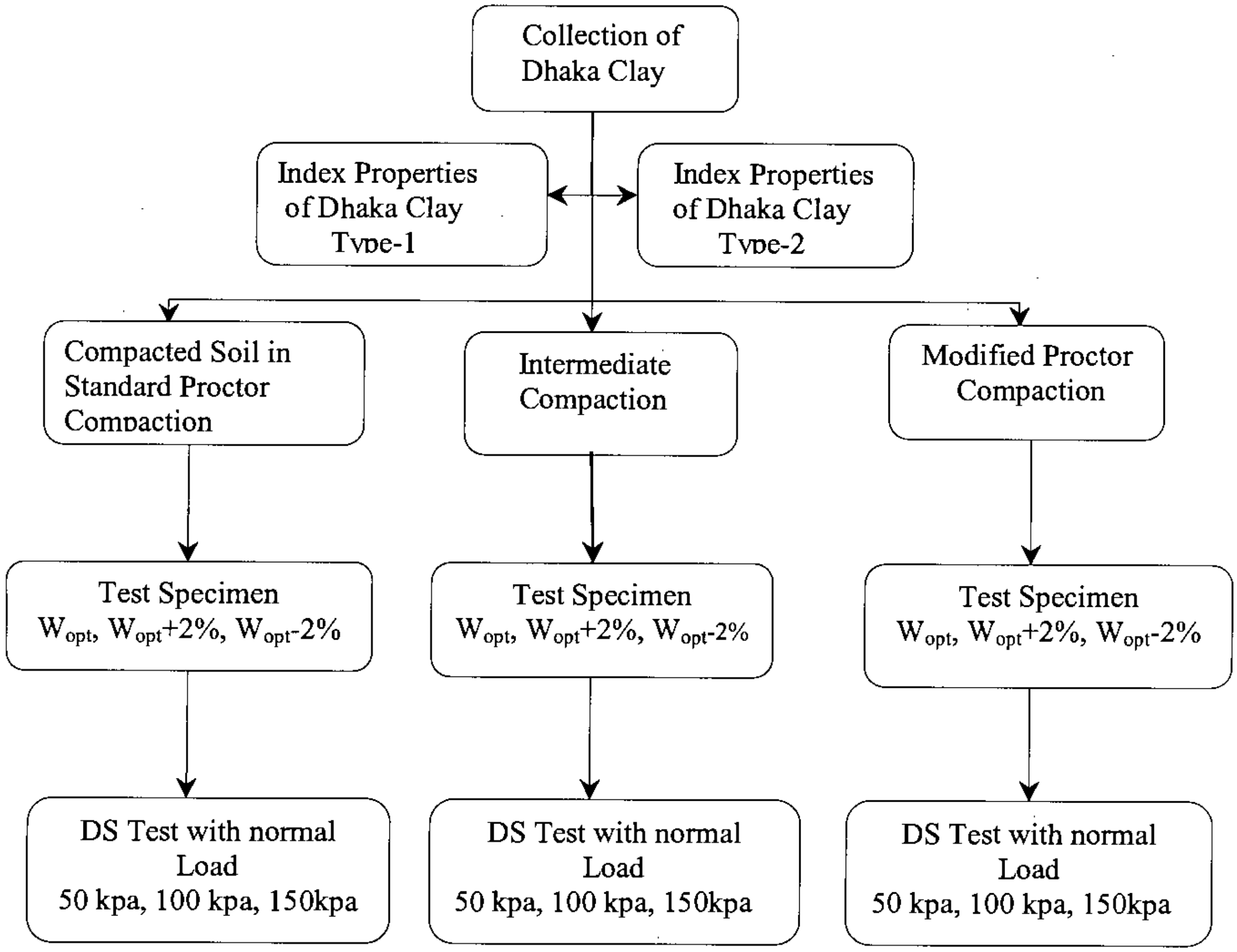


Fig. 3.2: Block diagram of the testing program.

Chapter 4
Compacted Dhaka Clay

4.1 Introduction

Roads, Railways, Reservoir bunds, Earth dams, Reclaimed lands are constructed by filling/ compacting soils found cheaply available near the project areas. Clay and silty clay soils are mostly used for their availability and cost effectiveness of such projects. In the development of such projects, construction methods followed are filling the site using loose soils and compacting it to increase the density so as to improve the strength characteristics and decrease the permeability. Literature survey showed that the analysis and design of such development projects by compacted soils are performed considering the characteristics of the isotropic soils. The role of anisotropy is not generally considered in the stability analysis of such slopes because of simplicity. Seed and Chan (1961) and Rao et al. (1982) had reported a few works in this regard. The actual stability number of the anisotropic soil may decrease or increase depending on the nature of the anisotropy than that of the stability number found from the analysis considering the isotropic soil. Normally the analysis is based on the strength of a horizontally oriented specimen (S1). If the ratio of the strength (S1) of horizontally oriented specimen to the strength (Si) of vertically oriented specimen is greater than unity (i.e. $r = S1/Si > 1$), the stability number will be decreased so that the embankment slope will be in the more unsafe side than the estimated value based on isotropic strength characteristics and vice versa (Lo, 1965; Duncan & Seed, 1966; Rao et al., 1862; Su and Liao, 1999). The anisotropy of compacted clay are described in this chapter. The directional variations of shear strength evaluated from laboratory testing are described and the strength anisotropy are attempted to quantify. The effects of different moisture content (optimum moisture content, moisture content more than that of optimum moisture content hereby referred as optimum plus and moisture content less than that of optimum moisture content hereby referred as optimum minus moisture contents) are attempted to characterize. Attempts are also made to characterize the effects of different compactive efforts and different overburden pressure on the strength anisotropy of the soils used in this research.

4.2 Soil Samples

There are several factors influencing the anisotropy, such as clay mineralogy, the maximum pre-consolidation pressure, moisture contents at the time of compaction, etc. There were previous works by Islam, 1999 on compacted clay, Ko-consolidated clay and natural clay to investigate the presence of anisotropy in undrained shear strength of clays. Therefore it was decided to give emphasis on the different compactive efforts, moisture contents and different overburden pressure on the compacted Dhaka clay to characterize the effects on the strength anisotropy. Two samples of Dhaka clay (collected from two locations) were used for testing. They were collected from different depths with respect to the mean sea level. Optimum moisture content of Dhaka clay type-1 was 16%, 13% and 14.5% in case of Standard Proctor compaction (Table 4.1), Modified Proctor compaction and Intermediate Compaction (Intermediate compaction is defined as the compaction energy in between Standard Proctor and Modified Proctor and mentioned in Table 4.1), respectively, whereas the maximum dry density was 17.71 kN/m^3, 19.73 kN/m^3 and 18.53 kN/m^3, respectively, as shown in Fig. 4.1a. Optimum moisture content of Dhaka clay type-2 was 14%, 12% and 13% in case of Standard Proctor compaction (Table 4.5), Modified Proctor compaction and intermediate compaction while the maximum dry density was 17.86 kN/m^3, 19.63 kN/m^3 and 18.75 kN/m^3, respectively, as shown in Fig. 4.1b.

4.3 Sample Preparation

The disturbed soil samples collected from two different locations of Dhaka city as mentioned earlier was dried in air, powdered by grinding and sieved through #200 sieve to obtain the soil for preparation of the test specimen. After determination of the optimum moisture content for each type of test specimen, the soil was mixed thoroughly with the required quantity of water as per different moisture content considered in this research work and compacted in layers by the specified number of blows of the standard weight of the hammer as described in Table 4.1 and Table 4.5. The compacted soil mass in the form of soil cake was extruded from the mold and test specimens were retrieved at different orientations from the soil cake using scale, knife and cutting wire. For preparation of 0^0 degree specimen, i.e., horizontally oriented test specimen, it was taken directly from the top of the cake extruded from the mold as shown in Fig.4.2. For preparation of 30^0 degree specimen, inclined surface was prepared on the soil cake by 1:2 dimensions as vertical: horizontal and a small cake was taken out from the inclined surface of the main soil mass as shown in Fig. 4.2. Accordingly, for preparation of 45^0 degree specimen, inclined surface was prepared on the original soil cake extruded from the mold by 1:1 dimensions as vertical: horizontal and a small portion of cake was taken out from the inclined surface from which the 45^0 degree specimen was collected. Similarly, 60^0 degree specimen was collected from the inclined surface prepared by 2:1 dimensions as vertical: horizontal on the original soil cake extruded from the mold. Finally the 90^0 degree specimen was collected from the vertically oriented portion of the original soil cake.

The specimen designation CM1, CM2 and CM3 is used for Dhaka Clay Type-1 tested under normal stress of 50 kPa, 100 kPa and 150 kPa respectively in case of Standard Proctor compaction. The specimen designation CM4, CM5 and CM6 is used for Dhaka Clay Type-1 tested under normal stress of 50 kPa, 100 kPa and 150 kPa respectively in case of Modified Proctor compaction. Similarly, CM7, CM8 and CM9 specimen designations are used for Dhaka Clay Type-1 tested under normal stress of 50 kPa, 100 kPa and 150 kPa, respectively, in case of Intermediate compaction. Again the specimen group CM1 was divided into test specimens designated by CM1-1, CM1-2 and CM1-3 for different moisture content, i.e., Optimum minus two percent moisture content (OMC-2) %, Optimum moisture content (OMC) % and Optimum plus two percent moisture content (OMNC+2) %, respectively. Similarly CM2 group was divided into test specimens designated by CM2-1, CM2-2 and CM2-3 for different moisture content, i.e., (OMC-2) %, (OMC) % and (OMC+2) %, respectively. Again CM3 group was also divided into test specimens designated by CM3-1, CM3-2 and CM3-3 for different moisture content, i.e., (OMC-2) %, (OMC) % and (OMC+2) %, respectively.

The specimen designation CM-I, CM-II and CM-III is used for Dhaka Clay Type-2 tested under overburden pressure of 50 kPa, 100 kPa and 150 kPa respectively in case of Standard Proctor compaction. The specimen designation CM-IV, CM-V and CM-VI is used for Dhaka Clay Type-2 tested under overburden pressure of 50 kPa, 100 kPa and 150 kPa respectively in case of Modified Proctor compaction. Similarly, CM-VII, CM-VIII and CM-IX specimen designations are used for Dhaka Clay Type-2 tested under overburden pressure of 50 kPa, 100 kPa and 150 kPa respectively in case of Intermediate compaction.

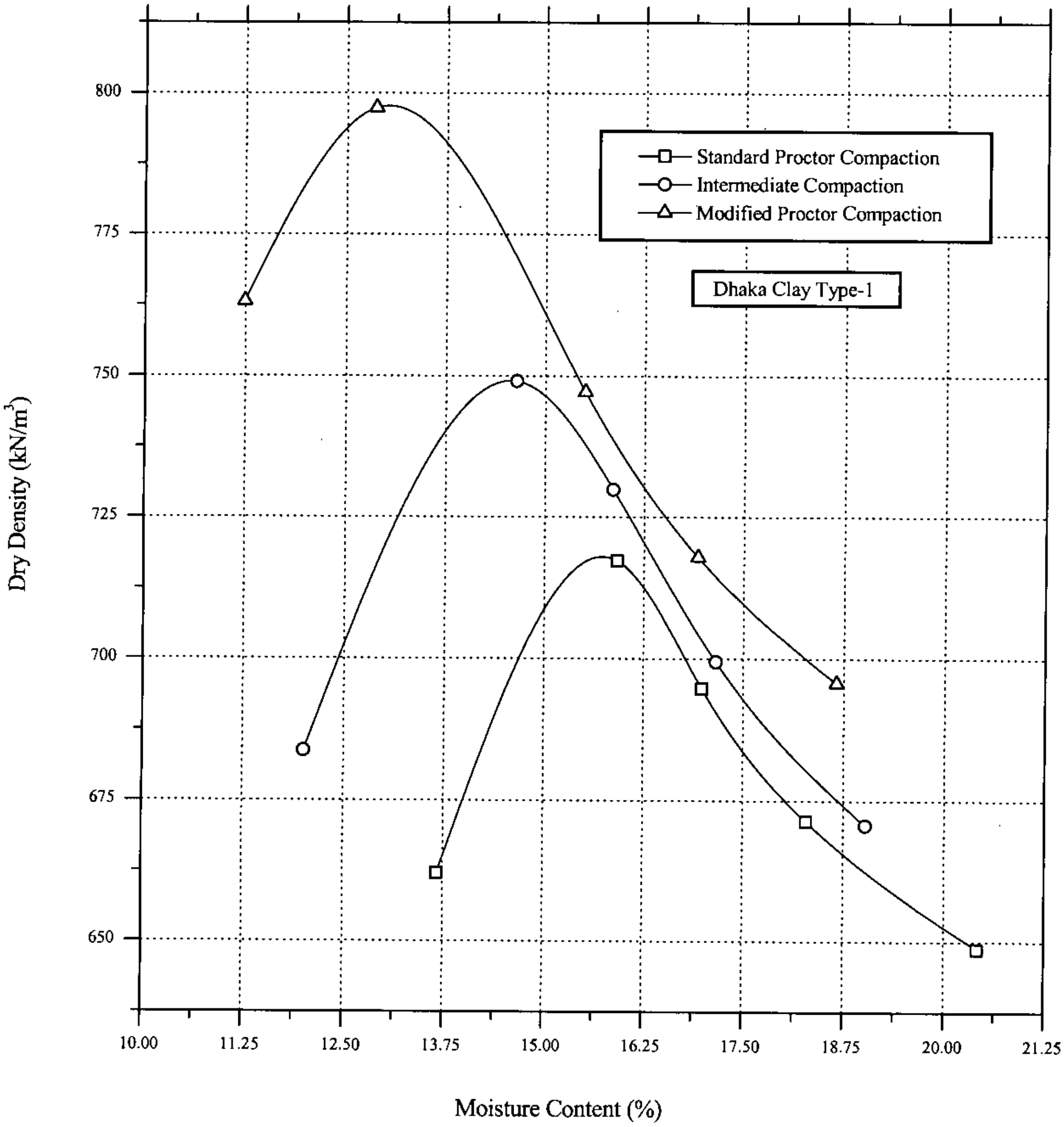


Fig. 4.1a: Dry Density Vs Moisture Content Diagram of Dhaka Clay Type-1

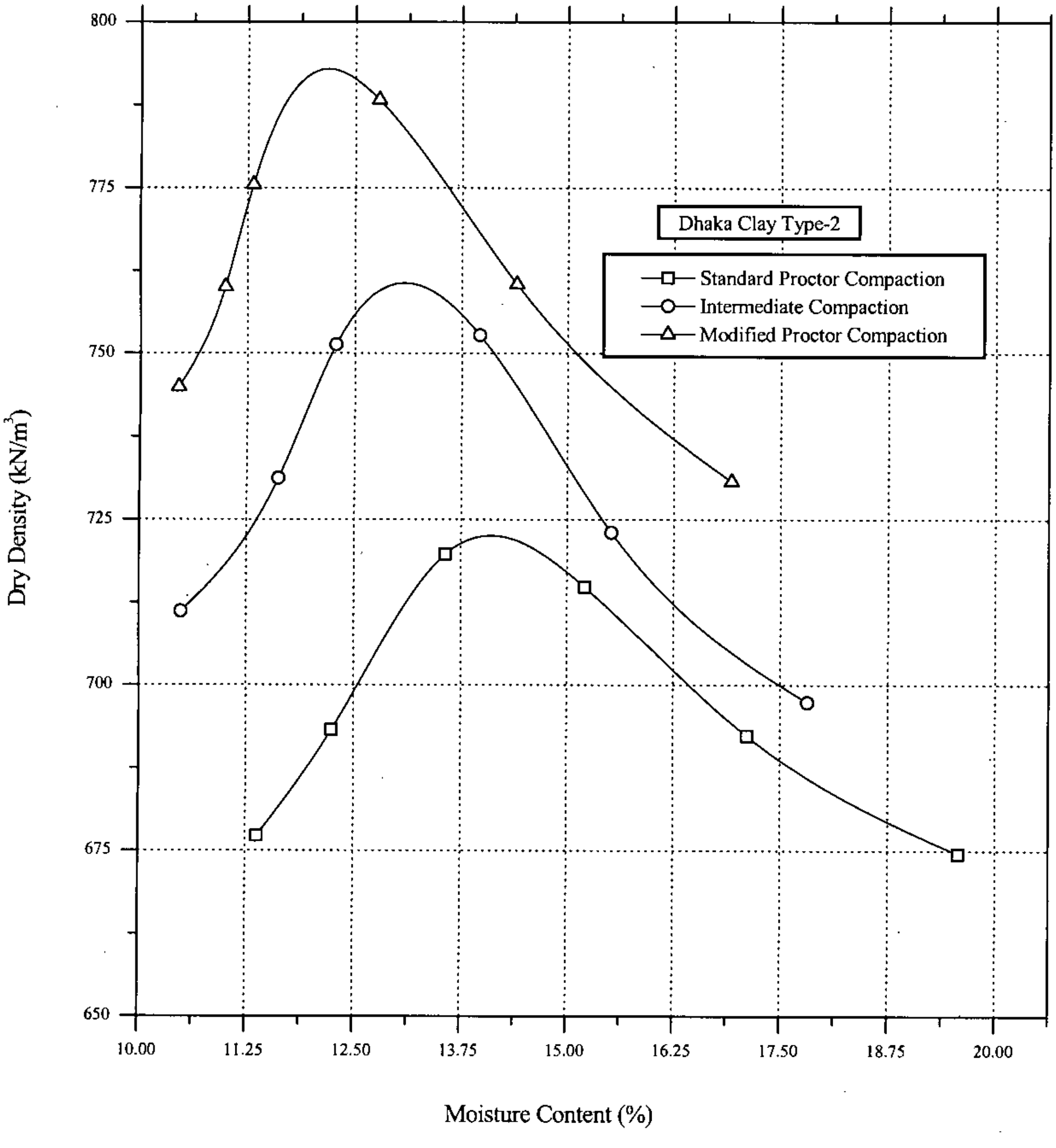
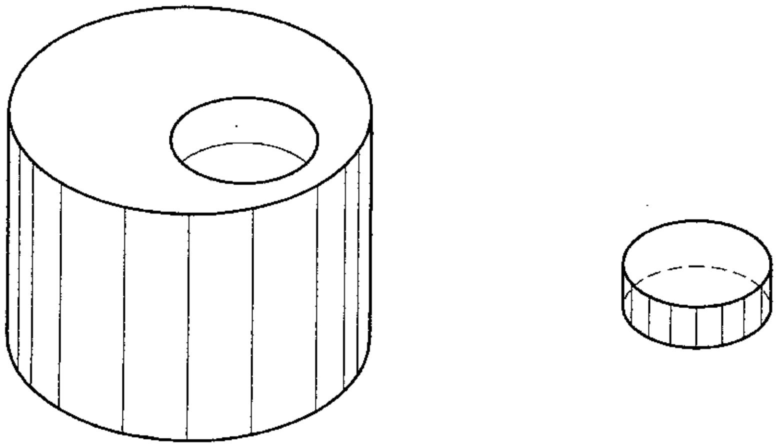
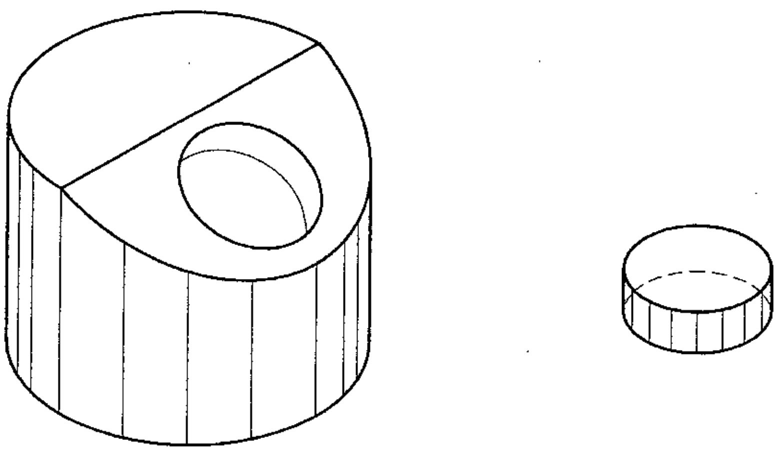


Fig. 4.1b: Dry Density Vs Moisture Content Diagram of Dhaka Clay Type-2



a) 0° Specimen extrusion



b) 30° Specimen extrusion

Fig:4.2: Specimen preparation for compacted soil mass

4.4 Test results of Dhaka Clay type-1

Specimens extracted at different angular orientation with respect to its compaction direction such as 0^0, 30^0, 45^0, 60^0 and 90^0 for each sample (Table 4.2) were tested in Direct Shear testing machine. Broadly, there are 3 groups of specimens prepared from Dhaka clay type-1 according to compaction energy utilized during reconstitution. Three levels of compaction energies are Standard Proctor compaction, Modified Proctor compaction and Intermediate compaction. Where, Intermediate compaction is the compaction energy chosen in between Standard Proctor and Modified Proctor energies (Table 4.1). Test results of the specimens reconstituted under these energy levels are discussed in the following articles 4.4.1, 4.4.2 and 4.4.3, respectively.

4.4.1 Results on Standard Proctor compaction

Groups of specimens extracted at different angular orientation with respect to its compaction direction such as 0^0, 30^0, 45^0, 60^0 and 90^0 for each sample (Table 4.2) were tested in Direct Shear testing machine. Each group was subdivided into three groups according to normal loading condition of 50 kPa, 100 kPa and 150 kPa during specimen testing. These groups were again subdivided according to different moisture content, i.e., specimens having optimum moisture content designated by OMC%, optimum plus two percent moisture content designated by (OMC+2)% and optimum minus two percent optimum moisture content designated by (OMC-2)%.

Figures 4.3a to c show the relationship between shear stress and shear displacement under normal loading of 50 kPa for different orientation angles of specimen groups designated by CM1-1, CM1-2 and CM1-3, respectively (Table 4.2). These specimens were reconstituted from Dhaka Clay type-1 and designated as CM1 group (i.e., reconstituted by Standard Proctor energy and tested under σ_v = 50 kPa, Table 4.2). Similar relationships obtained for CM2 and CM3 group specimens. For CM2 group specimens, the relationships between shear stress and shear displacements for specimens CM2-1, CM2-2 and CM2-3 groups are shown in Figs. 4.4a to c, respectively. Similarly for CM3 group specimens, the relationships between shear stress and shear displacements for specimens CM3-1, CM3-2 and CM3-3 groups are shown in Figs. 4.5a to c, respectively. In these Figs., moisture content during reconstitution was (OMC-2)% for

CM1-1, CM2-1 and CM3-1 groups, OMC% for CM1-2, CM2-2 and CM3-2 groups and (OMC+2)% for CM1-3, CM2-3 and CM3-3 group specimens. The following results can be seen from Figs. 4.3a to c, 4.4a to c and 4.5a to c:

Shear stress ~ displacement curve for a given orientation of compaction direction represents a typical stress ~ strain or stress ~ displacement curve of clay soil; that is, the shear stress increases as the shear displacement initially; after achieving the peak, shear stress starts to decrease with the further increase of shear displacement. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak shear strength. The above findings are not influenced by the magnitude of the normal loading specially in the range of investigation carried out (i.e., σ_v = 50~150 kPa).

Figures 4.6a to c show the relationship between normal displacement and shear displacement under σ_v = 50 kPa for different orientation angles of the specimen groups CM1-1, CM1-2 and CM1-3, respectively (Table 4.2). Similar relationship obtained for CM2 group specimens under σ_v = 100 kPa and CM3 group specimens under σ_v = 150 kPa were shown in Figs. 4.7a to c and 4.8a to c, respectively. It is seen that total volume of the specimens initially decreases as the shear displacement increases and after reaching a minimum magnitude it starts to increase with the further increase of shear displacement. For a given set of plots (for example, CM1-1 of Fig. 4.6a, CM1-2 of Fig. 4.6b, etc.), the variation of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves are similar. That is if one looks into the values of normal displacement for various θ for a given value of shear displacement, the values are not similar to each other except at the initial point. It indicates that anisotropy exists in the dilatancy characteristics.

Figure 4.9a to c show the variation of peak shear strength with the orientation angle θ. Three curves are plotted in each Fig.4.9a, Fig. 4.9b and Fig. 4.9c. All these specimen groups were reconstituted according to Standard Proctor compaction energy. During shear testing, normal vertical stress was σ_v = 50 kPa for specimen groups CM1-1, CM1-2 and CM1-3. The moisture content for CM1-1 group specimens was (OMC-2)% and that of CM1-2 and CM1-3 groups was OMC% (i.e., optimum moisture content) and (OMC+2)%, respectively. Similar plots are drawn for CM2 group specimens (i.e., CM2-

1, CM2-2 and CM2-3, Table 4.2) and CM3 group specimens (i.e., CM3-1, CM3-2 and CM3-3, Table 4.2) and are shown in Fig. 4.9b and c, respectively. The following findings can be drawn from Figs. 4.9a to c:

a) for a given specimen group (say, CM1-1, that is the groups with σ_v = 50 kPa and reconstituted for the moisture content (OMC-2)%, Fig. 4.3a), the peak shear strength is not a constant irrespective of orientation angle θ. This indicates the existence of strength anisotropy due to directional compaction.

b) for a given specimen group (say, CM1-1, Fig. 4.3a), the peak shear strength initially increases with the increase of orientation angle (θ), attains a peak at $\theta = 45^\circ$ and starts to decrease at $\theta > 45^\circ$. Similar relationship is observed for CM1-2 (see Fig. 4.3b) groups of specimens. However, for specimens of groups CM1-3 (i.e., the specimens reconstituted of moisture content (OMC+2)%), the peak is obtained at $\theta = 0^\circ$ specimens. But in no cases, the magnitude of the deviation of the peak shear strength value for θ was more than 5 to 8%.

c) for any given θ, the peak shear strength varies for different moulding moisture contents. The strength is maximum for specimens of dry moulding moisture content (i.e., for (OMC-2) %) and minimum for specimens of wet sample (i.e., (OMC+2)%).

Anisotropy: Stress induced and strain induced

As mentioned earlier, Fig. 4.9a to c show the relationship between peak shear strength (τ_{max}) and the orientation angle θ for CM1-1 group specimens. This specimen group is compacted according to Standard Proctor compaction energy and eventually sheared at normal stress σ_v = 50 kPa. Similar $\tau_{max} \sim \theta$ relationships are obtained for specimen subjected to σ_v = 100 kPa (CM1-2 group) and σ_v = 150 kPa (CM1-3 group) under otherwise similar conditions. That is, these three groups of specimen CM1-1, CM1-2 and CM1-3 are reconstituted for similar compaction energy, moisture content and also subjected to similar loading rate during shearing. A new term, the degree of strength anisotropy is defined, as the ratio of peak strength at any orientation angle θ to the corresponding strength for $\theta = 0^\circ$. That is, the term $\tau_{max,\ \theta=30^\circ} / \tau_{max,\ \theta=0^\circ}$ indicates the degree of anisotropy in strength at $\theta = 30^\circ$, where $\tau_{max,\ \theta=30^\circ}$ denotes the peak strength of a specimen that is compacted at $\theta = 0^o$ but sheared at $\theta = 30^o$ and $\tau_{max,\ \theta=0}{}^o$ represents the peak strength of the specimen for which compaction and shearing directions are the same, that is $\theta = 0^o$.

The degree/ coefficient of strength anisotropy, I, as defined above is plotted in Fig. 4.10a to d (using data of Fig. 4.9a to c) as variation of orientation angle θ. All three groups of Dhaka Clay type-1, namely, CM1-1 of w= (OMC-2) %, CM1-2 of w= OMC % and CM1-3 w= (OMC-2) % are shown in Fig.4.10a. It can be seen from Fig. 4.10a to d that all the I~θ graphs are started from I = 1.0 at $\theta = 0^o$. It is because that $\theta = 0^o$ corresponds to the conventional direction of shearing after sampling a specimen conventionally (i.e. vertically). Since the first point ($\theta = 0^o$) of each curve represent this $\theta = 0^o$ sample strength after normalized by $\tau_{max,\ \theta=0}{}^o$ (i.e. the same peak strength), all the four plots of Fig. 4.10a to d will start from I = 1.0. However, any other point along a given curve (say, CM1-1 group), if the value of I becomes unity (i.e. = 1) it indicates that the particular sample is isotropic with respect to strength for the particular direction.

Figure 4.10a shows that none of the curves follows I = 1.0 line irrespective of any value of θ, indicating the existence of anisotropy of peak shear strength. The shapes of the three curves are dissimilar to each other. For CM1-1 groups, for example, the value of I varies from 1.00~1.05, whereas it was 1.04~0.95 and 1.03~0.92, respectively, for CM1-2 and CM1-3 groups of specimens (Fig. 4.10a). On the other hand, the value of I was 1.00~0.97, 1.00~0.93 and 1.00~0.90 for CM2-1, CM-2-2 and CM2-3 (Fig. 4.10b) groups of specimens, respectively, and that was 1.00~0.97, 1.00~0.98 and 1.00~1.04 for CM3-1, CM3-2 and CM3-3 (Fig.4.10c) groups of specimens, respectively. The value of I was 1.04~0.95, 1.00~0.93 and 1.00~0.98 for CM1-2, CM2-2 and CM3-2 (Fig.4.10d) groups of specimens, respectively, at optimum moisture content and different σ_v (i.e., σ_v = 50~150 kPa). A value of I more than unity is not found along CM1-1 group plot. It indicates that if the design of soil strength, as conventionally done, is based on only $\tau_{max,\ \theta=0}{}^o$ will put the structure into more safe side, i.e., the structure will be said to be over designed. But if at any point along any curve (say, CM1-2 and CM1-3) the value of I becomes less than unit, then the conventional design of strength based on $\tau_{max,\ \theta=0}{}^o$ will put the superstructure in unsafe condition, i.e., the standard factor of safety practiced in design shall be reduced which may cause failure of the structure.

4.4.2 Results on Modified Proctor compaction

A soil mass is reconstituted basically with OMC moisture content under Modified Proctor compaction energy. Each test specimen was extracted from the reconstituted soil mass at different angular orientation with respect to its compaction direction such as 0^0, 30^0, 45^0, 60^0 and 90^0 (Table 4.2) and tested in Direct Shear testing machine. For each group of specimen, the normal stress σ_v was varied for σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, during direct shear testing, while the moisture content was maintained at OMC%. Figures 4.11a to c show the relationship between shear stress and shear displacement under normal loading of 50 kPa, 100 kPa and 150 kPa respectively for different orientation angles of specimen groups designated by CM4, CM5 and CM6, respectively (Table 4.2). The following results can be seen from Figs. 4.11a to c:

Shear stress ~ displacement curve for a given orientation of compaction direction represents a typical stress ~ strain or stress ~ displacement curve of clay soil; that is, the shear stress increases as the shear displacement initially; after achieving the peak, shear stress starts to decrease with the further increase of shear displacement. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak strength. The above findings are not influenced by the magnitude of the normal loading specially in the range of investigation carried out (i.e., σ_v = 50 to 150 kPa).

Figures 4.12a to c show the relationship between normal displacement and shear displacement under σ_v = 50 kPa, σ_v = 100 kPa and σ_v = 150 kPa respectively, for different orientation angles of the specimens of the groups CM4, CM5 and CM6 respectively (Table 4.2). It is seen that total volume of the specimens initially decreases as the shear displacement increases and after reaching a minimum magnitude it starts to increase with the further increase of shear displacement. For a given set of plots (for example, CM4 of Fig. 4.12a, CM5 of Fig. 4.12b, etc.), the variation of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves are similar. That is if we look into the values of normal displacement for various θ for a given value of shear displacement, the values are not similar to each other except at the initial point. It indicates that anisotropy exists in the dilatancy characteristics.

Figure 4.13 show the variation of peak shear strength with the orientation angle θ. Three curves are plotted in Fig.4.13. All these specimens of the groups were reconstituted according to Modified Proctor compaction energy. The peak shear strength of specimens CM4 (i.e., data of Fig. 4.11a), CM5 (i.e., data of Fig. 4.11b) and CM6 (i.e., data of Fig. 4.11c) were plotted against the orientation angle θ and thus yield three curves in Fig. 4.13. In all these cases, the moisture content was maintained at OMC%. The following findings can be drawn from Figs. 4.13:

a) for a given specimen group (say, CM4, that is the groups with σ_v = 50 kPa and reconstituted for the moisture content OMC%, Fig. 4.11a), the peak shear strength is not a constant irrespective of orientation angle θ. This indicates the existence of strength anisotropy due to directional compaction.

b) for a given specimen group (say, CM4, Fig. 4.11a), the peak shear strength initially increases with the increase of orientation angle (θ), attains a peak at $\theta = 30^o$ and starts to decrease at $\theta > 30^o$. Similar relationship is observed for CM6 (see Fig. 4.11c) groups of specimens. However, for specimens of groups CM5 (i.e., the specimens reconstituted of moisture content OMC%, Fig. 4.11b), the peak is obtained at $\theta = 45^o$ specimens. But in no cases, the magnitude of the deviation of the peak shear strength value for θ was more than 10 to 20%.

The degree/ coefficient of strength anisotropy, I, as defined earlier is plotted in Fig. 4.14 (using data of Fig. 4.13) as variation of orientation angle θ. All three groups of Dhaka Clay type-1, namely, CM4 (σ_v = 50 kPa), CM5 (σ_v = 100 kPa) and CM6 (σ_v = 150 kPa) are shown in Fig.4.14. It can be seen that none of the curves follows I = 1.0 line irrespective of any value of θ, indicating the existence of anisotropy of peak strength. The shapes of the three curves are dissimilar to each other. For CM4 groups, for example, the value of I varies from 1.00~1.11, whereas it was 1.00~1.26 and 1.00~1.26 for CM5 and CM6 groups of specimens (Fig. 4.14). A value of I less than unity is not found along any plot of the groups of specimens of CM4 groups, CM5 groups and CM6 groups respectively.

4.4.3 Results on Intermediate compaction

The specimens were reconstituted under compaction energy, which is in between Standard and Modified Proctor energy (Table 4.1). The moisture content was fixed at

OMC%. The test specimens were extracted from the reconstituted mass at different 'θ" with respect to its compaction direction, such as 0^0, 30^0, 45^0, 60^0 and 90^0. Normal stress was varied from 50 kPa to 150 kPa during direct shear (DS) testing of the specimens.

Figures 4.15a to c show the relationship between shear stress and shear displacement under normal loading of 50 kPa, 100 kPa and 150 kPa respectively for different orientation angles of specimen groups designated by CM7, CM8 and CM9, respectively (Table 4.2). The following results can be seen from Figs. 4.15a to c:

Shear stress ~ displacement curve for a given orientation of compaction direction represents a typical stress ~ strain or stress ~ displacement curve of clay soil; that is, the shear stress increases as the shear displacement initially; after achieving the peak, shear stress starts to decrease with the further increase of shear displacement. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak strength. The above findings are not influenced by the magnitude of the normal loading specially in the range of investigation carried out (i.e., σ_v = 50 to 150 kPa).

Figures 4.16a to c show the relationship between normal displacement and shear displacement under σ_v = 50 kPa, σ_v = 100 kPa and σ_v = 150 kPa respectively, for different orientation angles of the specimens of the groups CM7, CM8 and CM9 respectively (Table 4.2). It is seen that total volume of the specimens initially decreases as the shear displacement increases and after reaching a minimum magnitude it starts to increase with the further increase of shear displacement. For a given set of plots (for example, CM7 of Fig. 4.16a, CM8 of Fig. 4.16b, etc.), the variation of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves are similar. That is if we look into the values of normal displacement for various θ for a given value of shear displacement, the values are not similar to each other except at the initial point. It indicates that anisotropy exists in the dilatancy characteristics.

Figure 4.17 show the variation of peak shear strength with the orientation angle θ. Three curves are plotted in Fig.4.17. All these specimens of the groups were reconstituted according to Intermediate compaction energy. The peak shear strength of specimens CM7 (i.e., data of Fig. 4.15a), CM8 (i.e., data of Fig. 4.15b) and CM9 (i.e., data of Fig. 4.15c) were plotted against the orientation angle θ and thus yield three curves in Fig. 4.17. In all these cases, the moisture content was maintained at OMC%. The following findings can be drawn from Figs. 4.17:

a) for a given specimen group (say, CM7, that is the groups sheared with σ_v = 50 kPa and reconstituted at the moisture content OMC%, Fig. 4.15a), the peak shear strength is not a constant irrespective of orientation angle θ. This indicates the existence of strength anisotropy due to directional compaction.

b) for a given specimen group (say, CM7, Fig. 4.15a), the peak shear strength initially increases with the increase of orientation angle (θ), attains a peak at θ = 30^o and starts to decrease at θ > 30^o. Similar relationship is observed for CM8 (see Fig. 4.15b) groups of specimens and specimens of groups CM9 (i.e., the specimens reconstituted of moisture content OMC%, Fig. 4.15c). But in no cases, the magnitude of the deviation of the peak shear strength value for θ was more than 12 to 20%.

The degree/ coefficient of strength anisotropy, I, as defined earlier is plotted in Fig. 4.18 (using data of Fig. 4.17) as variation of orientation angle θ. All three groups of Dhaka Clay type-1, namely, CM7 (σ_v = 50 kPa), CM8 (σ_v = 100 kPa) and CM9 (σ_v = 150 kPa) are shown in Fig.4.18. It can be seen from Fig. 4.18 that all the I ~ θ graphs are started from I = 1.0 at θ = 0^o. It can be seen that none of the curves follows I = 1.0 line irrespective of any value of θ, indicating the existence of anisotropy of peak strength. The shapes of the three curves are dissimilar to each other. For CM7 groups, for example, the value of I varies from 0.96~1.13, whereas it was 0.87~1.01 and 1.00~0.90 for CM8 and CM9 groups of specimens (Fig. 4.18).

4.5 Test results of Dhaka Clay type-2

Clay-2 is also typical Dhaka clay collected from different location (description provided at Chapter 3). The reconstituted samples for testing in this investigation were prepared following the same procedure as employed in clay-1. The test sample was reconstituted using optimum moisture content (OMC %) only, while the compacting efforts were Standard Proctor, Modified as well as Intermediate compaction energies. For a given group of testing, normal stress during DS testing was varied from 50 to 150 kPa, while the angular orientation θ with respect to its compaction direction (which was always vertical) was 0^0, 30^0, 45^0, 60^0 and 90^0.

Figures 4.19a to c show the relationship between shear stress and shear displacement for different orientation angles θ under normal loading of 50 kPa (designated as sub-group CM-I), 100 kPa (CM-II) and 150 kPa (CM-III), respectively (Table 4.6). The following results can be observed:

Shear stress ~ displacement curve for a given orientation of compaction direction represents a typical stress ~ displacement curve of clay soil under DS testing; that is, the shear stress increases as the shear displacement initially; after achieving the peak, shear stress starts to decrease with the further increase of shear displacement. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak strength. The above findings are not influenced by the magnitude of the normal loading, especially in the range of investigation carried out (i.e., σ_v = 50 to 150 kPa).

Figures 4.20a to c show the corresponding relationship between normal displacement and shear displacement under σ_v = 50 kPa, σ_v = 100 kPa and σ_v = 150 kPa respectively, for different orientation angles. Typically the volume of the specimens initially decreases as the shear displacement increases and after reaching a minimum magnitude it starts to increase with the further increase of shear displacement. For a given set of plots (for example, CM-I of Fig. 4.20a, CM-II of Fig. 4.20b, etc.), the variation of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves are similar. That is if we look into the values of normal displacement for various θ for a given value of shear displacement, the values are not similar to each other except at the initial point. It indicates that anisotropy exists in the dilatancy characteristics.

Figure 4.21a to c show the variation of orientation angle θ with the degree of strength anisotropy I (as defined earlier). Among them, Fig. 4.21a shows I~θ relationship obtained using the test results data from the relationship between peak shear strength (τ_{max}) and the orientation angle θ for specimens of CM-I (σ_v = 50 kPa), CM-II (σ_v = 100 kPa) and CM-III (σ_v = 150 kPa) sub-groups (i.e., Figs. 4.19a~c). Note that compaction energy during

DS testing for above data was Standard Proctor. On the other hand, the corresponding data (I~θ) obtained from specimens subjected to Modified Proctor and Intermediate energies, for otherwise similar conditions, are shown in Figs. 4.21b and c, respectively.

It can be seen from Fig. 4.21a to c that all I ~ θ graphs are started from I = 1.0 at $\theta = 0^{o}$. It is because that $\theta = 0^{o}$ corresponds to the conventional direction of shearing after sampling a specimen conventionally (i.e. vertically). Since the first point ($\theta = 0^{o}$) of each curve represent this $\theta = 0^{o}$ sample strength after normalized by $\tau_{max,\ \theta=0}{}^{o}$ (i.e. the same peak strength), all the nine plots of Fig. 4.21a to c will start from I = 1.0. However, any other point along a given curve (say, CM-I groups), if the value of I becomes unity (i.e. = 1) it indicates that the particular sample is isotropic with respect to strength for the particular direction. However, none of the curves follows I = 1.0 line irrespective of any value of θ, indicating the existence of anisotropy of peak strength. The shapes of the curves are dissimilar to each other. For CM-I, CM-II and CM-III groups, for example, the value of I varies from 1.00~0.90, 1.00~0.90 and 1.00~0.85, respectively, whereas it was 0.99~1.11, 1.00~1.37 and 1.00~1.31 for CM-IV, CM-V and CM-VI, respectively. On the other hand, the value of I varies within the range of 0.93~1.03, 1.00~1.14 and 1.00~1.13 for CM-VII, CM-VIII and CM-IX groups of specimens, respectively. It can be mentioned that the specimens reconstituted under Modified Proctor energy exhibits the maximum anisotropy in strength. Similar behavior was also observed in case of Clay-1.

4.6 Effect of σ_v (i.e., normal stress) on anisotropy

Effects of σ_v (i.e., normal stress) on the degree of strength anisotropy were investigated for various compaction energies. The degree of anisotropy is slightly influenced by the variation of σ_v (i.e., normal stress) in case of both the soils of Dhaka clay (type-1 and type-2) reconstituted at optimum moisture content under Standard Proctor and Intermediate compaction (Figs. 4.10d & 4.18 for Dhaka clay type-1 and Figs. 4.21a & 4.21c for Dhaka clay type-2). In case of Standard Proctor compaction, the value of I varied in the range of 1.04~0.95, 1.00~0.93 and 1.00~0.98 for σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, for Dhaka clay type-1 (Fig. 4.10d) and 0.899~1.00, 0.896~1.00 and 0.85~1.00 for σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, for Dhaka clay type-2 (Fig. 4.21a). In case of Intermediate compaction, the value of I varied in the range of 1.13~0.96, 1.01~0.87 and 1.00~0.90 for σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, for Dhaka clay type-1 (Fig. 4.18) and 1.03~0.93, 1.14~1.00 and 1.13~1.00 for σ_v = 50 kPa, 100 kPa and 150 kPa, respectively, for Dhaka clay type-2 (Fig. 4.21c). That is, the largest value of anisotropy was about 14%.

On the other hand, the degree of anisotropy is slightly influenced by the variation of σ_v at lower normal stresses (i.e., σ_v = 50 kPa) in case of Modified Proctor compaction for both the soils. The value of I varied in the range of 1.00~1.11 and 0.99~1.11 for Clay-1 (Fig. 4.14) and Clay-2 (Fig. 4.21b), respectively, under σ_v = 50 kPa in case of Modified Proctor compaction. But the value of I increases noticeably with the increase of σ_v (i.e., 100 kPa and 150 kPa) for both the soils reconstituted by Modified Proctor compaction. The value of I varied in the range of 1.00~1.26 and 1.00~1.37 for Clay-1 (Fig. 4.14) and Clay-2 (Fig. 4.21b), respectively, under σ_v = 100~150 kPa for Modified Proctor compaction.

4.7 Summary of the test results

The observations from these series of testing program are as follows;

- Optimum moisture content was observed 16%, 14.5% and 13% in case of Standard Proctor, Modified Proctor and Intermediate compaction, respectively, for Dhaka Clay Type-1 and it was 14%, 13% and 12% for the same levels of compaction energy, respectively, in case of Dhaka Clay Type-2. That is optimum moisture content of Dhaka Clay varies with the compaction effort. However it is observed that optimum moisture content evaluated for a given energy level compaction is independent of the mold size used for reconstitution of the sample.

- Both the two types of Dhaka Clay Type-1 and Type-2 are showing strength and strain anisotropy. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak shear strength. The above findings are not influenced by the magnitude of the normal loading. Similarly, the variation of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves is similar and the testing conditions are otherwise similar. It can be realized if one looks into the values of normal displacement for various θ for a given value of shear displacement, the values are not similar to each other except at the initial point. It indicates that anisotropy exists in the dilatancy characteristics.

- It is observed that the volume of the test specimen decreases with the increase of stress at initial stages of loading in all cases of normal loading. The volume was increased at failure in case of normal loading of 50 kPa. However, the volume was not increased to the similar level as it did in case of smaller normal stress after initial decrease in case of $\sigma_v = 100$ and 150 kPa. This variation yields large variation in the stiffness characteristics.

- The magnitude of the deviation of the peak shear strength value for different θ lies within 10%, 26% and 13% in case of Standard Proctor compaction, Modified Proctor compaction and Intermediate compaction, respectively, for Dhaka Clay Type-1 and the same was within 15%, 37% and 14% in case of Standard Proctor compaction, Modified Proctor compaction and Intermediate compaction, respectively, for Dhaka Clay Type-2.

- The coefficient of anisotropy I is defined as the ratio of peak shear strength at any orientation angle θ to the corresponding strength for $\theta = 0^\circ$ (i.e. S_θ°/S_0°). For Dhaka Clay Type-1 reconstituted at optimum moisture content under Standard Proctor compaction energy, the value of I varied in the range of 1.04~0.95 for $\sigma_v = 50$ kPa, 1.00~0.93 for 100 kPa and 1.03~0.98 for 150 kPa. However, when the moisture content was changed to (OMC+2) % under otherwise similar conditions, the value of I varied in the range of 1.03~0.92 for $\sigma_v = 50$ kPa, 1.00~0.90 for 100 kPa and 1.00~1.04 for 150 kPa. Similarly for moisture content (OMC-2)%, the value of I varied in the range of 1.00~1.05 for $\sigma_v = 50$ kPa, 1.00~0.97 for 100 kPa and 1.02~0.97 for 150 kPa.

- Comparing the values of I, resulted from otherwise similar conditions, it can be said that the variation of degree of anisotropy is not significant with the variation of moisture content.

- For Dhaka Clay Type-1 reconstituted at optimum moisture content under Modified Proctor compaction, the value of I varied in the range of 1.00~1.11 for σ_v = 50 kPa, 1.00~1.26 for 100 kPa and 1.00~1.26 for 150 kPa. Similar testing was employed for Dhaka Clay Type-1 under Intermediate compaction energy defined as the energy level, which is in between Standard and Modified Proctor energy levels. For the latter case, the value of I varied in the range of 1.13~0.96 for σ_v = 50 kPa, 1.01~0.87 for 100 kPa and 1.00~0.90 for 150 kPa. Note that moisture content was not changed for the test specimens to (OMC+2)% and (OMC-2)% in case of tests conducted under Modified Proctor and Intermediate compaction energy levels.

- Limited tests were performed for Dhaka clay type-2. The variables were orientation angle, compaction energy and normal stress. The tests were conducted only at optimum moisture content. Test results show that the value of I varies in the range of 1.00~0.90 for σ_v = 50 kPa, 1.00~0.90 for 100 kPa and 1.00~0.85 for 150 kPa in case Standard Proctor compaction. Similarly for Modified Proctor compaction the value of I varies in the range of 1.11~0.99 for σ_v = 50 kPa, 1.00~1.37 for 100 kPa and 1.00~1.31 for 150 kPa. For Intermediate compaction, I varies in the range of 1.03~0.93 σ_v = 50 kPa, 1.14~1.00 for 100 kPa and 1.00~1.13 for 150 kPa.

- Comparing the values of I (see Figs.4.10a to c, Fig. 4.14 and Fig. 4.18) resulted from different energy levels of compaction, under other wise similar conditions, it can be said that the magnitude of strength anisotropy in case of Intermediate compaction lies between the magnitudes of the strength anisotropy obtained from Standard Proctor compaction and Modified Proctor compaction. That means a tendency of increasing anisotropy was observed (with minor exceptions) with the increase of compaction energy during reconstitution of specimens.

- Shear Strength, S_0 (i.e., $\theta = 0^\circ$) is used conventionally in design assuming isotropic strength of soil. Test results show that most of the specimens oriented other than $\theta = 0^\circ$, under otherwise similar conditions, are having shear strength higher than S_0 by a margin of maximum 37%. In these cases, the conventional design will keep the foundation of the structure at more conservative side. On the other hand, a few specimens, under otherwise similar conditions, are having shear strength lower than

S_0 by a margin of maximum 15%. In these cases, the factor of safety in the conventional design will be reduced but the structure will not be in unsafe condition because the factor safety used in case of strength of soil is usually varied in the range of 2~2.5.

- For both the clays, the degree of anisotropy was observed to be influenced slightly by the variation of σ_v (i.e., normal stress), when the specimens were reconstituted at optimum moisture content under Standard Proctor and Intermediate compaction. Similar results were observed in specimens reconstituted under Modified Proctor compaction energy and tested at lower σ_v (i.e., $\sigma_v = 50$ kPa).

- However the value of I was observed to increase noticeably with the increase of σ_v (i.e., 100 kPa and 150 kPa) for both the soils reconstituted by Modified Proctor compaction. In this case, the maximum anisotropy was about 37%.

Table - 4.1: Details of compaction efforts for Dhaka clay type-1

Mold Size (mm)	Hammer Weight (kg)	Nos. of Layer	Nos. of Blows/ Layer	Height of fall of hammer (mm)	Compaction Energy /fall (kN-m/m^3)	Compaction Type	Optimum Moisture Content (%)
Diameter 152.40 Height 114.30	2.50	3	55	300	1950	Standard Proctor	16.00
Diameter 152.40 Height 114.30	4.53	5	55	450	8850	Modified Proctor	13.00
Diameter 152.40 Height 114.30	4.53	5	25	450	4050	Intermediate Proctor	14.50%

Table - 4.2: List of test specimens and designations of Dhaka clay type-1

Sl No.	Specimen Designation	Compaction	Clay Mass	Normal Stress	Moisture Content	Dry density (kN/m^3)
1	CM1-1	Standard Proctor	Clay-1	50 kPa	14.0%	16.90
2	CM1-2				16.0%	17.61
3	CM1-3				18.0%	17.21
4	CM2-1	Standard Proctor	Clay-1	100 kPa	14.0%	16.90
5	CM2-2				16.0%	17.61
6	CM2-3				18.0%	17.21
7	CM3-1	Standard Proctor	Clay-1	150 kPa	14.0%	16.90
8	CM3-2				16.0%	17.61
9	CM3-3				18.0%	17.21
10	CM4	Modified Proctor	Clay-1	50 kPa	13.0%	19.60
11	CM5			100 kPa	13.0%	19.60
12	CM6			150 kPa	13.0%	19.60
13	CM7	Intermediate Compaction	Clay-1	50 kPa	14.0%	18.42
14	CM8			100 kPa	14.0%	18.42
15	CM9			150 kPa	14.0%	18.42

Table – 4.3: Some typical test results of Dhaka clay type-1

Sample Designation	Specimen Designation	Peak Shear Stress (kPa)				
		S_0	S_{30}	S_{45}	S_{60}	S_{90}
Standard Proctor Compaction						
CM1	CM1-1	92.80	96.23	97.43	97.16	94.91
	CM1-2	76.21	78.81	79.69	77.81	72.65
	CM1-3	74.09	76.33	76.17	73.67	67.99
CM2	CM2-1	166.50	163.88	163.41	162.31	160.93
	CM2-2	152.78	150.69	149.32	146.63	141.94
	CM2-3	97.99	96.00	93.89	90.74	88.11
CM3	CM3-1	143.60	146.07	145.21	143.01	140.03
	CM3-2	138.60	142.37	143.69	139.55	135.49
	CM3-3	126.67	130.40	132.48	127.65	130.25
Modified Proctor Compaction						
CM4	CM4	70.05	78.17	76.83	74.02	73.65
CM5	CM5	87.43	108.64	110.34	106.13	101.29
CM6	CM6	96.69	122.56	119.24	118.79	108.67
Intermediate Compaction						
CM7	CM7	104.37	118.02	108.56	104.72	100.43
CM8	CM8	141.00	143.29	139.23	136.23	123.25
CM9	CM9	147.21	147.75	143.34	135.67	132.14

Table - 4.4: Coefficient of Anisotropy, $I = S_\theta/S_0$ in Dhaka clay type-1

Sample Designation	Specimen Designation	Coefficient of Anisotropy, $I = S_\theta/S_0$				
		S_0/S_0	S_{30}/S_0	S_{45}/S_0	S_{60}/S_0	S_{90}/S_0
Standard Proctor Compaction						
CM1	CM1-1	1.00	1.037	1.05	1.04	1.02
	CM1-2	1.00	1.034	1.04	1.02	0.95
	CM1-3	1.00	1.030	1.03	0.99	0.92
CM2	CM2-1	1.00	0.984	0.981	0.97	0.97
	CM2-2	1.00	0.986	0.977	0.96	0.93
	CM2-3	1.00	0.979	0.96	0.93	0.90
CM3	CM3-1	1.00	1.02	1.01	0.99	0.97
	CM3-2	1.00	1.027	1.03	1.006	0.98
	CM3-3	1.00	1.029	1.04	1.008	1.03
Modified Proctor Compaction						
CM4	CM4	1.00	1.11	1.09	1.05	1.05
CM5	CM5	1.00	1.24	1.26	1.21	1.16
CM6	CM6	1.00	1.26	1.23	1.23	1.12
Intermediate Compaction						
CM7	CM7	1.00	1.13	1.04	1.00	0.96
CM8	CM8	1.00	1.01	0.98	0.96	0.87
CM9	CM9	1.00	1.00	0.97	0.92	0.90

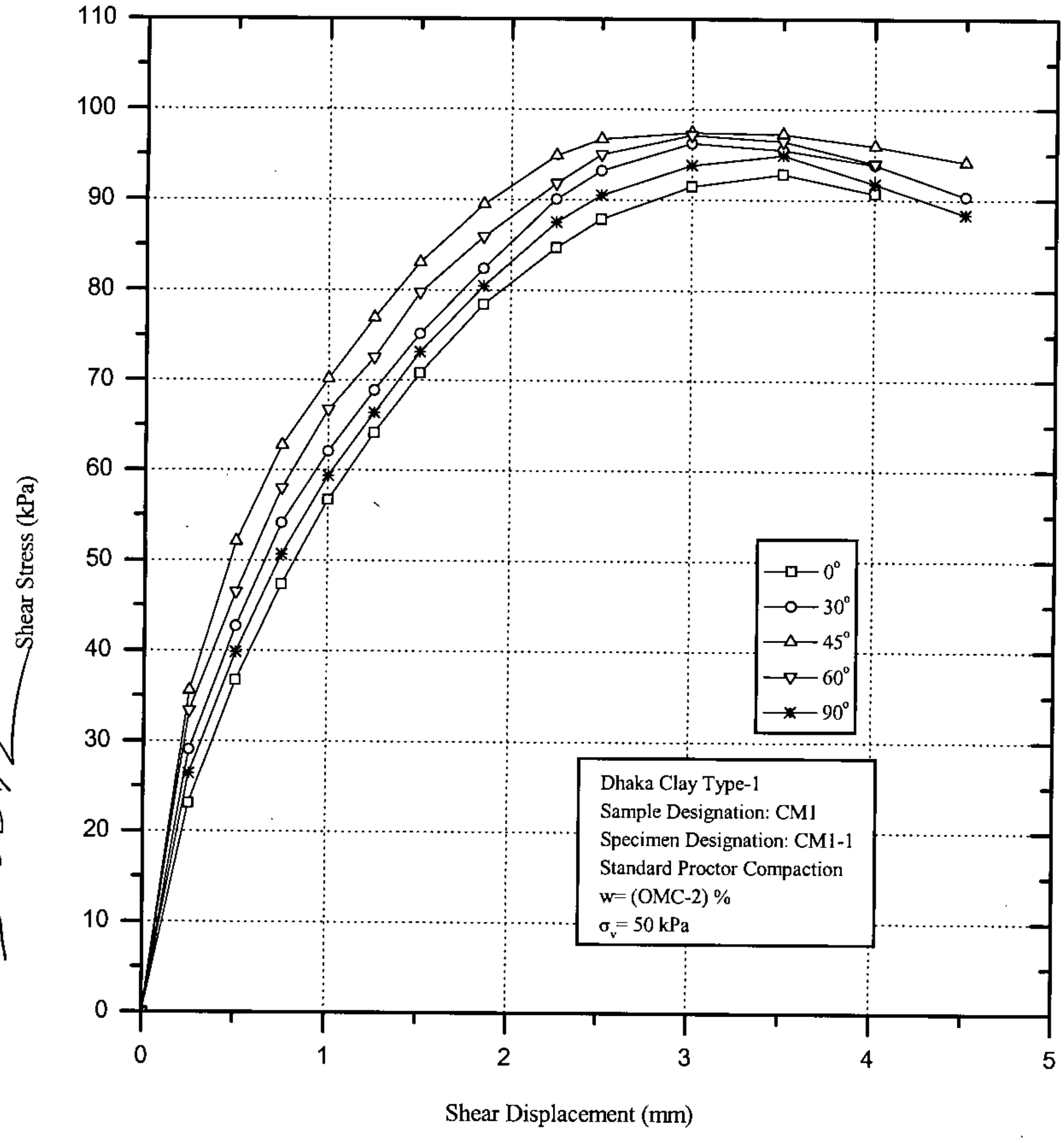


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Fig. 4.3a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM1-2 for σ_v= 50 kPa and w= (OMC-2) %

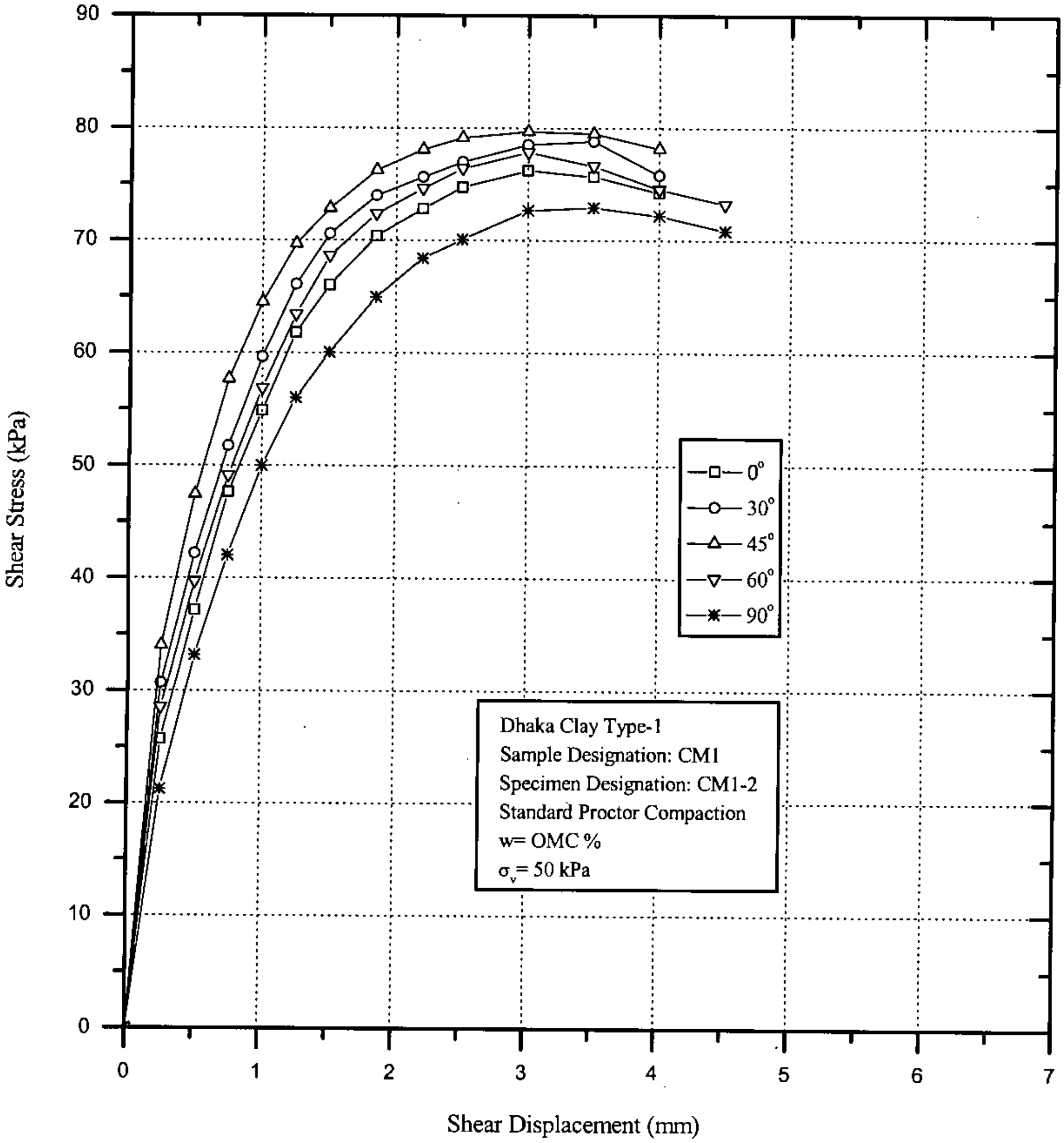

Fig. 4.3b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM1-2 for σ_v= 50 kPa and w= OMC %

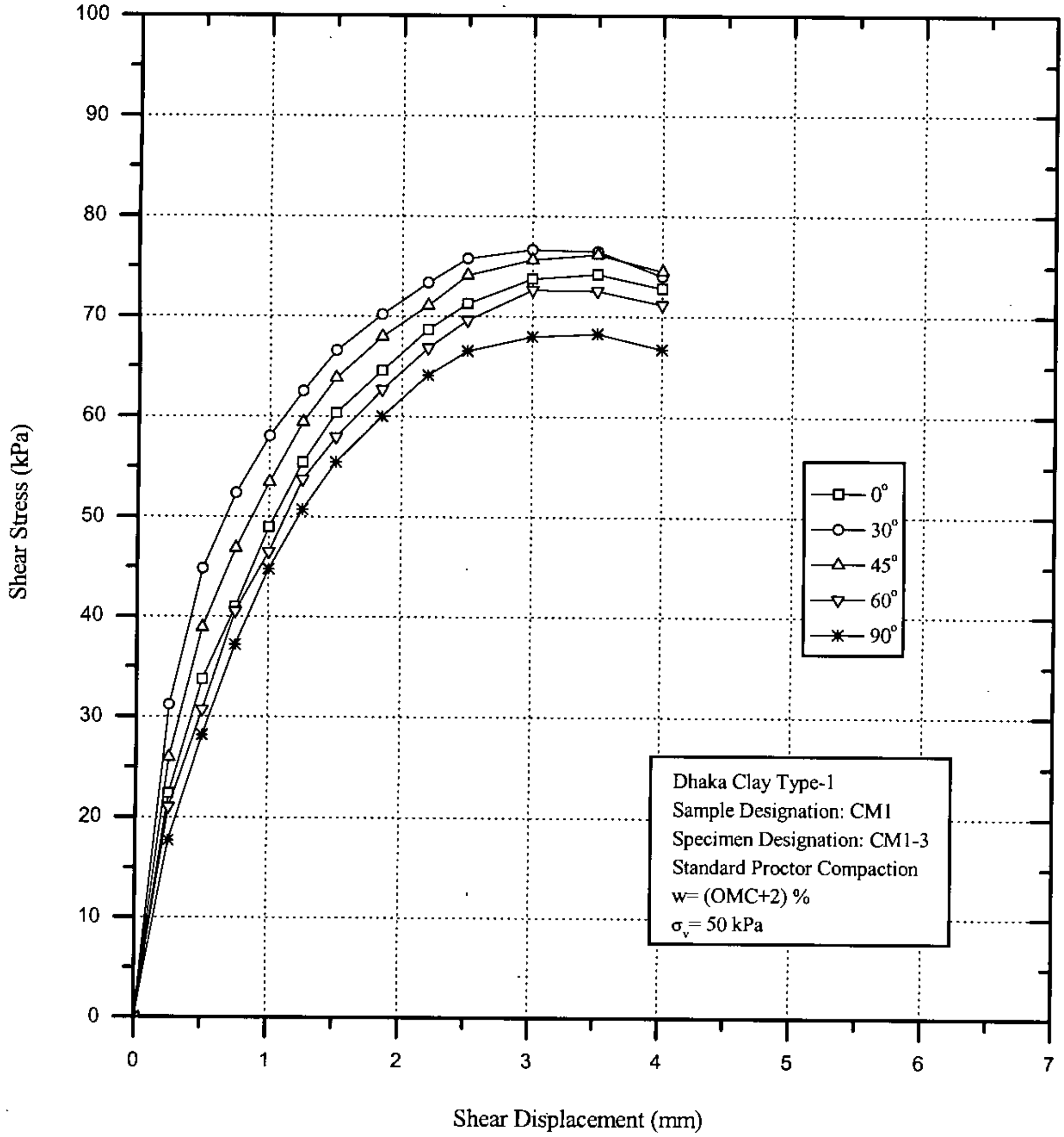

Fig. 4.3c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM1-3 for σ_v= 50 kPa and w= (OMC+2) %

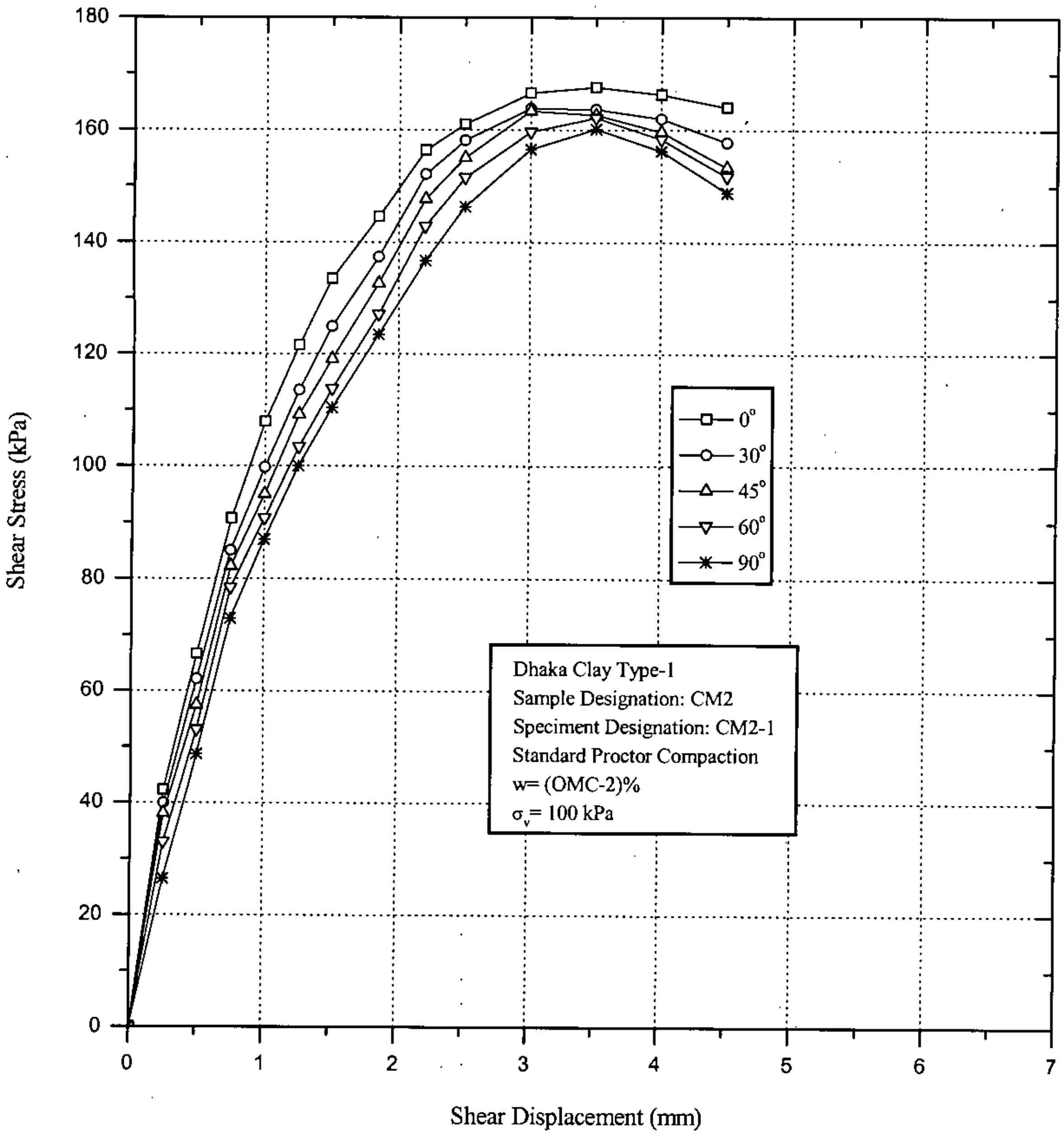

Fig. 4.4a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM2-1 for σ_v= 100 kPa and w= (OMC-2) %

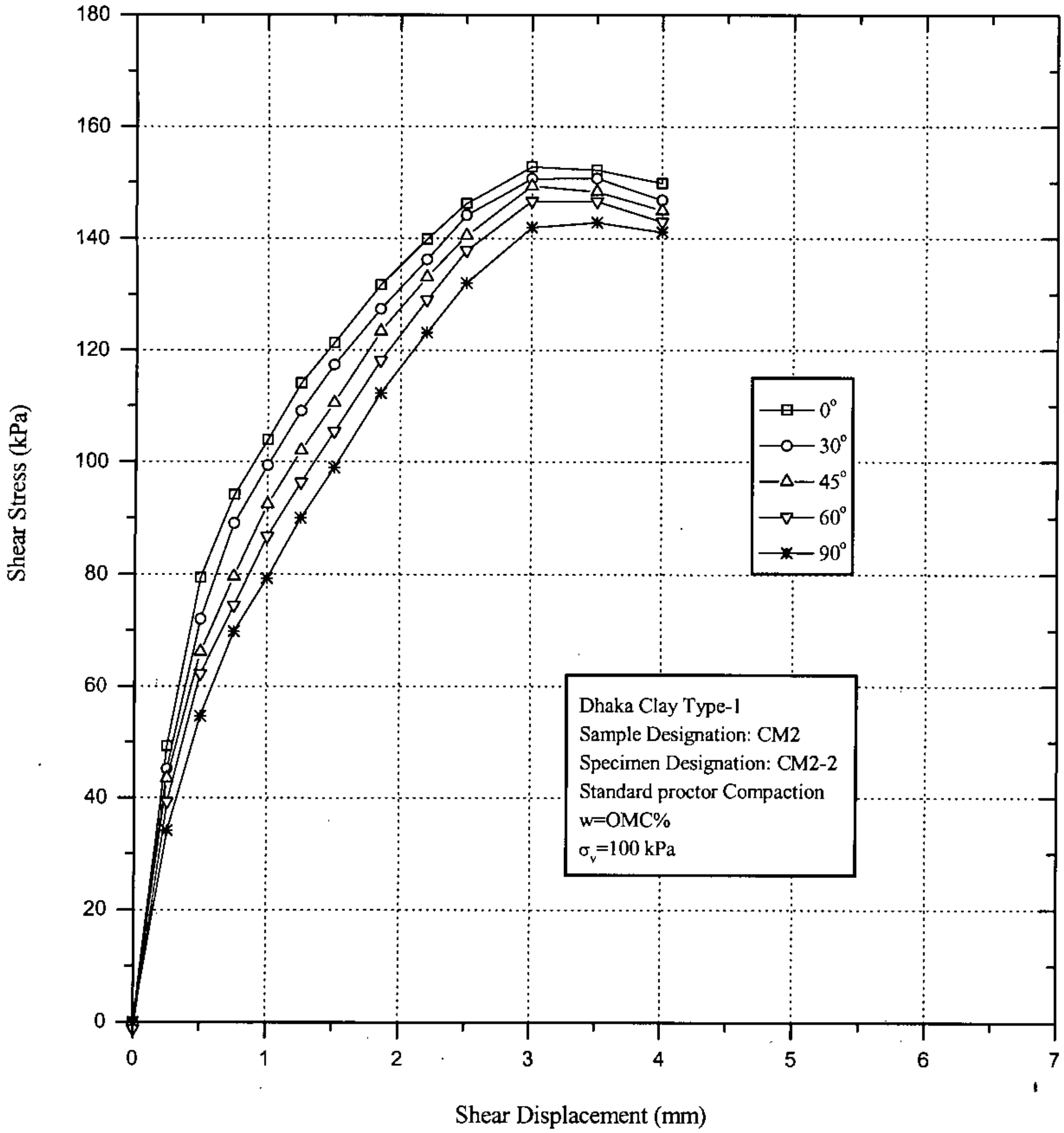

Fig. 4.4b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM2-2 for σ_v= 100 kPa and w= OMC %

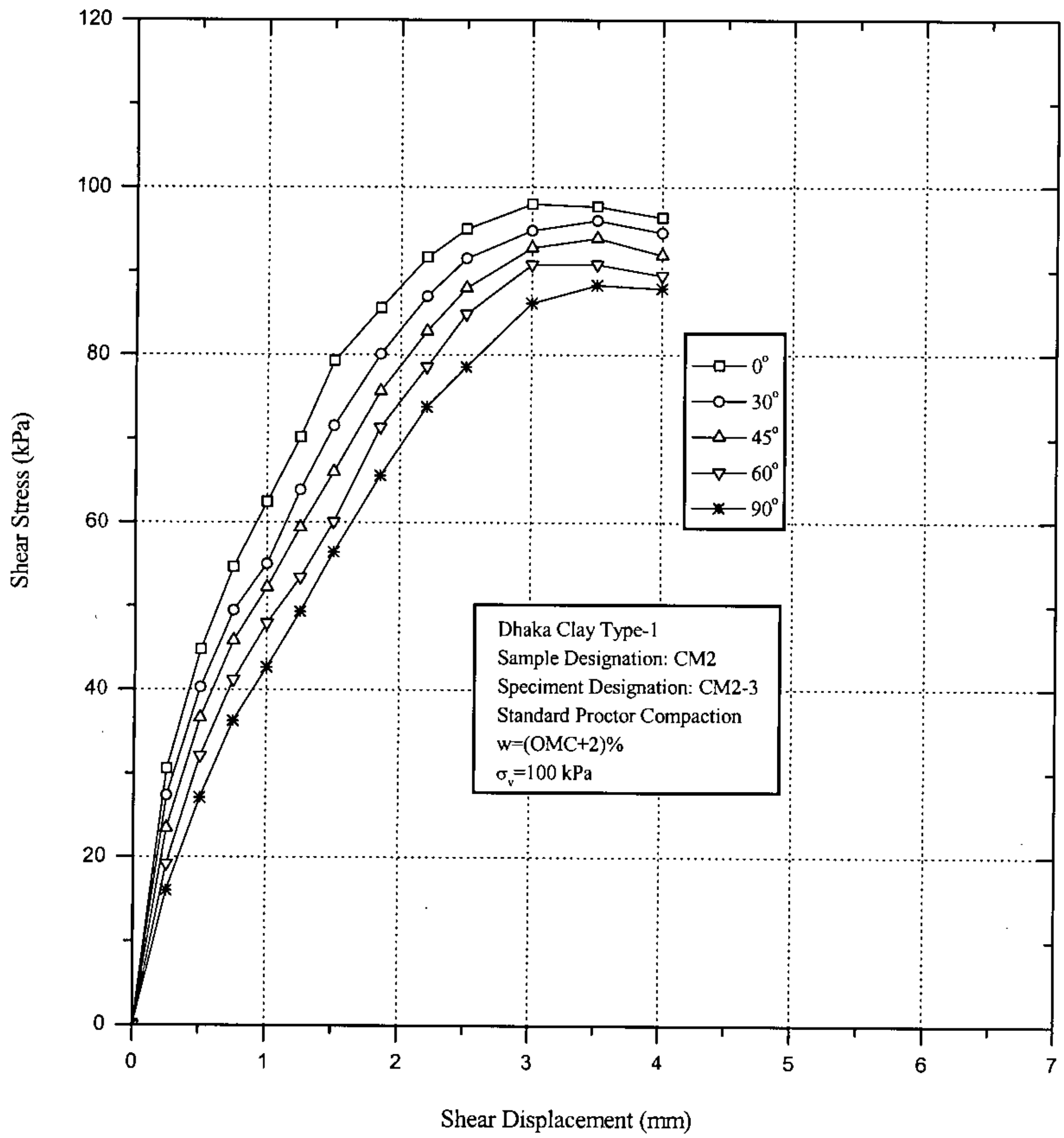

Fig. 4.4c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM2-3 for σ_v= 100 kPa and w= (OMC+2) %

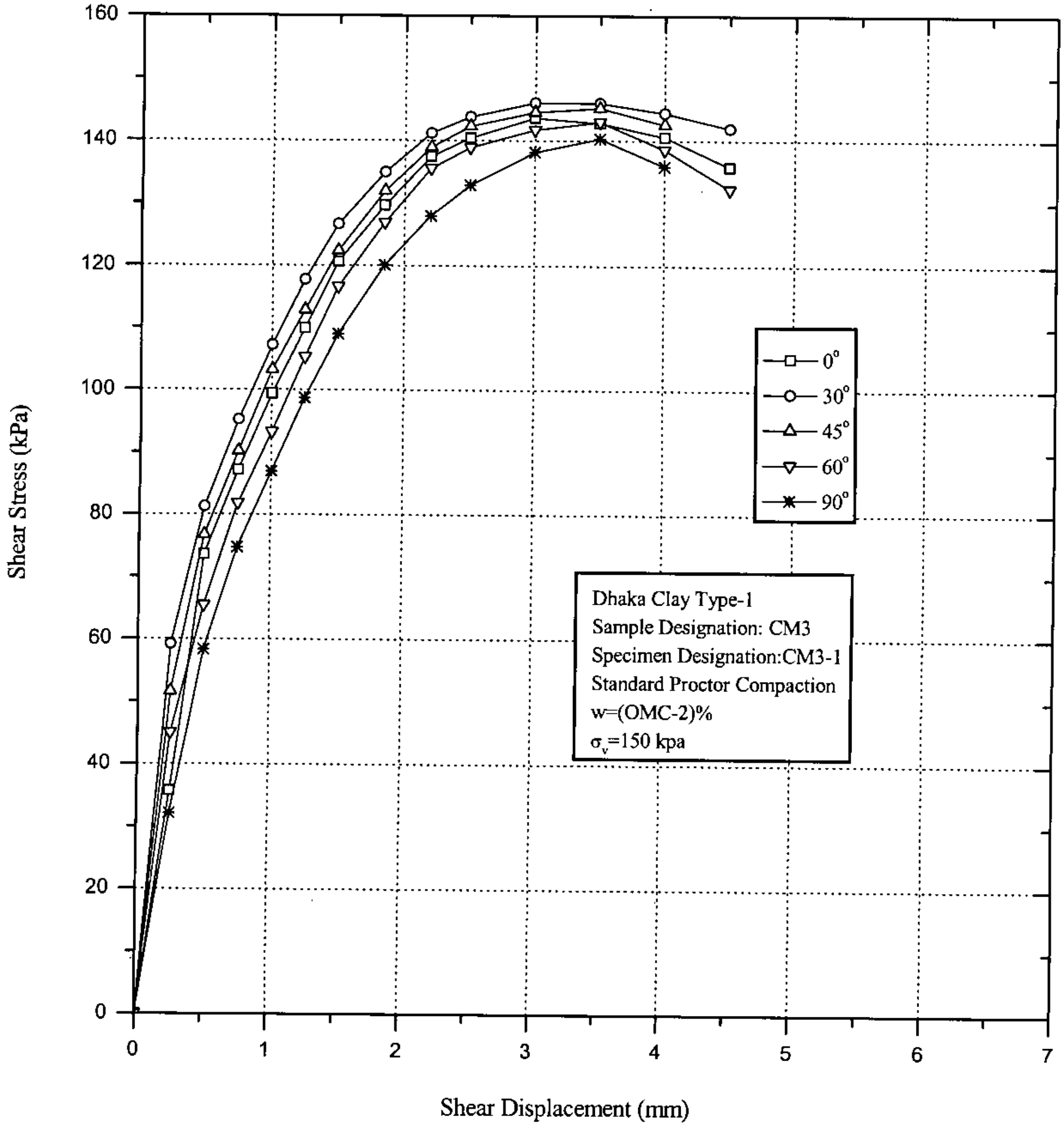

Fig. 4.5a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM3-1 for σ_v= 150 kPa and w= (OMC-2) %

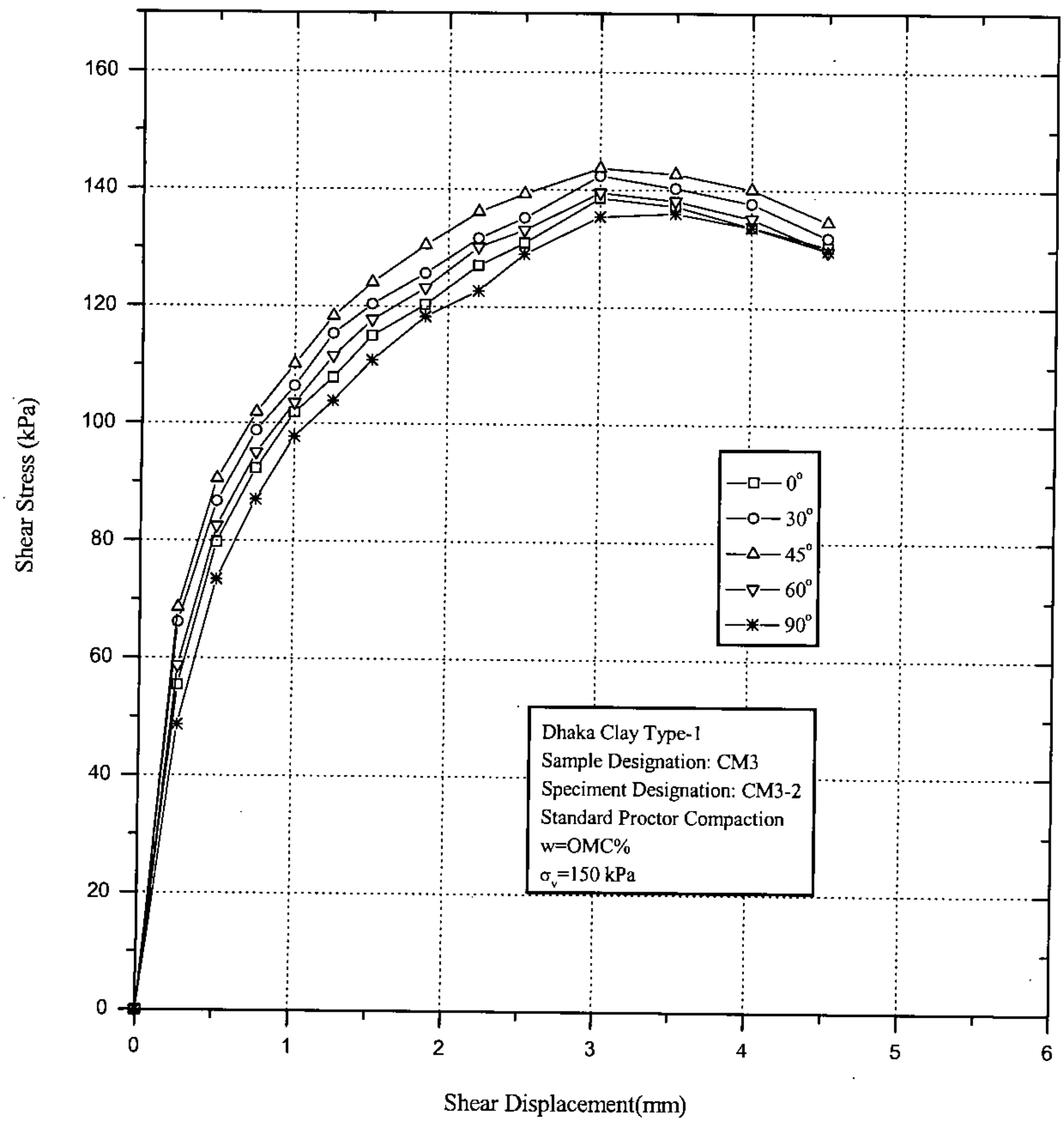

Fig. 4.5b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM3-2 for σ_v= 150 kPa and w= OMC %

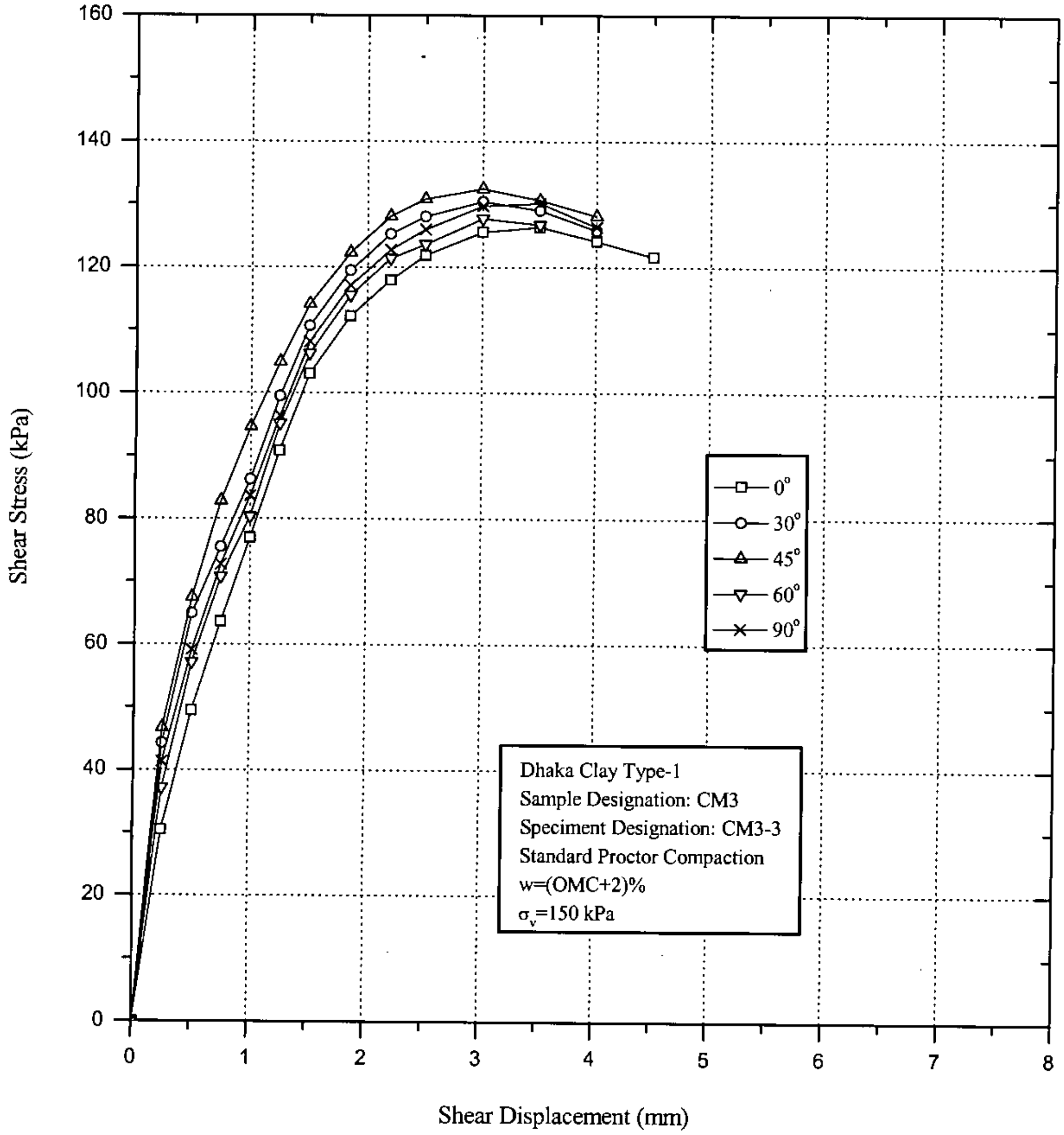

Fig. 4.5c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM3-3 for σ_v= 150 kPa and w= (OMC+2) %

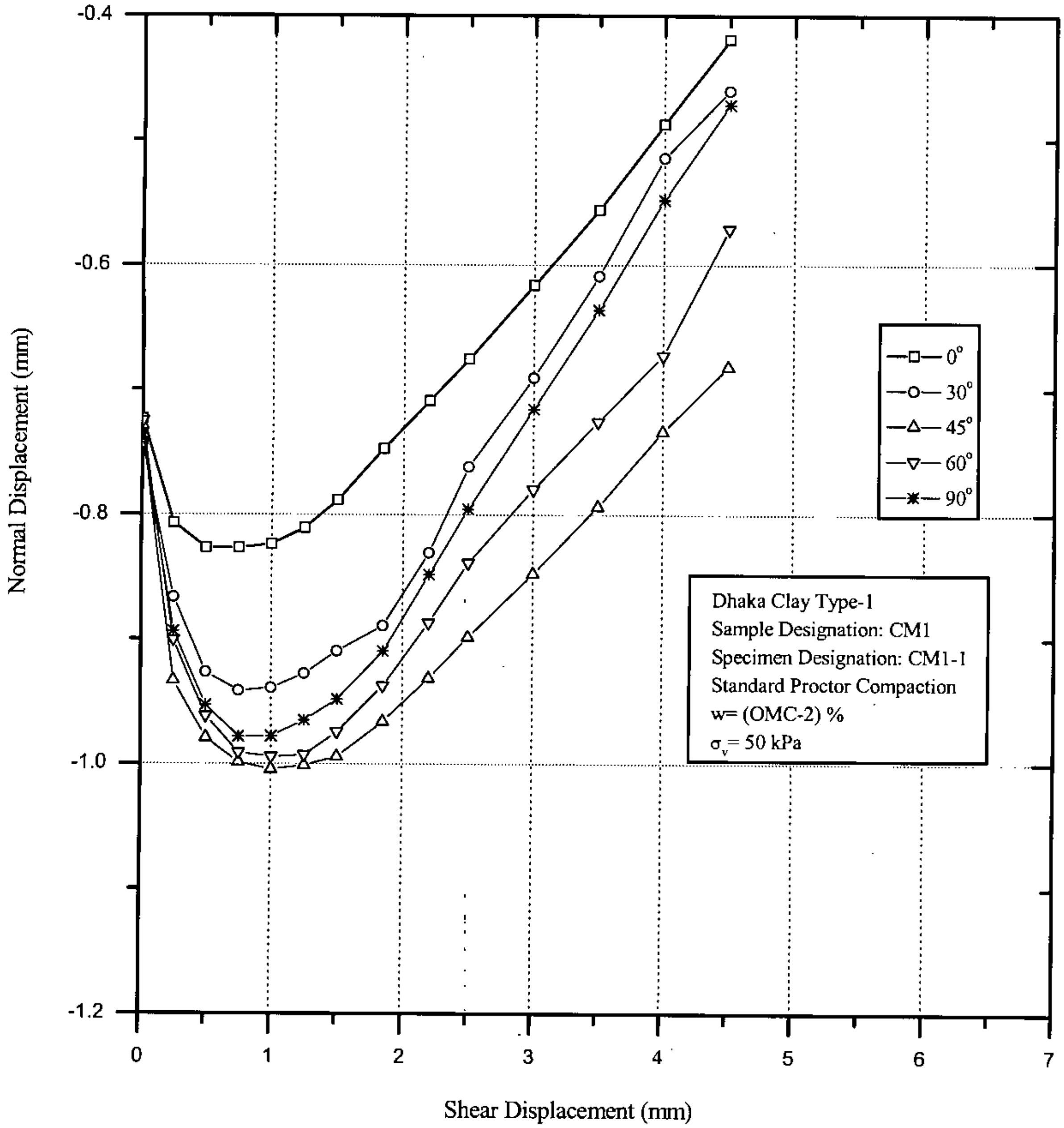

Fig. 4.6a: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM1-1 for σ_v= 50 kPa and w= (OMC-2) %

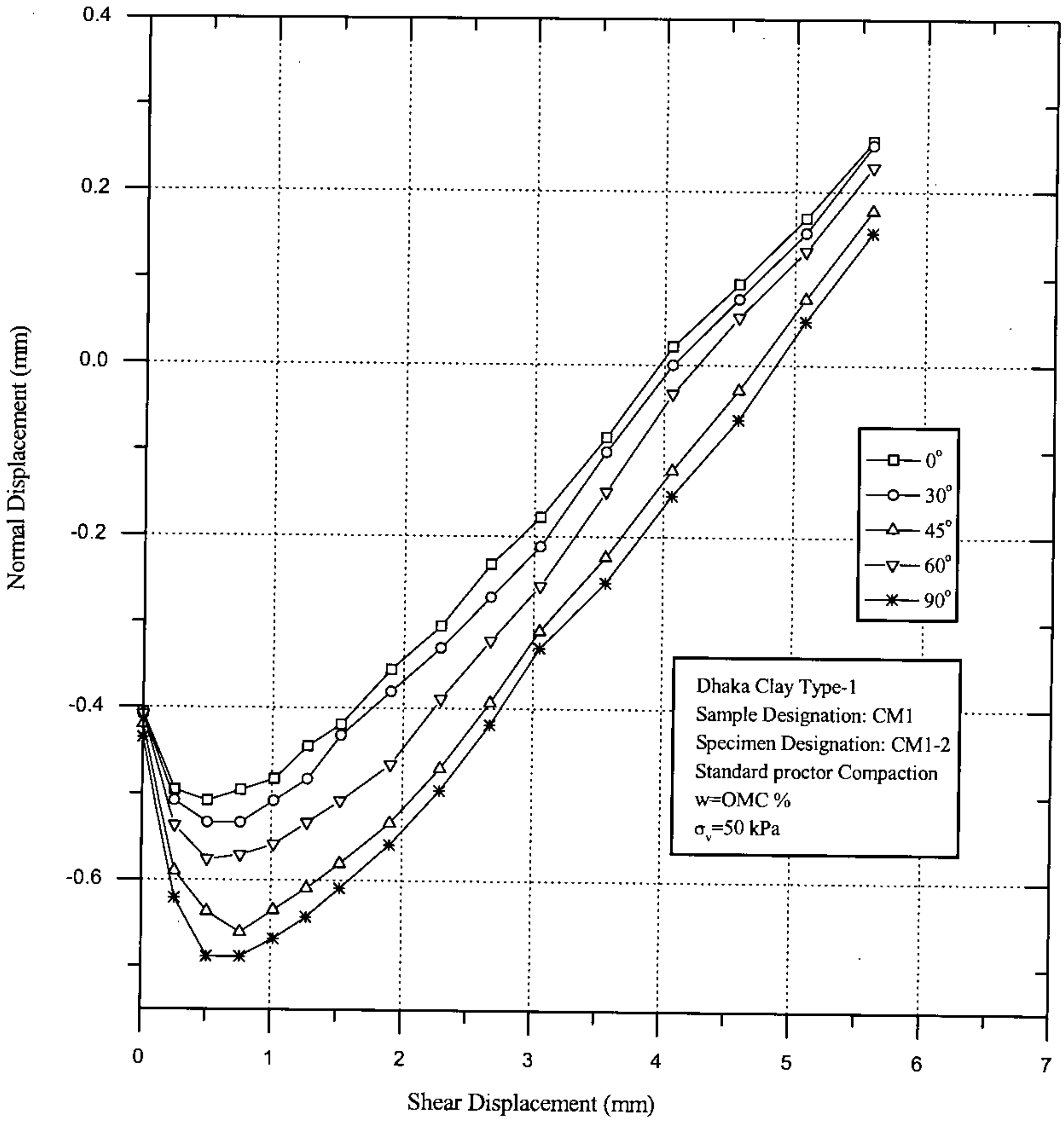

Fig. 4.6b: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM1-2 for σ_v= 50 kPa and w= OMC %

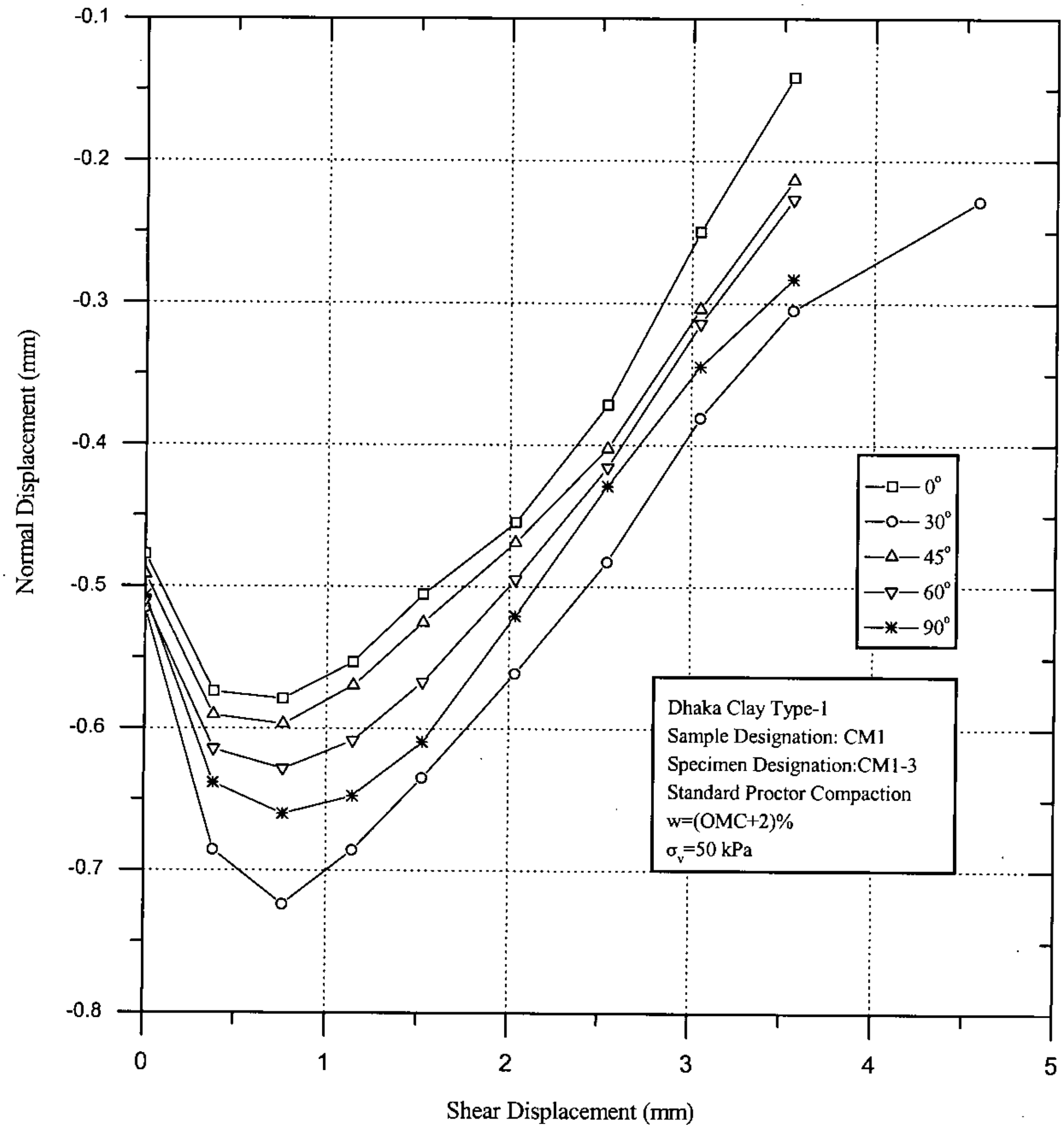

Fig. 4.6c: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM1-3 for σ_v = 50 kPa and w= (OMC+2) %

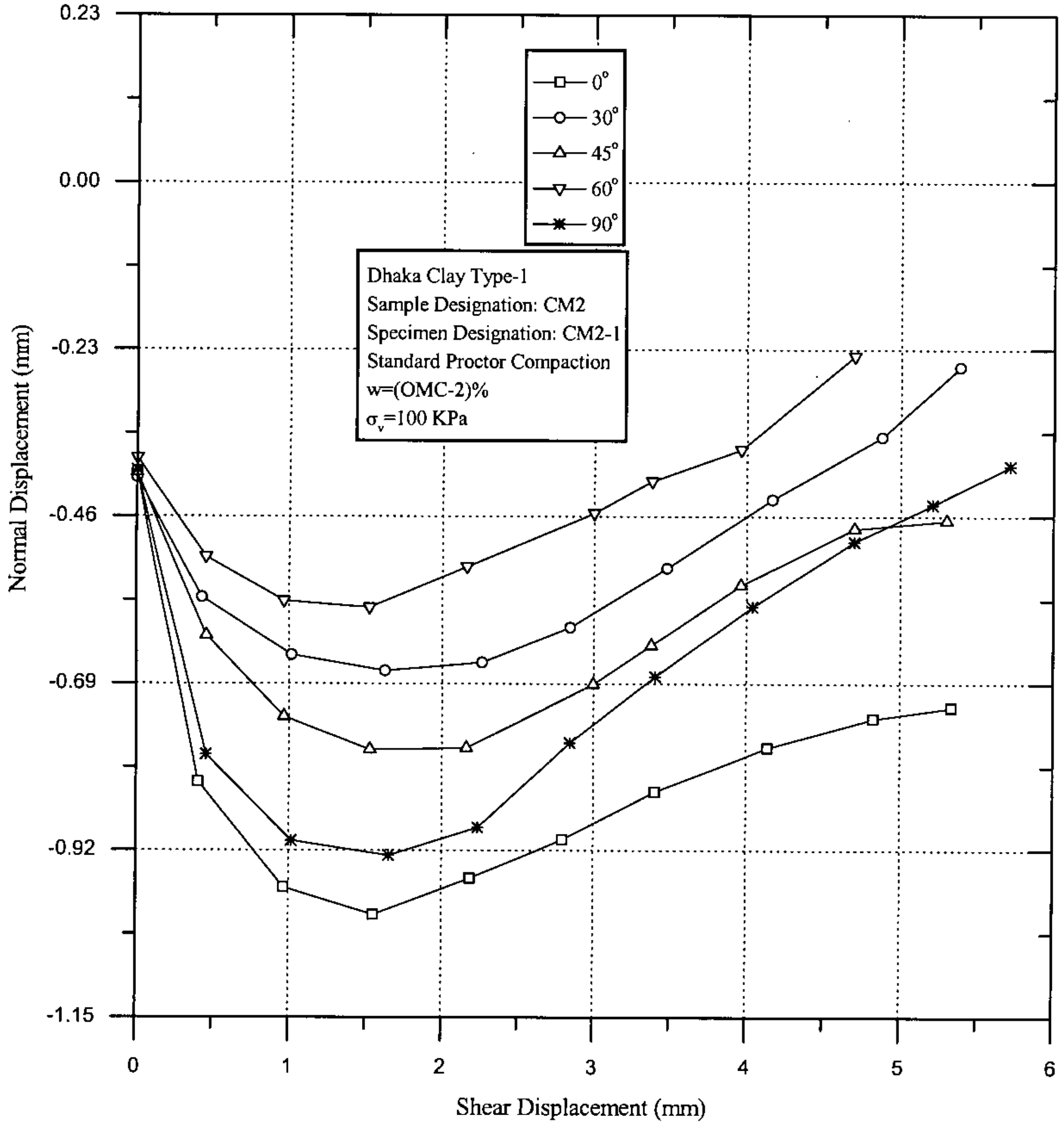

Fig. 4.7a: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM2-1 for σ_v= 100 kPa and w= (OMC-2) %

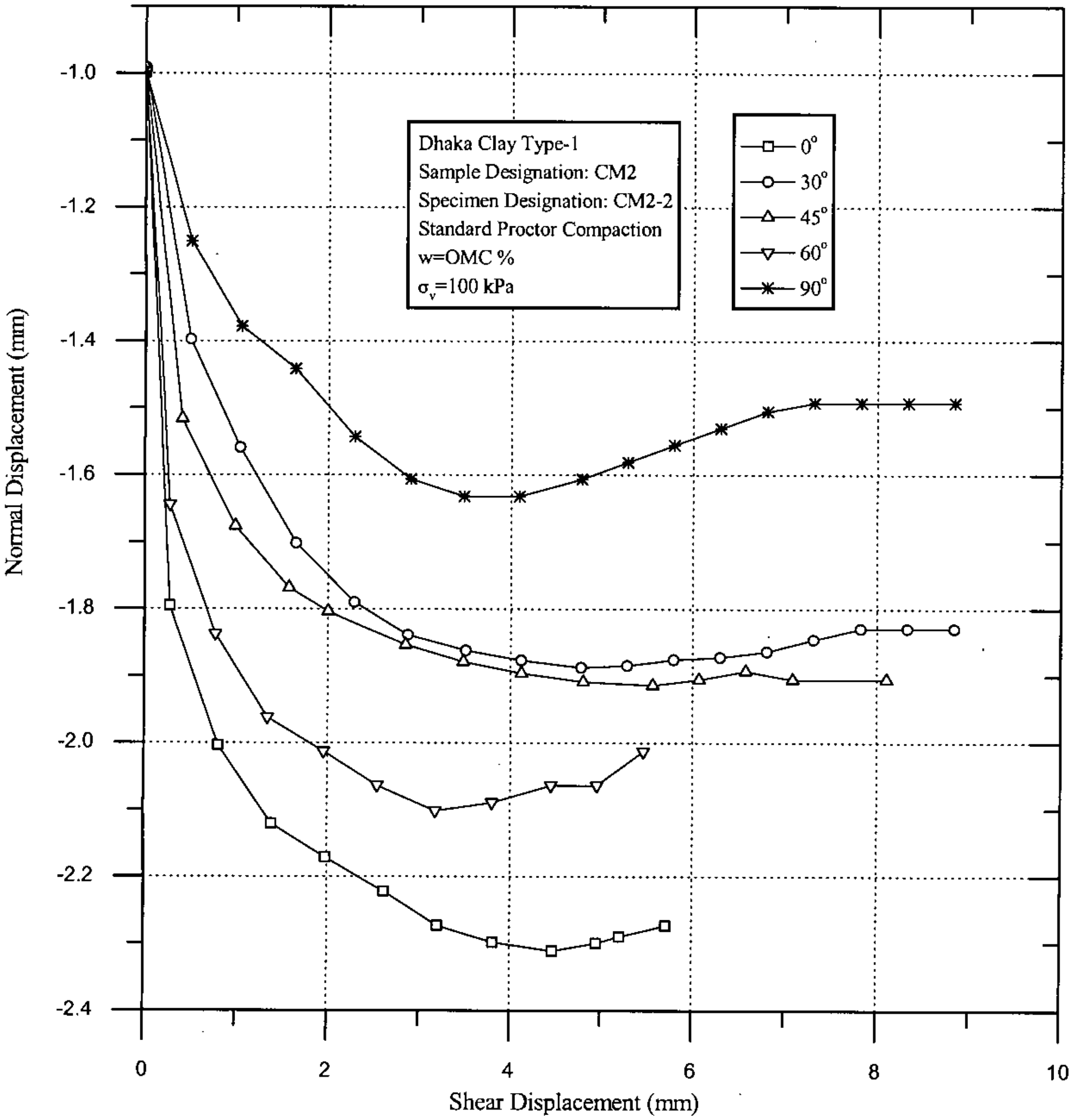

Fig. 4.7b: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM2-2 for σ_v= 100 kPa and w= OMC %

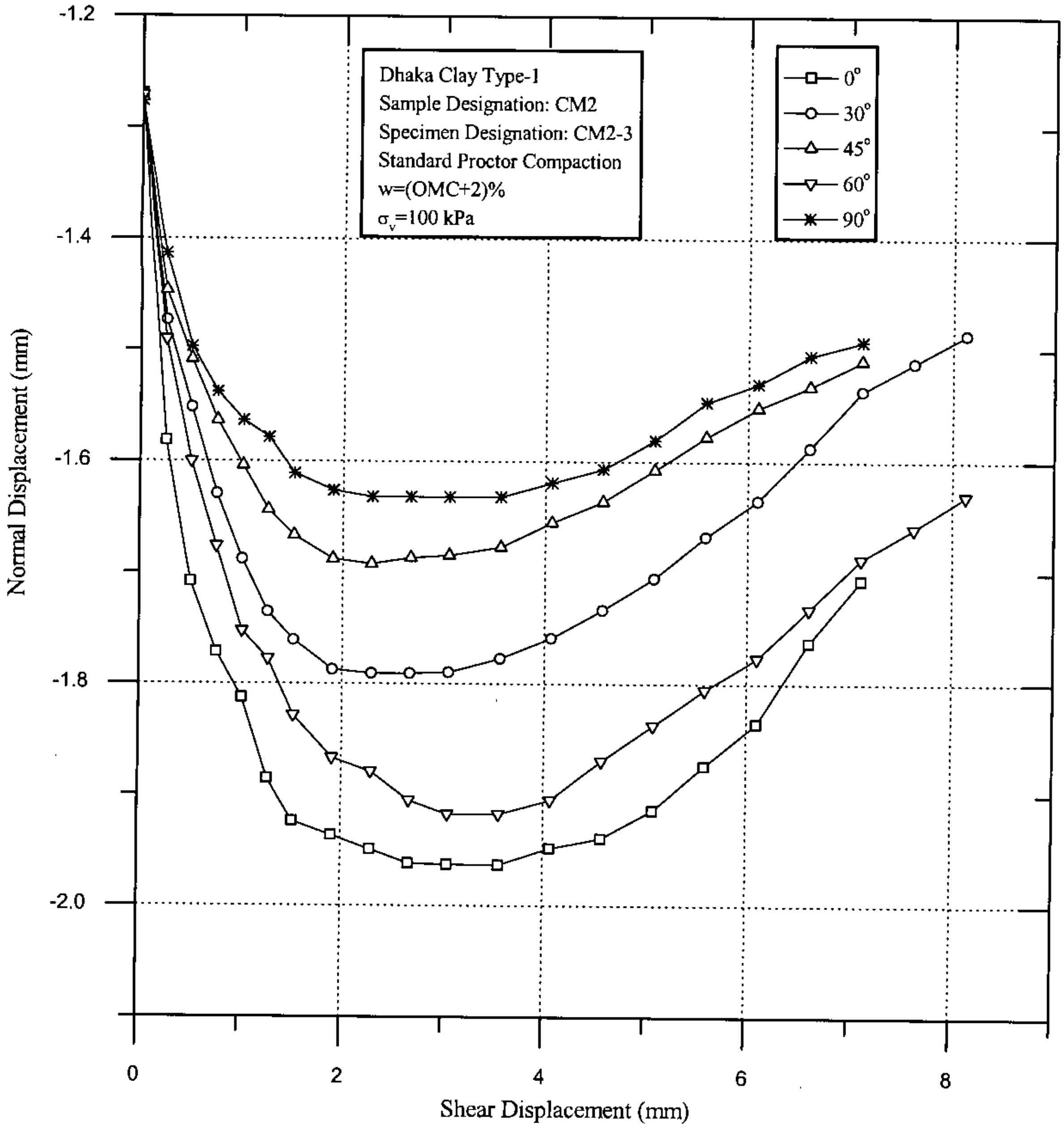

Fig. 4.7c: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM2-3 for σ_v= 100 kPa and w= (OMC+2) %

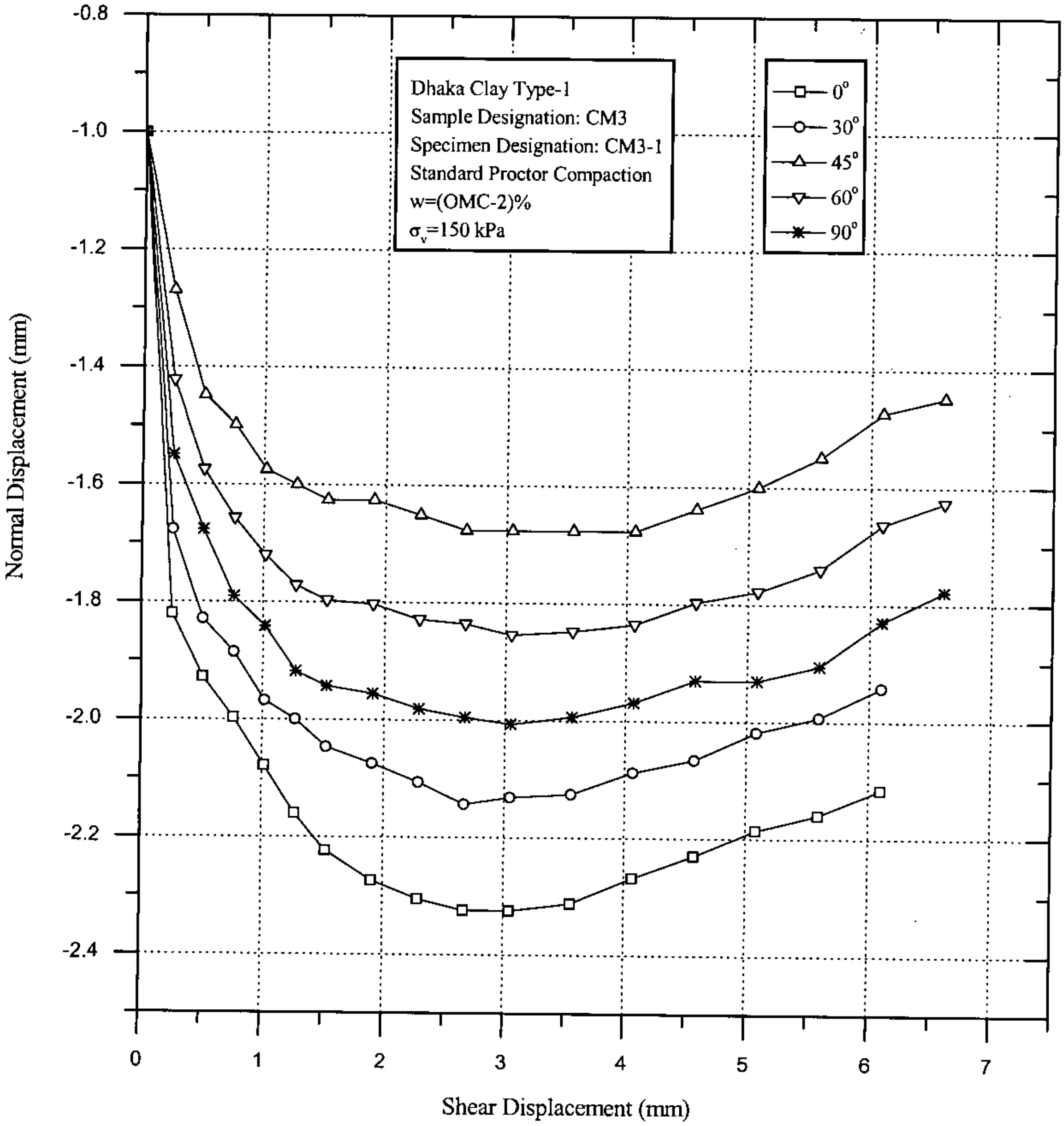

Fig. 4.8a: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM3-1 for σ_v= 150 kPa and w= (OMC-2) %

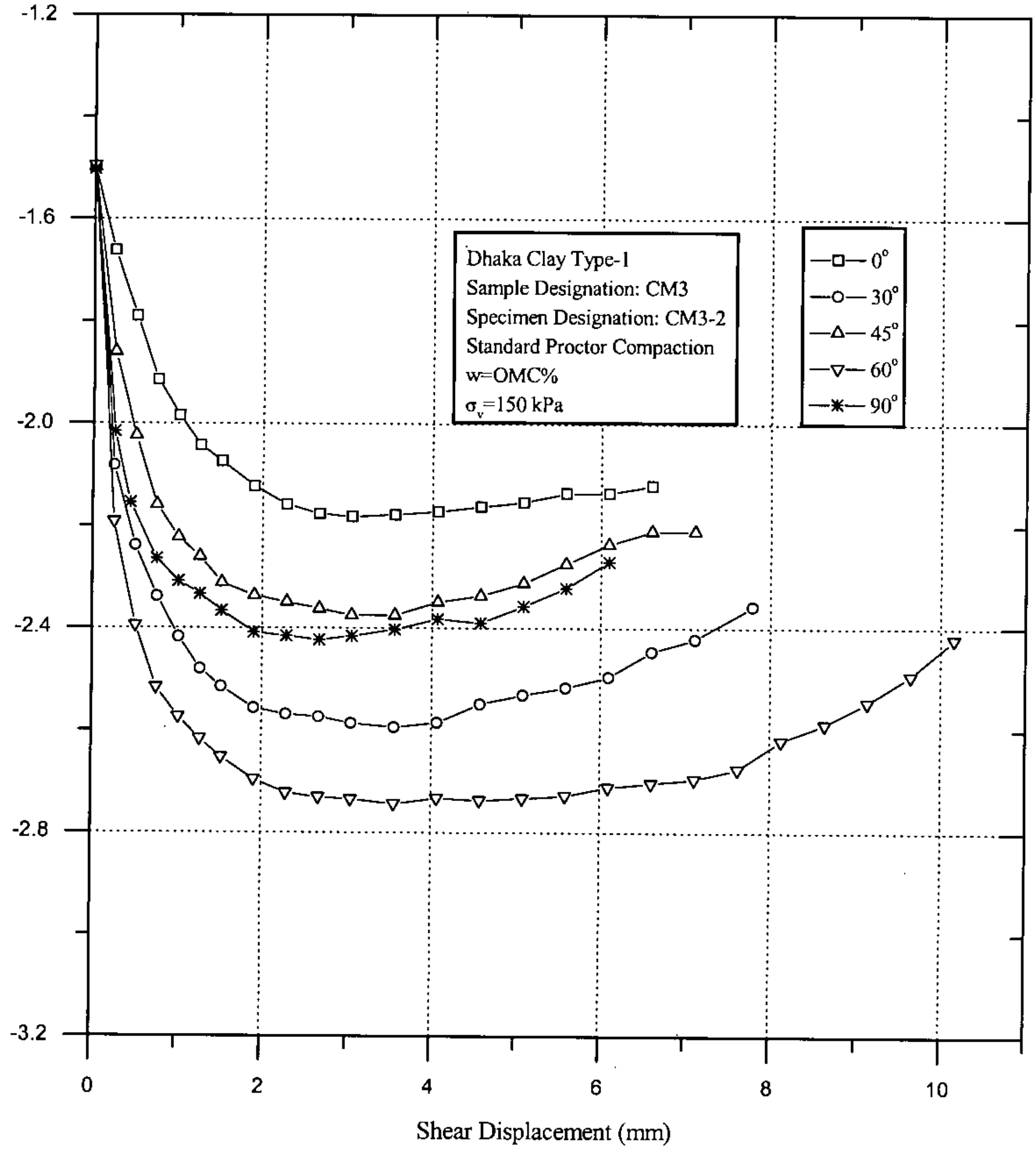

Fig. 4.8b: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM3-2 for σ_v= 150 kPa and w= OMC %

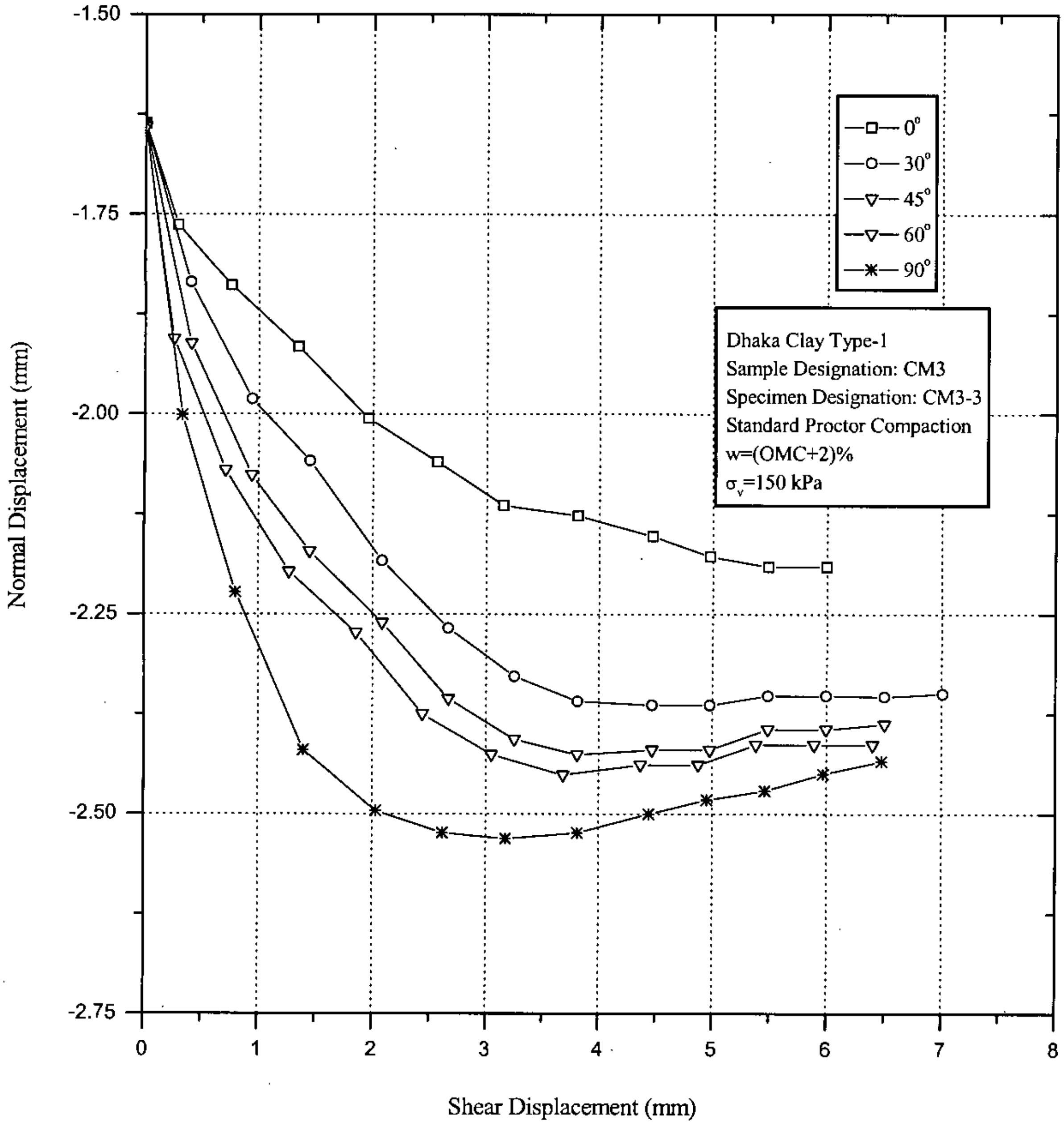

Fig. 4.8c: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM3-3 for σ_v= 150 kPa and w= (OMC+2) %

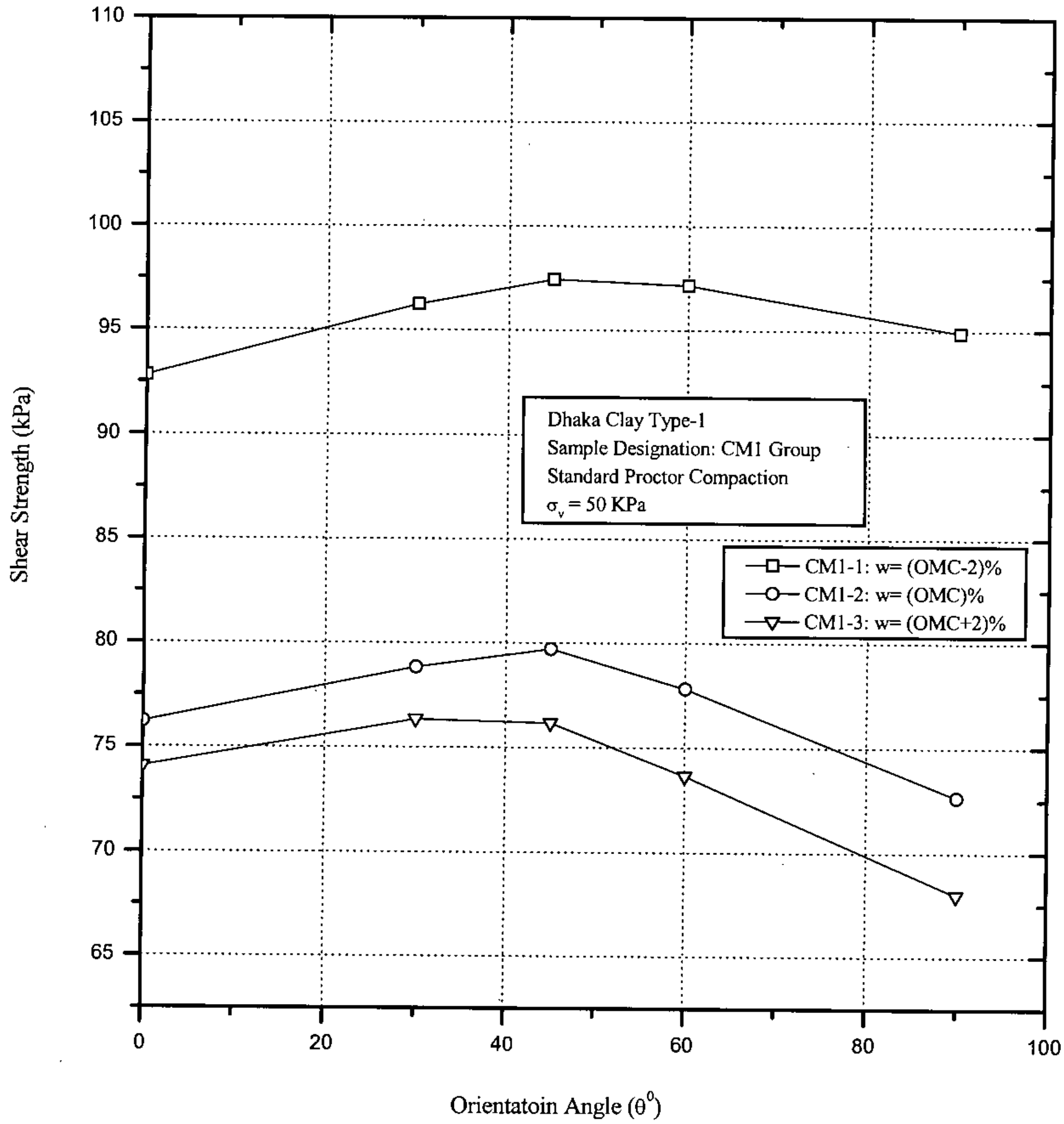

Fig. 4.9a: Relationship between shear strength and orientation angle θ of Dhaka Clay Type-1 of specimen group CM1 for σ_v= 50 kPa and different moisture content

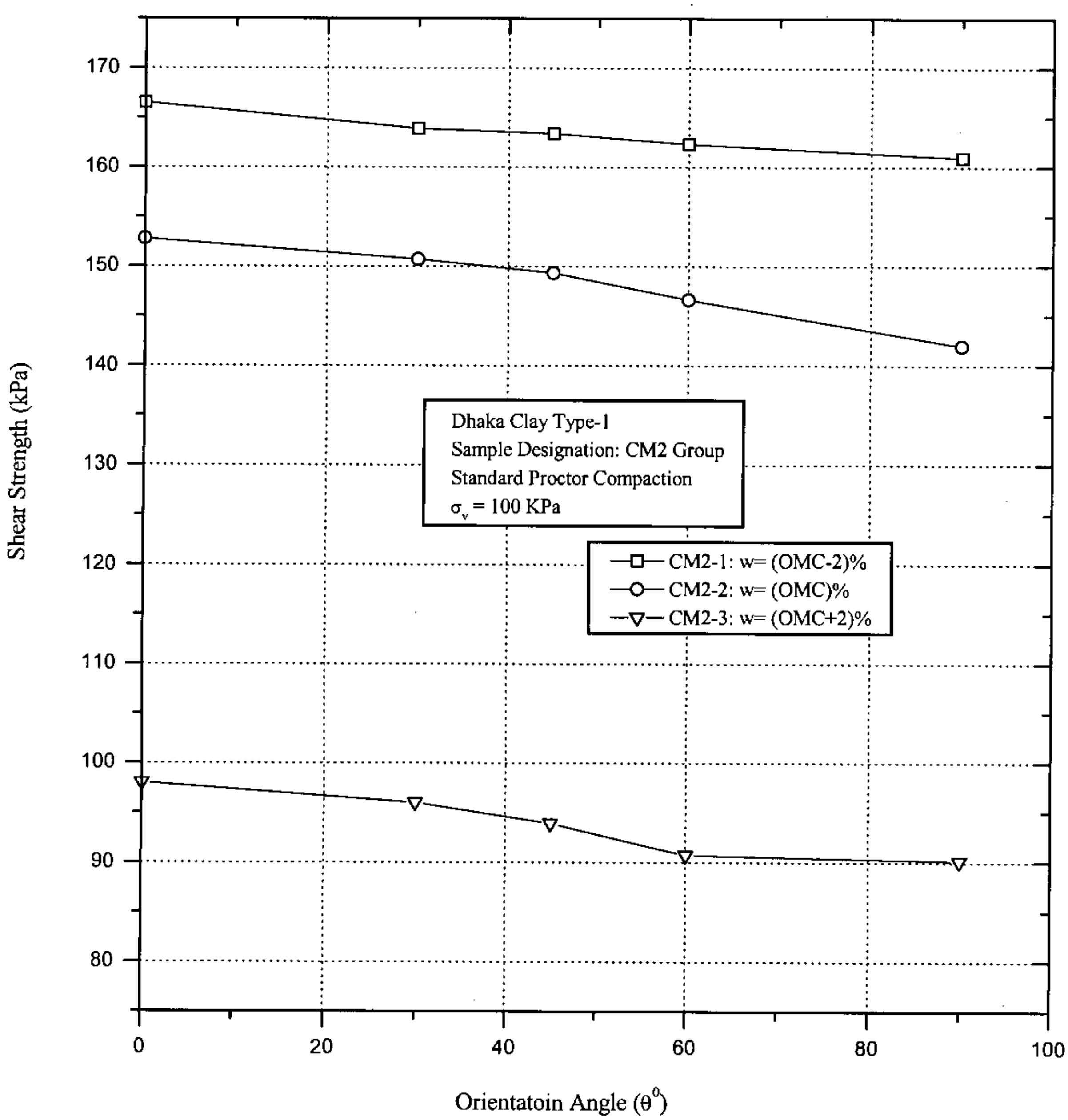

Fig. 4.9b: Relationship between shear strength and orientation angle θ of Dhaka Clay Type-1 of specimen group CM2 for σ_v= 100 kPa and different moisture content

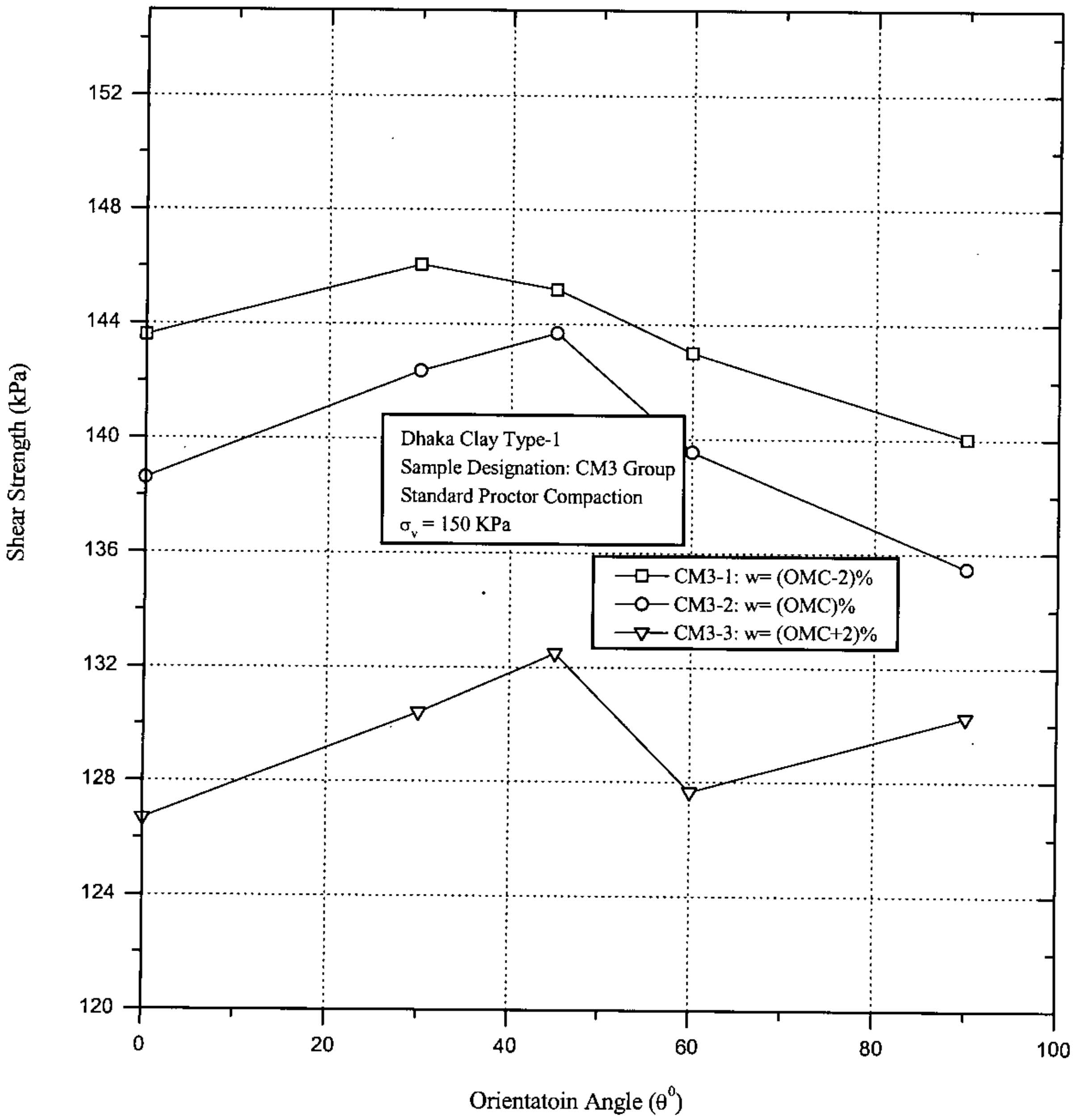

Fig. 4.9c: Relationship between shear strength and orientation angle θ of Dhaka Clay Type-1 of specimen group CM3 for σ_v= 150 kPa and different moisture content

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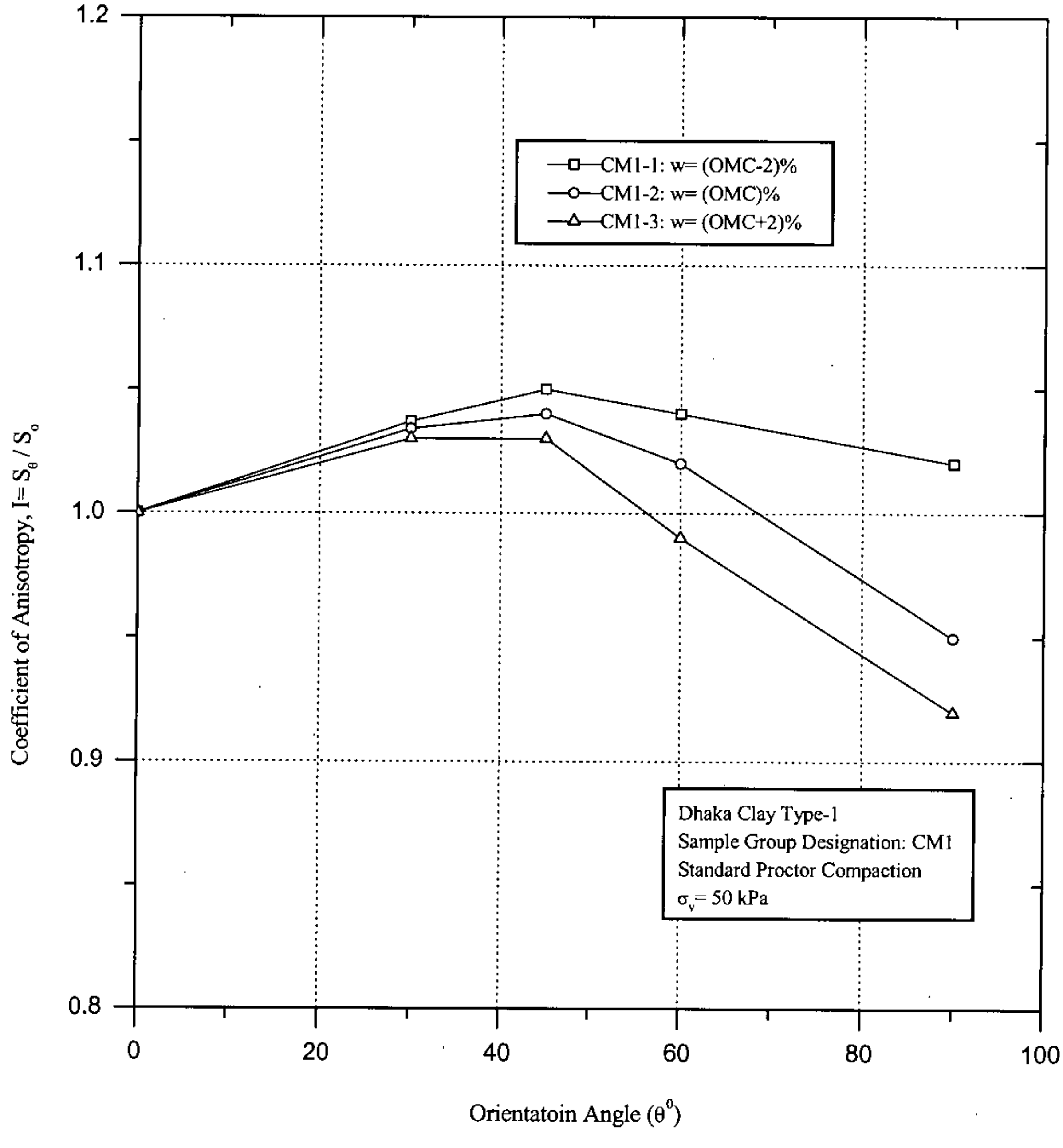


Fig. 4.10a: The variation of I~θ for Standard Proctor compaction effort on Dhaka Clay Type-1 of specimen group CM1 for σ_v= 50 kPa and different moisture content

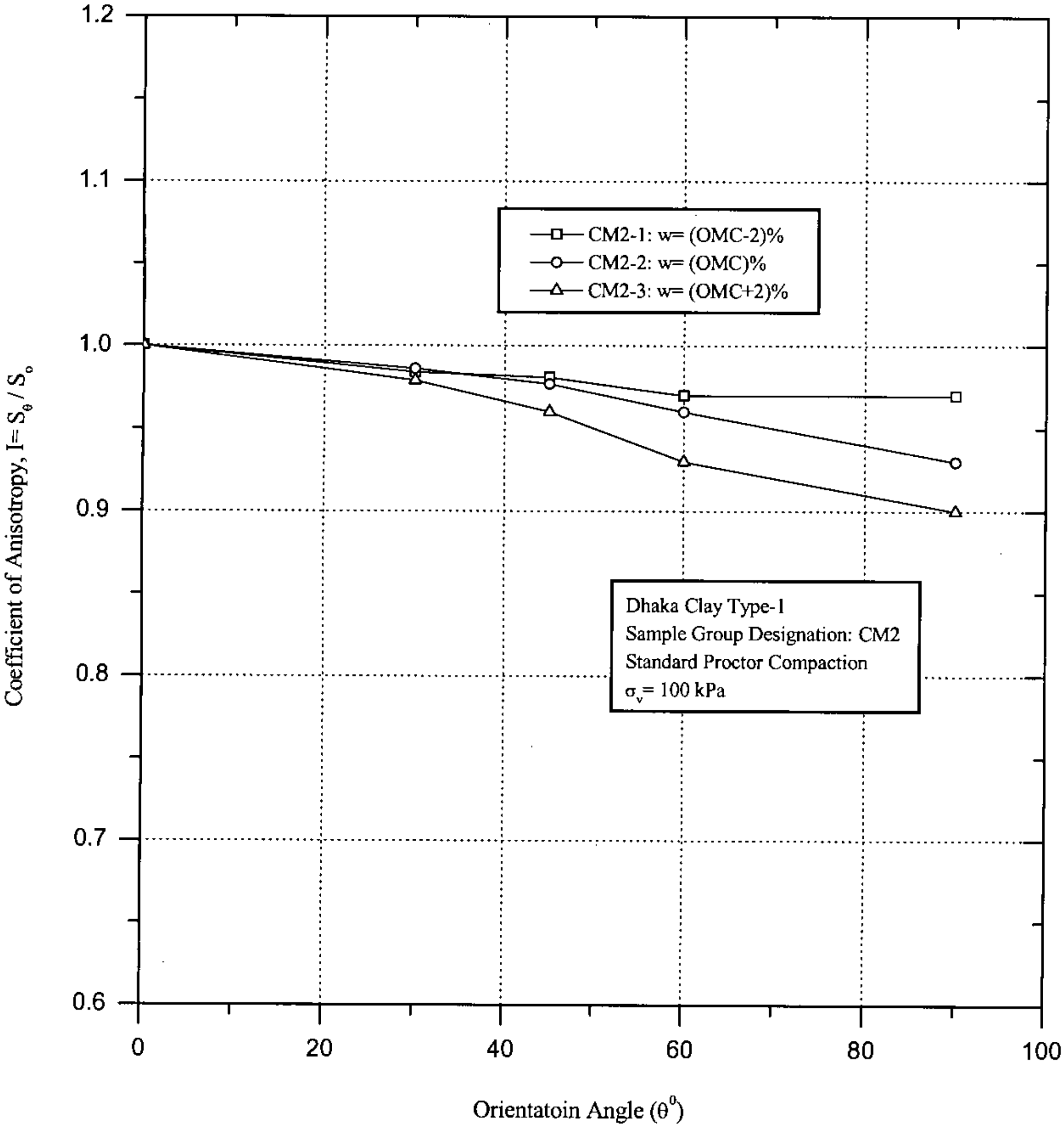


Fig. 4.10b: The variation of I~θ for Standard Proctor compaction effort on Dhaka Clay Type-1 of specimen group CM2 for σ_v= 100 kPa and different moisture content

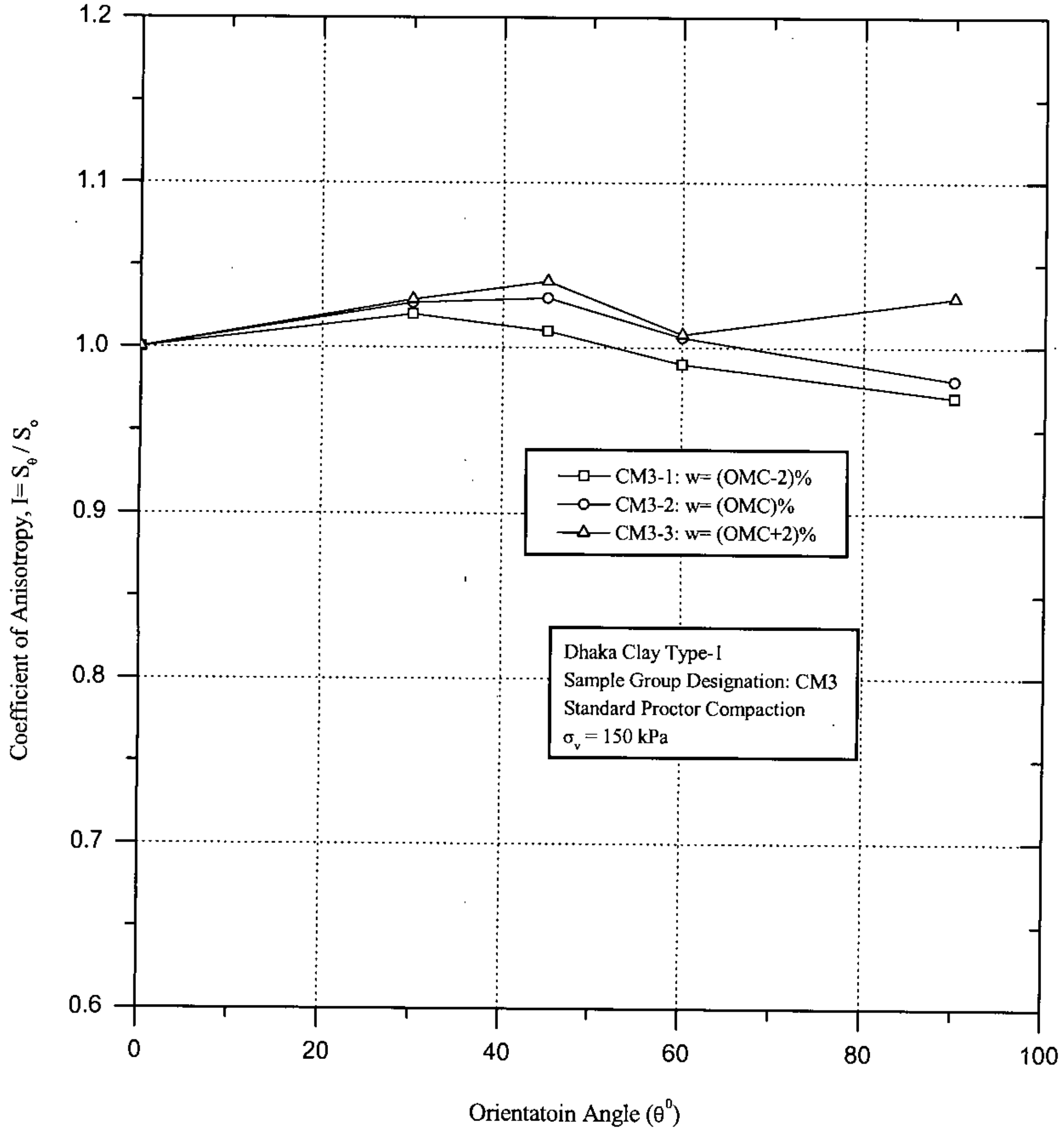


Fig. 4.10c: The variation of I~θ for Standard Proctor compaction effort on Dhaka Clay Type-1 of specimen group CM3 for σ_v= 150 kPa and different moisture content

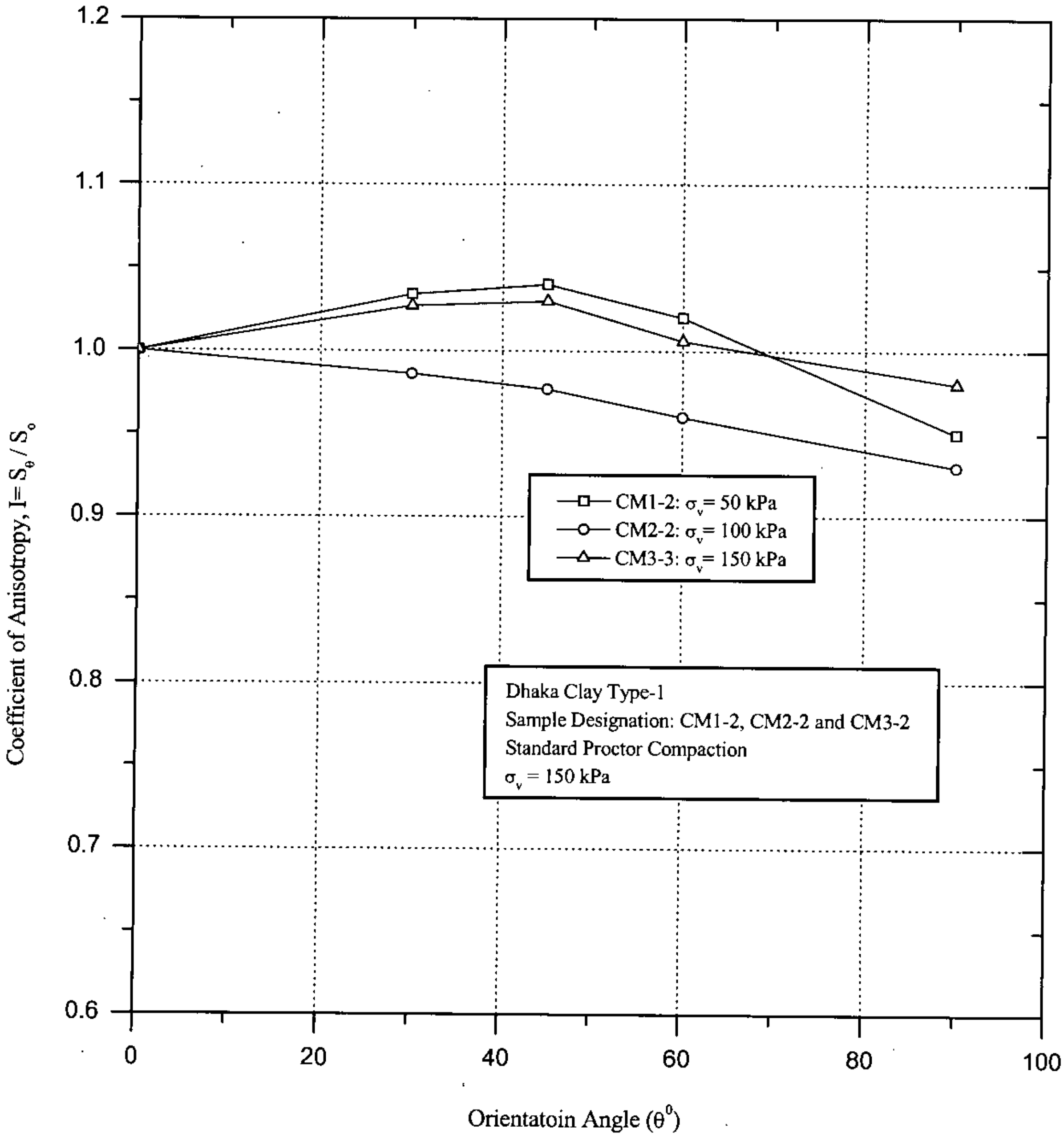
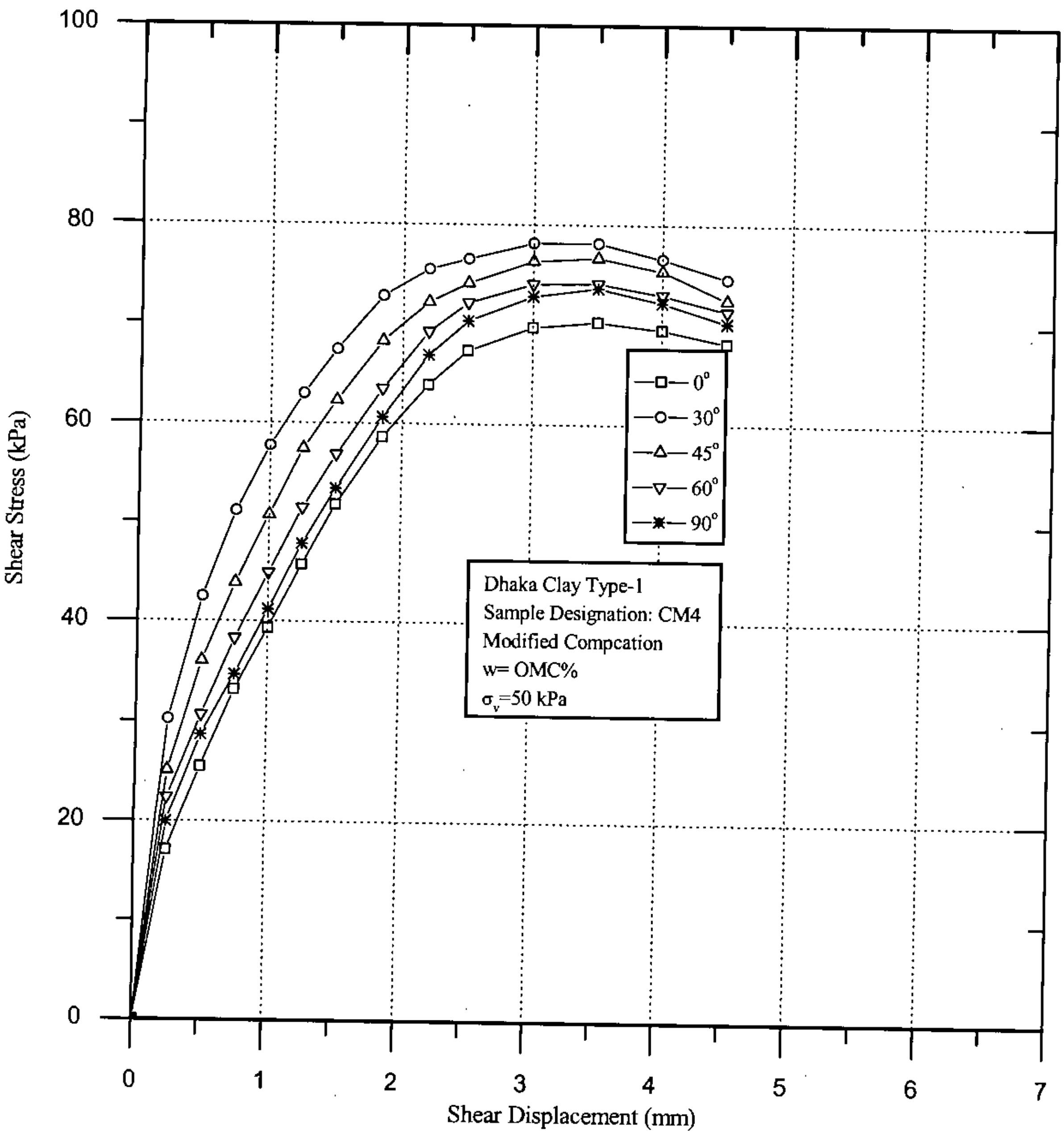


Fig. 4.10d: The variation of I~θ for Standard Proctor compaction effort on Dhaka Clay Type-1 of specimens CM1-2, CM2-2 and CM3-2 for σ_v= 50~150 kPa at optimum moisture content



g. 4.11a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation CM4 for σ_v= 50 kPa and w= OMC %

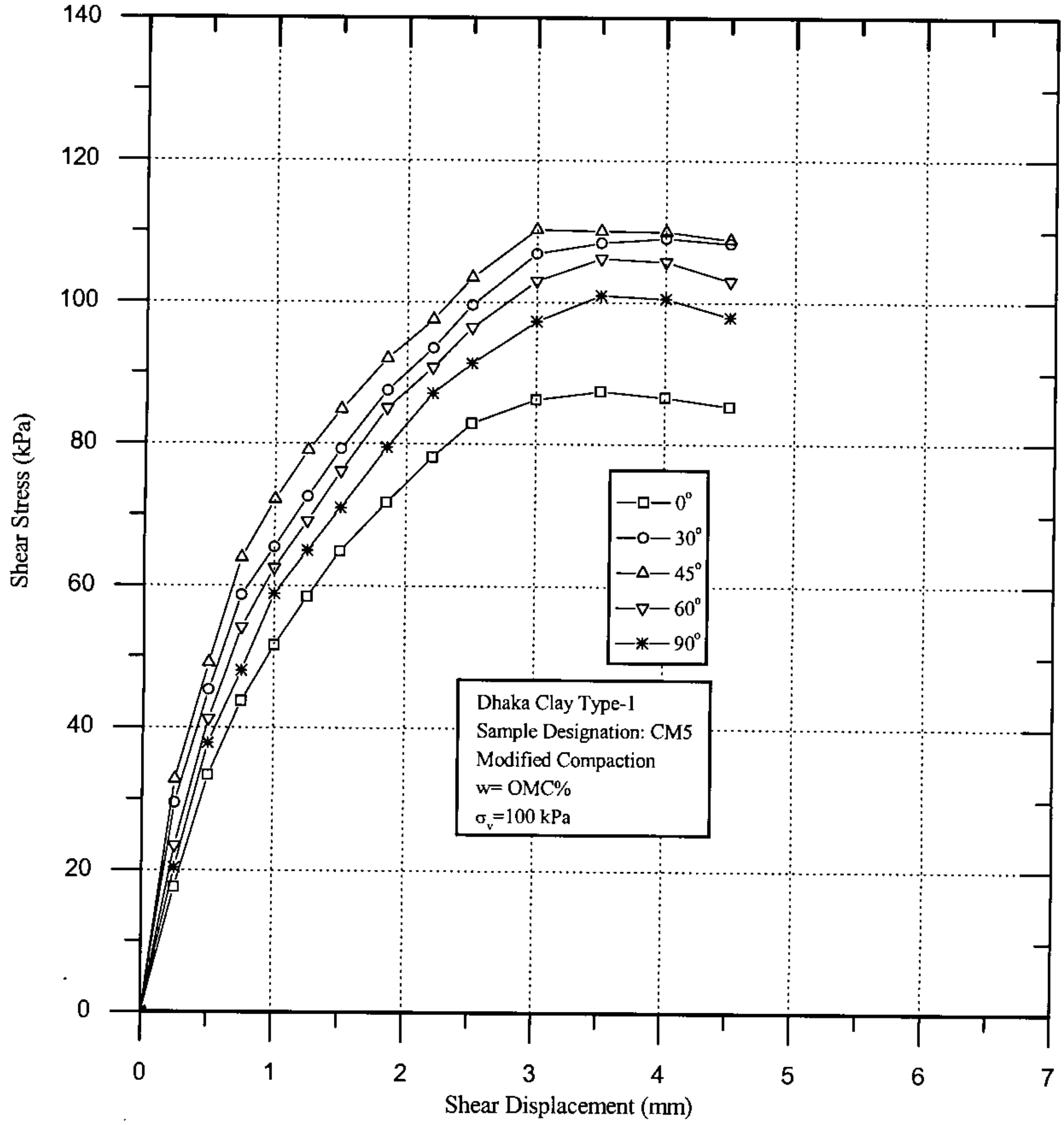


Fig. 4.11b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation for σ_v= 100 kPa and w= OMC %

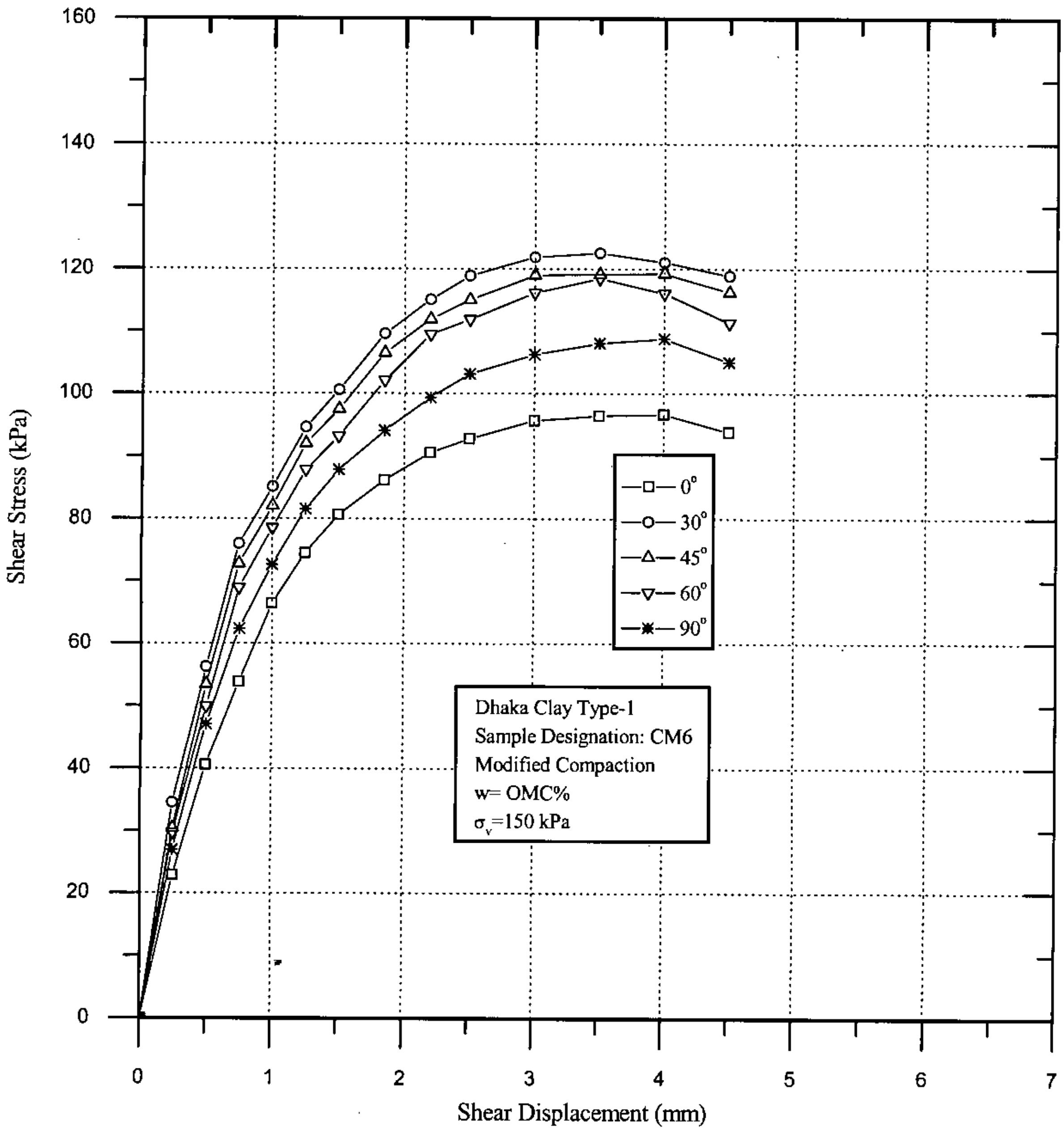


Fig. 4.11c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation for σ_v= 150 kPa and w= OMC %

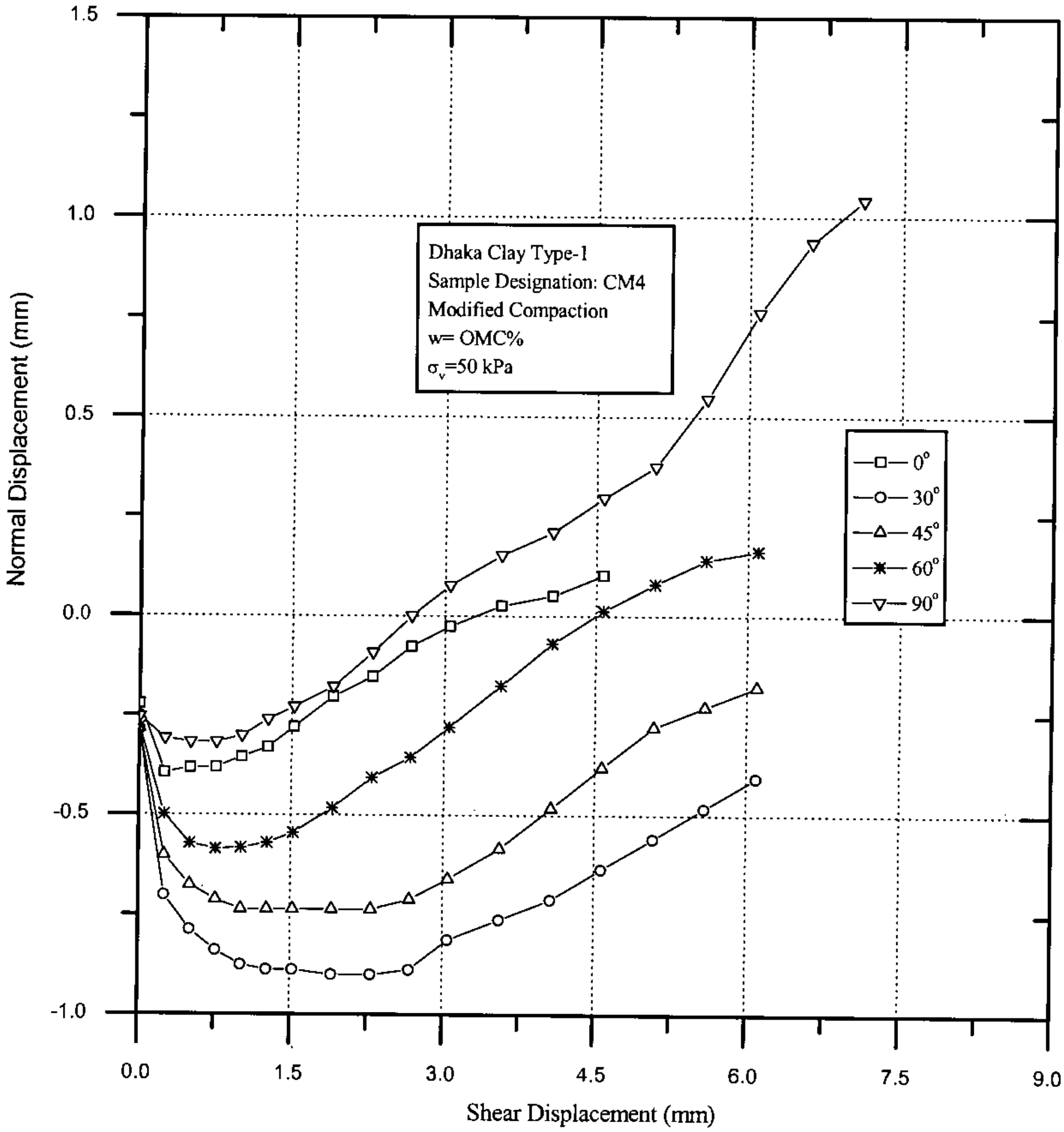


Fig. 4.12a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation for σ_v= 50 kPa and w= OMC %

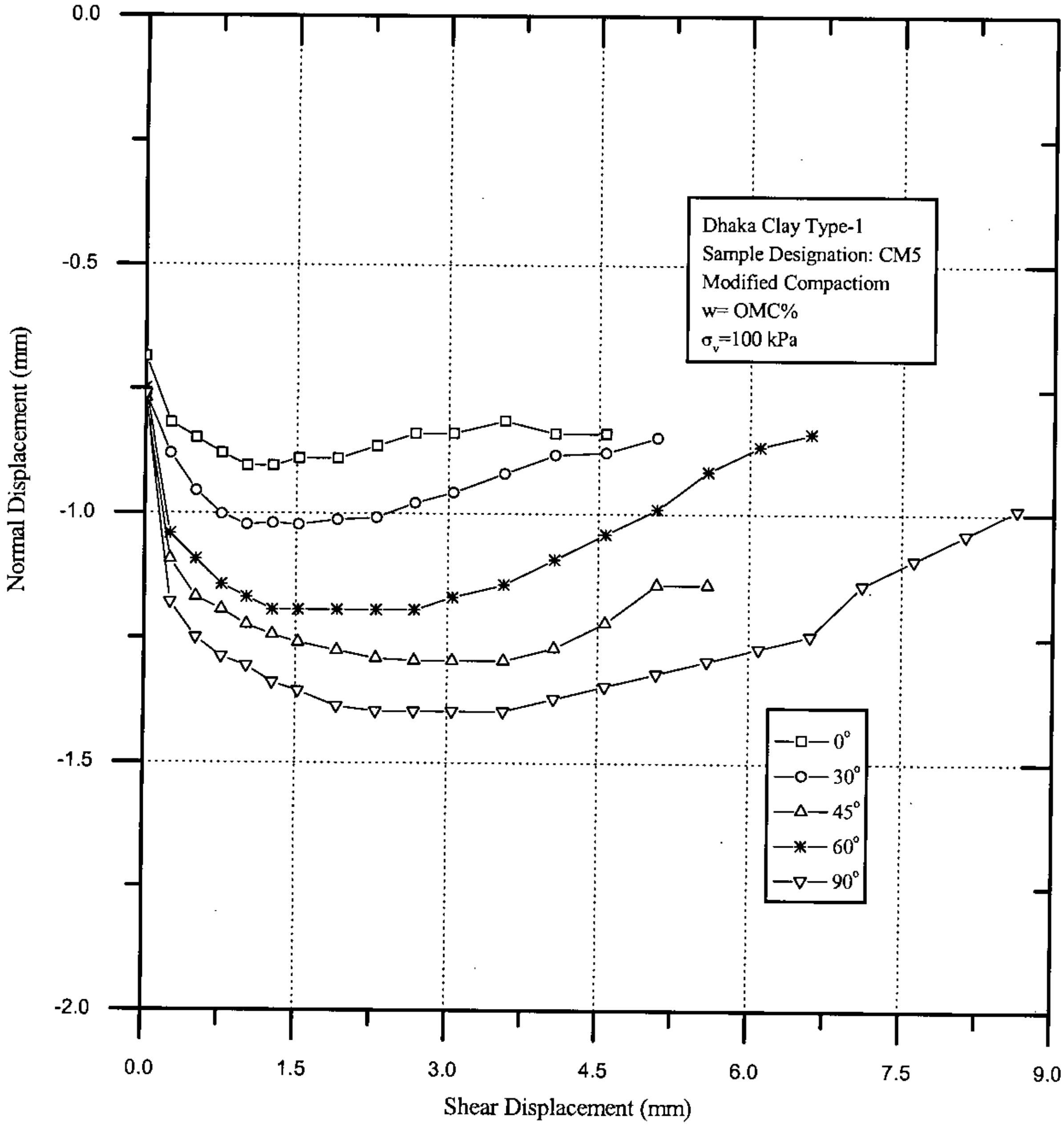


Fig. 4.12b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation for σ_v= 100 kPa and w= OMC %

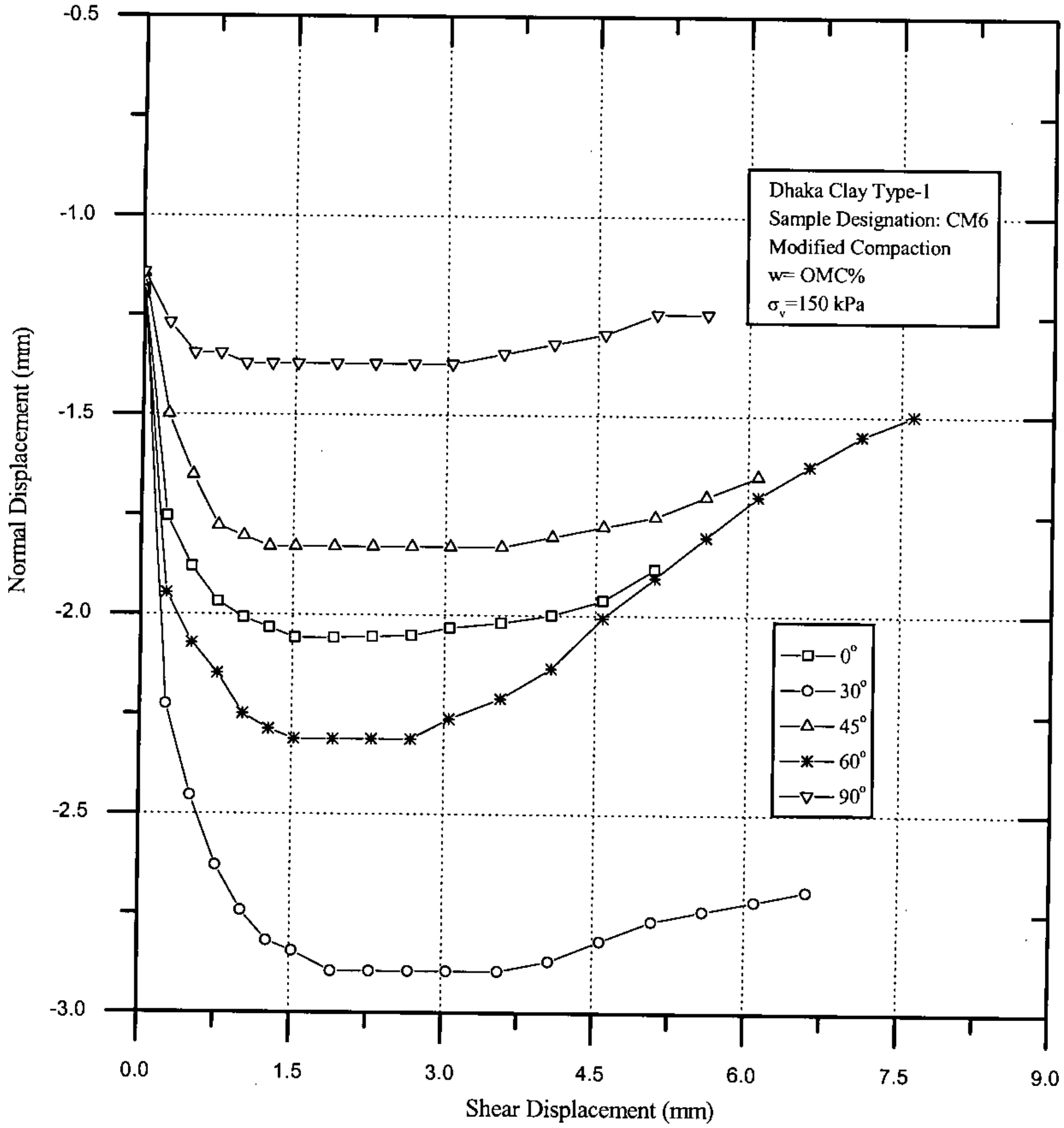


Fig. 4.12c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of sample designation for σ_v= 150 kPa and w= OMC %

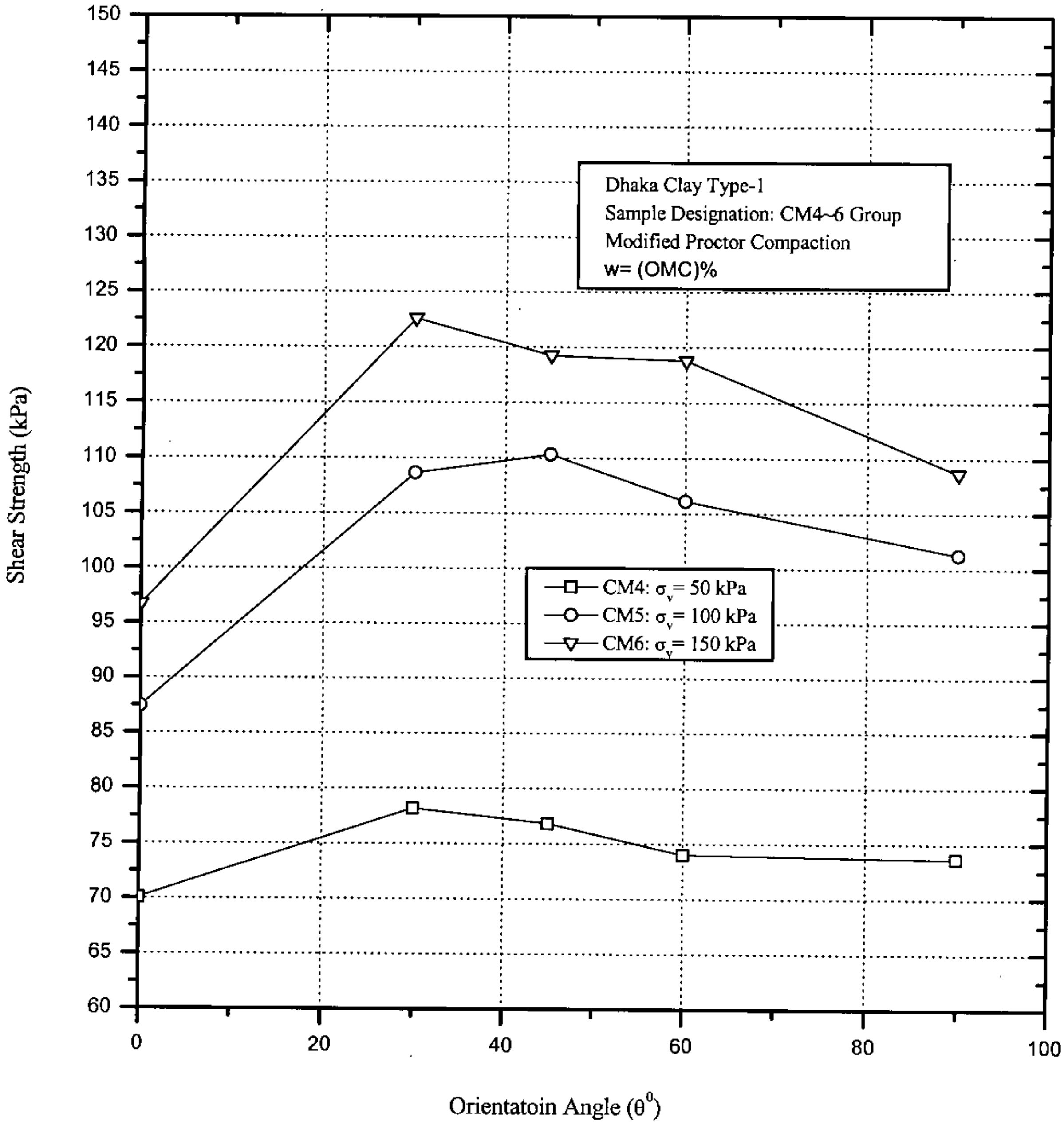


Fig. 4.13: Relationship between shear strength and orientation angle θ of Dhaka Clay Type-1 of specimen group CM4~6 for σ_v= 50~150 kPa at optimum moisture content

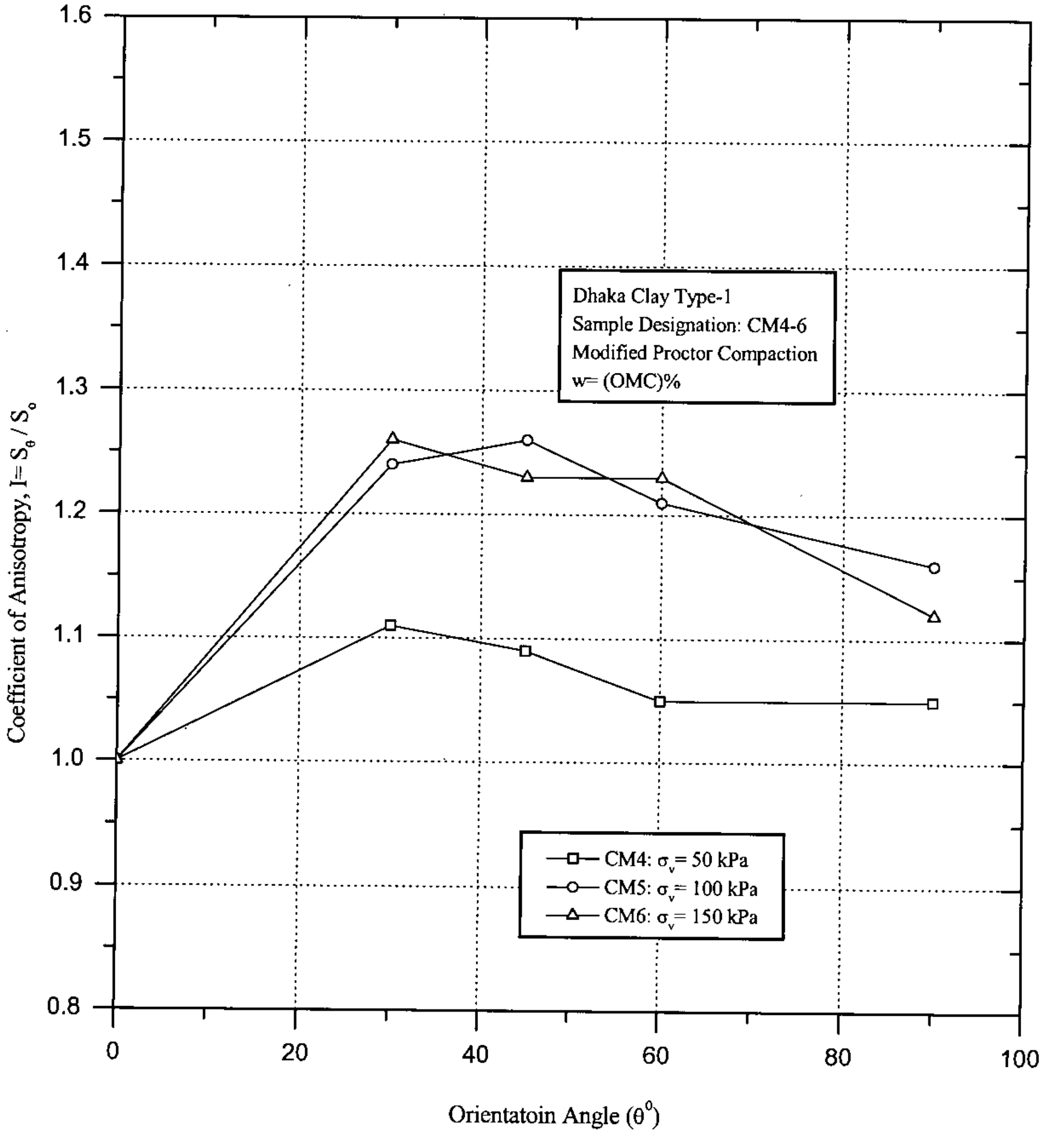


Fig. 4.14: The variation of I~θ for Modified Proctor compaction effort on Dhaka Clay Type-1 of specimen group CM4~6 for σ_v= 50~150 kPa at optimum moisture content

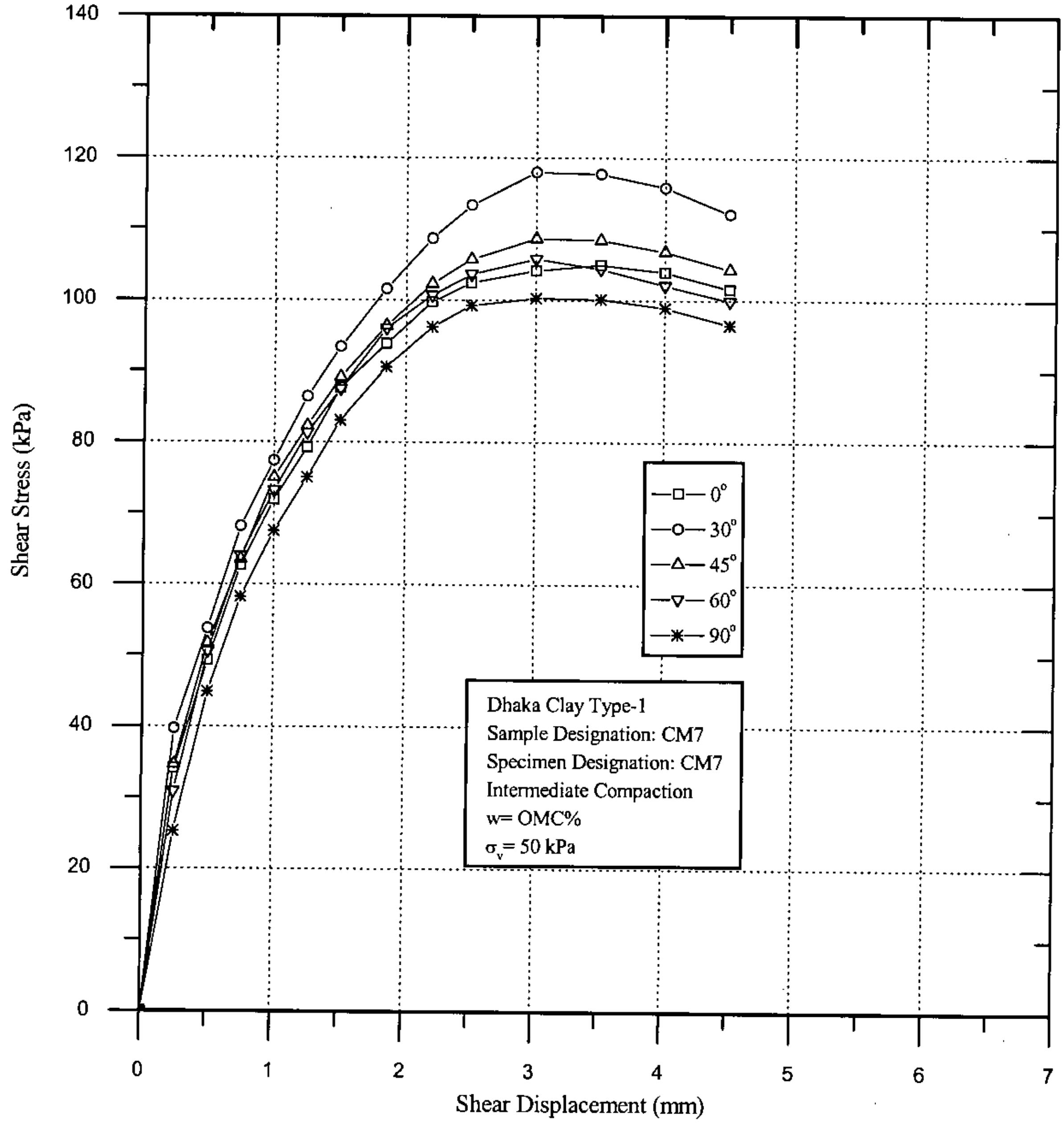


Fig. 4.15a: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM7 for σ_v= 50 kPa and w= OMC %

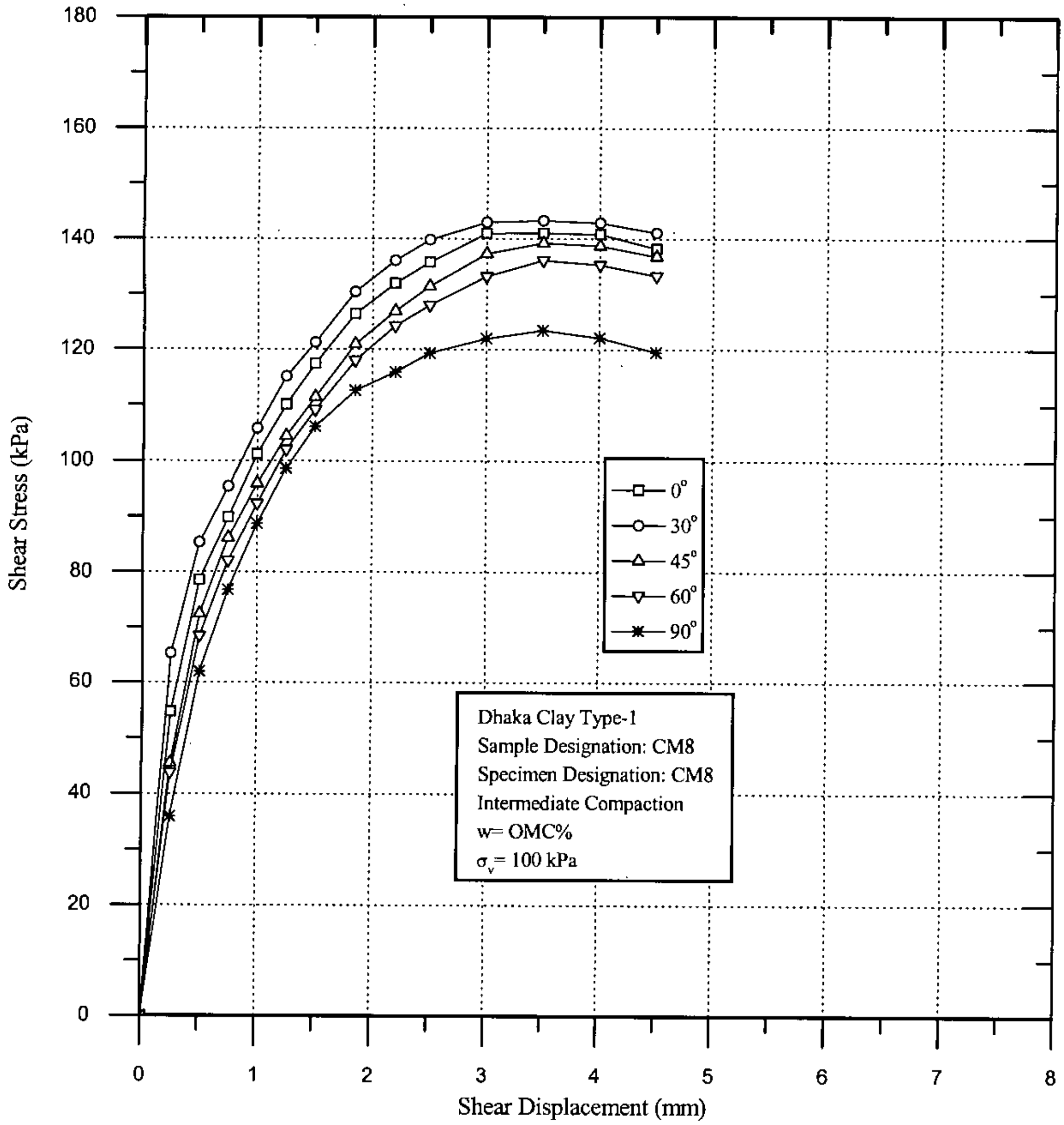


Fig. 4.15b: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM8 for σ_v= 100 kPa and w= OMC %

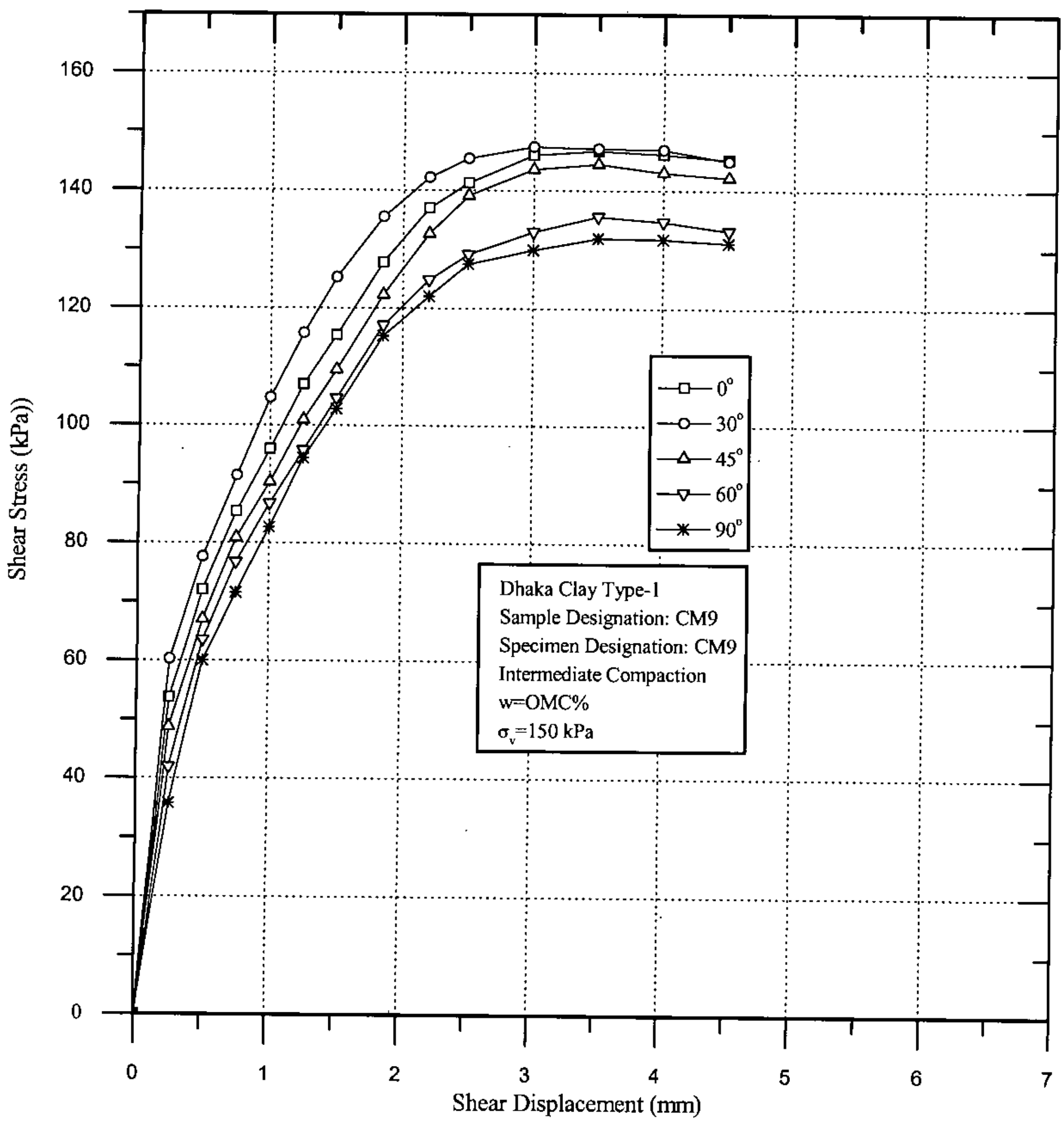


Fig. 4.15c: Relationship between shear stress and shear displacement of Dhaka Clay Type-1 of specimen CM9 for σ_v= 150 kPa and w= OMC %

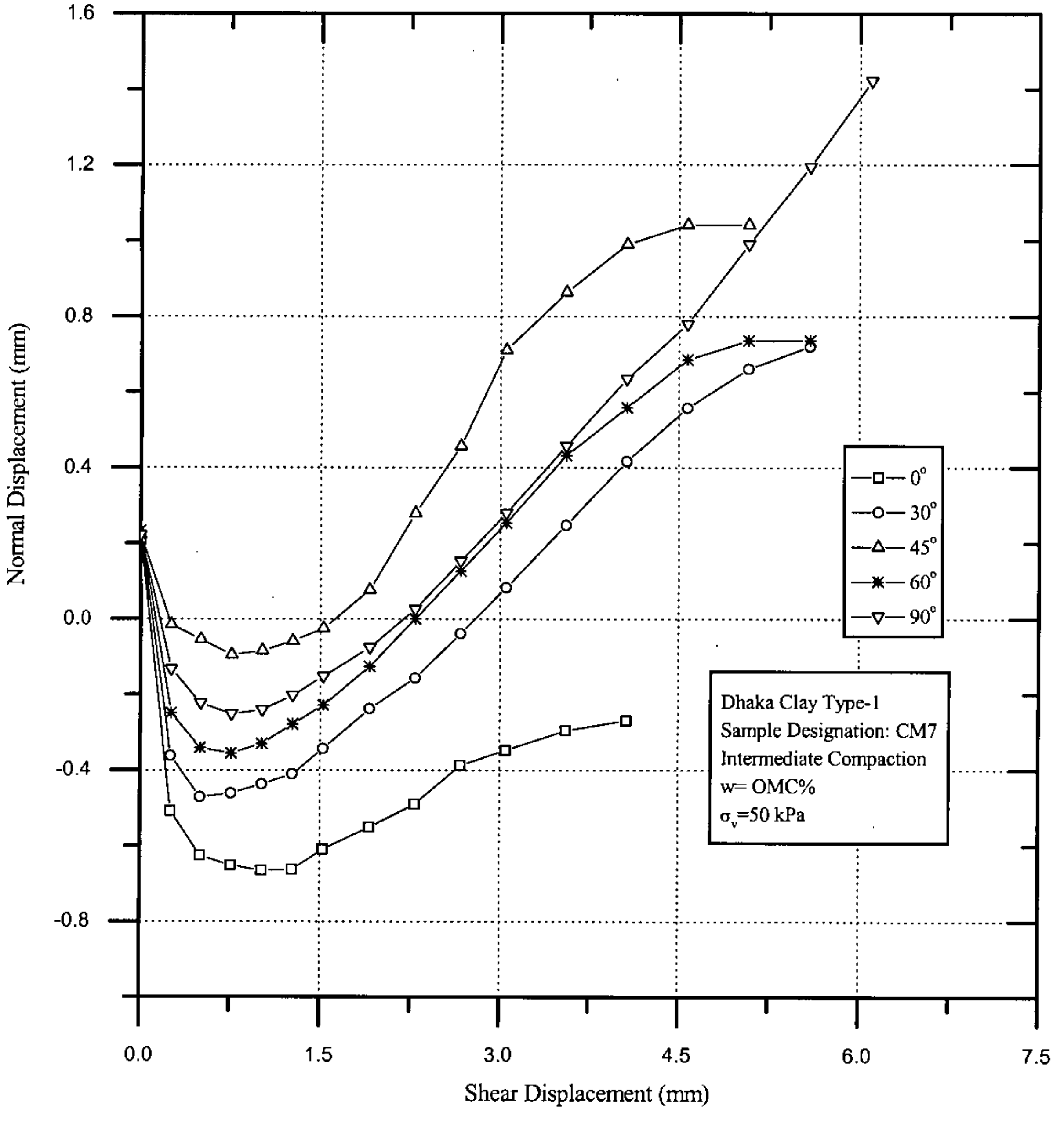


Fig. 4.16a: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM7 for σ_v= 50 kPa and w= OMC %

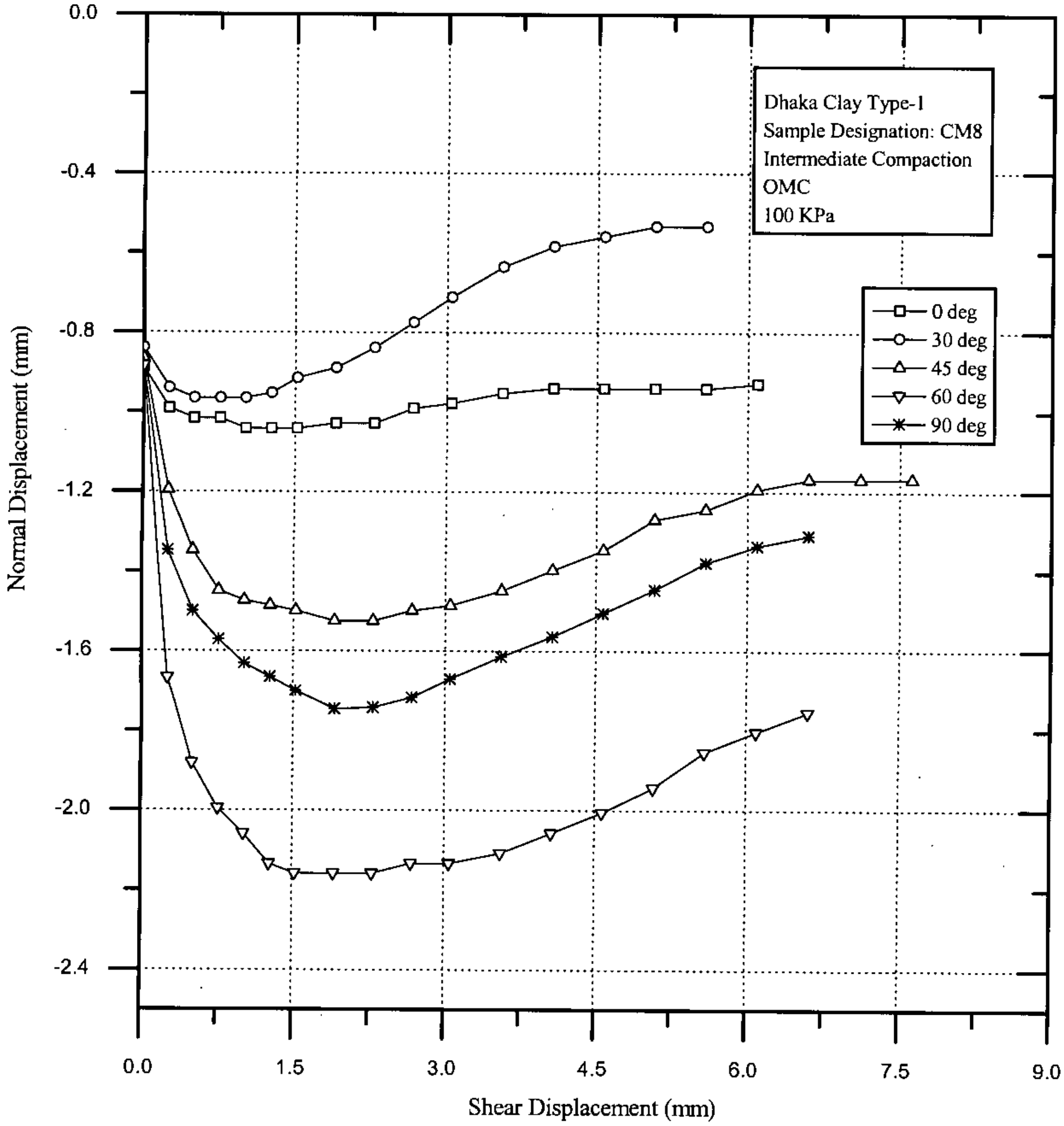


Fig. 4.16b: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM8 for σ_v= 100 kPa and w= OMC %

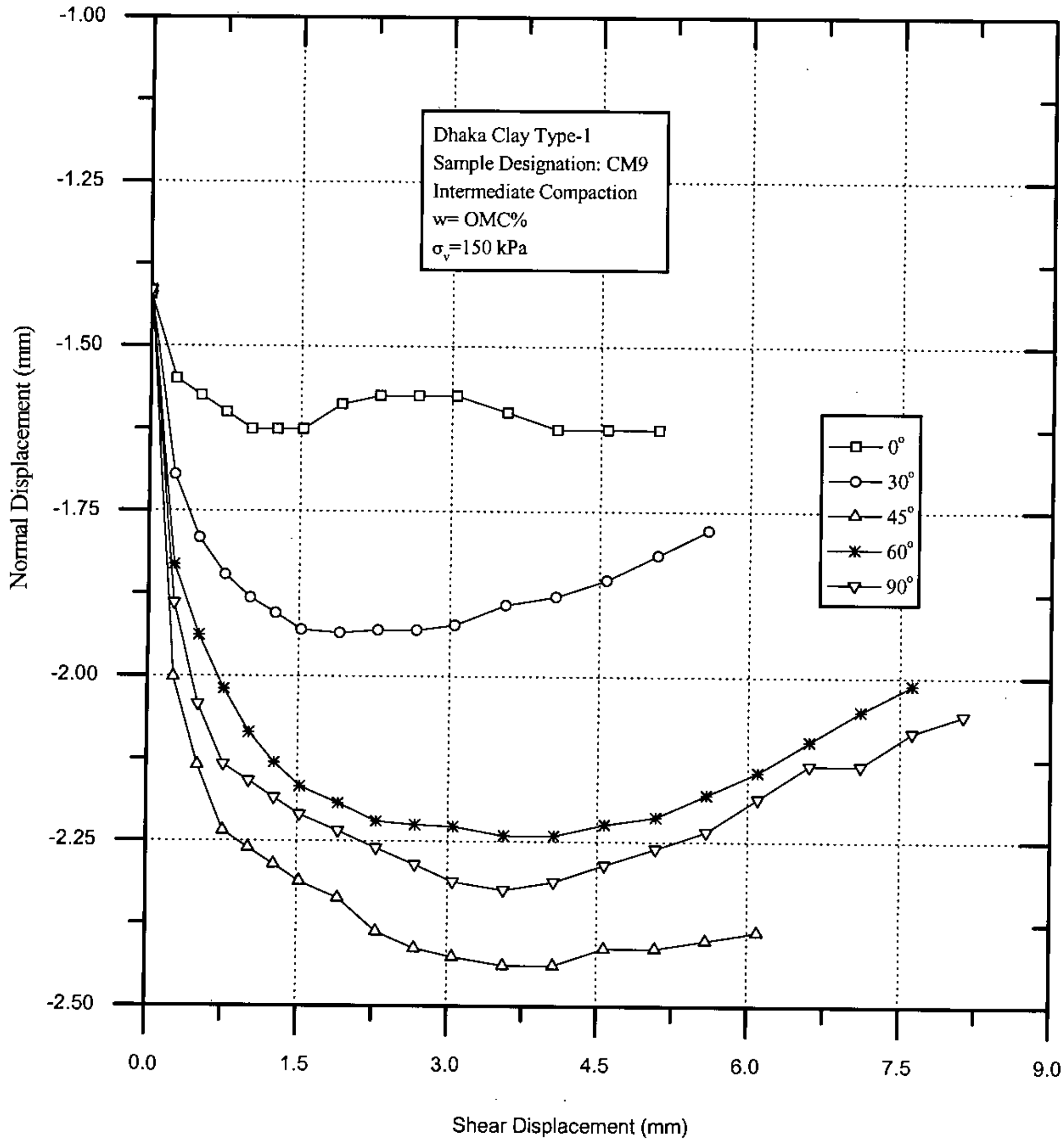


Fig. 4.16c: Relationship between normal and shear displacements of Dhaka Clay Type-1 of specimen CM9 for σ_v= 150 kPa and w= OMC %

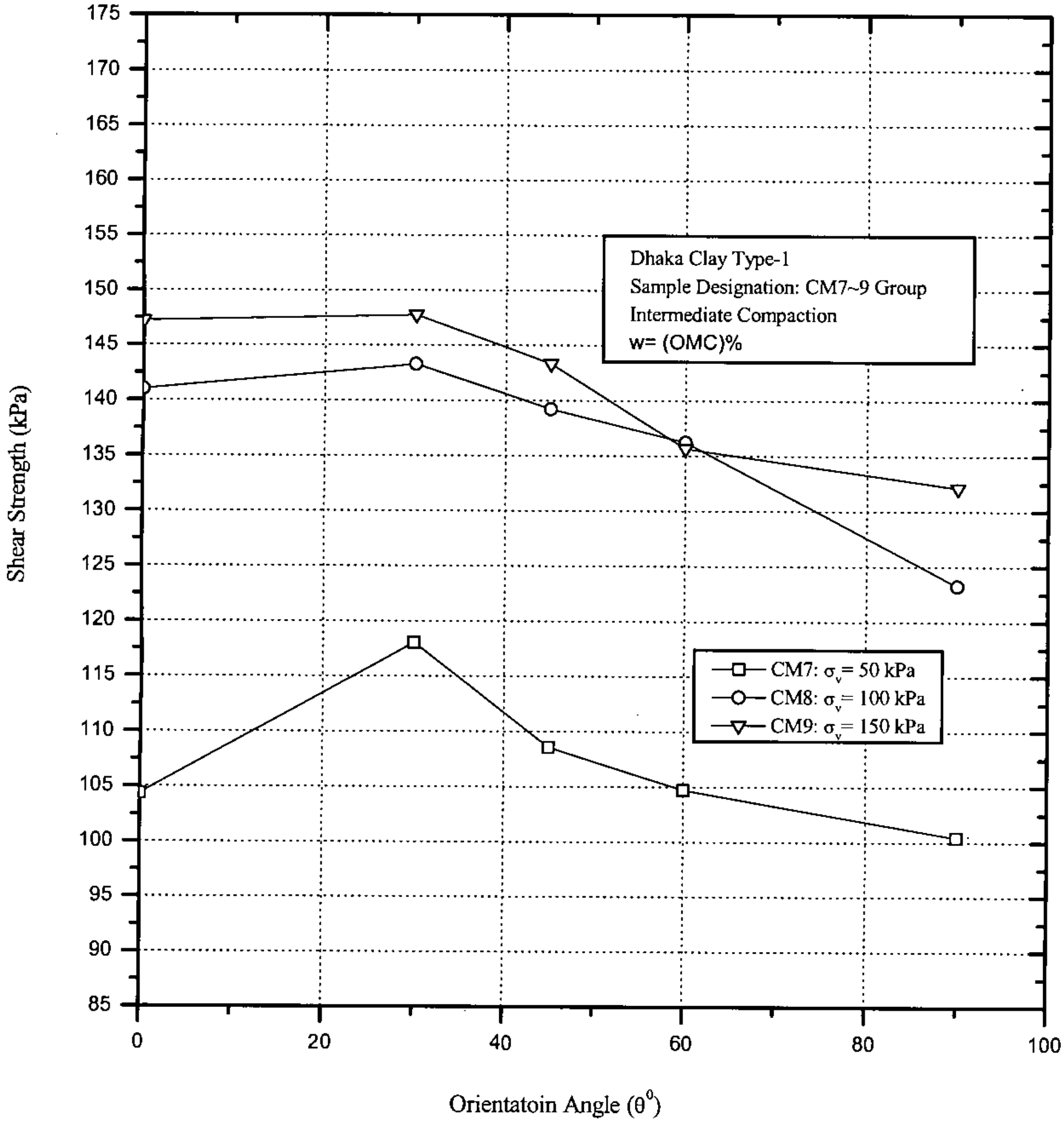


Fig. 4.17: Relationship between shear strength and orientation angle θ of Dhaka Clay Type-1 of specimen group CM7~9 for σ_v= 50~150 kPa at optimum moisture content

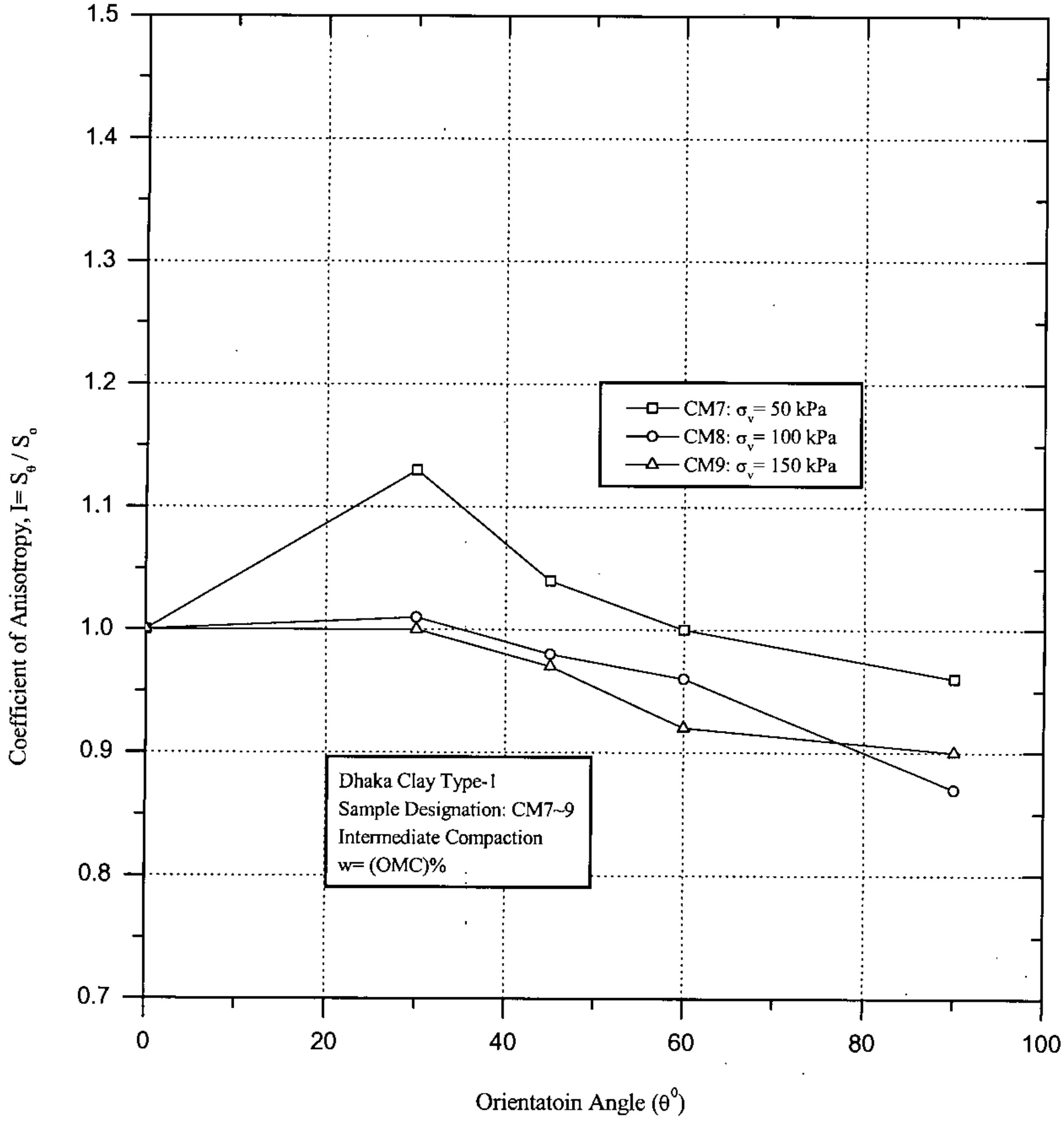


Fig. 4.18: The variation of I~θ for Intermediate compaction effort on Dhaka Clay Type-1 of specimen group CM7~9 for σ_v= 50~150 kPa at optimum moisture content

Table - 4.5: Details of compaction efforts for Dhaka clay type-2

Mold Size (mm)	Hammer Weight (kg)	Nos. of Layer	Nos. of Blows/ Layer	Height of fall of hammer (mm)	Compaction Energy /fall (kN-m/m^3)	Compaction Type	Optimum Moisture Content (%)
Diameter 152.40 Height 114.30	2.50	3	55	300	1950	Standard Proctor	14.00
Diameter 152.40 Height 114.30	4.53	5	55	450	8850	Modified Proctor	13.00
Diameter 152.40 Height 114.30	4.53	5	55	450	4050	Intermediate Proctor	12.00

Table - 4.6: List of test specimens and designations of Dhaka clay type-2

S/n	Specimen Designation	Compaction	Clay Mass	Normal Stress	M/c	Dry density (kN/m^3)
Standard Proctor Compaction						
1	CM-I	Standard Proctor compaction	Clay-2	50 kPa	14%	17.89
2	CM-II			100 kPa	14%	17.89
3	CM-III			150 kPa	14%	17.89
Modified Proctor Compaction						
4	CM-IV	Modified Proctor compaction	Clay-2	50 kPa	12%	19.65
5	CM-V			100 kPa	12%	19.65
6	CM-VI			150 kPa	12%	19.65
Intermediate Compaction						
7	CM-VII	Intermediate compaction	Clay-2	50 kPa	13%	18.79
8	CM-VIII			100 kPa	13%	18.79
9	CM-IX			150 kPa	13%	18.79

Table – 4.7: Some typical test results of Dhaka clay type-2

Sample Designation	Specimen Designation	Peak Shear Stress (kPa)				
		S_0	S_{30}	S_{45}	S_{60}	S_{90}
Standard Proctor Compaction						
CM-I	CM-I	100.21	94.07	91.76	90.12	91.00
CM-II	CM-II	105.53	96.92	95.28	94.58	100.00
CM-III	CM-III	111.52	100.85	94.43	103.31	109.25
Modified Proctor Compaction						
CM-IV	CM-IV	60.30	63.40	67.25	64.52	60.00
CM-V	CM-V	82.92	108.63	107.75	100.65	114.40
CM-VI	CM-VI	94.44	124.32	118.83	108.63	119.27
Intermediate Compaction						
CM-VII	CM-VII	77.75	80.38	78.41	75.51	72.33
CM-VIII	CM-VIII	89.60	95.01	102.20	98.93	92.97
CM-IX	CM-IX	97.66	110.85	107.87	102.28	105.48

Table - 4.8: Coefficient of Anisotropy, $I = S_\theta/S_0$ in Dhaka clay type-1

Sample Designation	Specimen Designation	Coefficient of Anisotropy, $I = S_\theta/S_0$				
		S_0/S_0	S_{30}/S_0	S_{45}/S_0	S_{60}/S_0	S_{90}/S_0
Standard Proctor Compaction						
CM-I	CM-I	1.00	0.94	0.91	0.899	0.91
CM-II	CM-II	1.00	0.92	0.90	0.896	0.95
CM-III	CM-III	1.00	0.90	0.85	0.93	0.98
Modified Proctor Compaction						
CM-IV	CM-IV	1.00	1.05	1.11	1.07	0.99
CM-V	CM-V	1.00	1.31	1.30	1.21	1.37
CM-VI	CM-VI	1.00	1.31	1.26	1.15	1.26
Intermediate Compaction						
CM-VII	CM-VII	1.00	1.03	1.00	0.97	0.93
CM-VIII	CM-VIII	1.00	1.06	1.14	1.10	1.04
CM-IX	CM-IX	1.00	1.13	1.10	1.05	1.08

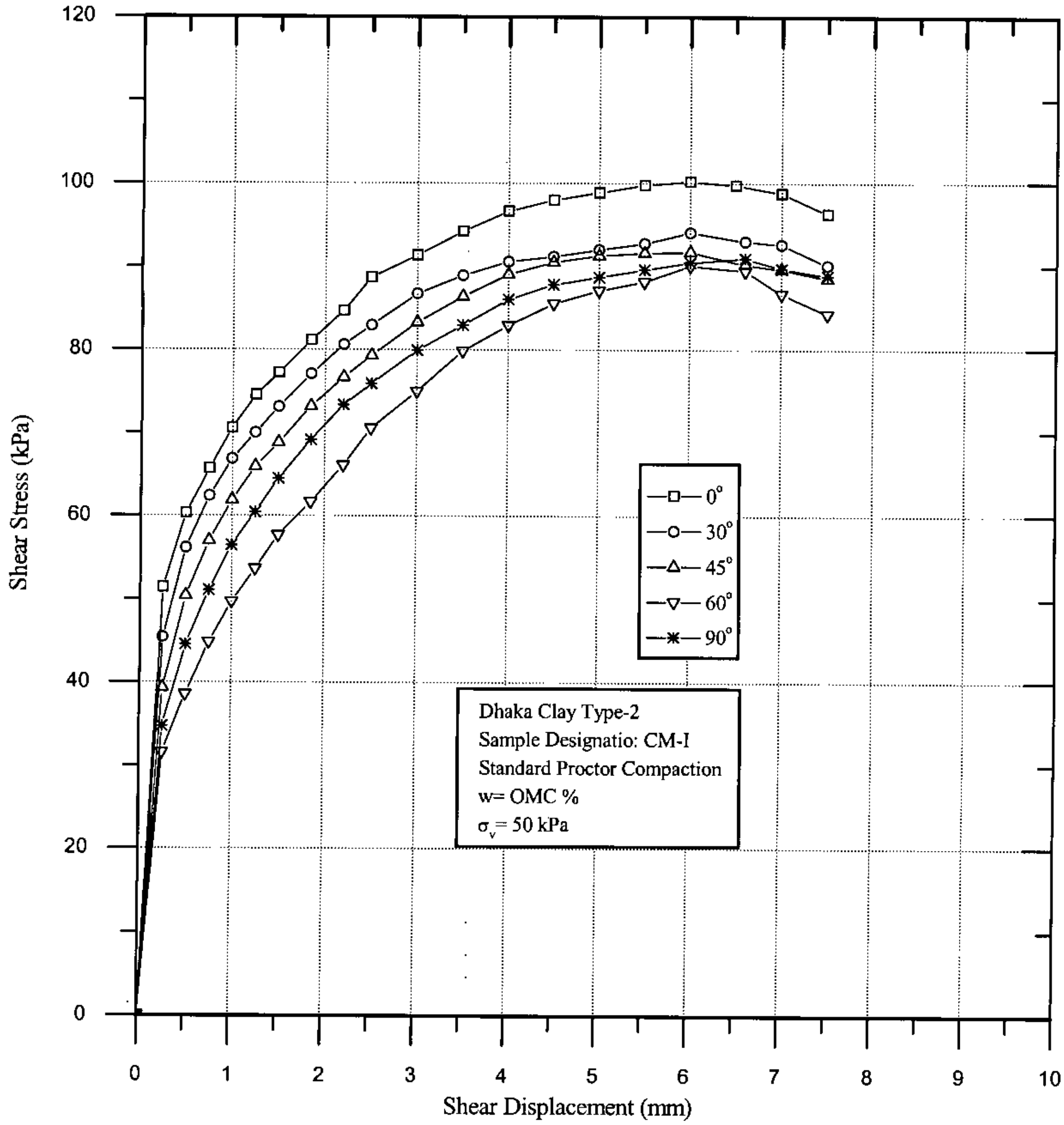


Fig. 4.19a: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-I for σ_v= 50 kPa and w= OMC %

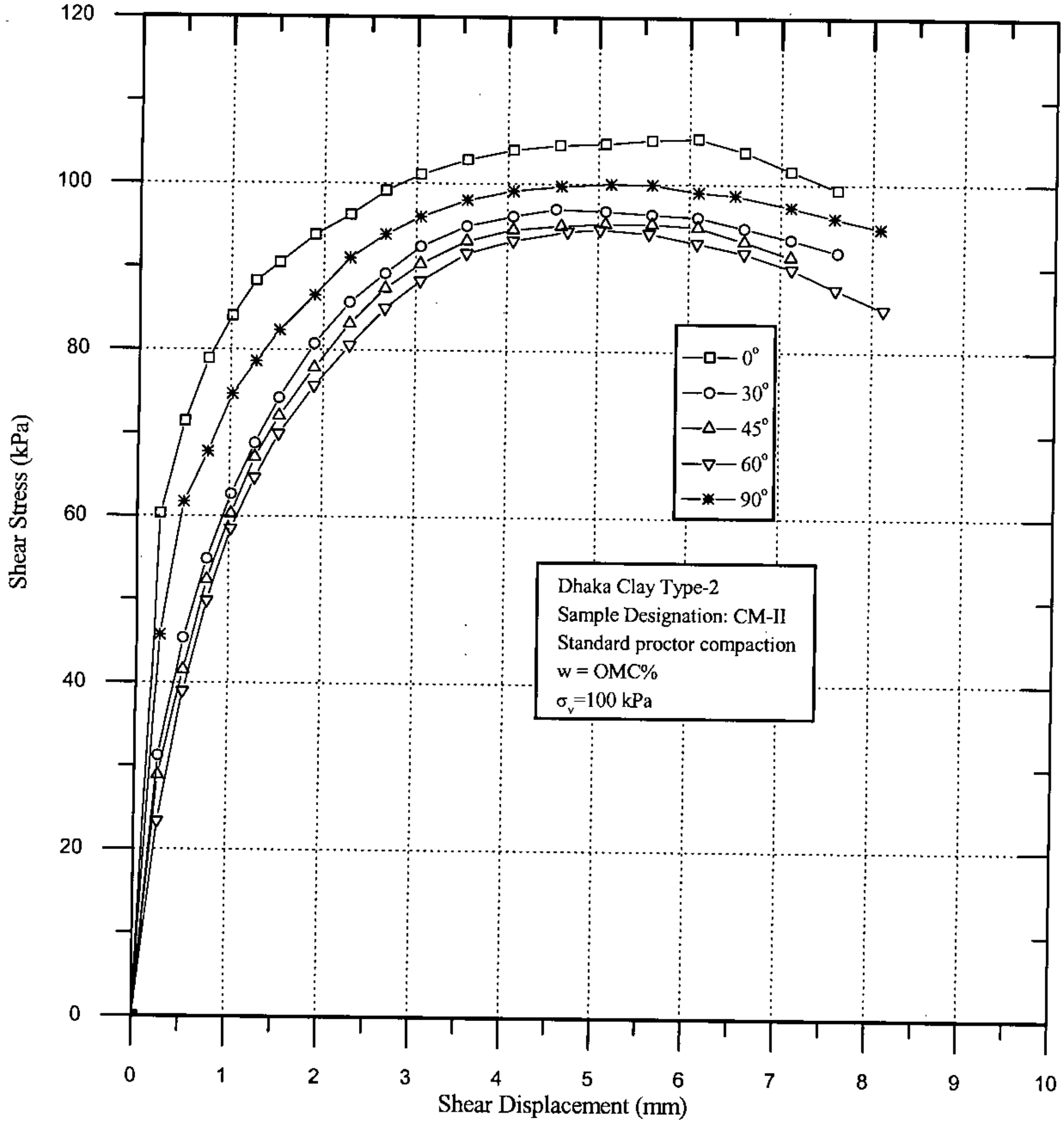


Fig. 4.19b: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-II for σ_v= 100 kPa and w= OMC %

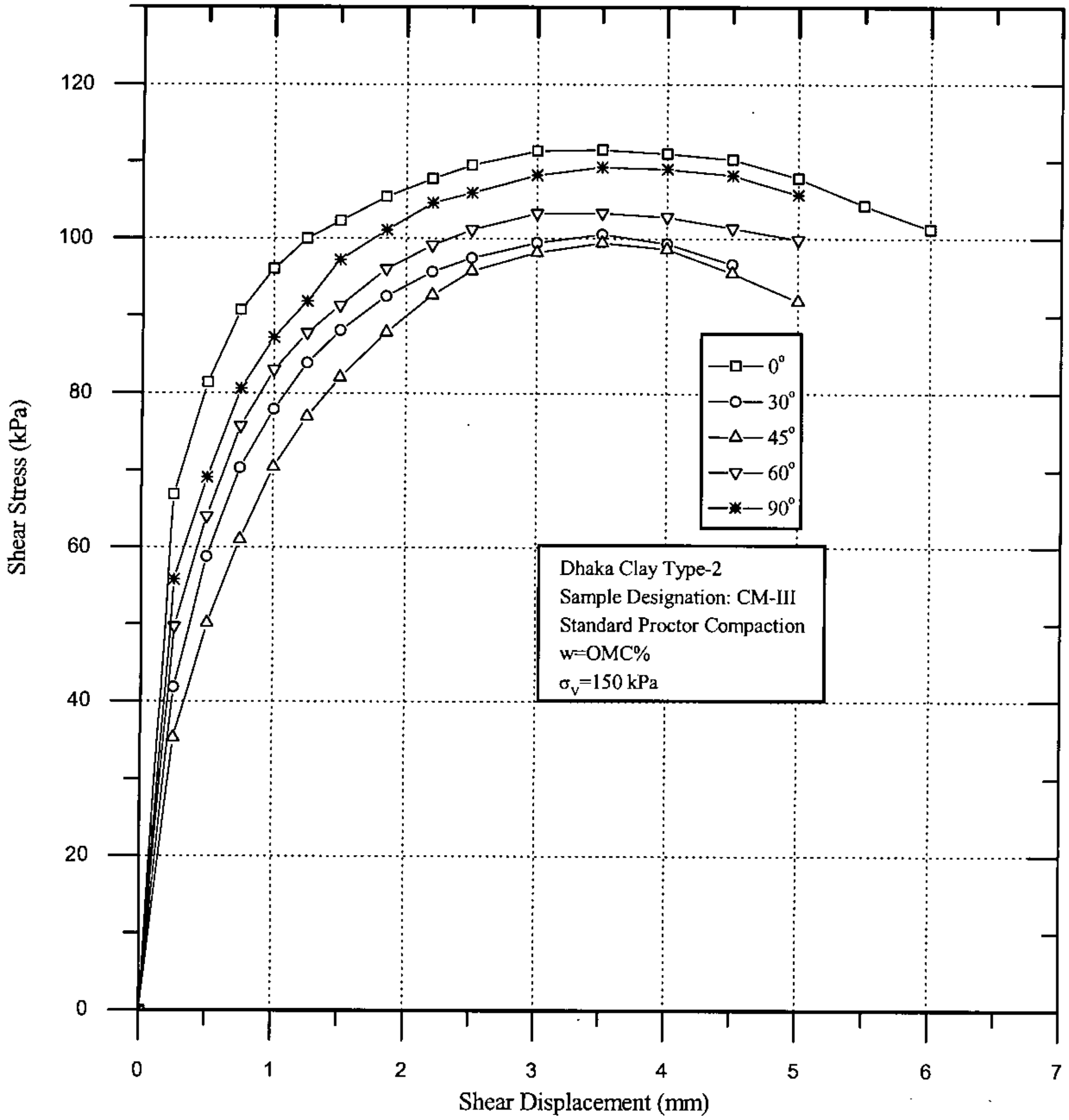


Fig. 4.19c: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-III for σ_v= 150 kPa and w= OMC %

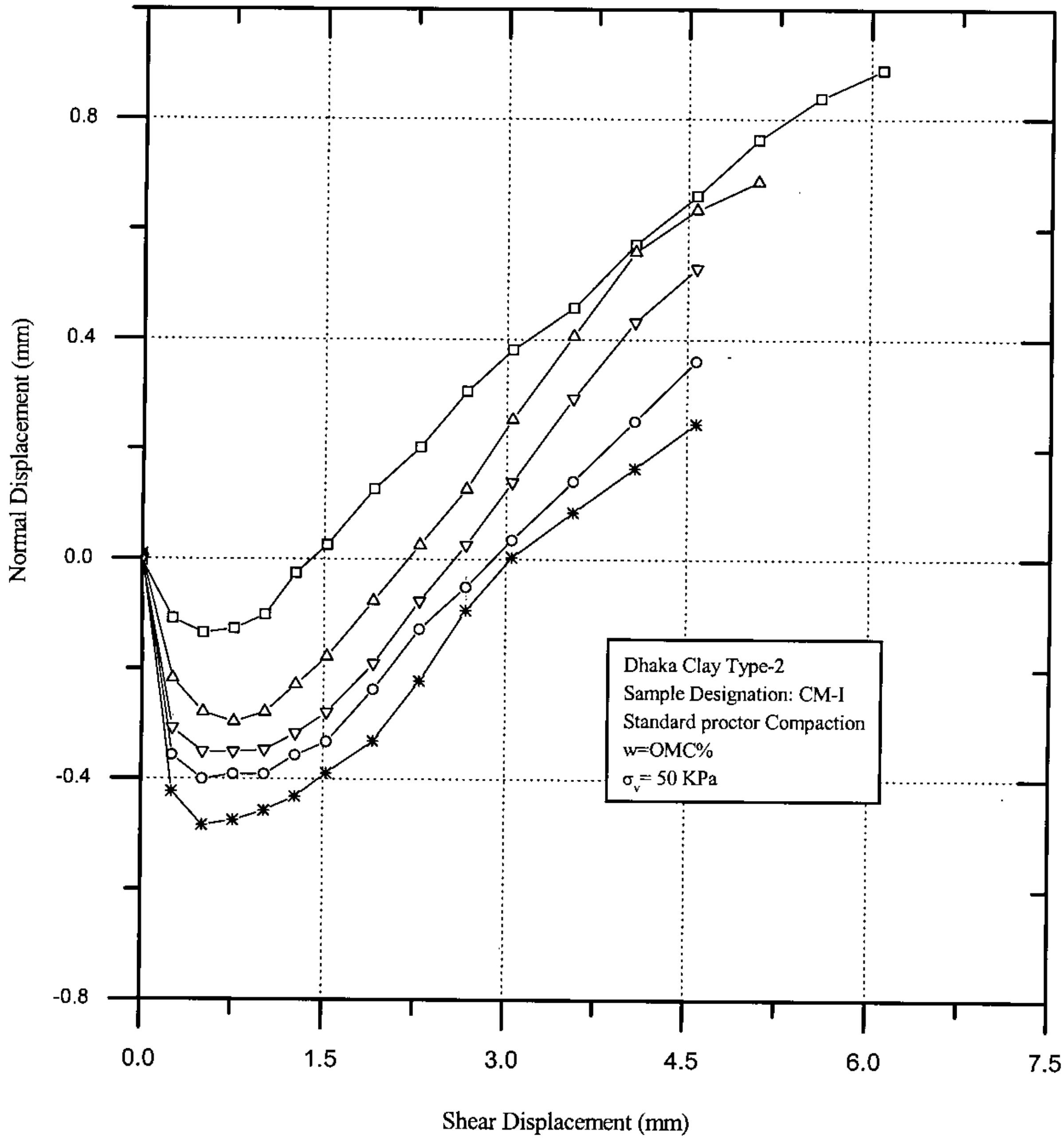


Fig. 4.20a: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-I for σ_v= 50 kPa and w= OMC %

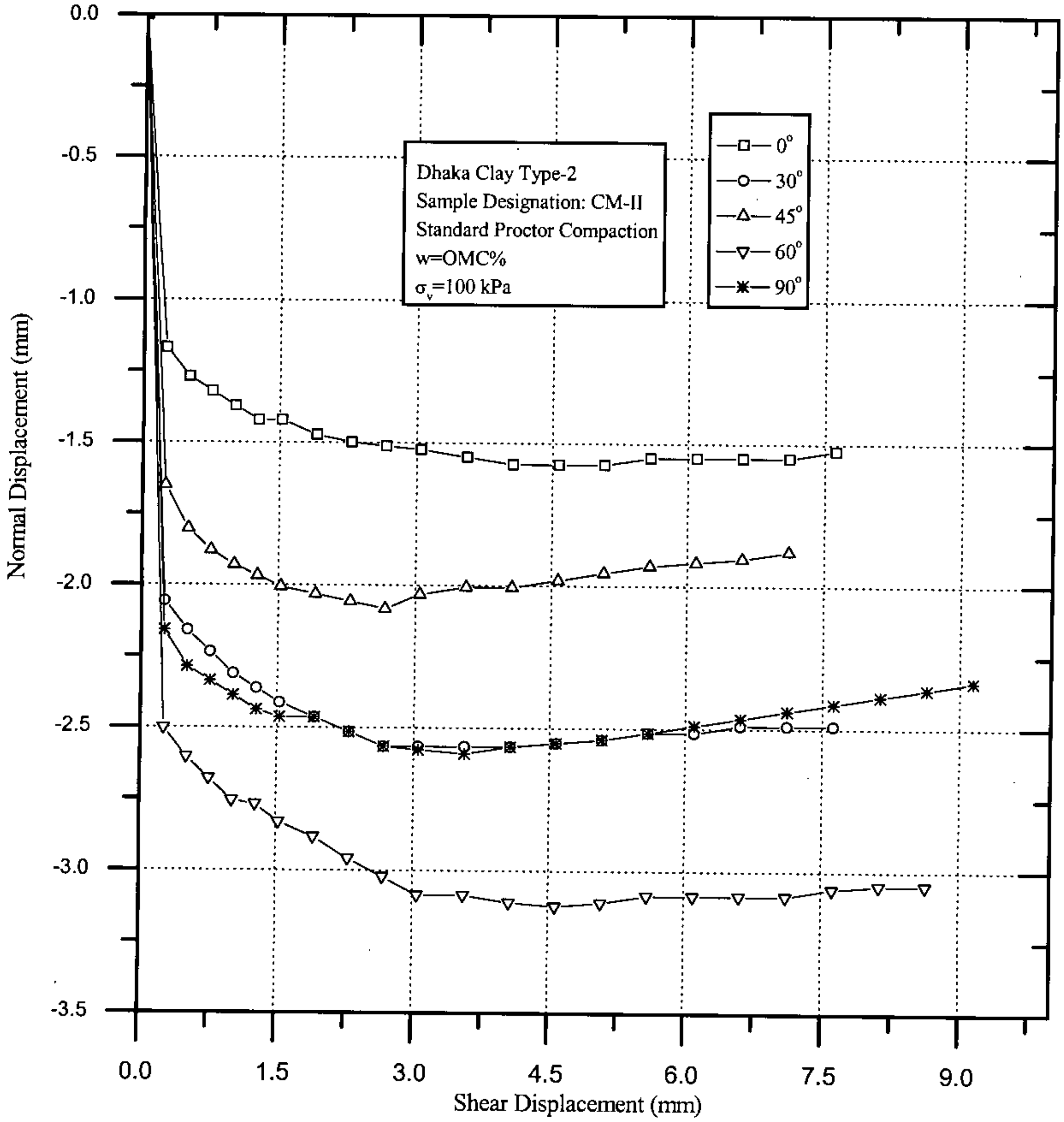


Fig. 4.20b: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-II for σ_v = 100 kPa and w= OMC %

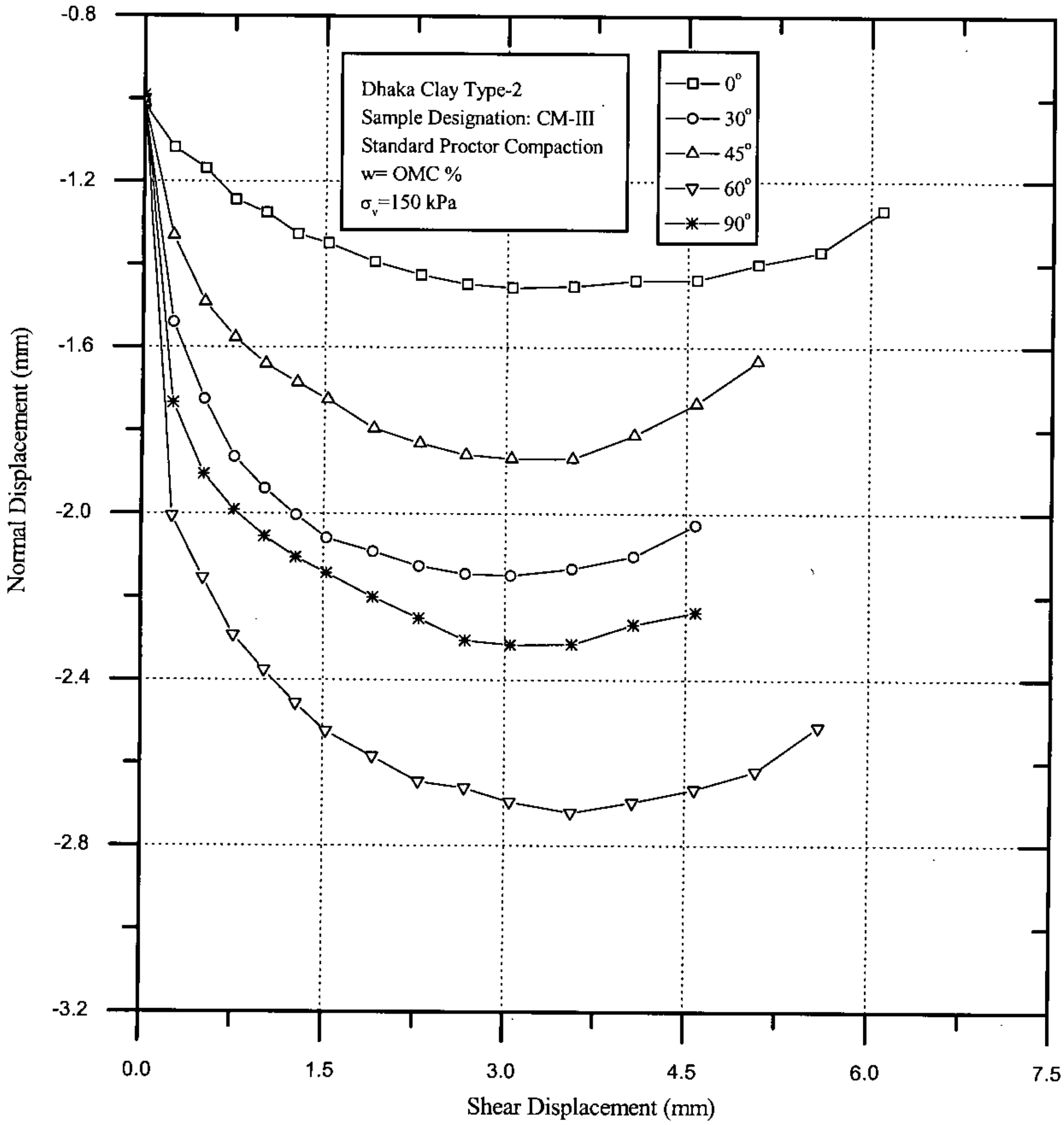


Fig. 4.20c: Relationship between shear stress and shear displacement of Dhaka Clay Type-2 of sample designation CM-III for σ_v= 150 kPa and w= OMC %

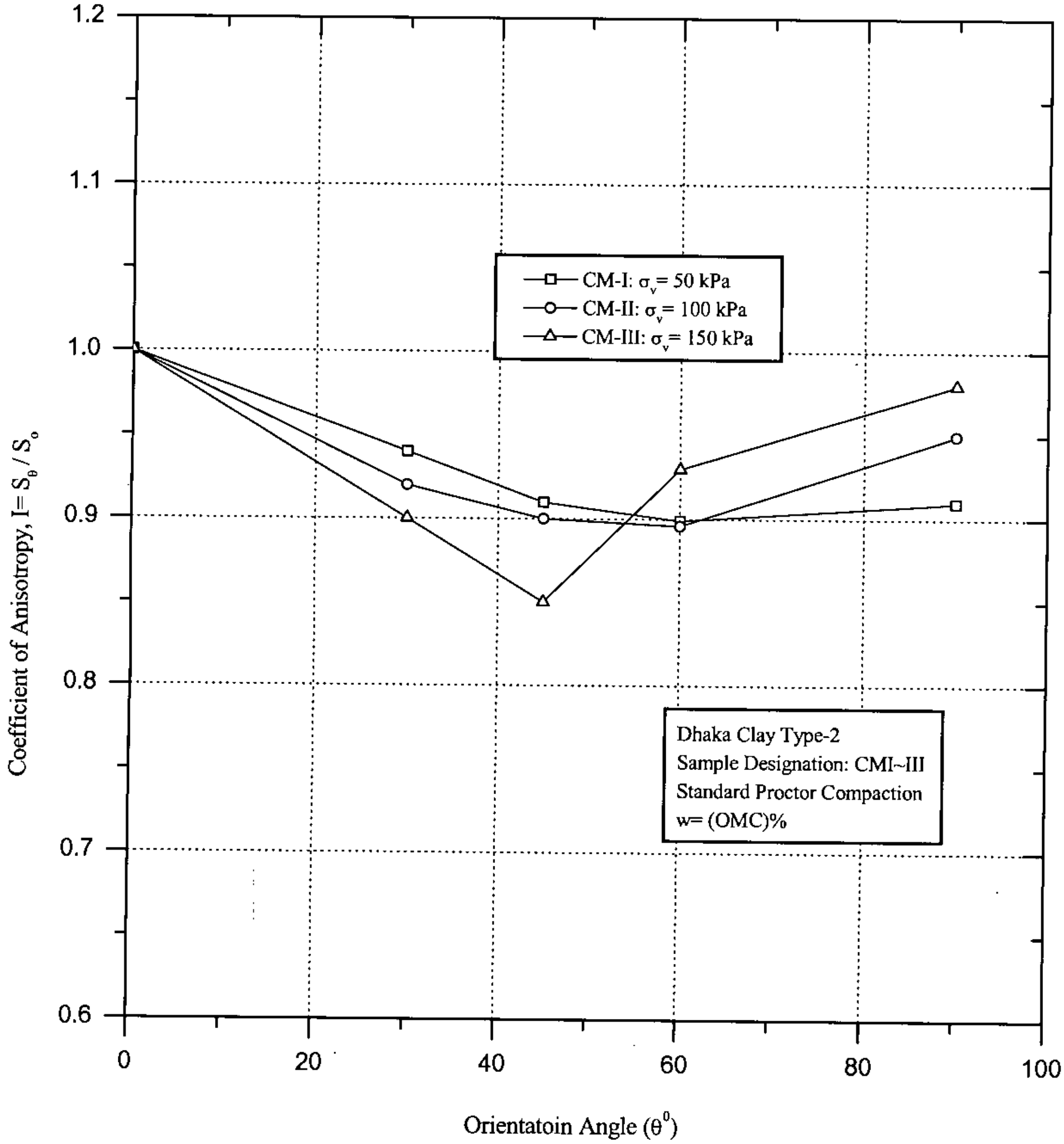


Fig. 4.21a: The variation of I~θ for Standard Proctor compaction effort on Dhaka Clay Type-2 of specimen group CMI~III for σ_v= 50~150 kPa at optimum moisture content

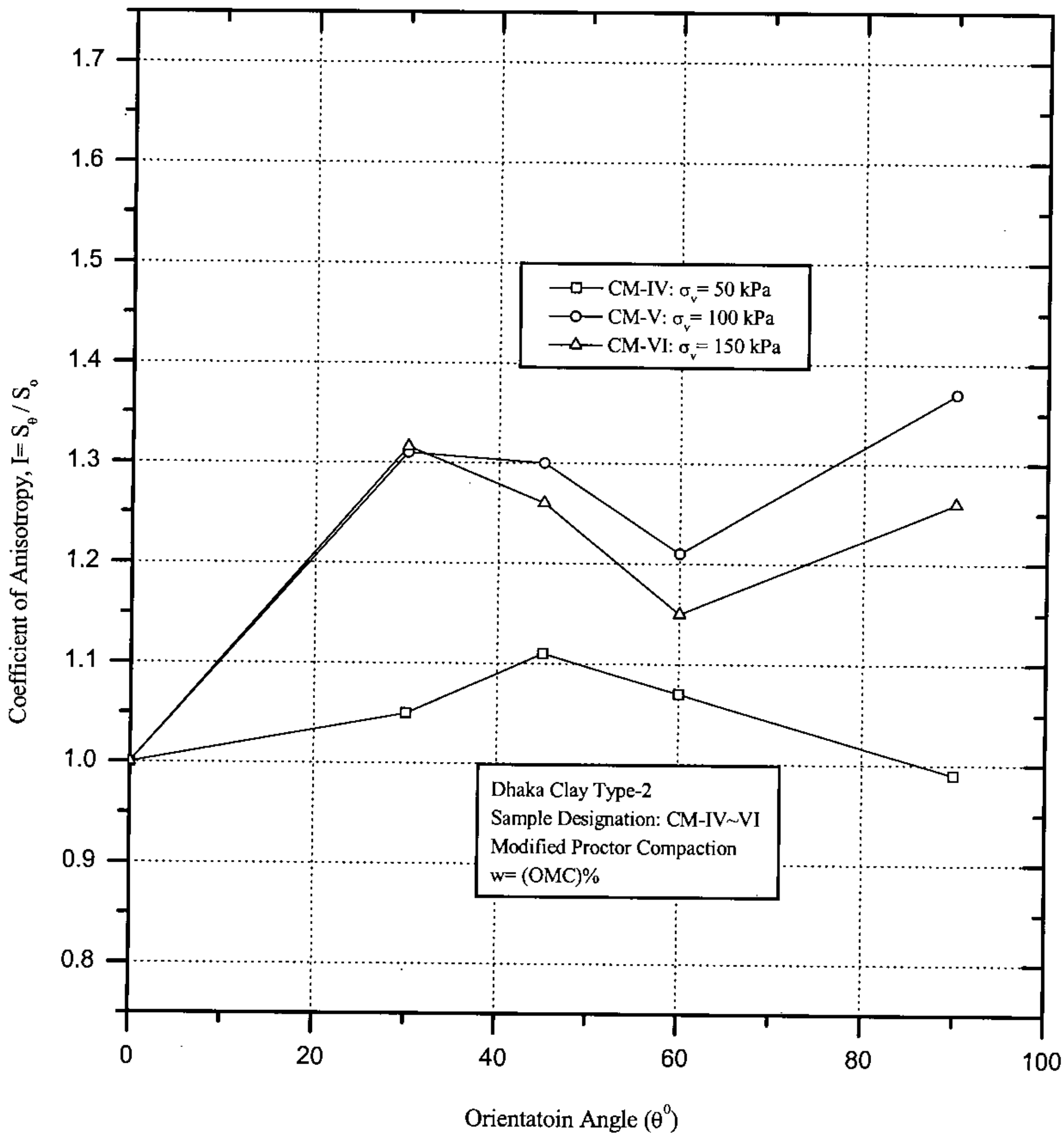


Fig. 4.21b: The variation of I~θ for Modified Proctor compaction effort on Dhaka Clay Type-2 of specimen group CM-IV~VI for σ_v= 50~150 kPa at optimum moisture content

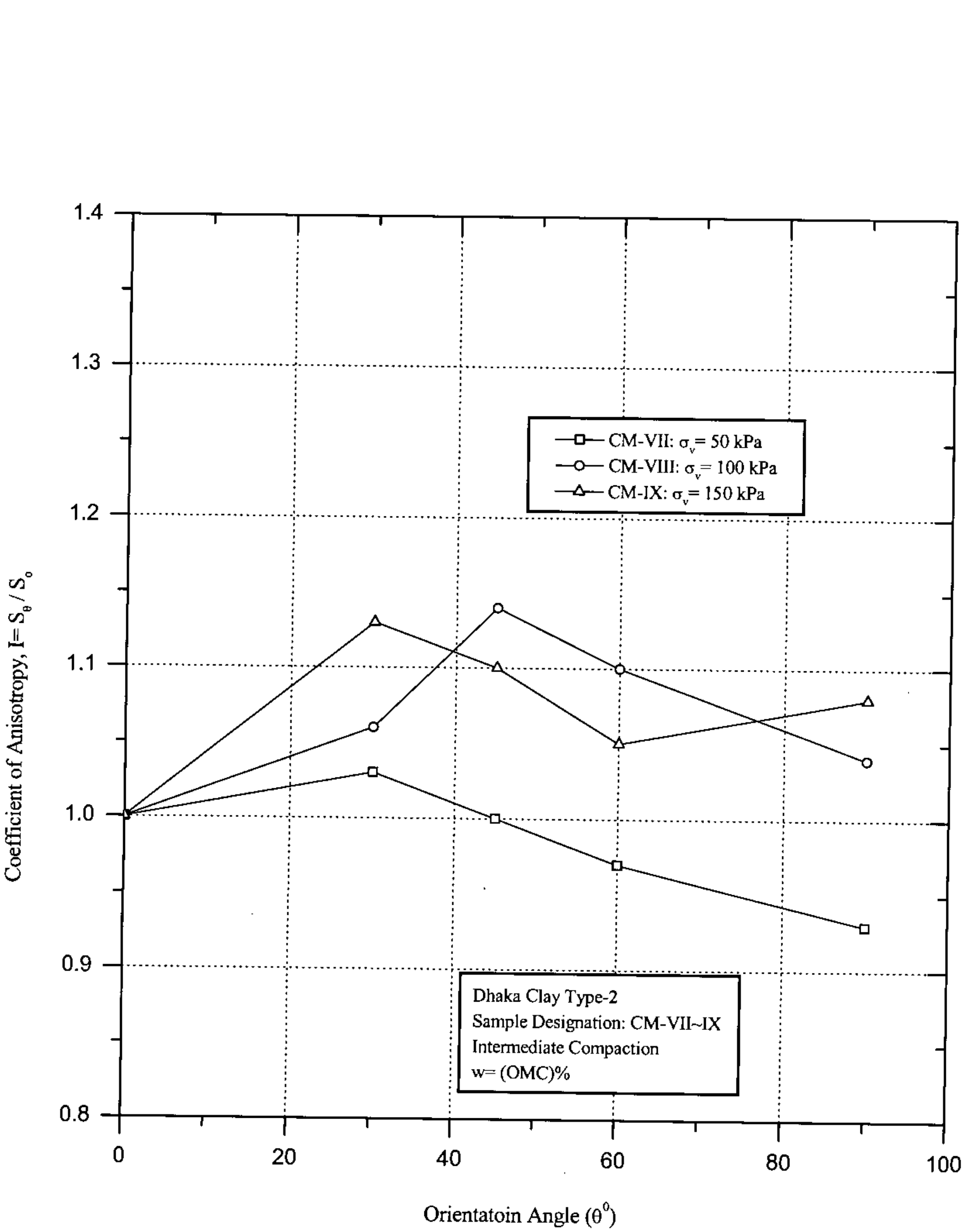


Fig. 4.21c: The variation of I~θ for Intermediate compaction effort on Dhaka Clay Type-2 of specimen group CM-VII~IX for σ_v= 50~150 kPa at optimum moisture content

Chapter 5
Conclusions and Recommendations for Future Study

5.1 General

The objective of the research work was investigation of the strength characteristics, deformation characteristics and anisotropy. The effect of moisture content, compaction effort and normal stress (overburden pressure) on the strength and deformation characteristics and anisotropy for Compacted Dhaka Clay was studied. There were previous works by Islam 1999 on compacted clay, Ko-consolidated clay and natural clay to investigate the presence of anisotropy in undrained shear strength of clays. Therefore it was decided to give emphasis on the effect of different compaction efforts, moisture contents and normal stress on the Compacted Dhaka Clay to characterize the strength and deformation anisotropy. Different energies used for reconstitution of the Compacted Dhaka Clay were Standard Proctor, Modified Proctor and Intermediate compaction, where, Intermediate compaction energy was used as the energy in between Standard Proctor and Modified Proctor energy mentioned in Table 4.1. Similarly moisture contents used for the reconstitution of the soil mass for preparation of the test specimens were optimum moisture content (OMC%), optimum plus two percent moisture content (OMC+2)% and optimum minus two percent moisture content (OMC-2)%. The normal stresses applied on the test specimens during direct shear testing were $\sigma_v = 50$ kPa, 100 kPa and 150 kPa. Two samples of Dhaka Clay (collected from two locations) were used for testing. They were collected from different depths with respect to the mean sea level. The soil samples having different moisture content were separately compacted in a compaction mold at Standard Proctor, Modified Proctor and Intermediate Compaction (varying hammer weight, blows and height of fall to obtain different compaction effort). The orientation of a sample extracted from the soil with keeping sampler axis vertical (normal to the compaction direction) is defined as $\theta = 0^0$, while that collected with the sampler axis along in the horizontal direction is defined as $\theta = 90^0$. A change in the value of θ means a change in the sample orientation in the vertical plane.

Test specimens were extracted from the samples dissembled from the mold in different orientation like $\theta = 0^0, 30^0, 45^0, 60^0, 90^0$ for determination of shear strength under different normal loading condition (50 kPa, 100 kPa and 150 kPa) for considering overburden pressure effect on the soil. Undrained direct shear test was performed on each specimen to evaluate the shear strength and deformations. The conclusions derived from the detail laboratory test program as well as recommendations for future study related to this research are described in this chapter.

5.2 Conclusion

- Optimum moisture content was observed 16%, 14.5% and 13% in case of Standard Proctor, Modified Proctor and Intermediate compaction, respectively, for Dhaka Clay Type-1 and it was 14%, 13% and 12% for the same levels of compaction energy, respectively, in case of Dhaka Clay Type-2. That is optimum moisture content of Dhaka Clay varies with the compaction effort. However it is observed that optimum moisture content evaluated for a given compaction energy level is independent of the mold size used for reconstitution of the sample.

- Both the two types of Dhaka Clay Type-1 and Type-2 are showing strength and strain anisotropy. The shear stress ~ displacement plots for different orientation angles of compaction direction, although similar in nature, exhibit different peak indicating the existence of anisotropy in peak shear strength. The above findings are not influenced by the magnitude of the normal loading. Similarly, the variations of normal displacement and shear displacement for different orientation angle θ are not similar to each other although the shape of the curves is similar and the testing conditions are otherwise similar indicates that anisotropy exists in the dilatancy characteristics.

- The volume of the test specimen decreases with the increase of stress at initial stages of loading in all cases of normal loading. The volume was increased at failure in case of normal loading of 50 kPa. However, the volume was not increased to the similar level as it did in case of smaller normal stress after initial decrease in case of $\sigma_v = 100$ and 150 kPa. This variation yields large variation and hence anisotropy in the stiffness characteristics.

- The magnitude of the deviation of the peak shear strength value for different θ lies within 10%, 26% and 13% in case of Standard Proctor compaction, Modified Proctor compaction and Intermediate compaction, respectively, for Dhaka Clay Type-1 and the same was within 15%, 37% and 14%, respectively, in case of Standard Proctor compaction, Modified Proctor compaction and Intermediate compaction, respectively, for Dhaka Clay Type-2.

- The coefficient of anisotropy I is defined as the ratio of peak shear strength at any orientation angle θ to the corresponding strength for $\theta = 0^{o}$ (i.e. $S_{\theta}{}^{o}/S_{0}{}^{o}$). For Dhaka Clay Type-1 reconstituted at optimum moisture content under Standard Proctor compaction energy, the value of I varied in the range of 1.04~0.95 for σ_v = 50 kPa, 1.00~0.93 for 100 kPa and 1.03~0.98 for 150 kPa. However, when the moisture content was changed to (OMC+2) % under otherwise similar conditions, the value of I varied in the range of 1.03~0.92 for σ_v = 50 kPa, 1.00~0.90 for 100 kPa and 1.00~1.04 for 150 kPa. Similarly for moisture content (OMC-2)%, the value of I varied in the range of 1.00~1.05 for σ_v = 50 kPa, 1.00~0.97 for 100 kPa and 1.02~0.97 for 150 kPa.

- Under otherwise similar conditions, the degree of anisotropy is not significantly influenced by the variation of moisture content.

- For Dhaka Clay Type-1 reconstituted at optimum moisture content under Modified Proctor compaction, the value of I varied in the range of 1.00~1.11 for σ_v = 50 kPa, 1.00~1.26 for 100 kPa and 1.00~1.26 for 150 kPa. Similar testing was employed for Dhaka Clay Type-1 under Intermediate compaction energy defined as the energy level, which is in between Standard and Modified Proctor energy levels. For the latter case, the value of I varied in the range of 1.13~0.96 for σ_v = 50 kPa, 1.01~0.87 for 100 kPa and 1.00~0.90 for 150 kPa.

- For Dhaka Clay Type-2, the variables were orientation angle, compaction energy and normal stress. Test results show that the value of I varies in the range of 1.00~0.90 for σ_v = 50 kPa, 1.00~0.90 for 100 kPa and 1.00~0.85 for 150 kPa in case Standard Proctor compaction. Similarly for Modified Proctor compaction the value of I varies in the range of 1.11~0.99 for σ_v = 50 kPa, 1.00~1.37 for 100 kPa and 1.00~1.31 for

150 kPa. For Intermediate compaction, I varies in the range of 1.03~0.93 σ_v = 50 kPa, 1.14~1.00 for 100 kPa and 1.00~1.13 for 150 kPa.

- Comparing the values of I it is observed that under other wise similar conditions, the magnitude of strength anisotropy in case of intermediate compaction lies between the magnitudes of the strength anisotropy obtained from Standard Proctor compaction and Modified Proctor compaction. That means a tendency of increasing anisotropy was observed (with minor exceptions) with the increase of compaction energy during reconstitution of specimens.

- Shear Strength, S_0 (i.e., $\theta = 0^o$) is used conventionally in design assuming isotropic strength of soil. Test results show that most of the specimens oriented other than $\theta = 0^o$, under otherwise similar conditions, are having shear strength higher than S_0 by a margin of maximum 37%. In these cases, the conventional design will keep the foundation of the structure at more conservative side. On the other hand, a few specimens, under otherwise similar conditions, are having shear strength lower than S_0 by a margin of maximum 15%. In these cases, the factor of safety in the conventional design will be reduced but the structure will not be in unsafe condition because the factor safety used in case of strength of soil is usually varied in the range of 2~2.5.

- For both the clays, the degree of anisotropy was observed to be influenced slightly by the variation of σ_v (i.e., normal stress), when the specimens were reconstituted at optimum moisture content under Standard Proctor and Intermediate compaction. Similar results were observed in specimens reconstituted under Modified Proctor compaction energy and tested at lower σ_v (i.e., σ_v = 50 kPa).

- However the value of I was observed to increase noticeably with the increase of σ_v (i.e., 100 kPa and 150 kPa) for both the soils reconstituted by Modified Proctor compaction. In this case, the maximum anisotropy was about 37%.

5.3 Recommendation for Future Study

The direct shear testing program in the laboratory condition was under taken for characterization of strength anisotropy for compacted Dhaka clay. The tests performed were not enough to end the topics considering its importance. The properties of Dhaka clay vary at some extent for different locations and depths up to lower strata. Moreover Dhaka clay should get the highest priority for its location, urbanization and industrialization of Bangladesh centering the Dhaka city. The properties of every type of soil located in different areas in Bangladesh will vary and more research will reveal the science of the soil to characterize the parameters accurately and the design will be more realistic in respect of safety, cost and durability.

In this regard the following aspects are recommended for future study and consideration;

- Research on similar topics involving more field and laboratory testing program for confirming the outcome of this research.
- Effects of age of compacted clay on strength anisotropy.
- Effects of clay mineralogy on strength anisotropy.
- Strength anisotropy of other type of soils used in Bangladesh for construction/ development of geotechnical structures.
- Conducting research work on each type of soil for characterization of all other related parameters needed in design calculations for the structures.
- Development of analyzing tools and application of these parameters in design and implementation to make the structures safe and economical.

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BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, DHAKA

OFFICE OF THE MEMBER SECRETARY OF THE COMMITTEE FOR ADVANCED STUDIES AND RESEARCH, BUET, DHAKA

Application for the approval of M. Sc. Engineering Thesis Proposal

1.	Name of the Student	:	Md. Monayem Hossain Roll No. 9504234 (P) Session - 1994-95-96
2.	Present Address	:	Engineer Assoconsult Ltd. Proshika Bhaban, Section-2 Mirpur-2, Dhaka Tel - 9004662-3
3.	Name of the Department	:	Civil Engineering (Geotechnical Division)
4.	Date of the first Enrolment into M. Sc. Engg. Programme	:	October, 1997
5.	Tentative Title	:	Anisotropic Strength Characterization of Compacted Dhaka Clay

6. **Background and Present State of the Problem**

The most soils are anisotropic in strength, deformation and hydraulic characteristics. Stress states in the field under loading condition are not unidirectional, rather the magnitude of stress differs from an element to another. Besides, the principal stresses direction, and hence the principal strain direction in soil mass under loaded condition rotates along the potential failure surface. Therefore, the geotechnical problems are three-dimensional (3-D), for which elastoplastic analysis are becoming popular. For 3-D characterization of elastoplastic deformation of geomaterials, their possible anisotropy should be properly accounted for all possible stress and stain paths. In the analysis of different Geotechnical structures, isotropic behavior of soil is considered. Thus the structures are becoming unsafe, where coefficient of anisotropy (defined as S_1/S_2, where S_1 is the strength of a vertical sample and S_2 is the strength of a horizontal specimen) is greater than one and factor of safety is becoming more in case of lower coefficient of anisotropy. Although a large amount of literature is available on directional variation in strength, it is solely devoted to in-situ soils and to laboratory samples under controlled mineralogy.

Substantial works on the anisotropy of soils are reported in literature, such as Oda, M. (1972), Freemen, W.S. and Sutherland, H.B. (1974), Baracos, A. (1977), Graham, J. (1979), Quigly, R.M. (1980), Graham and Houlsby (1983), Ampadu (1991), Jardine (1994) and Islam, M.S. (1999).

All these research work revealed the existence of strength anisotropy in clay. Rao and John (1982) investigated the role of anisotropy in compacted soils and quantified the directional variation in shear strength based on comprehensive laboratory testing program and concluded that maximum and minimum strengths for vertical and horizontal specimens, respectively varied from 15-40%.

Research done with clay of Bangladesh are: Stress-strain behavior of micaceous clay and structure of compacted clay (Ali, 1980), Deformation characteristics and geotechnical properties of Dhaka clay (Ameen, 1985), Permeability and consolidation characteristics of normally consolidated clays (Siddique, 1986), Analysis and design of plain and jacketed stone column in clays (Alamgir, 1989), Compressibility and shear strength of remolded Dhaka clay (Uddin, 1990) and Strength and deformation anisotropy of clays (Islam, 1999). Of them, the last one is related to the anisotropy of clay.

Islam (1999) investigated the presence of anisotropy in clay (natural clay as well as two types of reconstituted clays - compacted clay and Ko-consolidated clay). It was found the co-efficient of anisotropy S_1/S_2 (strength of vertical specimen/strength of horizontal specimen) was varied between 0.6 to 1.25 for compacted clay and the results were not similar for unconfined compression (UC) and unconsolidated undrained direct shear (UU_{DS}) tests. The co-efficient was 1.58 to 2.2 from tri-axial test and 0.88 to 1.08 from direct shear test for the constituted Ko-consolidated clay. For natural clay it was 0.75-1.25 in UC tests, 1.0-1.55 in UU_{DS} tests and 1.0-1.16 in consolidated undrained direct shear (CU_{DS}) tests. For that research program, emphasis was given for tests on the natural and the constituted Ko-consolidated samples. However, limited tests were performed to investigate anisotropy in compacted soils, although this type of soil could exhibit more anisotropy behavior than the other naturally deposited or reconstituted Ko-consolidated samples because of the preferred uni-directional energy application during reconstitution. Moreover sufficient data was not available to establish a correlation between the compaction effort and the coefficient of anisotropy of compacted samples.

Therefore, research is necessary to characterize the anisotropic behavior of compacted soils (as such soils are frequently used for land reclamation) found in different parts of Bangladesh and to apply the results to the analysis of various geotechnical problems (subject to principal stress and strain rotations), such as bearing capacity of footing and raft, slope stability, retaining wall, embankment, etc.

7. **Objectives with specific aims and possible outcome:**

The study will be carried out with the following objectives:

- Study shear strength characteristics of compacted Dhaka clay.
- Study the degree of directional dependency (i.e., anisotropy) of shear strength characteristics.
- Study the effects of moisture content on the strength anisotropy.
- Study the effects of compaction efforts on the anisotropic behavior.

8. **Outline of Methodology**

Static laboratory tests such as direct shear, unconfined compression and tri-axial tests will be performed to characterize anisotropy. Dhaka clay will be used for this purpose. The clay will be collected from a location of Dhaka city. The sample will be air dried and powdered by grinding in the laboratory. The particles passing through # 200 sieve will be used for sample reconstitution. Water will be added at the required percentage and the mass will be compacted in a mould applying a specified amount of energy. Moisture content will be the optimum moisture content (w_{opt}), a wet state (moisture content = w_{opt} + 2%) and a dry state (moisture content = w_{opt} - 2%) for a given compaction effort, while the compaction effort will vary as per Standard Proctor energy, Modified Proctor energy, the energy less than Standard Proctor and the energy more than Standard Proctor but less than Modified Proctor. After dissembling the mould, the samples with various orientations ($\theta = 0^\circ, 30^\circ, 45^\circ, 60^\circ$ and 90°) will be extracted from the compacted mass for testing. The orientation of a sample extracted from the compacted soil mass keeping the sampler axis vertical (parallel to the compaction direction) is defined as $\theta = 0^\circ$, while that collected with sampler axis along in the horizontal direction is defined as $\theta = 90^\circ$. Samples so collected will be subjected to undrained direct shear test, unconfined compression tests and triaxial tests to evaluate anisotropy. It is to be noted that the change in the value of θ means the change in sample orientation with respect to the vertical plane/compaction direction. If a clay mass is isotropic, strength ad deformation characteristics will be independent of the values of θ.

8. **References**

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10. **List of the Courses so far taken**

Course No.	Course Name	Credit Hours	Grade	Grade Points	G.P.A.
CE6401	Soil Mechanics I	3.0	A+	4.0	
CE6402	Soil Mechanics II	3.0	A	3.5	
CE6405	Slope Stability and Earth Dams	3.0	A	3.5	3.83
CE6407	Soil Dynamics	3.0	A+	4.0	
CE6410	Constitutive Modeling in Soil 4.0Mechanics	3.0	A+	4.0	
CE6411	Earthquake Engineering	3.0	A+	4.0	

Signature of the Tabulator

11. **Cost Estimate**

a.	Soil sample collection	:	Tk.	1000/-
b.	Procurement of;			
	i. Plastic bucket of 75 litre, 3 pcs, @ Tk. 350/-	:	Tk.	1050/-
	ii. Polythene, wax (for sample preservation), grease	:	Tk.	1500/-
	iii. Wodden mallet, plate for sample extruder and wooden extruder	:	Tk.	1500/-
c.	Soil mass reconstitution and test sample extraction Labour 1 person/day, 60 days @ Tk. 100/- day	:	Tk.	6000/-
d.	Conveyance	:	Tk.	1000/-
e.	Computer diskette, ribons, paper, photograph	:	Tk.	3000/-
f.	Typing, drafting and binding	:	Tk.	2000/-
g.	Miscellaneous	:	Tk.	1000/-
	Total	**:**	**Tk.**	**18,050/-**

12.	Name and Designation of the Supervisor	:	Dr. Eqramul Hoque Associate Professor Dept. of Civil Engineering, BUET, Dhaka
13.	Name Designation of Co-supervisor	:	Not applicable
14.	Date and Resolution No. of relevant BPGS committee	:	
15.	Number of post-graduate students working with the supervisor at present	:	3

Signature of the student

Signature of the supervisor

Signature of the Head of the Department