NON-LINEAR BEHAVIOR OF FEW SANDS AND ITS ENGINEERING IMPLICATION

A THESIS

SUBMITTED IN FULFILMENT OF THE REQUIREMENT

FOR THE

AWARD OF THE DEGREE OF

DOCTOR OF PHILOSPHY

IN

CIVIL ENGINEERING

SUBMITTED BY

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JUNE -2013

CERTIFICATE

It is certified that the work carried out in the thesis titled “NON-LINEAR BEHAVIOUR OF FEW SANDS AND ITS ENGINEERING IMPLICATION” which is being submitted by Sadanand Ojha (Registration No. 5364) for the fulfilment of the requirement of award of degree of “Doctor of Philosophy” in the Department of Civil Engineering, Faculty of Technology, University of Delhi, Delhi, is original work carried out by the candidate during the period from June 2006 to June 2013 under my supervision and guidance. The matter presented in this thesis has not been submitted for the award of any other degree in any university.

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This work is dedicated to my father who's preaching of

Bhagwad Gita

inspired me immensely and awaken my inner self for Karma

and to give me the energy required to complete the work

I would like to express my deepest appreciation and heartfelt adulation to my guide, Prof. Ashutosh Trivedi, Head Department of Civil Engineering, Delhi Technological University, Delhi (formerly Delhi College of Engineering) for the continuous support during my Ph.D. study and research, for his patience, motivation, enthusiasm, constructive suggestion and immense knowledge of the subject. His guidance helped me during the entire period of my research and writing of this thesis. Under his able guidance I successfully overcame many difficulties and learned a lot. Despite of his tight academic schedule, he used to review my thesis progress, given his valuable suggestions and made corrections and modifications overnight which facilitated me in the completion of the work. His regular messages on my mobiles kept me reminding my targets and kept my morale high during my low confidence level.

I am also highly obliged to Prof. P R Bose (Formerly Head, Department of Civil Engineering, FoT, University of Delhi, Delhi) who guided me through the initial stages of formation of this problem whose inspiration and continuous support in the initial stage helped me to coordinate my project and made this work possible.

My sincere thanks also goes to all the faculty member of the Delhi Technological University specially Prof. Awdhesh Kumar, Prof. R. Mehrotra, Prof. Naresh Kumar, Dr. A. K. Sahu, Dr. A.K. Srivastava, whose valuable suggestion, encouragement, insightful comments, and hard questions during my Pre Ph.D. submission seminar helped me to improve in writing this thesis.

I thank my fellow lab associates Shri Sikander Tuteja, Shri Paritosh Goel and other assisting staff who helped in carrying out the various tests for the work and specifically Shri Sikander Tuteja for my computer related problems.

I am indebted to my late father who continuously enquired about my progress and whose spiritual blessings have been a constant inspiration in making this thesis a reality. The love and affection showered by mother was the greatest source of energy during the entire period.

I deeply thank my wife, Veena, and my daughter Swasti and son Shubham for their unconditional trust, timely encouragement, and endless patience. It was their love that raised me up again when I got weary.

I am also very touched to my elder brother Shri Sampurna nand Ojha who gave me important tips as and when required. I am deeply beholden to my younger brother Shri Sachchida nand ojha who helped me in my presentation and took no time to help from USA itself.

This thesis is the end of my journey in obtaining my Ph.D. This thesis has been kept on track and been seen through to completion with the support and encouragement of numerous invisible people including my well-wishers, my friends, colleagues and various institutions.
At the end of my thesis I would like to thank all those people who made this thesis possible and an unforgettable experience for me and I feel very excited to express my thanks to all those who contributed in many ways to the success of this study and made it an unforgettable experience for me.

Above all, I express my indebtedness to the blessings of Bhagwan Shri Ram who inspired me to get this work done.

(Sadanand Ojha)

There are vast deposits of sands along the banks of river Yamuna flowing through various parts of northern India. These deposits of sand contain varied amount of fines classified as silt and clay. Engineering behaviour of clean sand has been well investigated in the past few decades whereas a little effort was made to characterise sand with varied proportion of fines. Hence proper characterization of these sands with varied proportion of fines becomes a matter of great importance, since it provides support to various structures such as structural rafts and deep foundations to multi-storeyed buildings and supporting ancillary services.

In the present work, a set of standard tests are performed on the sample of sands with varied proportions of fine collected from various construction sites to study their engineering behaviour. The soil samples are analysed for grain size analysis, moisture content, density tests including maximum and minimum void ratio, liquid limit, plastic limit, relative density, relative compaction and shear strength etc. The results thus obtained have been co-related with the field observation and site specific record of SPT test. A series of triaxial tests have been performed on different samples with fine contents in the range of 5 to 25% by weight. The shear stress and shear strain data are plotted to interpret the shear strength and dilatancy characteristics of Yamuna sand with varied proportion of fines.

The stress strain behaviour of silty sand varies significantly due to the presence of fines, relative density, and confining pressure. The presence of fines modifies the grain size distribution compared to clean sand which indirectly affects the peak and critical friction angle. On the basis of stress strain plots of silty sands at varied confining pressure, fine content and relative compaction, the non-linear behaviour of Yamuna sand was evaluated based on non-dimensional and non-linear shear strength parameters. The strength behaviour of silty sand can be interpreted from its relative compaction, mean confining pressure and relative dilatancy. Use of relative compaction has an advantage of using itself with respect to the maximum density instead of its relation with minimum density. It is relatively easy to interpret relative compaction since in the field test natural density can be directly obtained in relation to the maximum density.

The non-linear and non-dimensional strength parameters Q_{af} and R_{af} were evaluated and the results thus obtained are compared with some other sands and also with the back calculated

values from the previously published literature on sand containing fines. It has been concluded that the non-linearity of Yamuna sand containing significant proportion of fines can be fairly represented by correct interpretation of non-dimensional strength parameters Q_{af} and $R_{af.}$ It has been observed that the Yamuna sand containing silts has comparable values (Q_{af} =25 to 40) for evaluation of shear strength at varied relative compaction. The presence of silt affects the strength properties of Yamuna sand significantly as reflected by the changes in the values of Q_{af}. The value of Q_{af} ranges from 50 to 30 when silt content increases from 0 to 15%. With further increase in silt (beyond 15%) Q_{af} drops to 5 at a silt content of 25% which shows that behavior of sand containing nearly 25% fines is no longer simulates to clean sand but the soil behavior is nearer to that of silt. Based on the values of new strength parameter of Yamuna sand containing varied proportions of fines, engineering implication namely bearing capacity and liquefaction potential of Yamuna sand are evaluated.

The bearing capacity of Yamuna sand with fines can be easily evaluated on the basis of non-linear, non-dimensional shear strength parameters using the field value of relative compaction without performing the triaxial test. Bearing capacity of Yamuna sand with varied proportion of fines first increases when fine contents are up to 10% then decreases with increase in fine content due to reduction in the value of Q_{af}.

The data of SPT test from Yamuna basin was collected in NCR of Delhi. The liquefaction potential of silty sand has been evaluated by Seed & Idriss method. Simultaneously the liquefaction potential was evaluated using relative compaction based on the newly established shear strength parameters Q_{af} and R_{af}. It is shown that the model technique is convergent with that of Seed & Idriss method for sandy soils while it diverges for silty sand. Also the liquefaction potential has been validated using the empirical relation developed in the present study and relative compaction obtained using field SPT data. It is proposed to use the present technique of liquefaction potential evolution (CRR) based upon Q_{af}, R_{af} and R_c specially for silty sand.

Certificate		i
Acknowledgement		ii
Abstract		iv
List of Tables		vi
List of Figures		vii
List of Symbols		x
Contents		xiii
Chapter – I Introduction		**1-9**
1.0	General	1
1.1	Factors Affecting Behaviour of Silty Sand	4
1.2	Basics of Non-Linearity	5
1.3	Aim and Scope of Research	7
1.4	Engineering Implications of Present Work	8
1.5	Organisation of Thesis	8
Chapter – II Literature Review		**10-25**
2.0	General	10
2.1	Characteristics of Clean and Silty Sand	12
2.2	Shear Strength of Clean and Silty Sand	14
	2.2.1 Factors Controlling Shear Strength of Soils	15
	2.2.2 Undrained Strength	18
	2.2.3 Drained Shear Strength	18
	2.2.4 Critical State Theory	19
	2.2.5 Steady State	20
	2.2.6 Effect of Fines (Plastic and Non-Plastic)	21
2.3	Relative Density and Relative Compaction	25
Chapter – III Experimental Programme and Preliminary Relation		**26-33**
3.0	General	26
3.1	Preparation of Soil Sample	27
3.2	Estimation of Relative Density of Sample	27
3.3	Experimental Program	27
3.4	Grain Size Analysis	46
3.5	Specific Gravity	46
3.6	Atterberg Limit	46
3.7	Shear Strength	47
3.8	Field Test (SPT)	47
Chapter – IV Development of Preliminary Relation for Relative Compaction		**48-51**
4.0	General	48
4.1	Estimation of Relative Density of Soil Sample	48
	4.1.1 Limitation of Relative Density and Evolution of Relative Compaction	49
4.2	Development of Preliminary Relationships using Relative Compaction	49
4.3	Validation of Relative density- Relative compaction relationship using Bolton's Dilatancy Equation	51

Chapter –V Results and Discussions **52-81**
5.0 General 52
5.1 Classification of Yamuna Sand 52
5.2 Void Ratio and Critical State Friction Angle 53
5.3 Shear Strength 54
5.4 Comparison of Non-Linear Engineering Behaviour of Yamuna Sand with Ottawa Sand 70

Chapter – VI Engineering Implication of Present Study **82-104**
6.0 General 82
6.1 Bearing Capacity of Soil 82
6.2 Evaluation of Bearing Capacity of Yamuna Sand with Fines 83
6.3 Liquefaction 89
6.3.1 Liquefaction Resistance of Silty Sands: Background 91
6.3.2 Evaluation of Liquefaction Potential of Yamuna Sand 95
6.3.3 Relative Compaction (R_c) based Method for Evaluation of Liquefaction Potential using Non-Linear Strength Parameters Q_{af} and R_{af} 97
6.4 Validation of Liquefaction Potential Plots 97

Chapter – VII Conclusions and Future Scope of study **105-108**
7.0 General 105
7.1 Summary 105
7.2 Conclusions 106
7.3 Scope of Further study 108
List of Publications from the present work **109**
References **110-122**

Table 1.1	Hydro-Geological Data for Yamuna River
Table 2.1	Classification of soil as per IS 1498-1970
Table 2.2	Plasticity Characteristics of Yamuna Sand with varied proportion of fines and silt
Table 3.1 (a)	Test Schedule for Index Properties
Table 3.1 (b)	Work Schedule for Triaxial Test
Table 3.1 (c)	Index properties of sand and silt
Table 3.1 (d)	Schedule of SPT test
Table 5.1	Grain Size Characteristics of Few Sands with and without fine content
Table 5.2	Plasticity Characteristics of silt and Yamuna Sand
Table 5.3	Peak Friction Angle for Yamuna Sand
Table 5.4	Intrinsic Variables of Few Sands
Table 5.5	Dilatancy Parameters with reference to Relative Compaction for Silty Yamuna Sand [Present Work]
Table 5.6	Dilatancy Parameters for Toyoura Sand based on relative density [Chakraborty and Salgado, 2010]
Table 5.7	Dilatancy parameters for Ottawa sand based on relative density [Salgado et al., 2000]
Table 5.8 (a)	Comparison of Shear strength parameter based on relative density for silty Sand
Table 5.8 (b)	Comparison of shear strength parameter based on relative compaction for silty sand
Table 5.9	Dilatancy Parameters with reference to Relative Compaction for Silty Yamuna Sand [Present Work]
Table 5.10	Compressibility of Yamuna sand with fines based on volumetric strain

Fig 1.1	View of Yamuna at Yamunotri
Fig 1.2	Yamuna river drainage basin
Fig.1.3	Typical stress strain volume change plot of a tri-axial result
Fig. 2.1	Shear stress and normal stress plot showing failure envelope
Fig. 2.2 (a)	Typical Stress-Strain Curve (Bishop and Henkel,1972)
Fig. 2.2 (b)	Typical Stress-Strain Curve (Salgado, 2008)
Fig. 2.3	Critical state line showing the boundary between the dilatant and contractive states for soil
Fig. 2.4	Typical stress strain curve for a drained dilatant soil
Fig. 3.1 (a)	Photograph showing Tri-axial test apparatus
Fig. 3.1 (b)	Schematic diagram of Triaxial Test
Fig. 3.1 (c)	Location of bore holes for SPT
Fig. 5.1 (a)	Grain size distribution of clean Yamuna sand and silt
Fig. 5.1 (b)	Grain size distribution of Yamuna sand with varied proportion of fines
Fig. 5.2 (a)	Best fit for the variation of maximum and minimum void ratio with percent fines
Fig. 5.2 (b)	Variation of maximum and minimum void ratio with mean size
Fig. 5.2 (c)	Variation of critical friction angle with percentage fine
Fig. 5.2 (d)	Variation of critical friction angle with mean size
Fig. 5.3 (a)	Deviator stress v/s axial strain at R_c = 0.92
Fig. 5.3 (b)	Deviator stress v/s axial strain at R_c = 0.93
Fig. 5.3 (c)	Deviator Stress v/s Axial Strain at R_c = 0.94
Fig. 5.3 (d)	Deviator stress v/s axial strain at R_c = 0.95
Fig. 5.3 (e)	Deviator stress v/s axial strain at R_c = 0.96
Fig. 5.3 (f)	Volumetric strain v/s axial strain at R_c = 0.92
Fig. 5.3 (g)	Volumetric strain v/s axial strain at R_c = 0.93
Fig. 5.3 (h)	Volumetric strain v/s axial strain at R_c = 0.94
Fig. 5.3 (i)	Volumetric strain v/s axial strain at R_c = 0.95
Fig. 5.3 (j)	Volumetric strain v/s axial strain at R_c = 0.96
Fig. 5.4 (a)	I_{na} v/s R_c for clean sand ($D_m^{0.225}$) with 5 percent error bars
Fig. 5.4 (b)	I_{na} v/s R_c for clean sand at 5% fines ($Dm^{0.224}$) with 5 percent error bars

Fig. 5.4 (c) I_{na} v/s R_c for clean sand at 10% fines ($Dm^{0.222}$) with 5 percent error bars

Fig. 5.4 (d) I_{na} v/s R_c for clean sand at 15% fines ($Dm^{0.215}$) with 5 percent error bars

Fig. 5.4 (e) I_{na} v/s R_c for clean sand at 20% fines ($Dm^{0.213}$) with 5 percent error bars

Fig. 5.4 (f) I_{na} v/s R_c for clean sand at 25% fines ($Dm^{0.208}$) with 5 percent error bars

Fig. 5.5 Variation of relative compaction with relative density

Fig. 5.6 Variation of maximum and minimum void ratio with percent fines

Fig. 5.7 Critical friction angle vs. percent fines

Fig. 5.8 (a) Variation of R_c vs. D_r without silt content

Fig. 5.8 (b) Variation of R_c vs. D_r at 5 percent silt content

Fig. 5.8 (c) Variation of R_c vs. D_r at 10 percent silt content

Fig. 5.8 (d) Variation of R_c vs. D_r at 15 percent silt content

Fig. 5.8 (e) Variation of R_c vs. D_r at 20 percent silt content

Fig. 5.9 (a) Variation of I_{na} vs. R_C without silt content

Fig. 5.9 (b) Variation of I_{na} vs. R_C at 5 percent silt content

Fig. 5.9 (c) Variation of I_{na} vs. R_C at 10 percent silt content

Fig. 5.9 (d) Variation of I_{na} vs. R_C at 15 percent silt content

Fig. 5.9 (e) Variation of I_{na} vs. R_C at 20 percent silt content

Fig. 5.10 Variation of unit weight with angle of internal friction for few sands

Fig. 5.11 Variation of peak friction angle with relative compaction of Yamuna sand with varied proportion of fines

Fig. 6.1 Plot showing variation of Q_{af} with percent fine at varied R_{af}

Fig. 6.2 Variation of p′/qult-peak with peak friction angle p′ at varied relative compaction for Raf = 25

Fig. 6.3 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for Raf= 30

Fig. 6.4 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for R_{af}= 40

Fig. 6.5 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for Few Sands

Fig. 6.6 (a) Variation of CRR with mean confining pressure at 0-5 percent fine content

Fig. 6.6 (b) Variation of CRR with mean confining pressure at 10 percent fine content

Fig. 6.6 (c) Variation of CRR with mean confining pressure at 15 percent fine content

Fig. 6.6 (d) Variation of CRR with mean confining pressure at 20 percent fine content

Fig. 6.6 (e) Variation of CRR with mean confining pressure at 25 percent fine content

Fig. 6.7 Variation of CRR with mean confining pressure (For all types of sands)

Fig. 6.8 Variation of R_c vs. p' at $R_{af} = 25$

Fig. 6.9 Variation of R_c vs. p' at $R_{af} = 30$

Fig. 6.10 Variation of R_c vs. p' at $R_{af} = 35$

Fig. 6.11 Variation of R_c vs. p' at $R_{af} = 40$

C_u	coefficient of uniformity
C_c	coefficient of curvature
SP	poorly graded sand
S	shear strength
$б_1$	major principal stress
$б_3$	minor principal stress
$б_d$	deviator stress
ϕ	friction angle
ϕ_p	peak friction angle
ϕ_c	critical friction angle
Ψ	dilatancy angle
G_s	specific gravity
ϵ_{axial} ϵ_v	axial and volumetric strain
$d\epsilon_v$	volumetric strain increment
$d\epsilon_1$	major principal strain increment
e	natural void ratio
e_{max} and e_{min}	maximum and minimum void ratio
γ	bulk density
γ_d	dry density
γ_w	unit weight of water
γ_{min} and γ_{max}	minimum and maximum dry density
D_r	relative density
R_c	relative compaction
Rc-critical	critical relative compaction
p'	effective confining pressure
$p`_p$	mean effective stress at peak strength
P_A	reference stress
Q, R	fitting parameters based on relative density
Q_{af}, R_{af}	fitting parameters based on relative compaction
Q_s, R_s	Salgado's fitting parameters based on relative density

r^2	coefficient of determination
D_{10}, D_{30}, D_{50}, D_{60}	diameter of particle corresponding to 10, 30, 50 and 60 percent finer
D_m	mean diameter of particles
$б`_{3p}$	lateral confining pressure
$б`_{mp}$	mean pressure = $(б_1 + 2\ б_3)/3$
I_R	dilatancy index
τ_f	shear force
τ	shear stress
c'	effective cohesion in soil
ϕ'	effective friction angle
ϕ'_c	effective critical state friction angle
ϕ'_{peak}	effective peak friction angle
I_{na}	compaction based dilatancy index
q_u	unconfined compressive strength
q_{ult}	ultimate bearing capacity
q_n	net ultimate bearing capacity
q_o	overburden pressure at foundation level
q_s	safe bearing capacity
q_a	allowable bearing pressure
N_c, Nq, Nγ	bearing capacity factors
s_c, s_q, s_γ	shape factors
d_c, d_q, d_γ	depth factors
i_c, i_q, i_γ	inclination factors
I_{af}	relative dilatancy
CSR	cyclic stress ratio
CRR	cyclic resistance ratio
η_c	relative compaction correction factor
ξ_R	state parameter
N	observed SPT value
N_{60}	SPT value corrected to 60% energy level
LL	liquid limit
PL	plastic limit

PI	plasticity index
SPT	standard penetration test
CPT	cone penetration test
G_s	specific gravity
Hc	hardening contractile behaviour
Sd	softening dilative behaviour
Nc	non-contractile behaviour
a_{max}	maximum surface acceleration
r_d	Stress reduction factor
σ_o	initial effective overburden pressure
σ_o’	effective over burden pressure

1.0 General

Most of the sands found in nature contain fines which is either silt or clay or their mixture. These fines may be either plastic or non-plastic in nature depending upon its in situ index properties. Indian Subcontinent has large deposits of sand with varied proportion of fines along the banks of many perennial rivers originating from Himalaya namely Ganga, Yamuna, Ghaggar, Indus etc. These sand deposits contain varied proportion of silt depending upon geographical and hydrological catchment along the traverse of the river. The percentage of silt present may be locational and seen in relation to elevation of origin and fall of the river. The silt quantity also varies with depth. The silt present in the silty sand is either non-plastic or plastic varying with its location from the mouth of river and the height of the fall along the length. The sand in the present study is collected from Yamuna river basin containing varied percentage of silt from NCR region of Delhi and their non-linear behaviour has been evaluated under different confining pressure and state of denseness in terms of non-linear and non-dimensional strength parameters (Q_{af} and R_{af}). It has been observed that silty sand deposits near the origin of river contain non plastic silt and at considerable distance from source and at large difference in elevation the silt present is plastic in nature. Due to the rapid increase in demand in the housing sector, a large number of multistoried residential, industrial, institutional and commercial structures are being constructed in and around capital region of Delhi in the proximity of the river Yamuna. For a safe design and to ensure that these structures don't have any structural damage in their life time, the knowledge of strength behavior of silty sand obtained from the proximity of river Yamuna becomes a matter of great importance. Yamuna River is the largest tributary of the Ganges in northern India. It is treated as a very prominent and sacred river in this subcontinent due to perennial supply of water throughout the year. Its source is at Yamunotri, in the state of Uttarakhand in the Himalayan Mountains. It flows through the states of Haryana, Uttar Pradesh, and Delhi flows again through Uttar Pradesh. Starting from the Yamunotri glacier, Yamuna covers a distance of over 1376 km, before merging with the Ganges in Allahabad. The place where Yamuna merges with the Ganges in Allahabad is known as Sangam which is a sacred place for Hindus. Fig.1.1 and Fig.1.2 shows its origin at Yamunotri and drainage basin of Yamuna River.

Fig 1.1 View of Yamuna at Yamunotri
(Source: http://www.indianetzone.com/32/origin_yamuna_river_indian_river.htm)

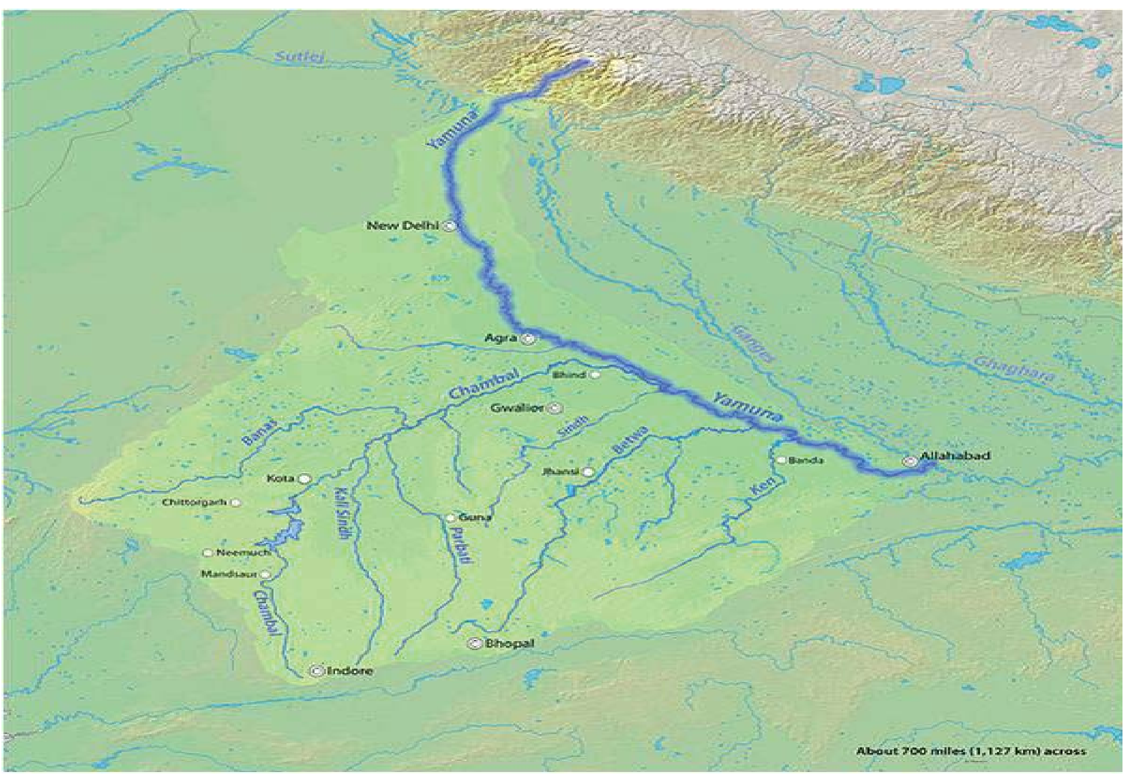


Fig 1.2 Yamuna river drainage basin
(Source: http://en.wikipedia.org/wiki/File:Yamunarivermap.jpg)

Hydro- geological data for Yamuna River is given in Table 1.1. The interpretation of this data along with other engineering properties is important for the correct evaluation of engineering properties and strength parameters for silty sand from Yamuna River basin.

Table 1.1 Hydro-Geological data for Yamuna River

S. No.	Description	Yamuna River
1.	Country	India
2.	Source	Yamunotri
3.	Mouth	Ganga
3.	Co-ordinates	31°01′0.12″N 78°27′0″E 31.0167°N 78.45°E
4.	Elevation at Source	3293m
5.	Elevation at Mouth	74m
6.	Approximate Length	1376 km
7.	Catchment Area	219663 km^2
8.	Discharge	317097 m^3/s

The human settlements along Yamuna river basin has taken place since the ancient time. River Yamuna, with a total length of 1376 km, originates from Yamunotri glacier in the Bandar punch range of Himalayas in the state of Uttarakhand. It merges with Ganga at Prayag (Allahabad) it forms the vast Ganga -Yamuna doab (flood plains) which are the most fertile plains of north India. River Yamuna basin, spread over some 219663 sq. km, lies in the states of Uttarakhand, Himachal Pradesh, Uttar Pradesh, Haryana, NCT of Delhi, Rajasthan and Madhya Pradesh.Various pilgrimage centres e.g. Yamunotri (Uttarakhand), Paonta Sahib (Himachal Pradesh), Mathura, Vrindavan, Bateshwar and Allahabad (Uttar Pradesh) are located on the bank of this river. Large urban centres e.g. Yamuna Nagar, Sonepat, Delhi, Gautama Buddha Nagar, Faridabad, Mathura, Agra and Etawa are also established on its banks. Large industrial centres have also been developed in its basin. Natural sand deposits contain significant amount of silt and clay. A soil exploration programme has been carried out at several sites in this region for erection of engineering structures which shows diverse depositional characteristics. Delhi has interesting geology on account of its being the edge of the exposed ancient Aravali mountain ranges extending NE in this area. Delhi and its adjoining region are surrounded in the north east by Indo-Gangetic plains, while in the west the extension of the great Indian Thar desert and in the south by the Aravali ranges. The rocks of Delhi have undergone multiple folding and different phases of metamorphism with some transverse features. The quartzite’s are bedded and highly jointed with intrusive pegmatite.

The Alwar series and the post Delhi intrusive are covered by the quaternary as Aeolian and alluvial deposits. Geologically, the alluvial deposits belong to the Pleistocene period, i.e., older alluvial deposits and of recent age i.e., newer alluvium. Older alluviums deposits consist of mostly inter bedded lenticula and inter fingering deposits of clay, silt and sand along with kankar. Based on the collected borehole data, soil profiles are made covering almost entire region to study the sub soil heterogeneity. In Trans-Yamuna area, silt is very predominant. In the eastern block the soils are sandy silts/silty sands with high percentage of medium to fine sand. The areas like Noida, Mayur Vihar, Yamuna Vihar, Abdul Fazal Enclave, Geeta Colony which falls in the eastern bloc has very soft soil deposits and high water table. Northern and Western blocks have silty sands with reasonable percentage of clay. The areas such as Rohini, Punjabi bagh, Paschim Vihar, Janakpuri and Dwarka falls in these blocks. The south and central blocks have gravelly sands with varied percentage of gravels (Ghitorni, Maidan Garhi, and Satbari). As per soil classification systems, these sands and silts are coarse and fine grained granular materials.

The behaviour of clean sands has been extensively investigated in the past by many researchers [Meyerhof, 1965; Vesic, 1973; Bolton, 1986; Thevanayagam et al., 1996; Jefferies, 2002.]. However there are little efforts to evaluate the engineering behaviour of silty sands. There is varied opinion in the literature to the effect of silt on the stress strain behaviour of silty sands [Vaid, 1994; Zlatovis and Ishihara, 1995; Salgado et al., 2000; Jefferies and Been, 2000; Gupta and Trivedi, 2009; Usmani et al., 2012; Ojha and Trivedi, 2013]. This material supports structural rafts and deep foundations for multi-storeyed buildings, underground excavation, tunnel and pipelines hence there is a great need to investigate the strength characteristics of silty sand and its engineering implication for design of structural foundation. The factor that controls the behaviour of silty sand is to be properly investigated.

1.1 Factors Affecting Behaviour of Silty Sand

Engineering characteristics of silty sand is governed by many factors such as amount of silt percentage, water content, void ratio, grain size distribution, confining pressure, methods of testing, nature of silt content (plastic or non-plastic), source of its origin and nature of deposits. The behaviour of silty Yamuna sand may be linear or non-linear in selected strain range depending upon the above factors. It has been observed that the stress strain behaviour of silty sand is non-linear even at very small strain [Been et al. 1985, 1991; Bardet 1986(a);

Salgado et al. 2000; Li, and Baus, 2005; Ayadat and Hanna, 2007; Chakraborty and Salgado, 2010].

1.2 Basics of Non-Linearity

(Source:http://www.colorado.edu/engineering/CAS/courses.d/NFEM.d/NFEM.Ch02.d/NFEM.Ch02.index.html)

Geotechnical models consistently indicate that the stress-strain relationship of soils is nonlinear even at very small strains though for all practical purpose it is considered linear for strain less than 10^{-5} [Salgado et al., 2000]. The main features of linear behaviour are given below:

1. The soil can sustain any magnitude of load and undergo any displacement magnitude.
2. There are no critical, yield or failure points.
3. Response to different load systems can be obtained by superposition.
4. After Removal of all loads the plot returns to the reference position.

The assumptions for linear model to be applicable are:

1. Perfect linear elasticity for any deformation
2. Infinitesimal deformations
3. Infinite strength

There are serious limitations placed on the validity of the linear model. These assumptions are not only physically unrealistic but mutually contradictory. For example, if the deformations are to remain infinitesimal for any load, the body must rigid rather than elastic, which contradicts the first assumption. Thus, there are necessarily limits placed on the validity of the linear model. Nonetheless, the linear model can be a good approximation of portions of the nonlinear response. However linear model is widely used in design calculations due to its simplicity and that the principle of superposition applies. Hence for accurate prediction of soil behaviour non-linear models are used. Over all non-linear behaviour of silty sand is characterized by a stress strain plot as typically shown in Fig. 1.3. In the Fig. 1.3 a typical stress strain plot of a tri-axial result is shown. The response curve shows non-linearity even at very small strain indicating that behaviour of soil as non-linear.

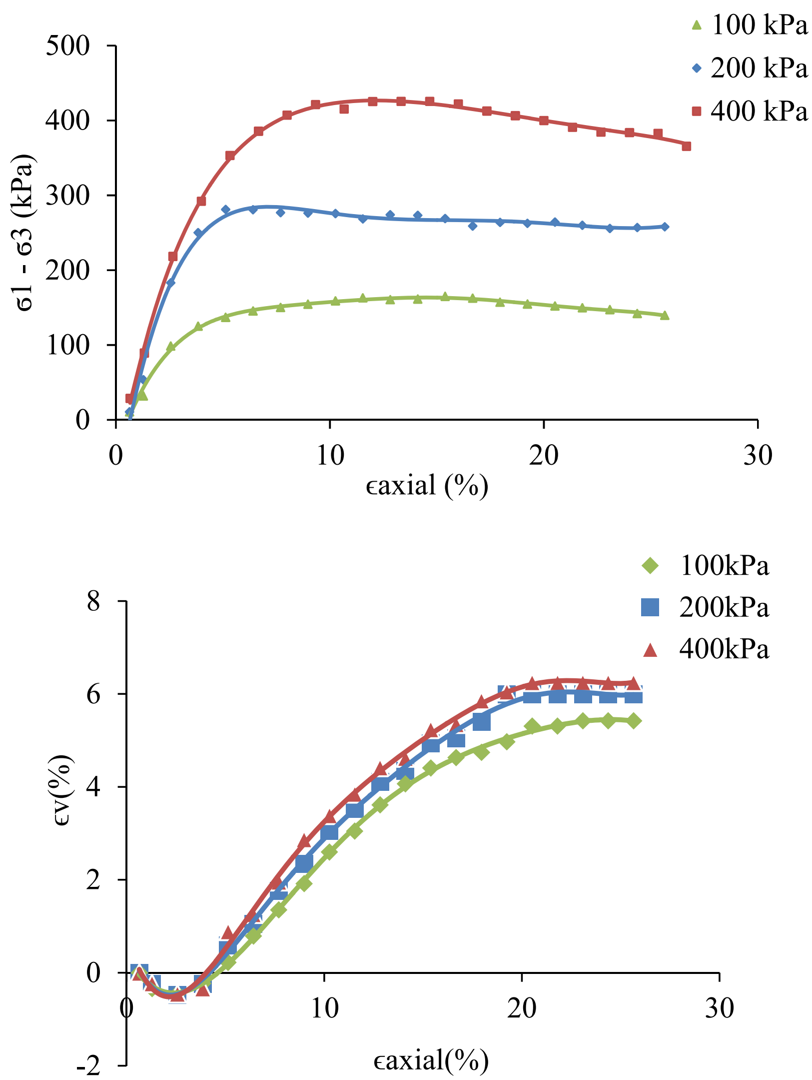


Fig.1.3 Typical stress strain volume change plot of a tri-axial result

There are normally two types of non-linearity observed in engineering application. They are listed below:

1. Material Non-Linearity:

Material behaviour depends on current deformation state and possibly past history of the deformation. Relative density, void ratio, and over consolidation ratio plays an important role for prediction of material non-linearity, while grain size distribution plays the crucial role [Vaid et al.1988, 1999; Yang and Gu, 2013].

2. Geometric Non-Linearity:

Change in geometry as the soil structure deforms is taken into account in setting up the strain displacement and equilibrium equations.

In the present work material non-linearity of silty sand has been considered for evaluation of non-linear shear strength parameters. Non-linear analysis is useful in the evaluation of following engineering applications [Chapter 2, A Tour of Nonlinear Analysis, Part I: Fundamentals of Nonlinear Structural Analysis, Course Content of University of Colorado at Boulder, US].

1. Strength Analysis: It gives an indication of the magnitude of load the soil can sustain before failure i.e. evaluation of safe bearing capacity of soil.

2. Deflection Analysis: Non-linear analysis is very useful in the settlement analysis of structural foundation under static and cyclic loading when deflection control is of primary importance.

3. Stability Analysis: Stability of slopes during deep excavation, earthquake excitation and under surcharge load due to vehicular movement also depends upon the non-linear behaviour of the supporting soil.

4. Reserve Strength Analysis: Evaluation of load carrying capacity beyond critical points to assess safety under abnormal conditions.

5. Progressive Failure Analysis: A variant of stability and strength analysis in which the progressive deterioration (e.g. cracking) is considered.

6. Envelope Analysis: A combination of analyses in which multiple parameters are varied and the strength information thus obtained is condensed into failure envelopes.

1.3 Aim and Scope of Research:

There are no past studies reported so far in the engineering literature which provide a direct relationship for prediction of the shear strength and dilatancy parameters of silty Yamuna sand obtained from the river Yamuna [Salgado et al. 2000; Vaid, 1994; Lo Presti et al. 2000]. To bridge this gap the main objectives of this work is listed below:

1. To evaluate the non-dimensional non-linear shear strength parameter of silty Yamuna sands with varied percentage of fines.

2. To predict the shear strength of Yamuna sand at varied confining pressure and relative compaction using non-linear strength parameter.
3. To carry out sensitivity analysis to evaluate the effect of variation of any one of the contributing factor like relative density, relative compaction, silt content and confining due to non-dimensional non-linear shear strength parameters (Q_{af} and R_{af})

1.4 Engineering Implications of Present Work

1. Shear strength
2. To evaluate bearing capacity of soil using non-linear, non-dimensional shear strength parameters.
3. To co-relate the liquefaction potential of silty sand with the softening parameters using relative compaction.

1.5 Organisation of Thesis

This thesis consists of seven chapters. Introduction of river Yamuna, its origin and destination, and various characteristics of silty sand are presented in Chapter–I. Basics of linearity and non-linearity is also described in this chapter. The limitation of linear soil behaviour and the necessity of the studying nonlinear behaviour of soil in the present research have been enumerated in the first chapter. The organisation of various chapters in the present work is also briefly described here. All the relevant literature and the research have been reviewed under chapter II titled as Literature Review. Critical evaluation of the past literature and need for the present research is also included in this chapter. Brief description of the factors contributing the non-linearity in silty Yamuna sand is also discussed. Limitation of determination of relative density which resulted in the introduction of the term relative compaction is also briefly mentioned. Chapter III deals with the details of the experimental programme carried out in the laboratory to determine the physical and engineering properties of Yamuna sand with varied proportion of fines. Methods of preparation of samples for various tests have also been given in this section. Chapter IV deals with the development of basic equations required for the interpretation of the non-linear and non-dimensional shear strength parameters using relative compaction. A relationship among relative dilatancy, relative compaction and mean confining pressure in terms of a new term I_{na} known as Compaction based Dilatancy index (abbreviated as I_{na}) has been derived in this chapter.

Conventional method of estimation of relative density has been briefly discussed. Advantage of using relative compaction in place of relative density has also been presented in this chapter. A relationship between relative density and relative compaction has been developed and validated by using Bolton's Dilatancy relation for shear strength parameters. Chapter V deals with the interpretation of test results and evaluation of the non-linear and non-dimensional shear strength parameters of Yamuna sand with varied proportion of fines. Chapter VI deals with the Engineering implication of the present study for estimation of bearing capacity of soil using non-linear non-dimensional parameters. A simple method based on shear strength parameters has been presented in this chapter. An empirical equation for evaluation of liquefaction potential has also been derived in this chapter. The new liquefaction plots have been developed for evaluation of the liquefaction potential of silty sand without going through the rigorous analysis available in the past literature. Conclusions and the scope for further study have been included in chapter VII. List of publications from present work has been given at the end of chapter VII. References for the present work are listed at the end of the thesis.

2.0 General

In this chapter, literature in the following main areas is reviewed:

1) Geological and Geotechnical study of silty sand
2) Research on the shear strength of sand and sand with fine material
3) Engineering implication of non-linear behaviour of silty sand namely bearing capacity and liquefaction study

The first two topics are independent but combining the knowledge from these areas will help to develop simple relations to estimate bearing capacity of silty sand and evaluate the liquefaction potential of silty sand taking into consideration of its non-linear engineering behaviour.

Soil mechanics has been developed in the beginning of the 20th century [Verruijt and Van Baars, 2007]. The need for the analysis of the behaviour of soils arose in many countries, due to the occurrence of accidents, such as landslides and failures of foundations [Wood, 1990]. The first important contributions to soil mechanics are due to Coulomb, who published an important treatise on the failure of soils in 1776, and to Rankine, who published an article on the possible states of stress in soils in 1857. In 1856 Darcy published his famous work on the permeability of soils, for the water supply of the city of Dijon where he proposed to empirically relate effective size with hydraulic conductivity of sands. The effect of material variables in the voids of clean sand such as significant proportion of silts and clays could not be fairly predicted by this relationship. In the 17th century, Newton worked on the principles of the mechanics of continua, including statics and strength of materials. Important pioneering contributions to the development of soil mechanics were made by Karl Terzaghi (1942), who described the influence of the pore water pressures on the behaviour of soils.

The sand is normally found in nature with varied proportion of fines. The soil in the lower reaches and plain area of river basin contains either silt or clay or both in certain proportion depending upon hydro-geological factor. The sand containing varied percentage of silt is called silty sand and is predominantly found in nature. The silty sand was one of the most prominent engineering materials ever since the human settlement started. Some of the preliminary references about the behaviour of silty sand is available in the Sanskrit text of early Aryan civilisation which states that the hydraulic conductivity of this material is such that it has free draining potential for the flow of water. The free flowing behaviour of silty sand indicates possibility of large void space available for the drainage of water through this material which made it a highly valuable media for cultivation and settlement of early civilization. The early Roman empires constructed road for the expansion and control of their kingdom. They removed surface material and replaced it by finished rocks.

Mohr was the first to present a generalised form of the theory in the year 1800 for the evaluation of the shear strength of the granular sand. Mohr theory of failure states that the failure of soil due to shear stress depends upon normal stresses on the potential failure plane and the failure is caused by a critical combination of normal and shear stresses. Mohr proposed that shear strength of soil at failure is a unique function of normal stress acting on that plane and is represented by following expression,

$$\tau_f = f(\sigma) \qquad (2.1)$$

A plot drawn between normal stress and shear stress at failure using Eq. (2.1) is called Mohr's envelope and represents a unique failure envelope for each soil. Coulomb (1776) introduces the idea that shear resistance developed on a soil mass under normal loading is a function of cohesion and frictional resistance between soil particles. Using the concept of Mohr, Coulomb developed a more general relation by considering cohesion between soil particle (c) and angle of internal friction (ϕ), known as Mohr–Coulomb failure criterion**.** This relation is a linear fit expressed as

$$\tau = c + \sigma \tan \Phi \qquad (2.2)$$

For cohesion less soil the Eq. (2.2) reduces to

$$\tau = \sigma \tan \Phi \qquad (2.3)$$

This is a form of Eq. (2.1). The failure occurs when the stresses are such that the Mohr circle has a common tangent as the failure envelope as shown in Fig. 2.1. It is clear that the failure occurs if the stresses σ and τ lie on or above the failure plane. Point lying below failure envelope represents a stable condition.

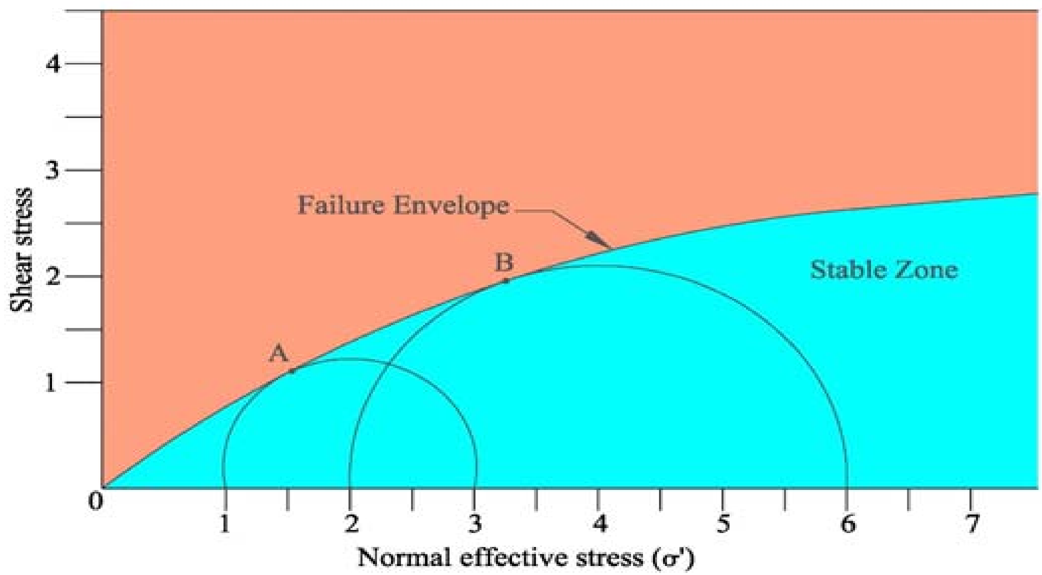


Fig. 2.1 Shear stress and normal stress plot showing failure envelope

Later the outstanding contribution of Rankine (1862) and Boussinesq (1885) to stress field analysis and latter's correct treatment of tension crack problem led to the evaluation of lower bound solution for the critical height of the vertical cut. Rankine (1862) worked out a unified approach to slope stability and earth pressure for sand and for clays in the long term condition using a simple and practical approach, popularly known as Rankin's theory of earth pressure.

Further the research done by various investigator showed that the parameter c and ϕ depends upon numbers of factors such as water content, drainage conditions, and methods of testing and is not necessarily the fundamental properties of soil. Terzaghi (1948) established that the normal stresses that controls the shear strength of the soil are effective stresses and not normal stresses, and modified the Eq. (2.2) as

$$\tau = c' + \sigma' \tan \Phi' \tag{2.4}$$

Here c' and ϕ' are the cohesion intercept and angle of internal friction in terms of effective stresses. Eq. (2.4) is known as revised Mohr-Coulomb equation for shear strength of soil.

2.1 Characteristics of Clean and Silty Sand

Soils can be classified on the basis of the grain size of the particles that constitute the soil [Trivedi and Sud, 2002, 2007; Shanthakumar et al., 2010]. Coarse granular material is often denoted as gravel and finer material as sand. Soil classification deals with the systematic categorization of soils based on distinguishing characteristics as well as criteria that dictate choices in use. There are many classification system based upon different parameters for classification of sand particles. AASHTO Soil Classification System was developed by the American Association of State Highway and Transportation Officials, and is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction purposes. The classification system was first developed by Hogentogler and Terzaghi (1929) but has been revised several times since then.

According to AASHTO Soil Classification System if the material passing through 0.075 mm sieve size are equal to or less than 35 % then the soil is grouped under categories A1-A3 as Granular material and if more than 35% material passes through 0.075 mm sieve then the soil is classified as Silt-Clay and is grouped under category A4-A7 (AASHTO M 145 or ASTM D3282).

In order to have uniformity IS 1498-1970 gives guidelines based on sieve analysis and particle size. Particles size greater than 4.75 mm, but less than 80 mm have been classified as gravel. Larger particles are denoted as stones. Sand is the material consisting of particles smaller than 4.75 mm, but larger than 0.075 mm. Particles smaller than 0.075 mm and larger than 0.002 mm is denoted as silt. Soil consisting of even smaller particles, smaller than 0.002 mm, is denoted as clay as given in Table 2.1.

Table 2.1 Classification of soil as per IS 1498-1970

Soil type	Minimum particle size (mm)	Maximum particle size (mm)
Clay	-	0.002 mm
Silt	0.002 mm	0.075 mm
Sand	0.075 mm	4.75 mm
Gravel	4.75 mm	80 mm

According to the Unified Soil Classification System USCS (ASTM D 2487), silty and clayey sands are soils that contain at least 50% of particles larger than 4.75 mm and more than 12% particles smaller than 75 μm, by weight. Depending on the plasticity characteristics of the fraction smaller than 75 μm, sand is classified as silty sand (SM) if the fines classify as silt. Sands with 5 to 25% fines require dual symbols based on the USCS. In the present work Yamuna sand containing varied proportion of fines (0 to 25%) has been considered and its grain size distribution curve has been drawn. The basic difference between silty and clean sands is in the careful choice of methods to quantify density for a logical and correct evaluation of shear strength and liquefaction of these soils. Secondly, analysis based exclusively on density and stress state are not sufficient for these materials, as the soil fabric (and thus specimen preparation) is a key determinant of their behaviour [Leroueil and Vaughan (1990)]. Also, not only the content of fines, but also their plasticity, needs to be properly accounted for proper evaluation of their strength property. Plasticity Characteristics of few sands used in the present work are given in the Table 2.2.

Table 2.2 Plasticity characteristics of Yamuna Sand with varied proportion of fines and silt

S.No.	Sample Description	Liquid Limit	Plastic Limit	Plasticity Index
1	Silt	22.4%	10%	12.4%
2	Yamuna Sand+ 0% Silt	-	-	-
3	Yamuna Sand + 5% Silt	-	-	-
4	Yamuna Sand + 10% Silt	15.0%	No thread of 3-mm could be formed (Non Plastic)	-
5	Yamuna Sand + 15% Silt	18.0%	Non Plastic	-
6	Yamuna Sand + 20% Silt	20.0%	Non Plastic	-
7	Yamuna Sand + 25% Silt	22.0%	Non Plastic	-

2.2 Shear Strength of Clean and Silty Sand

Shear strength of a soil is defined as the limiting value of the shearing stress that a soil can bear before failure. The shear strength of the soil is mainly due to the frictional forces of the inter particle contacts between the soil grains and meshing of particles, cementation and bonding at particle contacts. The soil behaviour may be dilative or contractive in volume, when subjected to shear strains depending upon the extent of interlocking of the particles. The expansion of volume of soil results in reduction of its density as well as strength and the peak strength is followed by a reduction of shear stress. The stress- strain curve becomes horizontal and the soil continues shearing at constant volume. This is the stage when the inter-particle bonds are broken. This state of soil at which the shear stress and density remains constant while the shear strain increases is called the critical state or steady state. A critical state line separates the dilatant and contractive states for soil as shown in Fig.2.3 [Bishop et al., 1965]. The volume change behaviour and inter-particle friction depend on the density of the particles; inter granular contact forces, and other factors such as the rate of shearing and the direction of the shear stress. The presences of pore water further results in the reduction of inter granular contact force. The net normal inter granular contact force per unit area is after considering pore water pressure is known as effective stress. A typical plot depicting various state of stress for loose sand, dense sand, soft clays and stiff clays are shown in the Fig. 2.2a [Bishop and Henkel,1972] and Fig. 2.2b [Salgado, 2008]

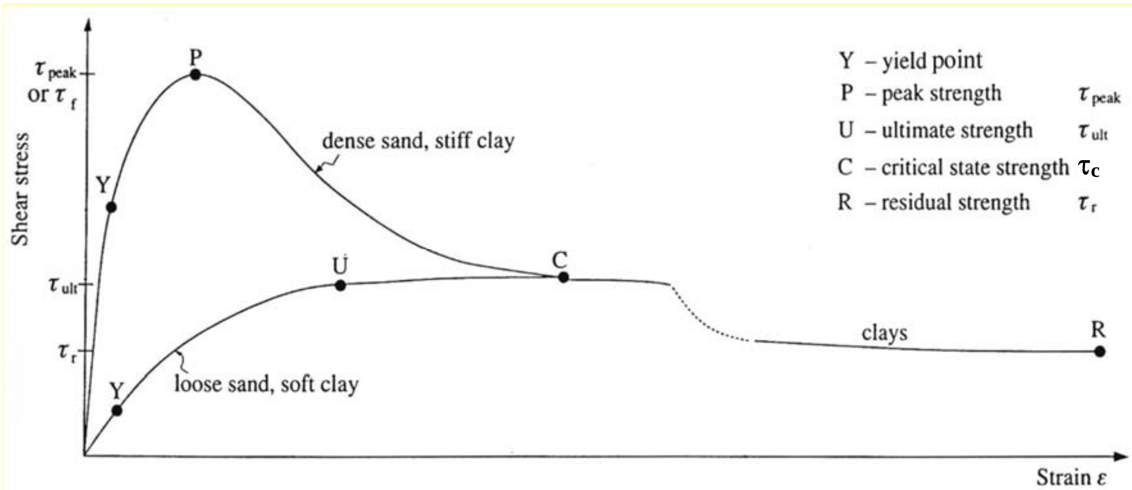


Fig. 2.2(a) Typical stress-strain curve for soils (Bishop and Henkel,1972)

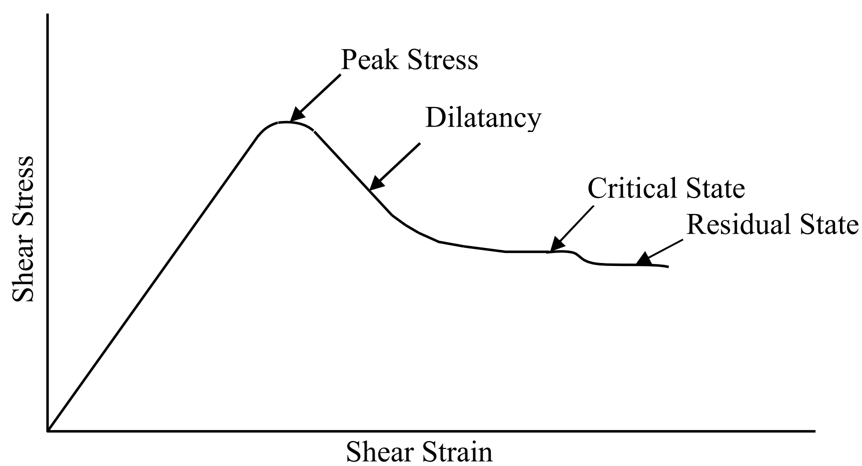


Fig. 2.2 **(**b) **T**ypical stress-strain curve for soils (Salgado, 2008)

2.2.1 Factors Affecting Shear Strength of Soils

The shear strength of the soil can be represented by the stress strain plot of a tri axial test output. The shear strength of soil depends upon many factors [Poulos, 1989]. The main factors that affect the stress-strain relationship of soils are given below:

1. Composition of Soil:

Mineralogy of soil grains, particle size distribution, shape of particles and water content in the soil mass greatly affects the shear strength of soil.

2. State of Soil:

The state of soil is expressed in terms of initial void ratio, effective principal stress and the effective shear stress. The state of a soil can be defined in terms of relative density or relative compaction. The soil can be described by terms such as loose soil, dense soil, over consolidated, normally consolidated, stiff, soft, contractive, dilative, etc. depending upon the index properties of the soil.

3. Soil Structure**:**

It depends upon the distribution of soil particles within the soil mass; the pattern of the packed particles and the particle size distribution. Structure of soils is mainly described by terms such as: undisturbed, disturbed, remoulded, compacted, layered, honey-combed, single-grained etc.

4. Pattern of Loading:

The shear strength of soil also depends upon type of loading and the condition under which the shearing takes place. Loading of soil sample under different loading condition like drained, undrained, consolidated drained, and unconsolidated undrained gives different shear strength. Magnitude of the load and the rate of loading also play an important role. It is further affected if the loading is static, dynamic, monotonic or cyclic.

5. Methods of Preparation of Sample:

The method selected for specimen reconstitution has a strong influence on the stress-strain response of sands containing fines. Moist tamping (MT), air or dry pluviation (AP), and water or wet pluviation (WP) or slurry deposition (SD) are the most widely used reconstitution techniques for these soils. Loose saturated MT specimens of sands with fines are typically the most contractive and show strain-softening response under undrained monotonic triaxial compression. Vaid (1994); Vaid et al.(1999); Thevanayagam et al.(2002) argued that the MT technique does not simulate the fabric of alluvial soil deposits and also indicated that specimens prepared with this technique may not be uniform.

The stress strain behaviour of clean sand has been investigated extensively in the past. A loose, saturated clean sand exhibit different type of behaviour such as contractive behaviour (Thevanayagam and Mohan, 2000), strain-softening (Pitman et al., 1994), pre-failure strain-softening (Chu et al., 2001) or flow liquefaction (Yang, 2002). The behaviour of medium dense sand is characterized by contraction followed by dilation (Thevanayagam & Mohan, 2000), strain-softening followed by strain-hardening (Pitman et al., 1994) or limited liquefaction (Vaid et al., 1990). However the behaviour of very dense sand is more or less is dilative. It is normally seen that clean sand will approach a Steady State Line (SSL) or

Critical State Line (CSL) at large strains before failure [Chu and Lo, 1993]. A plot drawn between mean effective stress and the void ratio is shown in Fig. 2.3. There is always a critical state line which divides the state of soil as dilative or contractive corresponding to a certain value of mean confining pressure and void ratio. Soil in dense state shows dilative behaviour in general. Fig. 2.4 (a) shows typical shear stress – shear strain curve for a drained dilatant soil.

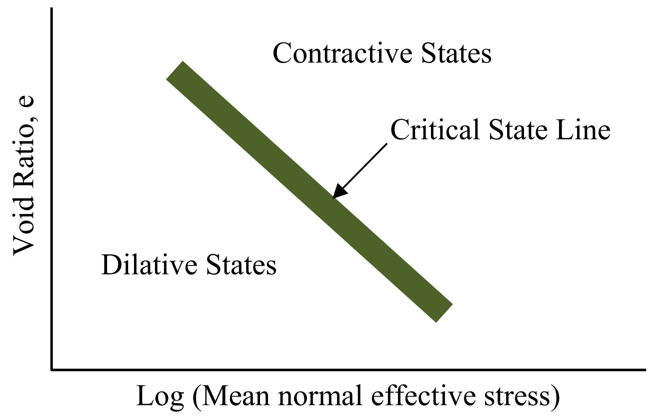


Fig. 2.3 Critical state line showing the boundary between the dilatant and contractive states for soil.

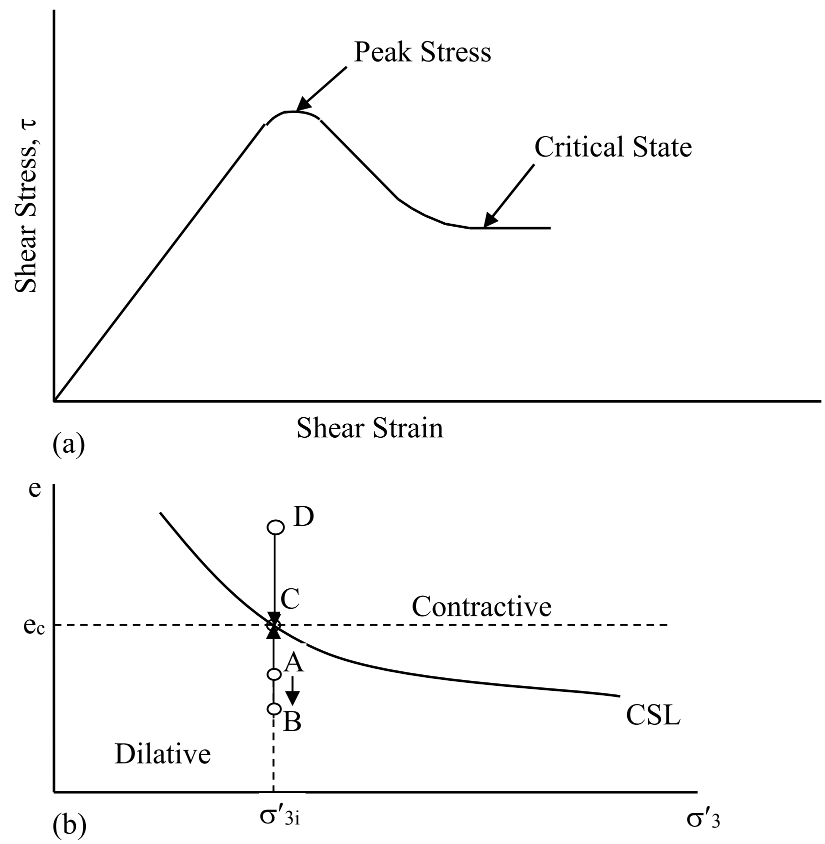


Fig. 2.4 (a) Typical shear stress - shear strain curve for a drained dilatant soil
(b) Volume change vs. pressure plot curve for a drained dilatant soil

Variation of volume change with confining pressure for a drained dilatant soil is shown in Fig. 2.4(b).

2.2.2 Undrained strength of soil

Undrained shear strength of soil is the peak strength at which soil is sheared in a triaxial test and the pore water pressure is not allowed to drain is called undrained strength of soil. This is a hypothetical term used to define shear strength of soil which is apparently seems to be different from drained strength but is impossible to simulate in actual field test. [Henkel, 1960; Henkel and Wade (1966); Konrad and Watts, 1995; Day and Thevanayagam, 1999; Cubrinovski and Rees, 2008; Murthy et al., 2007]. It depends upon a number of factors such as orientation of stresses, stress path, rate of shearing and quantity of material. Tresca theory is normally used to define undrained strength of soil, based on Mohr's circle as:

$\sigma_1 - \sigma_3 = 2\, c_u$ where σ_1 is the major principal stress; σ_3 is the minor principal stress; c_u is the shear strength.

It is commonly adopted in limit equilibrium analyses where the rate of loading is very much greater than the rate at which pore water pressures dissipates due to the shearing of the soil mass. During undrained condition, no elastic volumetric strains occur, and Poisson's ratio remains constant throughout shearing of sample. The undrained strength of soil is significant in the advanced analyses like finite element analysis. In finite element analysis methods, soil models other than Tresca may be used to model the undrained condition. Mohr-Coulomb and critical state soil models such as the modified Cam-clay model are normally used in such conditions.

2.2.3 Drained shear strength

The drained shear strength is defined as the shear strength of the soil when pore water pressures, generated during the course of shearing dissipate during shearing. It is also the case of a dry soil when there is no pore water present in the soil. It is estimated using the Mohr-Coulomb equation. Terzaghi (1942) provided a solution for estimation of shear strength of soil using principle of effective stress.

The shear strength is often approximated in terms of effective stresses by Eq. (2.4) as

$\tau = \sigma' \tan(\phi') + c'$

where $\sigma' = (\sigma - u)$, is defined as the effective stress. σ is the total stress applied normal to the plane of shearing, and u is the pore water pressure acting on the same plane. ϕ' = the effective stress friction angle, or the angle of internal friction after Coulomb friction. The curvature (nonlinearity) of the failure envelope in Fig. 2.1 occurs because the dilatancy of closely packed soil particles depends on confining pressure. Normally a loose sand contracts and dense sand expands as it approaches the critical sate, defined as the state at which the sand is sheared without changes in either shear strength or volume. However, whether a sample of sand is contractive or dilatant depends not only on density but also effective confining stress. In a critical state model, when a sample is sheared under high effective confining stress, the shear stress increases monotonically until it reaches a peak, after which the sample continues to undergo shear straining, without any change in shear stress or sample volume. The soil is then said to have reached the critical state and the corresponding friction angle is known as the critical state friction angle ϕ_c. At this stage the rate of dilation attains maximum value. With further increase in loading the shear stress drops until it reaches the residual state. For all practical purposes the critical state friction angle obtained from triaxial tests is a unique value for a given granular soil, regardless of the initial relative density and initial confining stress.

2.2.4 Critical state theory

A more advanced understanding of the behaviour of soil undergoing shearing lead to the development of the critical state theory of soil mechanics (Roscoe, Schofield and Wroth 1958). In critical state soil mechanics, distinct shear strength is identified where the soil continue to shearing at a constant volume, also called the 'critical state'. Thus there are three commonly identified shear strengths for a soil undergoing shear as shown in Fig. 2.2

1. Peak strength
2. Critical state or constant volume strength
3. Residual strength

The peak strength is followed by critical state, depending on the initial state of the soil particles being sheared. A loose soil will contract in volume on shearing, and will not develop any peak strength above critical state. In this case peak strength is same as critical state shear strength, once the soil has ceased contracting in volume. A dense soil may contract slightly before granular interlock prevents further contraction (granular interlock is dependent on the shape of the grains and their initial packing arrangement). In order to continue shearing once granular interlock has occurred, the soil must dilate (expand in volume). As additional shear force is required to dilate the soil, a 'peak' strength occurs. Once this peak strength caused by dilation has been overcome through continued shearing, the resistance provided by the soil to the applied shear stress reduces (termed "strain softening"). Strain softening will continue until no further changes in volume of the soil occur on continued shearing. Peak strengths are also observed in over consolidated clays where the natural fabric of the soil must be destroyed prior to reaching constant volume shearing. Other factors that result in peak strengths include cementation and bonding of particles. The constant volume (or critical state) shear strength is said to be intrinsic to the soil, and independent of the initial density or packing arrangement of the soil grains. The residual strength occurs for some soils where the shape of the particles that make up the soil become aligned during shearing (forming a slickenside), resulting in reduced resistance to continued shearing (further strain softening) as also observed in ring shear tests [Sadrekarimi and Olson, 2013]. This occurs for most clay that comprises plate-like minerals, but is also observed in some granular soils with more elongate shaped grains. Clays that do not have plate-like minerals do not exhibit residual strengths.

2.2.5 Steady state

An improvement of the critical state concept is the steady state concept. The steady state strength is defined as the shear strength of the soil when it is at the steady state condition. The steady state condition is defined as "that state in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant velocity" [Poulos, 1981]. Steady state based on Poulos principal in soil mechanics is called "Harvard soil mechanics". It is not the same as the "critical state" condition. A soil sample is said to have reached the steady state after breakage of particle contacts. All the particles are oriented in a statistically steady state condition and so that the shear stress needed to continue deformation at a constant velocity of deformation does not change [Riemer et al., 1990; Thevanayagam et al., 1996]. This condition applies to both the drained and the undrained case. The steady state has a slightly different value depending on the strain rate at which it is measured. There is a minor difference between the two states. At the steady state condition the grains position themselves in the steady state structure, whereas no such structure occurs for the critical state. In the case of shearing to large strains for soils with elongated particles, this steady state structure is one where the grains are oriented in the direction of shear. The condition where the soil particles are strongly aligned in the direction of shear, the steady state corresponds to the residual condition.

The extra angle of shearing of ‘dense’ soil is correlated to its rate of dilation and hence to its relative density and mean effective stress, combined in a new relative dilatancy index. The effect of grading and anisotropy on sand was examined by Dunstan et al (1977), who used a mixing apparatus to form sand samples of different grading and then sheared the sand samples in a direct shear box in different directions [Diego et al., 1992; Houlsby, 1991]. It was found that the strength of various grading of sands at a similar relative density was fairly constant. Interactions of particles of differing sizes have little effect on the difference in strength caused by the anisotropic packing. It seems likely an anisotropic strength component will exist whatever the grading.

2.2.6 Effect of fines (plastic and non-plastic)

Bolton (1986) investigated the angle of shearing resistance of sand. Extensive data of the strength and dilatancy of 17 sands in axisymmetric or plane strain at different densities and confining pressures were collated. It was stated the critical state angle of shearing resistance of soil which is shearing at constant volume is principally a function of mineralogy and can readily be determined experimentally. Effects of fines play a very important role in the prediction of shear strength of soil [Thevanayagam, 1998, 1999; Thevanayagam et al., 2000, 2002; Naeini and Baziar, 2004;].

Been & Jefferies (1985) proposed the state parameter Ψ as a semi empirical normalizing parameter for sand behaviour. This concept requires knowledge of the critical/steady state line, which provides a reference state from which the state parameter and the most important sand behaviours are derived

Jardine et al. (1986) studied the influence of non-linear stress-strain characteristics in soil-structure interaction. Non-linear behaviour of soil observed even at very low strain results in the concentration of strain and deformation towards the loading boundaries [Kokusho, 2004]. This has important consequences for soil-structure interaction problems such as settlement profiles, pile group interaction and contact stress distributions. Small strain nonlinearity also has a significant influence on the interpretation in terms of equivalent elastic moduli of in situ deformation tests and of field measurements. It is concluded that soil-structure interaction computations and the interpretation of field measurements can be misleading unless the non-linear nature of soils is taken into account.

Vaid et al. (1990) have investigated the effect of stress path on the steady state lines of liquefiable sand. Results from undrained triaxial compression and extension tests on water-deposited sands show that steady state line of a given sand, though unique in the effective stress space, is not so in the void ratio-effective stress space. The sand is contractive over a much larger range of void ratios in extension than in compression. While a single steady state line emerges for compression loading, extension loading yields several lines, each characteristic to a given deposition void ratio. All these extension lines lie to the left of the compression line in void ratio-effective stress space. Thus at a given void ratio, steady state strength is smaller in extension than in compression, the difference increasing as the sand becomes looser.

Åberg (1992) presented a theory for calculation of the void ratio of non-cohesive soils and similar materials. The most important variable is the grain-size distribution of the soil, but the grain shape and the degree of densification were also considered. It was stated that the void sizes in a graded soil are mainly determined by the fine grains, which successfully fill the space between the large grains. Based upon a simple stochastic model of the void structure and void sizes, theoretical equations are derived by means of which the void ratio of a soil can be calculated from its grain-size distribution. The calculations also give information about type of grain structure, and the grain size that separates fixed grains and possible loose grains is determined. The equations also consider grain shape, degree of densification, and size of compaction container. A result of numerous laboratory compaction tests on uniform to broadly graded sand, gravel, and crushed-rock materials confirm the general forms of the derived equations and form the basis for evaluation of certain parameters.

Maaza, et al. (2012) conducted a series of longitudinal resonance tests on dry sandy soils with different grain size distributions and different densities to identify the instability zone. The test results confirm the existence of a non-linearity zone represented by a "jump" just after the resonance for dense sand. This study also shows that the grains interact with the contact forces and with a slight increase in density induces more collisions and friction.

Atkinson (2000) found that the mechanical behaviour of sands in the small strains range is non-linear and depends upon the evolution of modulus of elasticity.

Lo Presti (2000) analysed a large number of drained triaxial compression tests conducted on Toyora, Quiou, and Ticino sands many of which were carried out with local strain measurement. The tests were performed on both isotropically and anisotropically consolidated specimens prepared by pluvial deposition in air. Using empirical fitting equations, the influences of different factors such as axial strain, vertical consolidation stress, consolidation stress ratio, and stress history on the secant Young's modulus under triaxial loading conditions at small and intermediate axial strains are singled out and presented. It is concluded that in the strain range of 0.01 to 0.1%, the soil stiffness under triaxial loading conditions exhibits a highly pronounced nonlinearity and is strongly influenced by the test conditions, in particular, the consolidation stress ratio and over consolidation ratio.

Salgado et al. (2000) studied the effects of non-plastic fines on the small-strain stiffness and shear strength of sands. Samples of Ottawa sand with varied proportion of fines were used in a series of triaxial test. The samples were prepared at different relative densities and were tested at varied confining pressure. The stress-strain responses were recorded and the shear strength and dilatancy parameters were obtained for each fines percentage. Bender element tests were also performed to assess of the effect of fines content on small-strain stiffness. Addition of non-plastic silt to clean sand considerably increases both the peak friction angle at a given initial relative density and the critical-state friction angle.

Evesq (2002) characterized the non-linear response of granular soil deposits and observed that their overall behavior is uncertain and complex.

Alejano & Alonso (2005) studies the importance of dilatancy in classical rock mechanics in post-failure problems such as tunnel or mine pillar design. They provided a detailed analysis of published test data with and proposed a very significant and conveniently simple formulation of the dilatancy angle that reflects its dependencies on plasticity experienced by the material and confining stress and that can be readily implemented in numerical codes. The model is then tested, demonstrating that it is capable of representing rock sample strain behaviour in compressive tests. Finally, the model is applied to the resolution of ground reaction curves for tunnels in poor-to-average-quality rock masses, showing a good correlation with results obtained using practical rock engineering techniques.

Yu et al. (2005) presented an experimental evaluation of the performances of a simple, unified critical state model CASM and its extension CASM-d for predicting the stress-strain behaviour of Portaway sand for a wide range of densities and confining pressure. It was concluded that the critical state line for Portaway sand established using the results of a series of triaxial tests is independent of drainage conditions, level of consolidation pressure and specimen preparation methods. The critical state model CASM has been modified so that hardening is controlled not only by volumetric plastic strain rates but also by deviatric plastic strain.

Yamamuro et al. (2009) investigated silty sand using scanning electron microscope (SEM) to find the effect of depositional method and silt content on the grain contact structure of silty sand. The controlled triaxial specimens containing Nevada sand with various quantities of non-plastic silt were isotropically consolidated to 25 kPa and then preserved through epoxy impregnation. Specimens were formed using two methods namely dry funnel deposition and water sedimentation. This procedure was developed to allow the microstructure to be quantified in terms of potentially stable and unstable grain contacts. The increasing silt content reduces the percentage of stable grain contacts. A more compressible particle structure contained higher percentage of unstable grain contacts formed by dry funnel deposition method than those reconstituted by water sedimentation. This effect became more pronounced as silt content increases.

Carraro et al (2009) presented the results of a systematic laboratory investigation on the static behavior of silica sand containing varied amounts of either plastic or non-plastic fines on remoulded samples. The fabric of sands containing fines was examined using the environmental scanning electron microscope ESEM. Static, monotonic, isotropically consolidated, drained triaxial compression tests were performed to evaluate the stress-strain-volumetric response of these soils. The intrinsic parameters that characterize critical state, dilatancy, and small-strain stiffness of clean, silty, and clayey sands were determined. All aspects of the mechanical behavior investigated in this study e.g., stress-strain-volumetric response, shear strength, and small-strain stiffness are affected by both the amount and plasticity of the fines present in the sand.

Schanz et al (1996) in their technical note for Hostun sand concluded that by using concepts of superposition it is possible to relate the angle of dilatancy to triaxial strain conditions. This yields an extended definition for the angle of dilatancy which applies to triaxial testing conditions as well as plane strain conditions. The extended theory is validated by the fact that data from plane strain and triaxial strain conditions yield the same angle of dilatancy at least near and beyond peak. In contrast to the angle of dilatancy, friction angles differ considerably when triaxial strain and plane strains are compared. This difference basically depends on the critical state friction angle, as by Bolton (1986) and other researchers.

Trivedi et al. (2009) established that the behavior of silty sand is affected by the content of non-plastic fine particles. The effect of non-plastic fines on the values of minimum and maximum void ratios, angle of internal friction and bearing capacity have been studied in detail. It is shown that the fines content plays an important role in determining the minimum and maximum void ratios, angle of internal friction and bearing capacity. From the results of the laboratory tests it has been established that the maximum and minimum void ratios of clean sand decreases with increase in fine content from 0 to 20% and increases if fines content exceeds 20%. It is also indicated that angle of internal friction and bearing capacity decreases on the addition of fines due to compressibility of fines.

Chakraborty et al. (2010) have analysed sample of Toyora sand in both plain strain and triaxial condition at low confining pressures. A relationship between peak friction angle, critical state friction angle and dilatancy angle has been evaluated. It is observed that sand dilates with shearing at a rate that increases with increasing relative density and decreases with increasing effective confining stress. The peak friction angle of sand depends on its critical-state friction angle and on dilatancy. Fitting parameters (Q and R) was evaluated and it was concluded that the rate of dilation decreases as the confining pressure increases.

2.3 Relative density and relative compaction

The relative density is used to define the state condition of silty sand when silt content is approximately less than 15 percent. It is based on the prediction of maximum void ratio; minimum void ratio and natural void ratio. Alternatively it can also be expressed in terms of maximum unit weight, minimum unit weight and natural unit weight of soil. The correct prediction of relative density is not possible since it is difficult to obtain maximum and minimum unit weight values within a definite accurate range. The definition is for the maximum and minimum values but average values are usually used for calculating relative density (D_r). This value range together with the uncertainty in obtaining the in situ value can give a potential error in computing relative density. Due to the above reason the authors has used the term relative compaction (R_c) in this thesis instead of relative density which is defined as the ratio of natural unit weight to maximum unit weight ($R_c = \gamma_d / \gamma_{max}$) which takes care of the uncertainty in computation of unit weight of soil since values of γ_d and γ_{max} can be accurately ascertained. Also an empirical relation between R_c and D_r has been established which is validated theoretically by using Bolton's equation.

3.0 General

Soil samples are taken from nearby area of Delhi and NCR during soil exploration using split spoon sampler as per IS 9640: 1980. Sieve analysis has been performed on the collected soil sample and the material passing through 4.75 mm sieve and retained on 75 micron was collected as per IS-2720 (Part IV). The sample is washed with water to remove any amount of silt and or clay. The dried sample was again repeated through the same procedure mentioned above and the clean Yamuna sand is obtained. The tests were conducted on this clean Yamuna sand thus obtained after washing and was designated as clean sand (CS). The material which passes through 75 μ was collected in a container and allowed to settle. Then the passing material is dried in the oven and pulverized. The pulverized material was again sieved through 75 μ sieve. Then a hydrometer analysis was carried out on the material passing through 75 micron sieve to segregate and remove amount of clay particles present if any as per IS-2720 (Part IV), 1985. The test was repeated till the amount of clay particles was found insignificant. The grain size analysis of clean Yamuna sand, silt and sand with varied proportions of fines was performed in conformity with IS-2720 (Part IV), 1985].

Yamuna sand is defined as SP according to the Unified Soil Classification System. The coefficient of uniformity Cu is 1.852, and the mean grain size D_{50} is 0.225 mm. The maximum and minimum void ratios e_{max} and e_{min} are 0.78 and 0.5, respectively. The specific gravity test was conducted on clean sand, silt and silty sand with varied proportion of fines in accordance with IS-2720[(part 3) sec 2-1980]. The specific gravity of the sand particles is 2.67. Yamuna sand particles are rounded to sub rounded in shape. The fines are silt content which passes through 75 micron sieve size. Its specific gravity is 2.63. The effective size (D_{10}), the mean grain size (D_{50}), coefficient of uniformity (Cu), and coefficient of curvature (Cc) are calculated and presented in the tabular form in the next chapter. The sample of sand and silt thus obtained is used for preparation of sample for triaxial testing.

3.1 Preparation of soil sample

Samples were prepared by first estimating the weights of sand and silt needed for fines content. These amounts of silt and sand were then mixed in a cylindrical Plexiglas tube completely filled with desired water. The silt and sand are thoroughly mixed by vigorous shaking of the Plexiglas tube for approximately 20 min to achieve sample uniformity. The rubber cap is then removed, a very small amount of desired water is added to raise the water level back to the top of the tube, and the tube is topped with the pieces of high-density polyethylene film. The contents of the tube are then released into the membrane by raising the tube. Densification of the sample is accomplished by carefully and symmetrically tapping the sides of the sample mold immediately after slurry deposition. Because the mass of sand and silt used in sample preparation can be accurately estimated, it is possible to obtain a relative density that is reasonably close to a target value by measuring the height of the sample as it gets compacted. The samples had heights of the order of 76 mm and diameters of the order of 38 mm.

3.2 Estimation of relative density of soil sample

Relative density is commonly used for evaluating the state of compactness of a given soil mass. The engineering properties, such as shear strength, compressibility, and permeability, of a given soil depend on the level of compaction. In the present work minimum density was obtained by pouring sand into a standard mould with a volume of 3000cm^3 using a thin sheet of paper from a height of approx. 25 mm above the mould top as loosely as possible. Spiralling motion has been kept just sufficient to minimize particle segregation. Maximum density was obtained by densifying dry sand in a standard mould of 3000cm^3 using a compaction rod. Empty weight of the mould was taken and the maximum and minimum density was calculated. The natural unit weight of soil was evaluated from undisturbed sample collected during soil exploration. The relative density of each sample was determined using the equation given below.

$$\mathrm{Dr} = \frac{\gamma_{max}}{\gamma_d}\left[\frac{\gamma_d - \gamma_{min}}{\gamma_{max} - \gamma_{min}}\right] \tag{3.1}$$

3.3 Experimental program

A series of tri-axial tests were performed to assess how the shear strength of sand changes when an increasing percent of silt is present in Yamuna sand. All tri-axial tests for this study were performed at axial strain rates that were slow enough to allow full dissipation of pore-water pressures during loading. The tests were discontinued at different percentages of axial strain when the failure occurs. Tests were performed to assess the effect of silt on angle of internal friction, minimum and the maximum void ratios of clean sand. Static drained triaxial compression tests were conducted on isotropically consolidated sand samples with 0, 5, 10, 15, 20 and 25% fines. The height of the samples has been kept as 76 mm and diameter is of the order of 38 mm. The confining pressures ranging from 100 kPa to 400 kPa were applied to the samples and the rate of strain was kept slow enough (1.25 per minute) to ensure uniformity of results. The volume change of the sample was measured using sensitive differential pressure techniques. The stress-strain data was recorded. The details of the sample preparation and testing procedures followed are as recommended by IS 2720 Part-1 (1983). The schedule of tests are given in Table 3.1(a) and Table 3.1(b). The test set up is shown in Fig.3.1 (a) and Fig. 3.1(b).

Fig. 3.1 (a) Photograph showing tri-axial test apparatus used in the present work

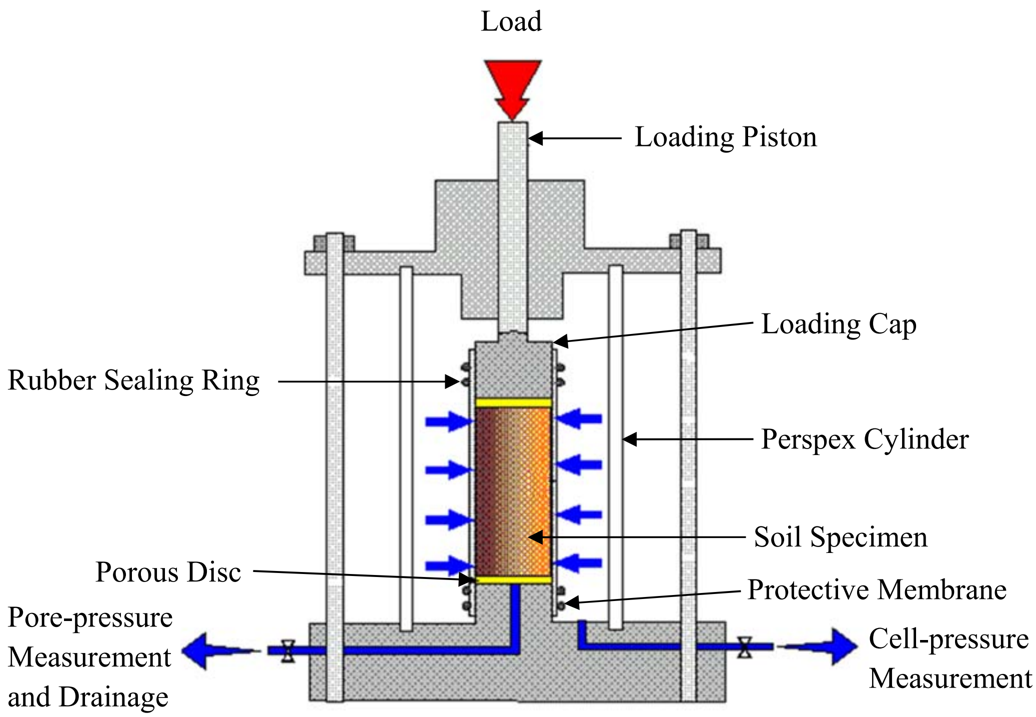


Fig. 3.1 (b) Schematic diagram of triaxial test

Table 3.1 (a) Test schedule for index properties

Sample Type	Total Test/ Accepted Result	Grain Size Analysis	Atterberg's Limits
Clean sand	5/5	Mechanical Sieve Analysis	-
Silty sand	12/10	Hydrometer Analysis	Liquid Limit/ Plastic Limit

Table 3.1 (b) Work schedules for triaxial tests and observed P'_P

Mean Diameter	Relative Compaction	Total Test	Accepted Result	Silt (%)	σ_3 (kPa)	P'_p (kPa)
0.225	0.92	2	1	0	100	147
0.225	0.92	3	1	0	200	293
0.225	0.92	2	1	0	400	563
0.225	0.93	3	1	0	100	159
0.225	0.93	1	1	0	200	306
0.225	0.93	1	1	0	400	610
0.225	0.94	1	1	0	100	153
0.225	0.94	1	1	0	200	302
0.225	0.94	2	1	0	400	586
0.225	0.95	3	1	0	100	177
0.225	0.95	2	1	0	200	307
0.225	0.95	2	1	0	400	590
0.225	0.96	2	1	0	100	179
0.225	0.96	3	1	0	200	310
0.225	0.96	3	1	0	400	595
0.224	0.92	3	1	5	100	155
0.224	0.92	3	1	5	200	294
0.224	0.92	2	1	5	400	542
0.224	0.93	2	1	5	100	147
0.224	0.93	1	1	5	200	295
0.224	0.93	1	1	5	400	535
0.224	0.94	1	1	5	100	143
0.224	0.94	2	1	5	200	306
0.224	0.94	2	1	5	400	580
0.224	0.95	3	1	5	100	185
0.224	0.95	3	1	5	200	310
0.224	0.95	3	1	5	400	584
0.224	0.96	3	1	5	100	189
0.224	0.96	2	1	5	200	312
0.224	0.96	2	1	5	400	592
0.222	0.92	1	1	10	100	184
0.222	0.92	1	1	10	200	303
0.222	0.92	1	1	10	400	622
0.222	0.93	2	1	10	100	175
0.222	0.93	2	1	10	200	318
0.222	0.93	3	1	10	400	563
0.222	0.94	3	1	10	100	192
0.222	0.94	3	1	10	200	329
0.222	0.94	2	1	10	400	560

Mean Diameter	Relative Compaction	Total Test	Accepted Result	Silt (%)	σ_3 (kPa)	P'_p (kPa)
0.222	0.95	2	1	10	100	196
0.222	0.95	1	1	10	200	332
0.222	0.95	1	1	10	400	565
0.222	0.96	1	1	10	100	199
0.222	0.96	2	1	10	200	338
0.222	0.96	2	1	10	400	571
0.215	0.92	3	1	15	100	135
0.215	0.92	3	1	15	200	294
0.215	0.92	2	1	15	400	574
0.215	0.93	1	1	15	100	129
0.215	0.93	1	1	15	200	284
0.215	0.93	1	1	15	400	586
0.215	0.94	2	1	15	100	134
0.215	0.94	2	1	15	200	303
0.215	0.94	3	1	15	400	600
0.215	0.95	3	1	15	100	167
0.215	0.95	3	1	15	200	305
0.215	0.95	3	1	15	400	604
0.215	0.96	2	1	15	100	172
0.215	0.96	2	1	15	200	309
0.215	0.96	2	1	15	400	609
0.213	0.92	1	1	20	100	135
0.213	0.92	1	1	20	200	271
0.213	0.92	1	1	20	400	501
0.213	0.93	1	1	20	100	146
0.213	0.93	2	1	20	200	281
0.213	0.93	3	1	20	400	513
0.213	0.94	3	1	20	100	148
0.213	0.94	3	1	20	200	285
0.213	0.94	3	1	20	400	514
0.213	0.95	2	1	20	100	155
0.213	0.95	2	1	20	200	286
0.213	0.95	1	1	20	400	549
0.213	0.96	1	1	20	100	161
0.213	0.96	1	1	20	200	298
0.213	0.96	1	1	20	400	559
0.208	0.92	1	1	25	100	147
0.208	0.92	2	1	25	200	280
0.208	0.92	2	1	25	400	511
0.208	0.93	2	1	25	100	155

Mean Diameter	Relative Compaction	Total Test	Accepted Result	Silt (%)	σ_3 (kPa)	P'_p (kPa)
0.208	0.93	3	1	25	200	291
0.208	0.93	3	1	25	400	543
0.208	0.94	3	1	25	100	161
0.208	0.94	3	1	25	200	296
0.208	0.94	3	1	25	400	550
0.208	0.95	2	1	25	100	168
0.208	0.95	2	1	25	200	303
0.208	0.95	3	1	25	400	558
0.208	0.96	3	1	25	100	171
0.208	0.96	3	1	25	200	311
0.208	0.96	3	1	25	400	524
Total	-	188	90	-	-	-

Table 3.1 (c) Index properties of sand and silt

Sample	% Fines	e_{max}	e_{min}	ϕ_c	Total Test/ Accepted Test
Silt	-	-	-	-	-
Yamuna sand	0	0.5	0.78	27	5/5
Yamuna sand	5	0.46	0.76	25.1	2/2
Yamuna sand	10	0.42	0.72	29.8	3/2
Yamuna sand	15	0.38	0.68	24.5	2/2
Yamuna sand	20	0.33	0.63	25.4	2/2
Yamuna sand	25	0.31	0.62	30.7	3/2

Table 3.1 (d) Schedule of SPT test and 'N' observed

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Bore hole: BH-1, Water table: 4.50 metre				
Latitude: 28°25'53.80"N Longitude: 77°30'20.87"E	1.95	1.95	8	Clayey Silt with Sand
	3.45	1.50	13	
	4.95	1.50	15	Silty Sand
	6.45	1.50	13	
	7.95	1.50	17	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	24	
	19.95	1.50	25	
	21.45	1.50	25	
	22.95	1.50	28	
	24.45	1.50	31	
	25.95	1.50	32	
	27.45	1.50	37	
	28.95	1.50	40	
	30.00	1.05	47	
Bore hole: BH-2, Water table: 4.60 metre				
Latitude: 28°25'53.93"N Longitude: 77°30'21.47"E	1.95	1.95	11	Clayey Silt
	3.45	1.50	15	Silty Sand
	4.95	1.50	16	
	6.45	1.50	13	
	7.95	1.50	18	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	24	
	19.95	1.50	26	
	21.45	1.50	24	
	22.95	1.50	29	
	24.45	1.50	32	
	25.95	1.50	37	
	27.45	1.50	41	

Location	Depth (m)	Interval (m)	‘N’ Observed	Soil Classification
	28.95	1.50	45	Fine Sand
	30.00	1.05	49	
Bore hole: BH-3, Water table: 4.50 metre				
Latitude: 28°25'53.77"N Longitude: 77°30'22.02"E	1.95	1.95	11	Clayey Silt
	3.45	1.50	17	Silty Sand
	4.95	1.50	15	
	6.45	1.50	11	
	7.95	1.50	14	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	28	
	18.45	1.50	30	
	19.95	1.50	26	
	21.45	1.50	31	
	22.95	1.50	34	
	24.45	1.50	36	
	25.95	1.50	38	
	27.45	1.50	42	
	28.95	1.50	43	
	30.00	1.05	46	
Bore hole: BH-4, Water table: 4.45 metre				
Latitude: 28°25'53.42"N Longitude: 77°30'22.16"E	1.95	1.95	13	Clayey Silt
	3.45	1.50	14	Silty Sand
	4.95	1.50	16	
	6.45	1.50	13	
	7.95	1.50	18	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	28	
	19.95	1.50	27	
	21.45	1.50	26	
	22.95	1.50	31	
	24.45	1.50	33	
	25.95	1.50	36	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	27.45	1.50	40	Fine Sand
	28.95	1.50	41	
	30.00	1.05	44	
Bore hole: BH-5, Water table: 4.60 metre				
Latitude: 28°25'53.40"N Longitude: 77°30'20.84"E	1.95	1.95	9	Clayey Silt
	3.45	1.50	14	Silty Sand
	4.95	1.50	17	
	6.45	1.50	14	
	7.95	1.50	19	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	26	
	16.95	1.50	29	
	18.45	1.50	31	
	19.95	1.50	34	
	21.45	1.50	38	
	22.95	1.50	39	
	24.45	1.50	41	
	25.95	1.50	42	
	27.45	1.50	45	
	28.95	1.50	47	
	30.00	1.05	52	
Bore hole: BH-6, Water table: 4.70 metre				
Latitude: 28°25'53.10"N Longitude: 77°30'20.99"E	1.95	1.95	8	Clayey Silt
	3.45	1.50	13	Silty Sand
	4.95	1.50	15	
	6.45	1.50	13	
	7.95	1.50	16	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	25	
	19.95	1.50	24	
	21.45	1.50	25	
	22.95	1.50	28	
	24.45	1.50	31	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	25.95	1.50	32	Fine Sand
	27.45	1.50	37	
	28.95	1.50	40	
	30.00	1.05	47	
Bore hole: BH-7, Water table: 4.60 metre				
Latitude: 28°25'52.69"N Longitude: 77°30'21.25"E	1.95	1.95	11	Clayey Silt
	3.45	1.50	11	Silty Sand
	4.95	1.50	16	
	6.45	1.50	16	
	7.95	1.50	14	
	9.45	1.50	15	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	25	
	19.95	1.50	28	
	21.45	1.50	31	
	22.95	1.50	31	
	24.45	1.50	35	
	25.95	1.50	39	
	27.45	1.50	42	
	28.95	1.50	45	
	30.00	1.05	46	
Bore hole: BH-8, Water table: 4.50 metre				
Latitude: 28°25'52.23"N Longitude:77°30'21.49'	1.95	1.95	10	Clayey Silt
	3.45	1.50	12	Silty Sand
	4.95	1.50	14	
	6.45	1.50	19	
	7.95	1.50	20	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	27	
	16.95	1.50	29	
	18.45	1.50	32	
	19.95	1.50	33	
	21.45	1.50	37	
	22.95	1.50	40	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	24.45	1.50	43	Fine Sand
	25.95	1.50	46	
	27.45	1.50	48	
	28.95	1.50	51	
	30.00	1.05	52	
Bore hole: BH-9, Water table: 4.50 metre				
Latitude: 28°25'52.39"N Longitude: 77°30'22.03'E	1.95	1.95	9	Clayey Silt
	3.45	1.50	12	Silty Sand
	4.95	1.50	13	
	6.45	1.50	18	
	7.95	1.50	16	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	27	
	19.95	1.50	30	
	21.45	1.50	33	
	22.95	1.50	37	
	24.45	1.50	39	
	25.95	1.50	47	
	27.45	1.50	48	
	28.95	1.50	52	
	30.00	1.05	54	
Bore hole: BH-10, Water table: 4.55 metre				
Latitude: 28°25'52.58"N Longitude: 77°30'22.73'E	1.95	1.95	10	Clayey Silt
	3.45	1.50	15	Silty Sand
	4.95	1.50	16	
	6.45	1.50	15	
	7.95	1.50	16	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	27	
	19.95	1.50	30	
	21.45	1.50	30	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	22.95	1.50	33	Fine Sand
	24.45	1.50	36	
	25.95	1.50	40	
	27.45	1.50	44	
	28.95	1.50	49	
	30.00	1.05	52	
Bore hole: BH-11, Water table: 4.60 metre				
Latitude: 28°25'54.38"N Longitude: 77°30'22.68'E	1.95	1.95	16	Clayey Silt
	3.45	1.50	18	Silty Sand
	4.95	1.50	10	
	6.45	1.50	14	
	7.95	1.50	18	
	9.45	1.50	21	Fine Sand
	10.95	1.50	24	
	12.45	1.50	23	
	13.95	1.50	25	
	15.45	1.50	24	
	16.95	1.50	24	
	18.45	1.50	26	
	19.95	1.50	24	
	21.45	1.50	29	
	22.95	1.50	32	
	24.45	1.50	33	
	25.95	1.50	36	
	27.45	1.50	40	
	28.95	1.50	42	
	30.00	1.05	46	
Bore hole: BH-12, Water table: 4.70 metre				
Latitude: 28°25'54.56" Longitude: 77°30'23.40'E	1.95	1.95	14	Clayey Silt
	3.45	1.50	16	Silty Sand
	4.95	1.50	12	
	6.45	1.50	13	
	7.95	1.50	15	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	24	
	18.45	1.50	24	
	19.95	1.50	25	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	21.45	1.50	30	Fine Sand
	22.95	1.50	30	
	24.45	1.50	32	
	25.95	1.50	36	
	27.45	1.50	40	
	28.95	1.50	42	
	30.00	1.05	48	
Bore hole: BH-13, Water table: 4.70 metre				
Latitude: 28°25'54.00"N Longitude: 77°30'22.77"E	1.95	1.95	14	Clayey Silt
	3.45	1.50	17	Silty Sand
	4.95	1.50	10	
	6.45	1.50	13	
	7.95	1.50	15	
	9.45	1.50	14	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	24	
	18.45	1.50	25	
	19.95	1.50	26	
	21.45	1.50	28	
	22.95	1.50	31	
	24.45	1.50	37	
	25.95	1.50	33	
	27.45	1.50	40	
	28.95	1.50	44	
	30.00	1.05	51	
Bore hole: BH-14, Water table: 4.50 metre				
Latitude: 28°25'54.22" Longitude: 77°30'23.57"	1.95	1.95	12	Clayey Silt
	3.45	1.50	16	Silty Sand
	4.95	1.50	10	
	6.45	1.50	13	
	7.95	1.50	13	
	9.45	1.50	14	Fine Sand
	10.95	1.50	12	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	29	
	18.45	1.50	24	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
	19.95	1.50	27	Fine Sand
	21.45	1.50	30	
	22.95	1.50	32	
	24.45	1.50	36	
	25.95	1.50	40	
	27.45	1.50	42	
	28.95	1.50	42	
	30.00	1.05	47	
Bore hole: BH-15, Water table: 4.50 metre				
Latitude: 28°25'53.64"N Longitude: 77°30'22.93"E	1.95	1.95	15	Clayey Silt
	3.45	1.50	18	Silty Sand
	4.95	1.50	10	
	6.45	1.50	13	
	7.95	1.50	14	
	9.45	1.50	12	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	25	
	15.45	1.50	29	
	16.95	1.50	32	
	18.45	1.50	26	
	19.95	1.50	28	
	21.45	1.50	32	
	22.95	1.50	36	
	24.45	1.50	35	
	25.95	1.50	40	
	27.45	1.50	41	
	28.95	1.50	45	
	30.00	1.05	49	
Bore hole: BH-16, Water table: 4.50 metre				
Latitude: 28°25'54.07"N Longitude: 77°30'24.26"E	1.95	1.95	14	Clayey Silt
	3.45	1.50	17	Silty Sand
	4.95	1.50	10	
	6.45	1.50	12	
	7.95	1.50	11	
	9.45	1.50	14	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	22	
	16.95	1.50	24	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Latitude: 28°25'54.07"N Longitude: 77°30'24.26"E	18.45	1.50	26	Fine Sand
	19.95	1.50	29	
	21.45	1.50	33	
	22.95	1.50	36	
	24.45	1.50	40	
	25.95	1.50	44	
	27.45	1.50	46	
	28.95	1.50	47	
	30.00	1.05	47	
Bore hole: BH-17, Water table: 4.40 metre				
Latitude: 28°25'53.76"N Longitude: 77°30'23.80"E	1.95	1.95	17	Clayey Silt
	3.45	1.50	16	Silty Sand
	4.95	1.50	10	
	6.45	1.50	10	
	7.95	1.50	12	
	9.45	1.50	11	Fine Sand
	10.95	1.50	12	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	25	
	18.45	1.50	24	
	19.95	1.50	26	
	21.45	1.50	29	
	22.95	1.50	32	
	24.45	1.50	31	
	25.95	1.50	32	
	27.45	1.50	34	
	28.95	1.50	37	
	30.00	1.05	42	
Bore hole: BH-18, Water table: 4.50 metre				
Latitude: 28°25'53.28"N Longitude: 77°30'23.15"E	1.95	1.95	15	Clayey Silt
	3.45	1.50	16	Silty Sand
	4.95	1.50	13	
	6.45	1.50	12	
	7.95	1.50	14	
	9.45	1.50	14	Fine Sand
	10.95	1.50	17	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Latitude: 28°25'53.28"N Longitude: 77°30'23.15"E	16.95	1.50	25	Fine Sand
	18.45	1.50	23	
	19.95	1.50	27	
	21.45	1.50	32	
	22.95	1.50	36	
	24.45	1.50	39	
	25.95	1.50	40	
	27.45	1.50	37	
	28.95	1.50	42	
	30.00	1.05	49	
Bore hole: BH-19, Water table: 4.50 metre				
Latitude: 28°25'53.73"N Longitude: 77°30'24.45"E	1.95	1.95	14	Clayey Silt
	3.45	1.50	16	Silty Sand
	4.95	1.50	12	
	6.45	1.50	13	
	7.95	1.50	23	
	9.45	1.50	21	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	
	15.45	1.50	24	
	16.95	1.50	28	
	18.45	1.50	28	
	19.95	1.50	26	
	21.45	1.50	28	
	22.95	1.50	31	
	24.45	1.50	39	
	25.95	1.50	38	
	27.45	1.50	40	
	28.95	1.50	45	
	30.00	1.05	49	
Bore hole: BH-20, Water table: 4.40 metre				
Latitude: 28°25'53.04"N Longitude: 77°30'23.62"E	1.95	1.95	12	Clayey Silt
	3.45	1.50	14	Silty Sand
	4.95	1.50	16	
	6.45	1.50	17	
	7.95	1.50	22	
	9.45	1.50	24	Fine Sand
	10.95	1.50	22	
	12.45	1.50	23	
	13.95	1.50	24	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Latitude: 28°25'53.04"N Longitude: 77°30'23.62"E	15.45	1.50	25	Fine Sand
	16.95	1.50	27	
	18.45	1.50	30	
	19.95	1.50	25	
	21.45	1.50	28	
	22.95	1.50	31	
	24.45	1.50	33	
	25.95	1.50	37	
	27.45	1.50	39	
	28.95	1.50	43	
	30.00	1.05	45	
Bore hole: BH-21, Water table: 4.40 metre				
Latitude: 28°25'53.44"N Longitude: 77°30'24.22"E	1.95	1.95	11	Clayey Silt
	3.45	1.50	14	Silty Sand
	4.95	1.50	16	
	6.45	1.50	18	
	7.95	1.50	23	
	9.45	1.50	25	Fine Sand
	10.95	1.50	30	
	12.45	1.50	32	
	13.95	1.50	36	
	15.45	1.50	41	
	16.95	1.50	48	
	18.45	1.50	50	
	19.95	1.50	54	
	21.45	1.50	52	
	22.95	1.50	57	
	24.45	1.50	56	
	25.95	1.50	59	
	27.45	1.50	63	
	28.95	1.50	60	
	30.00	1.05	64	
Bore hole: BH-22, Water table: 4.50 metre				
Latitude: 28°25'53.20"N Longitude: 77°30'24.67"E	1.95	1.95	7	Clayey Silt
	3.45	1.50	13	Silty Sand
	4.95	1.50	14	
	6.45	1.50	18	
	7.95	1.50	22	
	9.45	1.50	18	Fine Sand
	10.95	1.50	20	
	12.45	1.50	22	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Latitude: 28°25'53.20"N Longitude: 77°30'24.67"E	13.95	1.50	27	Fine Sand
	15.45	1.50	25	
	16.95	1.50	29	
	18.45	1.50	33	
	19.95	1.50	37	
	21.45	1.50	42	
	22.95	1.50	44	
	24.45	1.50	52	
	25.95	1.50	61	
	27.45	1.50	58	
	28.95	1.50	61	
	30.00	1.05	63	
Bore hole: BH-23, Water table: 4.50 metre				
Latitude: 28°25'51.82"N Longitude: 77°30'21.59"E	1.95	1.95	10	Clayey Silt
	3.45	1.50	13	Silty Sand
	4.95	1.50	14	
	6.45	1.50	16	
	7.95	1.50	21	
	9.45	1.50	24	Fine Sand
	10.95	1.50	28	
	12.45	1.50	32	
	13.95	1.50	36	
	15.45	1.50	40	
	16.95	1.50	43	
	18.45	1.50	45	
	19.95	1.50	50	
	21.45	1.50	54	
	22.95	1.50	57	
	24.45	1.50	56	
	25.95	1.50	59	
	27.45	1.50	60	
	28.95	1.50	56	
	30.00	1.05	58	
Bore hole: BH-24, Water table: 4.70 metre				
Latitude: 28°25'52.19"N Longitude: 77°30'22.71"E	1.95	1.95	8	Clayey Silt
	3.45	1.50	11	Silty Sand
	4.95	1.50	12	
	6.45	1.50	14	
	7.95	1.50	21	
	9.45	1.50	18	Fine Sand
	10.95	1.50	21	

Location	Depth (m)	Interval (m)	'N' Observed	Soil Classification
Latitude: 28°25'52.19"N Longitude: 77°30'22.71"E	12.45	1.50	29	Fine Sand
	13.95	1.50	27	
	15.45	1.50	33	
	16.95	1.50	35	
	18.45	1.50	41	
	19.95	1.50	44	
	21.45	1.50	54	
	22.95	1.50	66	
	24.45	1.50	70	
	25.95	1.50	72	
	27.45	1.50	52	
	28.95	1.50	57	
	30.00	1.05	64	
Bore hole: BH-25, Water table: 4.70 metre				
Latitude: 28°25'52.61"N Longitude: 77°30'23.52"E	1.95	1.95	9	Clayey Silt
	3.45	1.50	11	Silty Sand
	4.95	1.50	12	
	6.45	1.50	13	
	7.95	1.50	15	
	9.45	1.50	17	Fine Sand
	10.95	1.50	20	
	12.45	1.50	24	
	13.95	1.50	29	
	15.45	1.50	32	
	16.95	1.50	37	
	18.45	1.50	43	
	19.95	1.50	52	
	21.45	1.50	55	
	22.95	1.50	47	
	24.45	1.50	36	
	25.95	1.50	40	
	27.45	1.50	45	
	28.95	1.50	56	
	30.00	1.05	57	

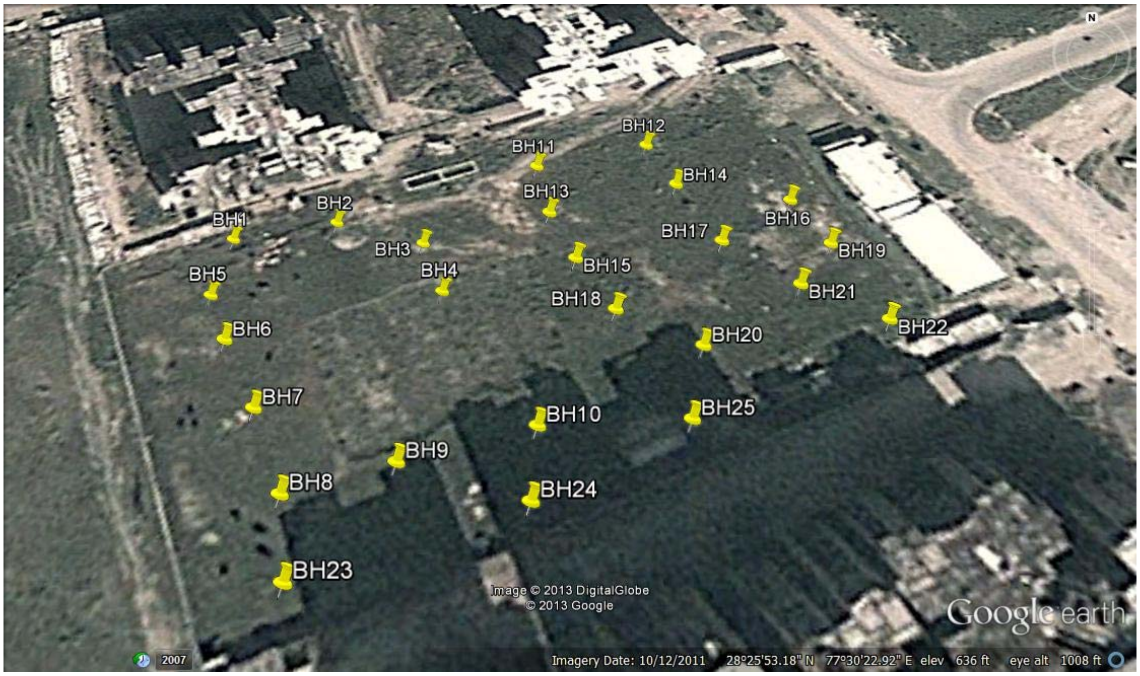


Fig. 3.1 (c) Location of bore holes for SPT

3.4 Grain size analysis

The grain size analysis has been conducted to estimate the variation in grain size of clean sand, silt and sand with varied proportion of fines as per IS -2720 (part 4),1985. For separation of silt content from soil, hydrometer analysis is performed.

3.5 Specific gravity

Specific Gravity test of clean sand and sand with varied proportion of fines were conducted in accordance of IS -2720 (part 3, sec-2), 1980. De-aeration of soil sample of sand containing varied proportion of fines takes longer duration (15-20 minutes) than for clean sand to achieve uniformity of results.

3.6 Atterberg limit

This test is performed to determine the liquid limit, plastic limit and the plasticity index of the fine grained soil and is determined as per I.S.2720 (part 5)-1985.The liquid limit (LL) is defined as the limiting percentage value of water content, at which a pat of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm when subjected to 25 blows from the cup being dropped 10 mm in a standard liquid limit apparatus (Casagrande apparatus) operated at a rate of two blows per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm diameter threads without crumbling. The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of zero (non-plastic) tend to have little or no silt or clay.

3.7 Shear strength

The shear strength of clean Yamuna sand and Yamuna sand with varied proportion of fines has been evaluated using tri axial test apparatus (Fig. 3.1-3.2). Confining pressure of 100, 200, and 400 kPa are applied on the soil sample for the test. Samples are prepared at relative compaction of 0.92, 0.93, 0.94, 0.95 and 0.96. Plots between deviatric stress and axial strain are drawn for each test. Variations of volumetric strain with shear strain are plotted. Friction angle, peak friction angle and critical state friction angle for each soil sample are evaluated using the test data and dilatancy angle has been calculated using Bolton’s strength dilatancy relation.

3.8 Field test (SPT)

A set of Standard Penetration Tests (SPT) have been carried out at 25 locations as given in Table 3.1(d).The locations are marked in Fig.3.1(c). The Standard Penetration Test (SPT) is an in-situ penetration test specified to collect subsurface information on the geotechnical engineering properties of soil. The test procedure is described in the IS 2131 - 1981(1997).

4.0 General

The tendency of loose sand to contract to a smaller volume and tendency of densely packed sand to dilate with increase in volume under shearing load is called dilatancy. Shear deformations of soils are thus accompanied with change in volume. In a tri axial test the angle at which the strain increases without further increase of stress at constant volume is called critical state friction angle. Attempts have been made by previous researcher to co-relate angle of dilatancy with the peak friction angle and critical state friction angle. In this chapter basic relationship for relative density, relative compaction and stress dilatancy relation using relative compaction shall be discussed. Also a relationship between relative compaction and relative density has been derived and validated Yamuna sand with varied proportion of fines.

4.1 Estimation of relative density of soil sample

Relative density of a soil is the ratio, expressed as a percentage, of the difference between the void ratios of cohesion less soil in its loosest state and existing natural state to the difference between its void ratio in the loosest and densest states.

$$\text{Relative Density (Dr)} = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{4.1}$$

Where,

e_{max} = maximum void ratio (loosest state).

e_{min} = minimum void ratio (densest state.)

e = natural void ratio of Yamuna sand in its natural existing state in the field.

Relative density has been commonly used for evaluating the state of compactness of a given soil mass. The engineering properties, such as shear strength, compressibility, and permeability, of a given soil depend on the level of compaction. In the present work minimum and maximum void ratios were determined according to ASTM D 4253 and ASTM D 4254. Minimum density was obtained by pouring sand into a standard mould with a volume of 3000cm3 using a thin sheet of paper from a height of approx. 25 mm above the mould top as loosely as possible. Spiralling motion should be just sufficient to minimize particle segregation. Maximum density was obtained by densifying dry sand in a standard mould of 3000cm3 and using a compaction rod. Empty weight of the mould was taken and the maximum and minimum density was calculated. The natural unit weight of soil was evaluated from undisturbed sample collected during soil exploration. The relative density of each sample was determined using the equation given below.

$$Dr = \frac{\gamma_{max}}{\gamma_d}\left[\frac{\gamma_d - \gamma_{min}}{\gamma_{max} - \gamma_{min}}\right] \qquad (4.2)$$

4.1.1 Limitation of relative density and evolution of relative compaction

The uncertainty in calculating the relative density for silty sands when fine contents is more than a certain limit (15%) gives error in computation of strength parameters of silty sand (Salgado et al.).The problem associated with the estimation of minimum density (γ_{min}) of the soil containing significant percentage of plastic or non-plastic fines, results in the inaccurate determination of relative density. The presence of plastic fines further makes it difficult due to segregation of fines on the periphery of sand grains. This error in determination of relative density for the soil containing significant percentage of fines often leads to in appropriate estimation of strength and deformation characteristics of silty sands. Hence in the present work, the term relative compaction has been used in place of relative density to interpret the strength parameter of the soil containing fines to eliminate the error involved in the determination of relative density. Relative compaction is defined as the ratio of dry unit weight to maximum unit weight ($R_c = \gamma_d / \gamma_{max}$) which takes care of the uncertainty in computation of unit weight of soil (γ_{min}) since values of γ_d and γ_{max} can be accurately ascertained. Previous researchers considered the stress strain response of silty sand based on critical state friction angle (ϕ_c), peak friction angle (ϕ_p), maximum and minimum void ratio (e_{max} and e_{min}) and empirical strength parameters Q and R based on relative density concept. In the present work the relative compaction has been used instead of relative density to obtain the strength parameters Q_{af} and R_{af} for Yamuna sand with varied proportion of fines to avoid the error involved due to the use of relative density. Also the plot between D_r and R_c at different confining pressures has been drawn and an empirical relation between R_c and D_r has been found which is validated theoretically by substituting in Bolton's equation.

4.2 Development of preliminary relationships using relative compaction

The data of a typical drained compression test on silty sand with cylindrical sample and frictionless ends is considered in the present work to interpret preliminary stress - dilatancy relations. Strains were inferred from boundary displacements and volume changes. The accurate determination of the ultimate conditions may considerably be hampered by the non-uniformity of the sample and the uncertainty related to membrane correction following the formation of a rupture plane. Nevertheless such evidence as exists suggests that soil in rupture zones will dilate fully to achieve a critical state, at which shear deformation will continue in the absence of any volume change [Rowe, 1962]. The point of peak strength is usually associated with the maxima of a ratio of volumetric and shears strain ($d\varepsilon_v/d\varepsilon_s$) [Ayadat & Hanna, 2007; Been et al., 1991; Bolton, 1986; Salgado et al., 2000; Trivedi, 2010; Ojha & Trivedi, 2012, 2013; Trivedi and Singh, 2004;]. Bolton reviewed a large number of tri-axial and plane-strain test results for 17 nos. of different clean sand and proposed a simple relationship between ϕ, ϕ_c and ψ as shown in the following Eq. (4.3)

$$\phi = 1.25\psi + \phi_c \tag{4.3}$$

The relationship between the peak friction angle ϕ_p and the critical-state friction angle ϕ_c can be expressed in terms of the similar quantity I_{af}, called as dilatancy index. For tri-axial condition it is expressed as

$$\phi_p - \phi_c = 3I_{af} \tag{4.4}$$

Where

$$I_{af} = R_c\,(Q_{af} - \ln 100\,p' / P_A) - R_{af} \tag{4.5}$$

Where, R_c is relative compaction defined as a ratio of dry unit weight to maximum unit weight, expressed as a number between 0 and 1, p' is mean effective stress at peak strength in kPa, P_A is reference stress (100 kPa) in the same units as p', Q_{af} and R_{af} are non-dimensional and non-linear shear strength parameters.

Further let us define a relationship among relative dilatancy, relative compaction and mean confining pressure in terms of a new term I_{na} known as Compaction based Dilatancy index (CbDi abbreviated as I_{na})

$$I_{na} = I_{af} + R_c \ln p' \tag{4.6}$$

Substituting Eq. (3) in Eq. (4)

$$I_{na} = R_c\,(Q_{af} - \ln p') - R_{af} + R_c \ln p' \tag{4.7}$$

Re-arranging Eq. (5), we get

$$I_{na} = R_c\,Q_{af} - R_{af} \tag{4.8}$$

Typical variations of I_{na} with relative compaction are shown in Fig. 5.3(a) - 4.3 (f). Using the relationships of Eq. (4.6), we obtained the values of Q_{af} and R_{af} and the results are presented in Table (5.5) for best fit and with varied R_{af} =25, 30, 40. It has been validated that values of Q_{af} corresponding to R_{af} = 40 is similar to that obtained by Bolton (1986) for clean sand (Q=10, R=1) using relative density. Hence using Eq. 4.3 to Eq. 4.6, the estimation of I_{af}, I_{na}, and dilatancy angle can be made for wide ranging granular materials namely sands, silty sands [Konrad, 1990; Chu and Lo, 1993; Salgado et al., 2000; Ayadat and Hanna, 2007; Bolton, 1986; Gupta and Trivedi, 2009; Chakraborty and Salgado, 2010], coal ash [Trivedi and Sud, 2007], and even rock masses [Trivedi, 2010].

4.3 Validation of relative density- relative compaction relationship using Bolton's dilatancy equation.

From the plot of relative density and relative compaction (Fig. 5.10), a relationship between R_c and D_R can be defined in parametric form as

R_c = m Dr +n (4.9)

Using Fig.4.5, we find that for clean sand, the values of m and n are calculated as m= 0.217 and n=0.789,

Using values of Q_{af} and R_{af} from Fig. 5(a), for clean sand [Q_{af}=49.883 and R_{af}= 40.105], and substituting values of R_c in Eq. (4.8), we get

I_{na} =10.77 Dr -1.19 (4.10)

where Q_{af}=10.77 and R_{af}=1.19

From Eq. (3.8) value of dilatancy parameter Q_{af} is 10.77 corresponding to R_{af} value of 1.19. It is very close to Bolton's fitting parameters for clean sand (Q=10 corresponding to R=1). Since Yamuna sand contains varied proportion of fines and is not clean sand, the estimation of D_R for Yamuna sand containing more than 15%, fines involves error [Salgado et al., 2000], hence it is recommended to use relative compaction (R_c) for Yamuna sand with varied proportion of fines in place of relative density (D_r). Hence the problem involved in estimation of relative density for silty soil is eliminated without affecting the results.

CHAPTER V

RESULTS AND DISCUSSION

5.0 General

The results of the various tests conducted on Yamuna sands with varied proportion of fines are given in this chapter. The main Emphasis of this chapter is to understand the nonlinear engineering behavior of Yamuna sand with varied proportion of fines. In the later part of this section strength characteristics of Yamuna sand with varied proportion of fines are compared with few other sands. Engineering implication like estimation of bearing capacity of Yamuna sand and evaluation of the liquefaction potential by using the values of Q_{af} and R_{af} for varied proportion of fines in the Yamuna sands are presented in Chapter VI of this work.

5.1 Classification of Yamuna sand

The grain size characteristics of Yamuna sand with varied proportion of fines and few other sand are given in Table 5.1. The mean size (D_{50}) of Yamuna sand with varied proportion of fines are in the range of 0.208mm to 0.225 mm. Average particle size finer than 60% (D_{60}) varies from 0.24 mm to 0.25 mm. Grain size finer than 30% lies between 0.15 to 0.19 mm, while effective size (D_{10}) is in the range of 0.08 to 0.13 mm. It shows that the material investigated with varied proportion of fines in the present work is significantly different from the material investigated by the previous investigators [Bishop and Green, 1965; Been and Jefferies, 1985; Bolton, 1986; Amini and Qi, 2000; Bandini et al., 2000; Trivedi and Sud, 2002; Ayadat and Hanna, 2007; Carraro et al, 2009; Gupta and Trivedi, 2009; Chakraborty and Salgado, 2010]. The coefficient of uniformity ranges from 1.852 to 3.307 for Yamuna sand with silt percentage up to 10 percent which shows that the sample characteristics changes very significantly for higher silt content. The index properties of the Yamuna sand and silt used in the present work were evaluated experimentally and are presented in Table 5.2. From the tabulated data it is clear that the silt present in Yamuna sand is plastic in nature. Also the results indicates that when the proportion of fines are equal to or more than 15%, the behavior of Yamuna sand is simulates more towards silty soil than of a sandy soil. The peak frictional angle for a range of relative compaction (0.92-0.96) at which the deviatric stress verses axial strain plot attains its peak is given in Table 5.3 corresponding to varied confining pressure of 100, 200 and 400kPa. The grain size distribution curve for clean Yamuna sand, silt and Yamuna sand with varied proportion of fines (silt) are shown in Fig. (5.1a-5.1b).

5.2 Void ratio and critical state friction angle

The effect of fines on extreme void ratio has been studied and it is established that maximum and minimum void ratio of silty sand deceases with increase in fine contents [Lade and Yamamuro, 1997; Lee, 1995]. The silt particles occupy the space between the sand particles when present up to a certain percent and thereby reducing the voids among the sand particles. This results in the decrease of void ratio. This trend continues up to a certain limit called limiting void ratio when the silt particles end up between the surface of adjacent sand particles and thereby increasing the void ratio. This process pushes the sand particles apart. Table 5.4 shows the value of extreme void ratio of Yamuna sand when the silt percentage is increased from 0% to 25% along with other intrinsic variables for few other sands investigated worldwide. These findings have wide ranging applications in estimation of dilatancy [Been et al., 1991; Bolton, 1986; Chakraborty and Salgado, 2010; Simoni et al., 2006; Trivedi & Singh, 2004; Trivedi and Sud, 2002,2004], hardening-softening [Alejano and Alonso, 2005; Yang et al., 2008; Trivedi, 2012; Gajo et al., 1999], collapse behavior [Ayadat and Hanna, 2007; Sachan and Rao, 2010], bearing-capacity of fills [Bean and Jefferies, 1985; Perkins and Madson, 2000; Singh, 2010] and prediction of engineering behavior of granular materials namely silts [Gupta and Trivedi, 2009; Thevanayagam et al., 1996; Simoni and Houlsby, 2006], and rock masses [Alejano and Alonso, 2005; Trivedi, 2012; Igwe et al., 2012; Shukla et al., 2012]. Variation of maximum and minimum void ratio due to the presence of varied proportion of fines and mean size from previous published data (Table 5.4) including that of Yamuna sand is shown in Fig. 5.2(a) and Fig. 5.2(b) respectively. From the plot it has been established that the maximum and minimum void ratio decreases as the proportion of fines increases.

The critical sate is defined as the state at which the sand is sheared without changes in either shear strength or volume. Normally a loose sand contracts and a dense sand expands as it approaches critical state. However, nature of sand to contract or dilate is dependent not only on density but also effective confining stress. According to critical state model, when a sample is sheared under high effective confining stress, the shear stress increases monotonically until it reaches a plateau, after which the sample continues to undergo shear straining, without any change in shear stress or sample volume. The sample is then said to have reached the critical state and the corresponding friction angle is known as the critical state friction angle ϕ_c. During the shearing of dense sand, the sample contracts initially and then dilates. The effective principal stress ratio ($\sigma'1/\sigma'3$) reaches a peak associated with a peak friction angle at which the dilation rate is maximum. Further loading causes the shear stress to drop until it reaches the critical state. The critical state friction angle obtained from triaxial tests is commonly taken as a unique value for a given granular soil, regardless of the initial relative density and initial confining stress. Fig.5.3 (a) and Fig. 5.3(b) shows variation of critical state friction angle due to the presence of fines and mean size respectively. The critical state friction angle increases with the increase in proportion of fines in the Yamuna sand.

5.3 Shear strength

The shear strength of a soil is defined as the maximum value of shear stress that the soil can resist before failure occurs. The shear strength within a soil mass is mainly due to the frictional resistance between inter particle contact surface. The analyses of shear strength of any soil depend on the methods of estimation of appropriate values of c´ and ϕ´ accurately [Sladen and Handford, 1987]. Values of c´ and ϕ´ depend on soil history, type of soil, loading condition and drainage. The purpose of shear strength testing is to determine values for the shear strength parameters c´ and ϕ´. The drainage conditions during the test influence the measured values considerably [Soni and Jain, (2008)]. The analysis of a number of consolidated drained tri-axial compression tests were carried out with volumetric strain measurement. The tests were performed on consolidated specimens of clean Yamuna sand and Yamuna sand with varied proportion of fines. Plots between deviatric stress and axial strain, variation of volumetric strain with shear strain at varied relative compaction (0.92-0.96) and varied confining pressures (100kPa -400kPa) are presented in Fig.5.3 (a-j). The data of Fig.5.3 (a-j) were used to draw the plots between I_{na} and R_c in Fig. 5.4 (a-f). The non-dimensional and non-linear shear strength parameters (Q_{af}, R_{af}) were evaluated using Eqs. (3.1-3.6) for Yamuna sand with fines ranging 0-25%.and is presented in Table 5.5.The results are obtained at the confining pressure of 100, 200 and 400kPa. Values of Q_{af} and R_{af} obtained by other investigators for few other sands based on relative density approach is given in Table (5.6-5.7) [Chakraborty and Salgado, 2010; Salgado et al., 2000]. Plot between relative compaction and relative density has been drawn for Yamuna sand with varied proportion of fines in Fig.5.5. Validation of Eq. 3.6 in chapter III was done using the relationship of Fig.5.5. Shear strength parameters for Yamuna sand (Present Work) and Ottawa sand (Salgado et al.,

2000) as evaluated based on relative density concept has been given in Table 5.8 (a) and later modified for relative compaction is presented in Table 5.8 (b) for comparison. Plots were drawn between effective peak angle of internal friction obtained from triaxial tests and the unit weight of Yamuna sand with varied proportion of fines. It has been observed that peak friction angle increases with increase in density of Yamuna sand or with increase in relative compaction as shown in Fig. 5.10 and Fig. 5.11 respectively. However peak friction angle increase with increase in fine content up to 10% then reduces with further increase in fines due slippage of silt particles adjacent to sand particles.

Table 5.1 Grain size characteristics of few sands with and without fine content

Sand type	(%) Fines	D_{10} (mm)	D_{30} (mm)	D_m (mm)	D_{60} (mm)	C_c	C_u	q_u (kPa)	Reference
Yamuna Sand	0	0.13	0.19	0.225	0.25	1.07	1.852	-	Present work
Yamuna Sand	5	0.12	0.19	0.224	0.25	1.14	1.992	23	Present work
Yamuna Sand	10	0.08	0.18	0.222	0.25	1.84	3.307	24	Present work
Yamuna Sand	15	-	0.18	0.215	0.25	-	-	25	Present work
Yamuna Sand	20	-	0.17	0.213	0.24	-	-	27	Present work
Yamuna Sand	25	-	0.15	0.208	0.24	-	-	42	Present work
Ghaggar Sand	0	0.19	0.33	0.50	0.56	1.00 7	2.9	-	Gupta and Trivedi (2009)
Ghaggar Sand	5	0.18	0.29	0.488	0.54	1.87	3.02	-	Gupta and Trivedi (2009)
Ghaggar Sand	10	0.11	0.24	0.45	0.51	1.17	4.7	-	Gupta and Trivedi (2009)
Ghaggar Sand	15	-	0.23	0.438	0.50	-	-	-	Gupta and Trivedi (2009)
Ghaggar Sand	20	-	0.22	0.44	0.51	-	-	-	Gupta and Trivedi (2009)
Ghaggar Sand	25	-	0.18	0.37	0.48	-	-		Gupta and Trivedi (2009)
Ottawa sand	0	0.18	-	0.39	0.27	1.09	1.48	-	Salgado et. al. al.(2000)
Toyora Sand	0	-	-	0.16	-	-	1.3	-	Chakraborty & Salgado (2010)
Ticino sand	0	-	-	0.55	-	-	1.6	-	Lo Presti et. al. (1987)
Ham river	0	-	-	0.22	-	-	-		Bishop and Green (1965)

Table 5.2 Plasticity characteristics of silt and Yamuna sand

S.No.	Soil type	Fines (%)	Liquid Limit	Plastic Limit	Plasticity Index
1	Yamuna Sand	0	Slip occurs	Thread not formed	-
2	Yamuna Sand	5	Slip occurs	Thread not formed	-
3	Yamuna Sand	10	15.0%	Non Plastic	-
4	Yamuna Sand	15	18.0%	Non Plastic	-
5	Yamuna Sand	20	20.0%	Non Plastic	-
6	Yamuna Sand	25	22.0%	11.5%	10.5%
7	Silt	100	22.4%	10%	12.4%

Table 5.3 Peak friction angle for Yamuna sand

S.No.	Soil type	Fines (%)	Confining Pressure (kPa)	Relative Compaction (%)				
				0.92	0.93	0.94	0.95	0.96
1	Yamuna Sand	0	100	24.40	27.98	26.37	32.51	32.79
			200	24.28	26.30	25.63	26.37	26.84
			400	22.32	26.14	24.30	24.56	24.99
2	Yamuna Sand	5	100	26.89	32.39	31.14	34.15	34.81
			200	24.38	24.61	26.34	26.84	27.15
			400	20.31	19.59	23.75	24.13	24.73
3	Yamuna Sand	10	100	33.92	31.87	35.42	36.21	36.72
			200	25.79	27.95	29.51	29.78	30.59
			400	27.05	22.32	21.99	22.48	23.02
4	Yamuna Sand	15	100	20.19	17.55	19.80	30.15	31.38
			200	24.39	22.81	25.91	26.07	26.68
			400	23.24	24.24	25.34	25.67	26.06
5	Yamuna Sand	20	100	20.21	24.25	24.71	26.98	28.61
			200	20.37	22.14	22.88	23.03	25.06
			400	15.92	17.32	17.40	21.01	21.90
6	Yamuna Sand	25	100	24.46	26.94	28.62	30.40	31.01
			200	22.04	23.99	24.80	25.90	26.98
			400	17.11	20.39	21.12	21.84	22.85

Table 5.4 Intrinsic Variables of Few Sands

Sand type	% Fines	e_{min}	e_{max}	ϕ_c	G_s	D_m	Reference
Yamuna Sand	0	0.50	0.78	24.2	2.67	0.225	Present work
Yamuna Sand	5	0.46	0.76	25.1	2.668	0.224	Present work
Yamuna Sand	10	0.42	0.72	26.0	2.666	0.222	Present work
Yamuna Sand	15	0.38	0.68	26.5	2.664	0.215	Present work
Yamuna Sand	20	0.33	0.63	28.5	2.662	0.213	Present work
Yamuna Sand	25	0.31	0.62	30.7	2.66	0.208	Present work
Ottawa Sand	0	0.48	0.78	29.0	-	-	Salgado et. al. (2000)
Ottawa Sand	5	0.42	0.70	30.5	-	-	Salgado et. al. (2000)
Ottawa Sand	10	0.36	0.65	32.0	-	-	Salgado et. al. (2000)
Ottawa Sand	15	0.32	0.63	32.5	-	-	Salgado et. al. (2000)
Ottawa Sand	20	0.29	0.62	33.0	-	-	Salgado et. al. (2000)
Ham river sand	0	0.92	0.59	33	-	0.22	Bishop & Green (1965)
Monterey Sand	0	0.57	0.86	37	-	-	Houlsby (1993)
Toyoura Sand	0	0.61	0.99	35.1	2.65	0.16	Chakraborty & Salgado (2010)
Sacramento river sand	0	0.53	0.87	33.2	-	0.3	Lade & Yamamuro (1997).

Table 5.5 Dilatancy parameters with reference to relative compaction for silty Yamuna sand [Present work]

Silt (%)	D_{50} (mm)	Best Fit		Tread Line with $R_{af} = 40$	Tread Line with $R_{af} = 30$	Tread Line with $R_{af} = 25$
		Q_{af}	R_{af}	Q_{af}	Q_{af}	Q_{af}
0	0.225	49.883	40.105	49.77	39.14	33.83
5	0.224	38.091	29.115	49.64	39.03	33.72
10	0.222	33.863	24.821	49.99	39.36	34.05
15	0.215	34.700	25.977	49.63	38.98	33.66
20	0.213	10.689	4.372	48.71	38.03	32.70
25	0.208	5.023	1.094	48.73	38.09	32.77

Table 5.6 Dilatancy parameters for Toyoura sand based on relative density [Chakraborty and Salgado, 2010]

$б'_{3p}$ (kPa)	$б'_{mp}$ (kPa)	Best Fit			Trend Line with R =1	
		Q_s	R_s	r^2	Q_s	r^2
4	9.3	6.9	0.47	0.92	7.7	0.914
6.2	14.3	6.2	-0.23	0.94	8.1	0.839
11.2	25.8	7.4	0.13	0.99	8.7	0.954
20.8	47.2	7.5	0.03	0.987	9	0.945
50.3	108.4	8.9	0.79	0.999	9.3	0.997

Table 5.7 Dilatancy parameters for Ottawa sand based on relative density [Salgado et al., 2000]

Silt (%)	Best Fit			Trend Line; $R_s = 0.5$	
	Q_s	R_s	r^2	Q_s	r^2
0	9.0	0.49	0.93	9.0	0.93
5	9.0	-0.50	0.98	11.0	0.92
10	8.3	-0.69	0.97	10.6	0.87
15	11.4	1.29	0.97	10.3	0.96
20	10.1	0.85	0.95	9.50	0.95
25	-	-	-	-	-

Table 5.8 (a) Comparison of Shear strength parameter based on relative density for silty Sand

Silt (%)	Yamuna Sand (Present work)		Ottawa Sand (Salgado et. al 2000)		Tread line with $R_a = 0.5$ Yamuna Sand		Tread line with $R_s = 0.5$ Ottawa Sand	
	Q_a	r^2	Q_s	r^2	Q_a	r^2	Q_s	r^2
0	11.674	0.721	9	0.93	7.096	0.502	9	0.93
5	11.397	0.790	9	0.98	7.047	0.673	11	0.92
10	10.934	0.705	8.3	0.97	7.433	0.632	10.6	0.87
15	11.041	0.553	11.4	0.97	6.817	0.471	10.3	0.96
20	5.435	0.356	10.1	0.95	5.426	0.332	9.5	0.95

Table 5.8 (b) Comparison of shear strength parameter based on relative compaction for silty sand

Silt (%)	Yamuna Sand (Present work)			Ottawa Sand (Salgado et. al 2000)			Trend Line with $R_{af} = 40$ Yamuna Sand		Trend Line with $R_{sf} = 40$ Ottawa Sand	
	Q_{af}	R_{af}	r^2	Q_{sf}	R_{af}	r^2	Q_{af}	r^2	Q_{sf}	r^2
0	49.60	39.78	0.797	51.18	42.40	0.925	49.83	0.797	48.53	0.922
5	44.34	34.85	0.880	52.95	43.15	0.976	49.79	0.867	49.43	0.972
10	45.62	35.73	0.856	45.09	35.87	0.962	50.16	0.847	49.72	0.952
15	20.65	12.52	0.784	40.67	31.77	0.739	49.80	0.779	49.74	0.702
20	19.84	12.95	0.380	45.03	35.72	0.965	48.74	0.427	49.88	0.953

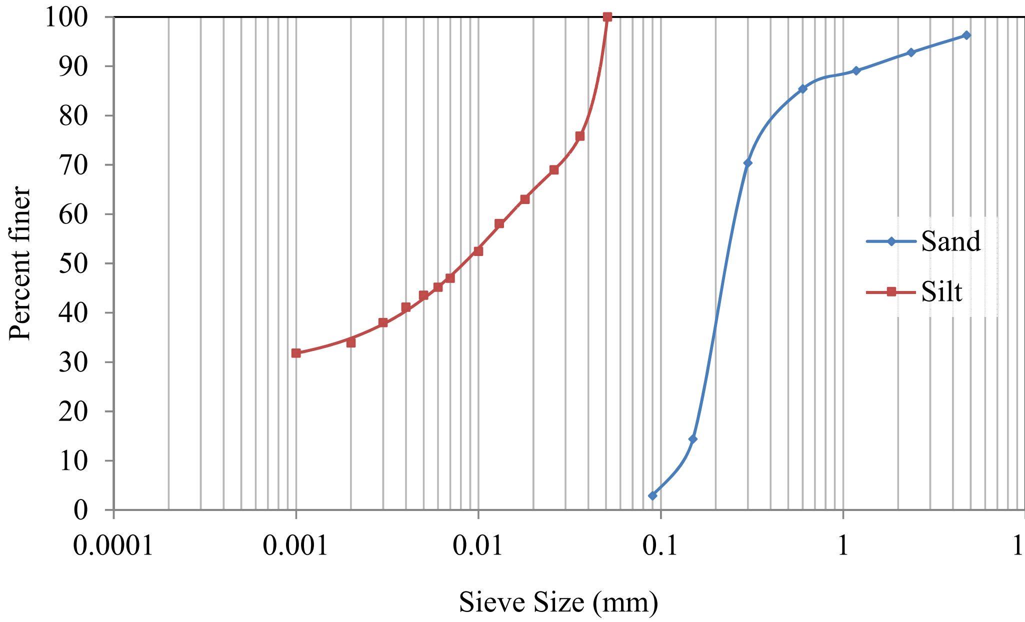


Fig. 5.1 (a**)** Grain size distribution of clean Yamuna sand and silt

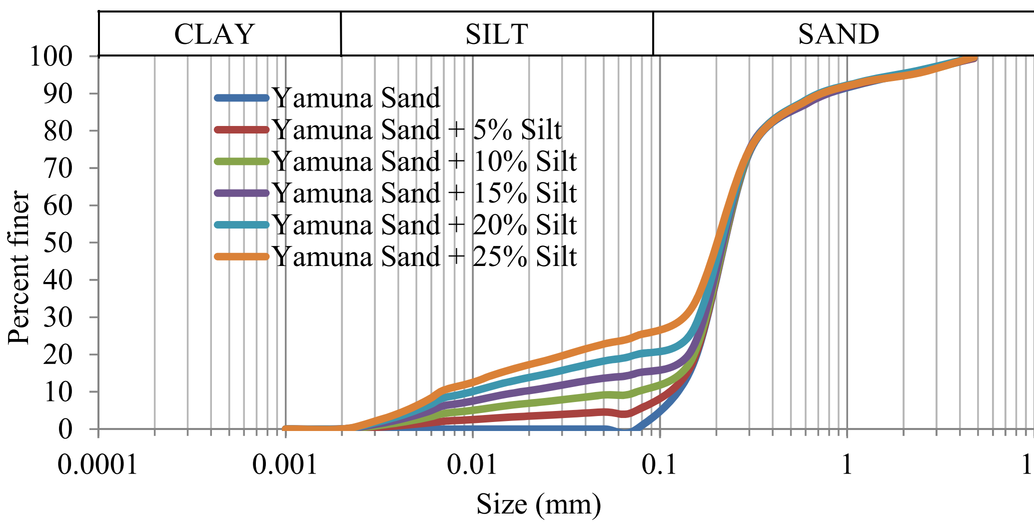


Fig. 5.1 (b) Grain size distribution of Yamuna sand with varied proportion of fines

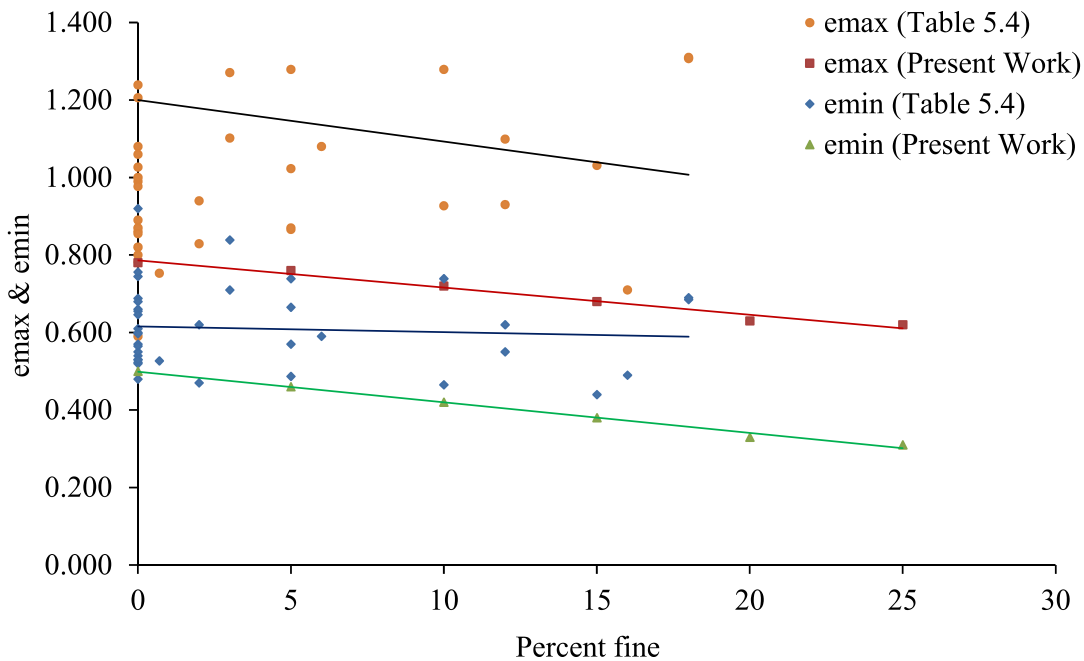


Fig. 5.2 (a) Best fit for the variation of maximum and minimum void ratio with percent fines

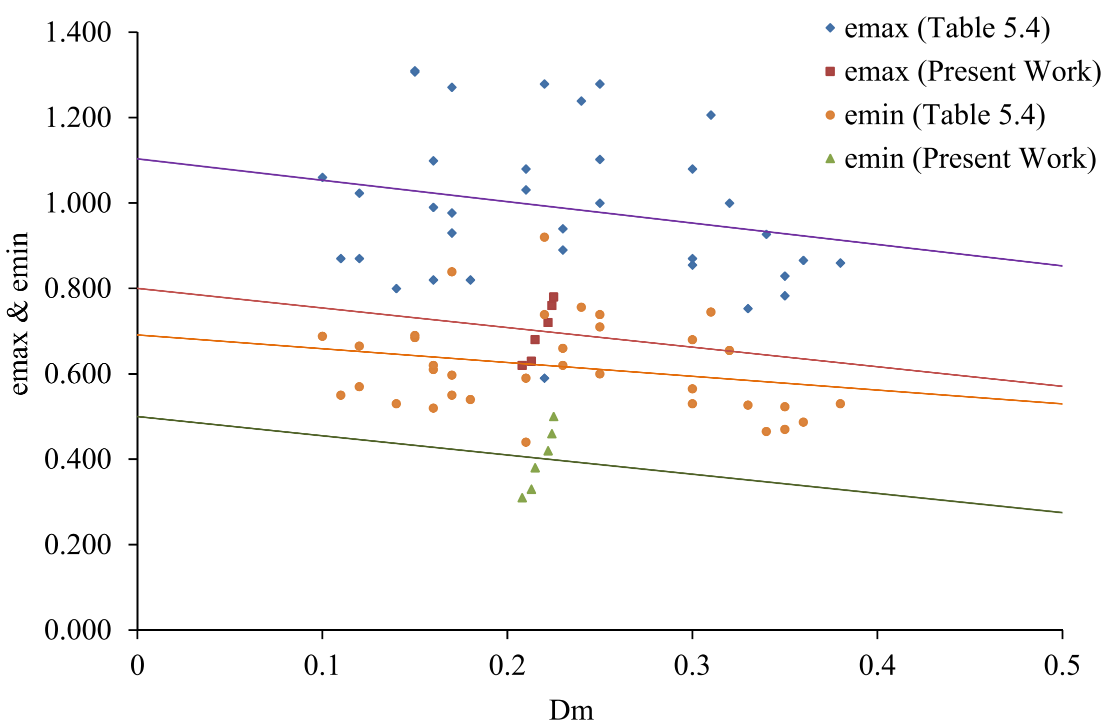


Fig. 5.2 (b) Variation of maximum and minimum void ratio with mean size

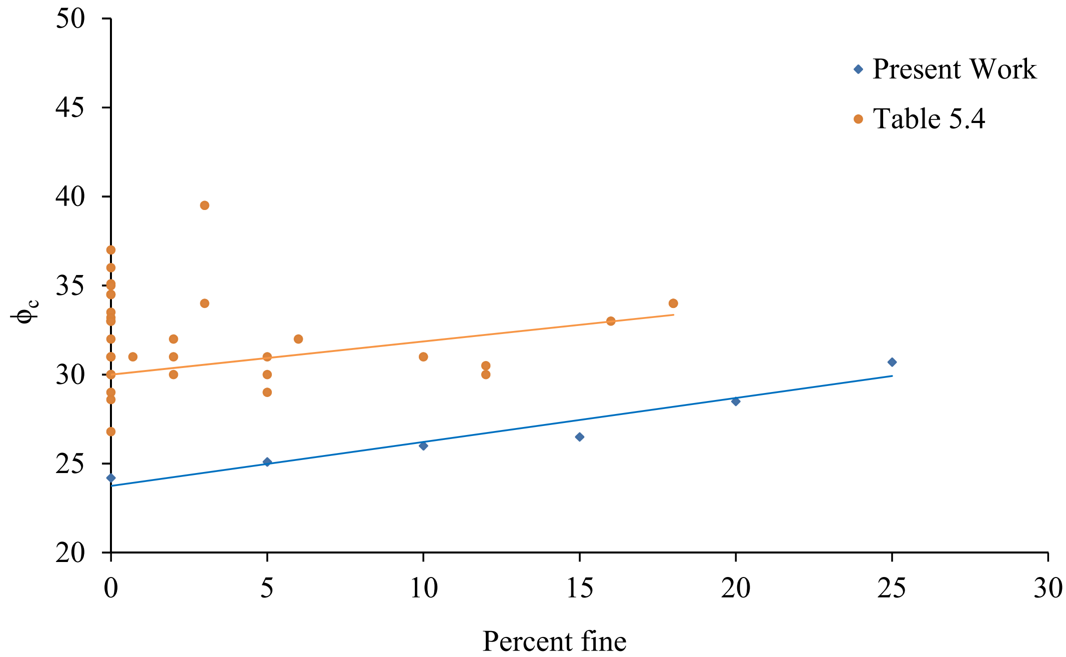


Fig. 5.2 (c) Variation of critical friction angle with percentage fine

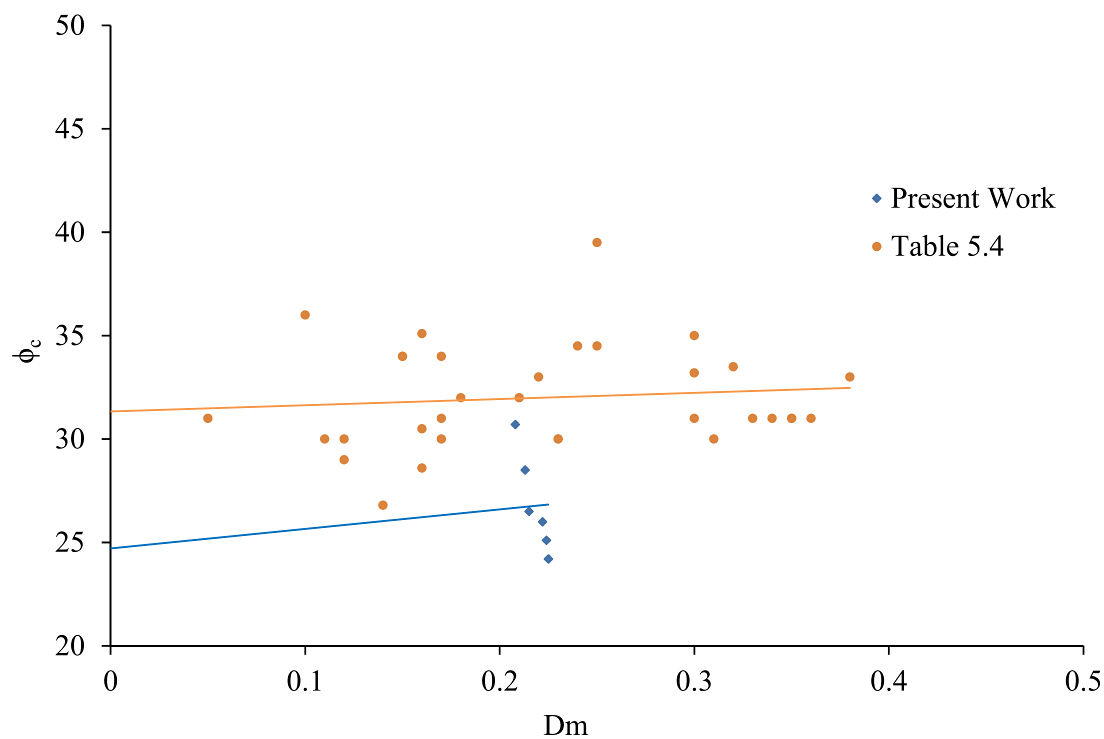


Fig. 5.2 (d) Variation of critical friction angle with mean size

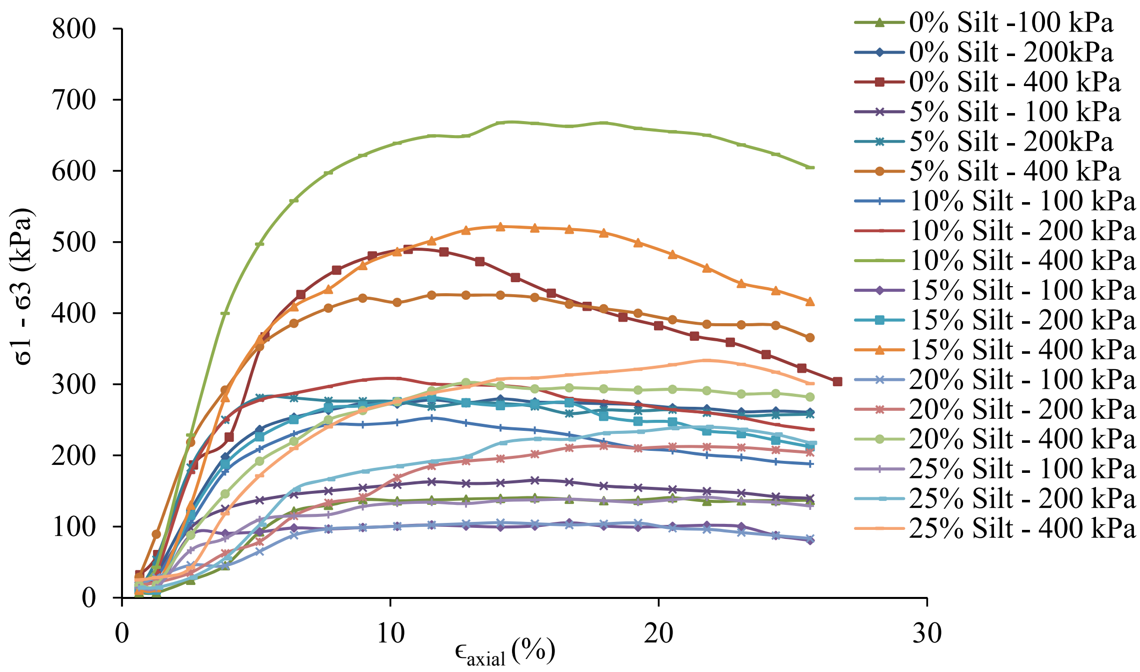


Fig. 5.3 (a) Deviator stress v/s axial strain at $R_c = 0.92$

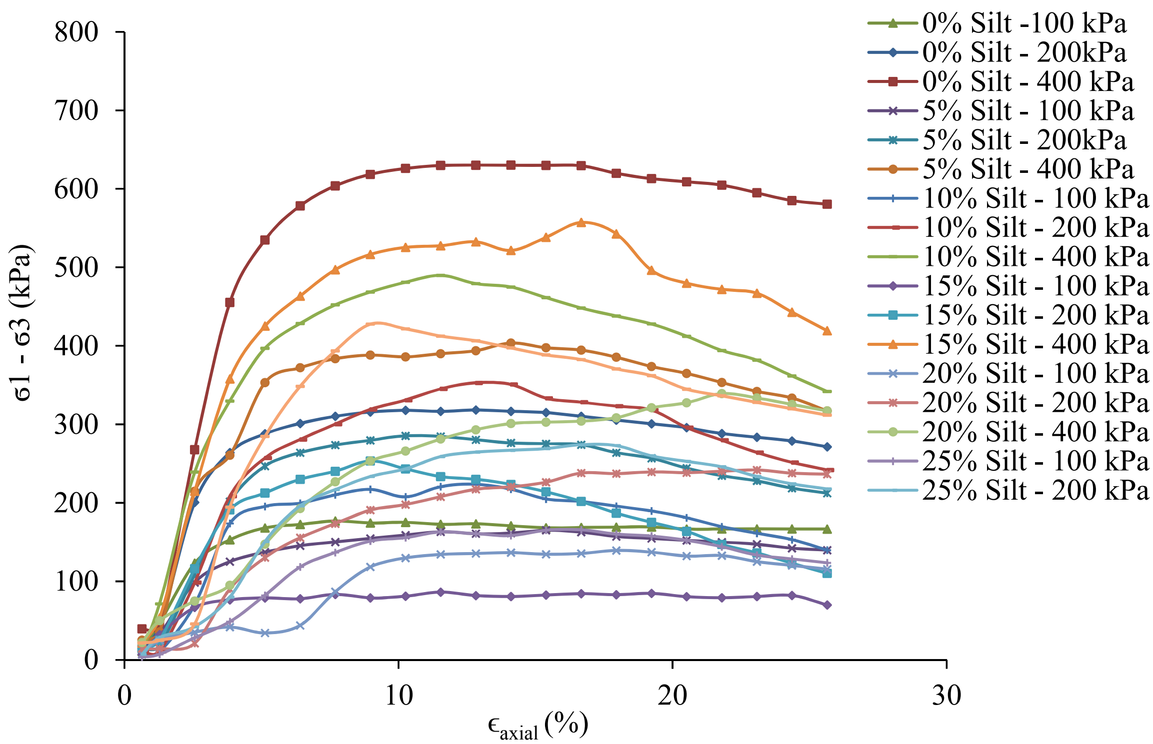


Fig. 5.3 (b) Deviator stress v/s axial strain at $R_c = 0.93$

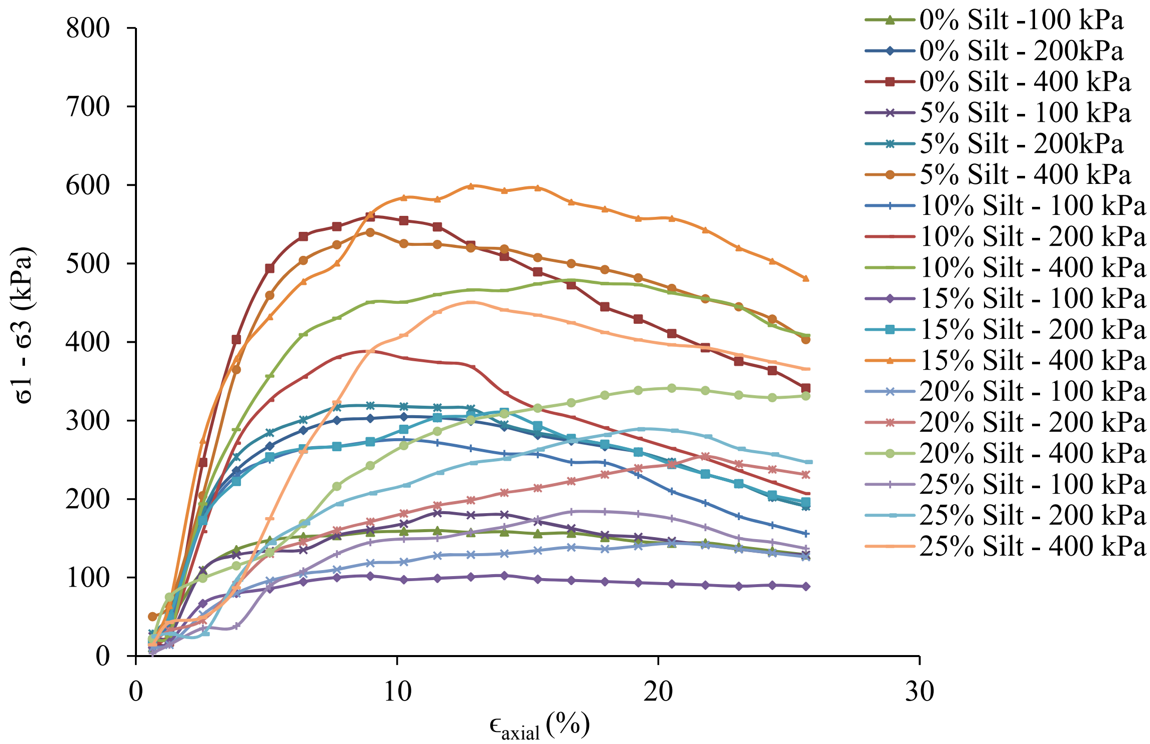


Fig. 5.3 (c) Deviator Stress v/s Axial Strain at $R_c = 0.94$

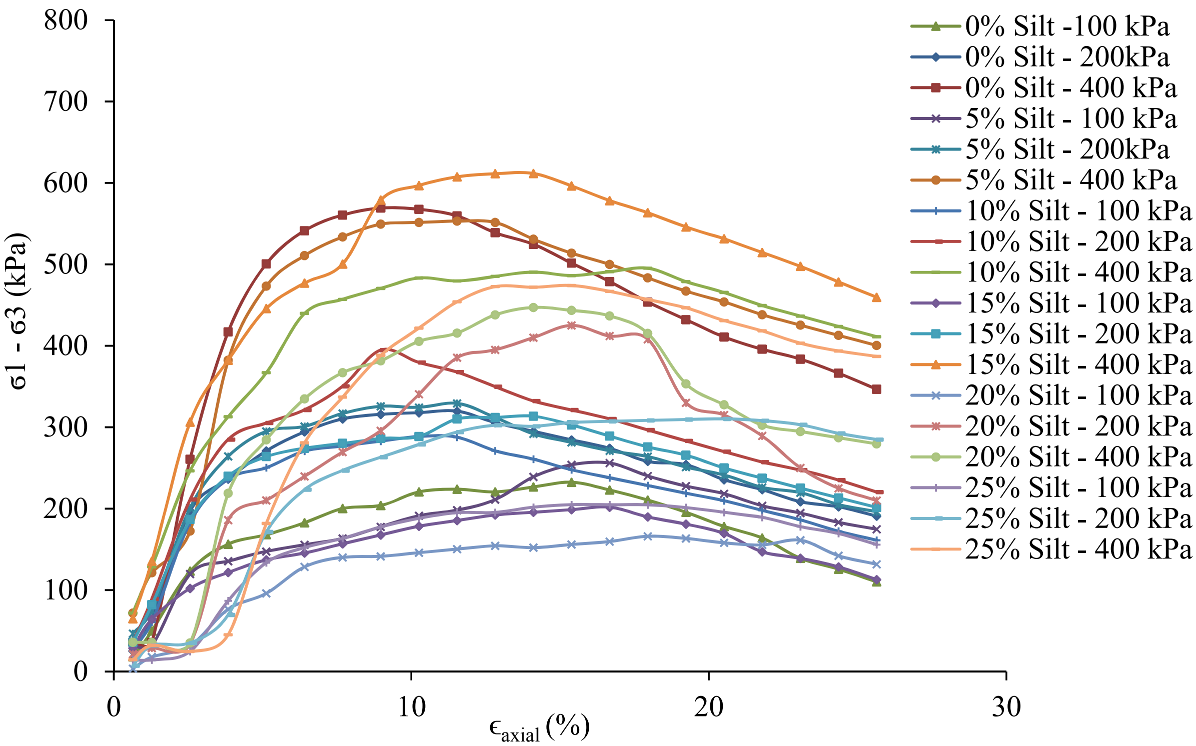


Fig. 5.3 (d) Deviator stress v/s axial strain at $R_c = 0.95$

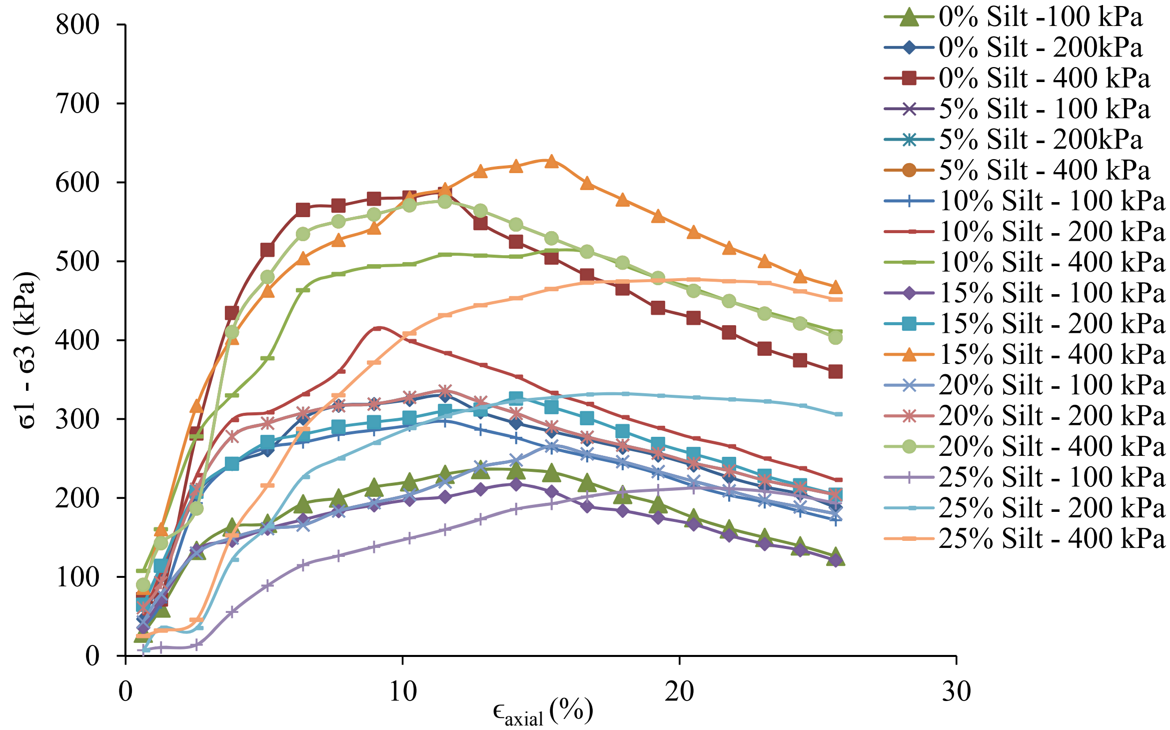


Fig. 5.3 (e) Deviator stress v/s axial strain at $R_c = 0.96$

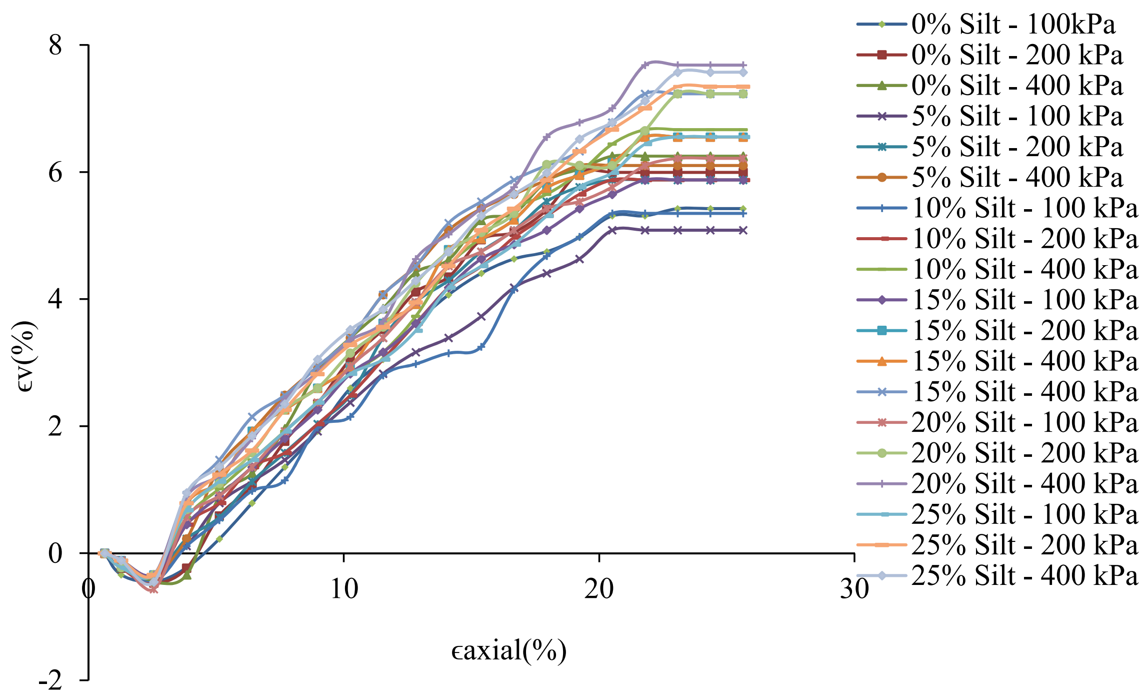


Fig. 5.3 (f) Volumetric strain v/s axial strain at $R_c = 0.92$

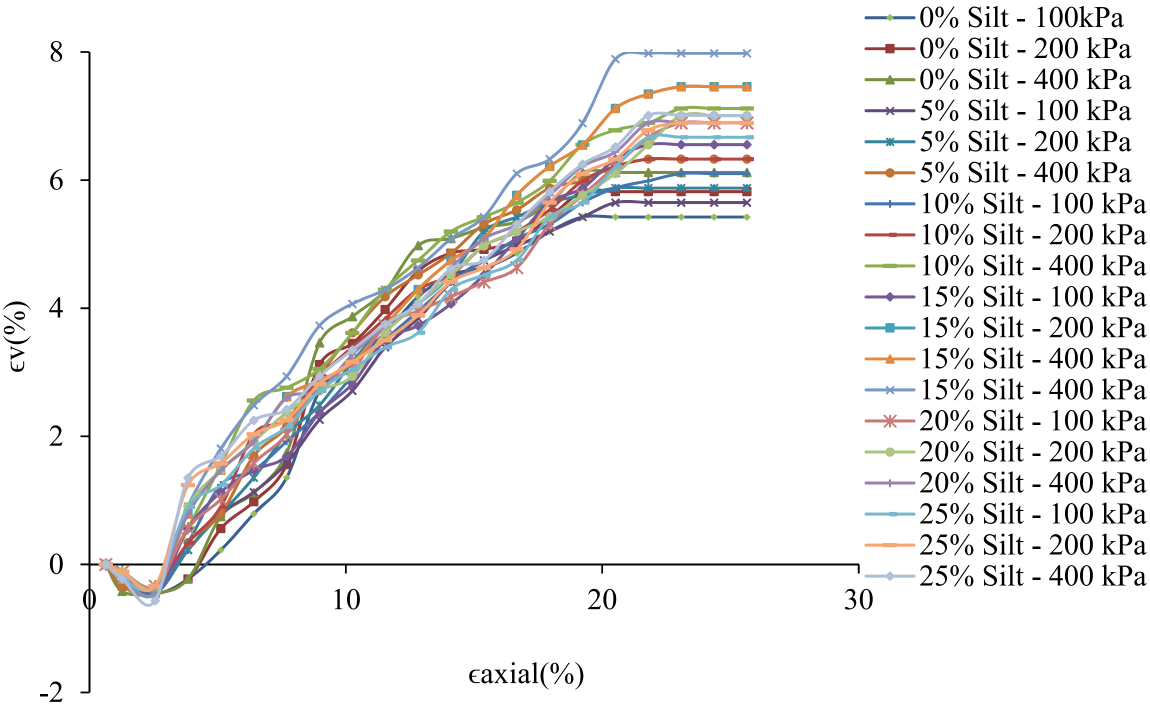


Fig. 5.3 (g) Volumetric strain v/s axial strain at Rc = 0.93

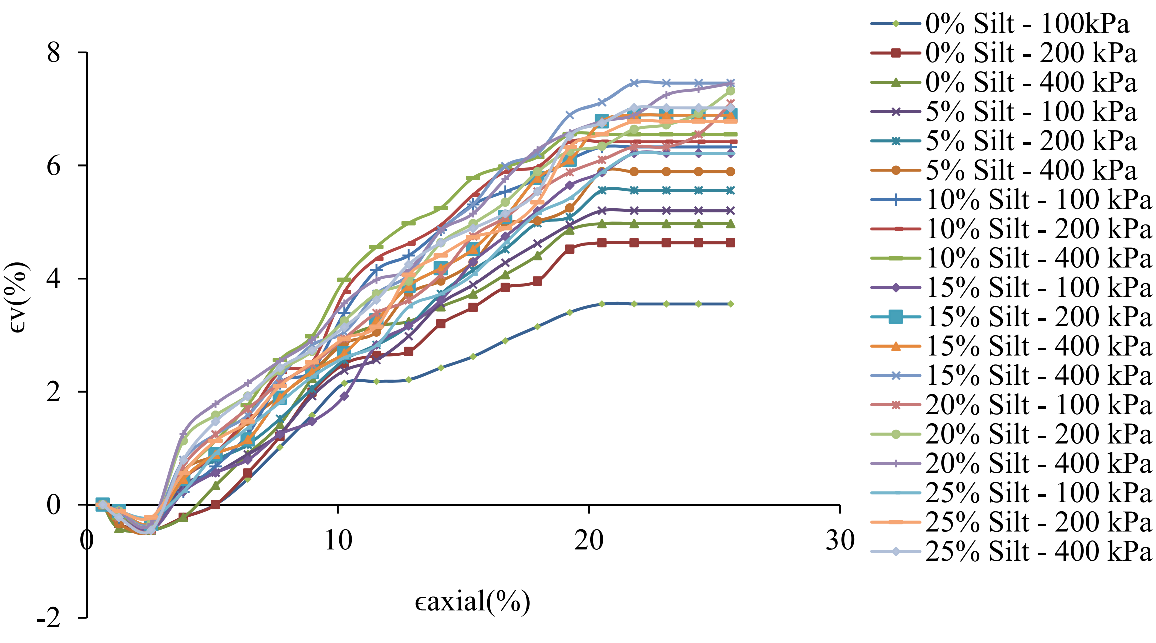


Fig. 5.3 (h) Volumetric strain v/s axial strain at R_c = 0.94

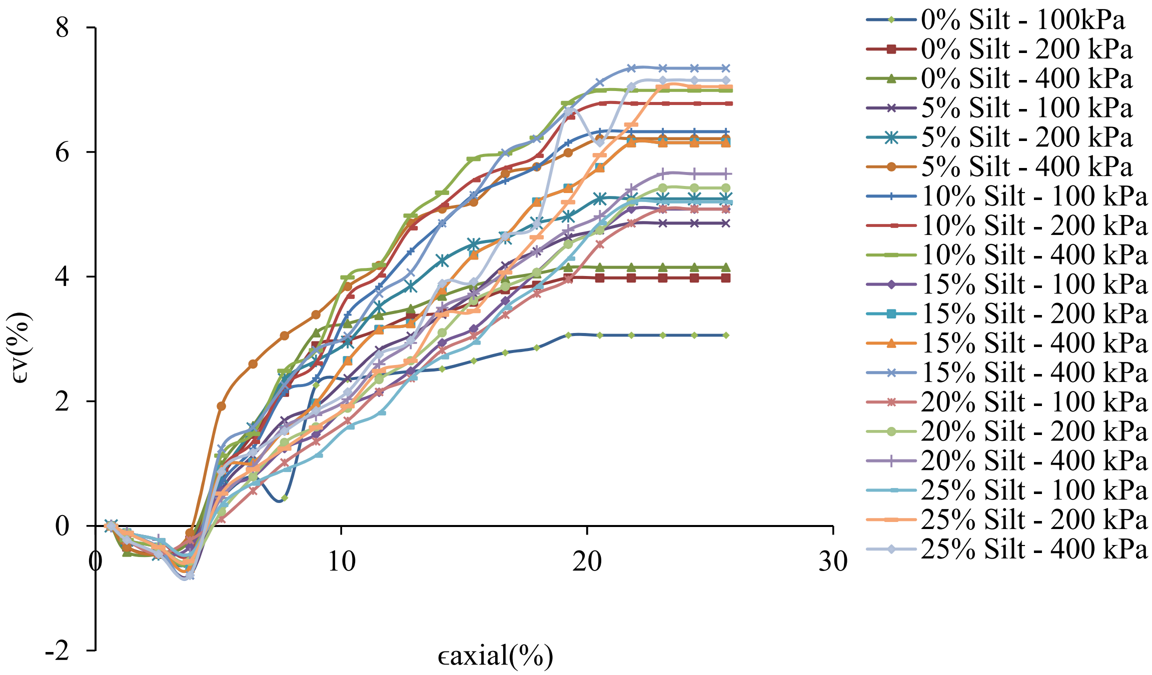


Fig. 5.3 (i) Volumetric strain v/s axial strain at $R_c = 0.95$

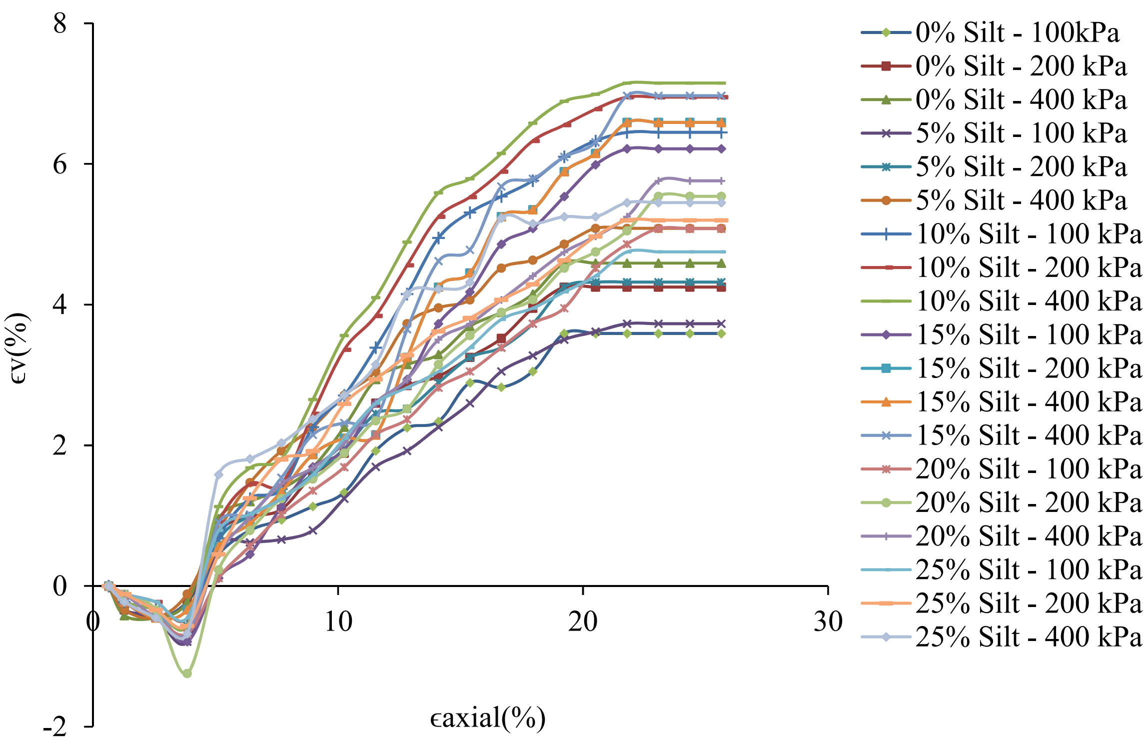


Fig. 5.3 (j) Volumetric strain v/s axial strain at $R_c = 0.96$

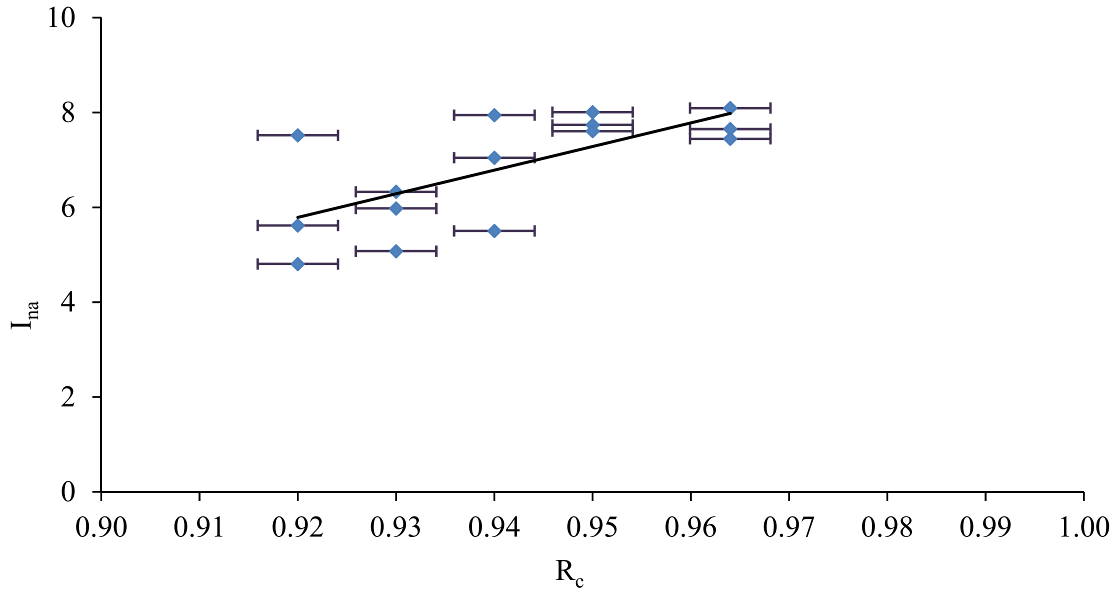


Fig. 5.4 (a) I_{na} v/s R_c for clean sand ($D_m^{0.225}$) with 5 percent error bars

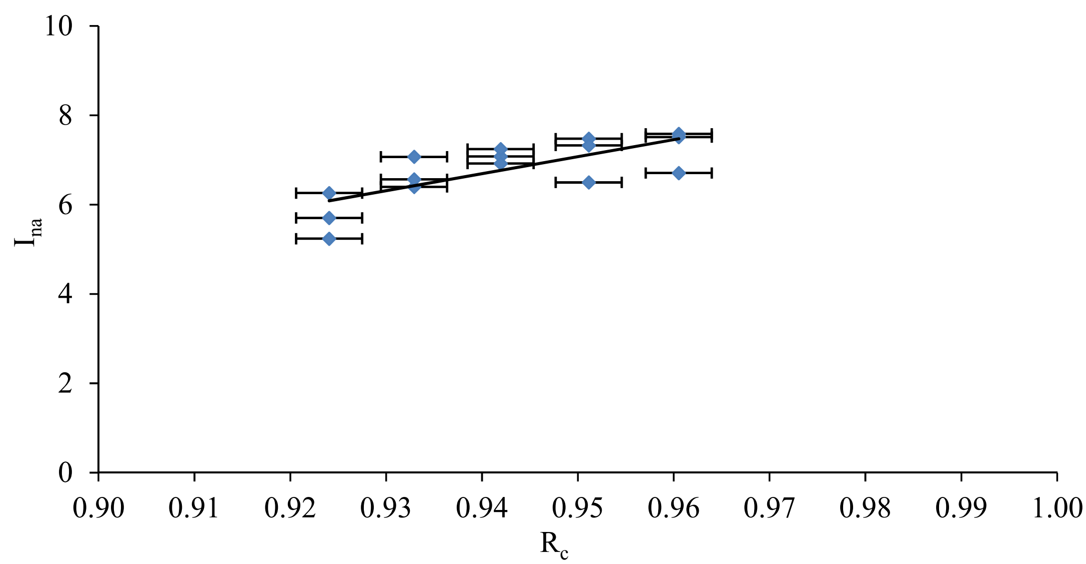


Fig. 5.4 (b) I_{na} v/s R_c for clean sand at 5% fines ($Dm^{0.224}$) with 5 percent error bars

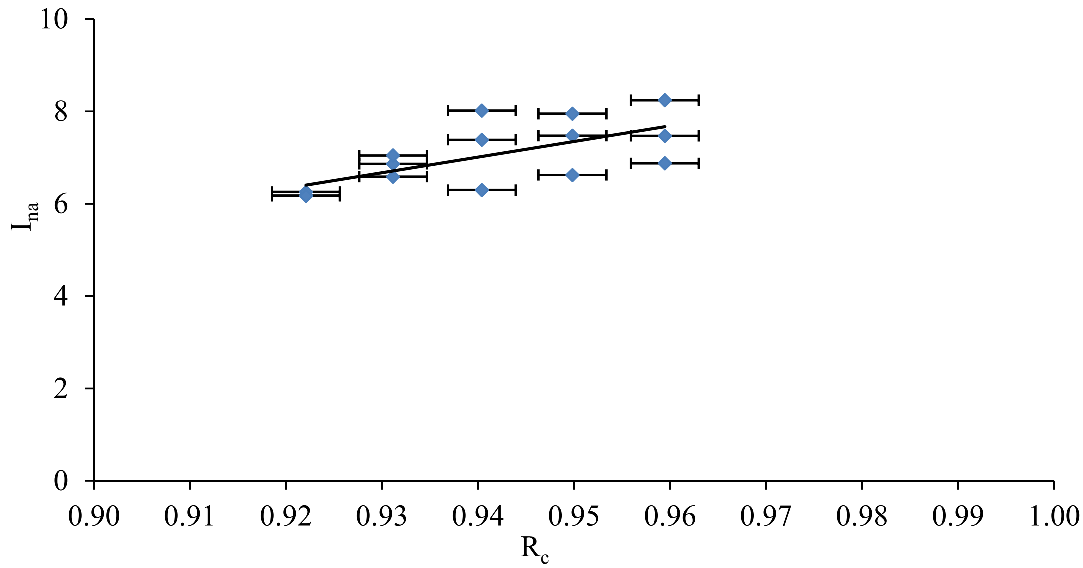


Fig. 5.4 (c) I_{na} v/s R_c for clean sand at 10% fines ($Dm^{0.222}$) with 5 percent error bars

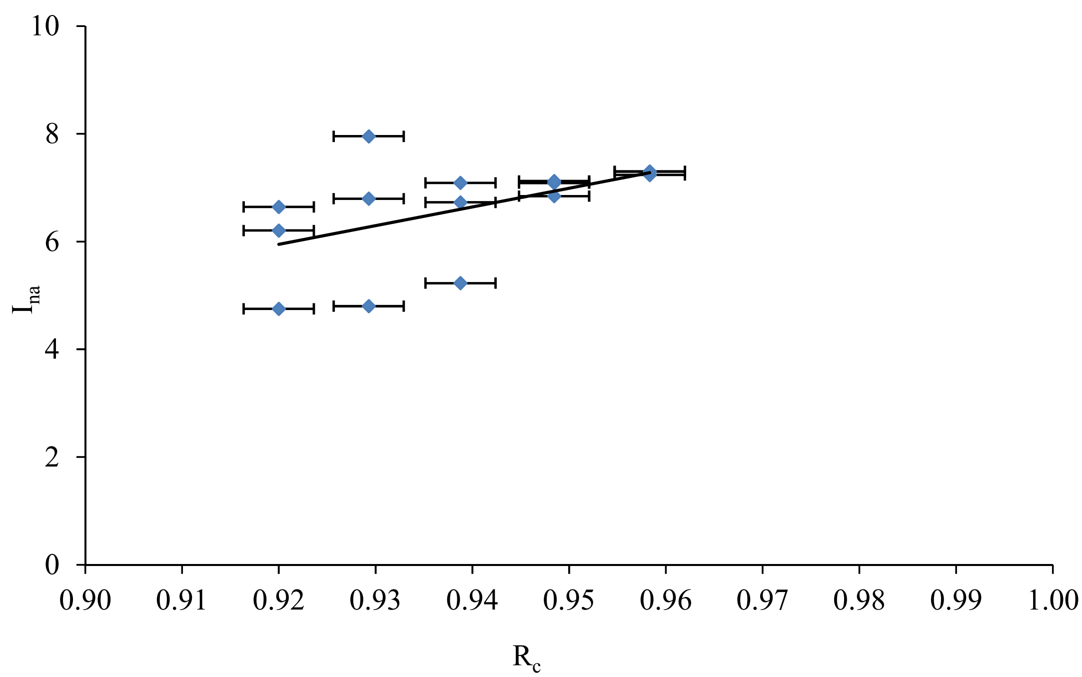


Fig. 5.4 (d) I_{na} v/s R_c for clean sand at 15% fines ($Dm^{0.215}$) with 5 percent error bars

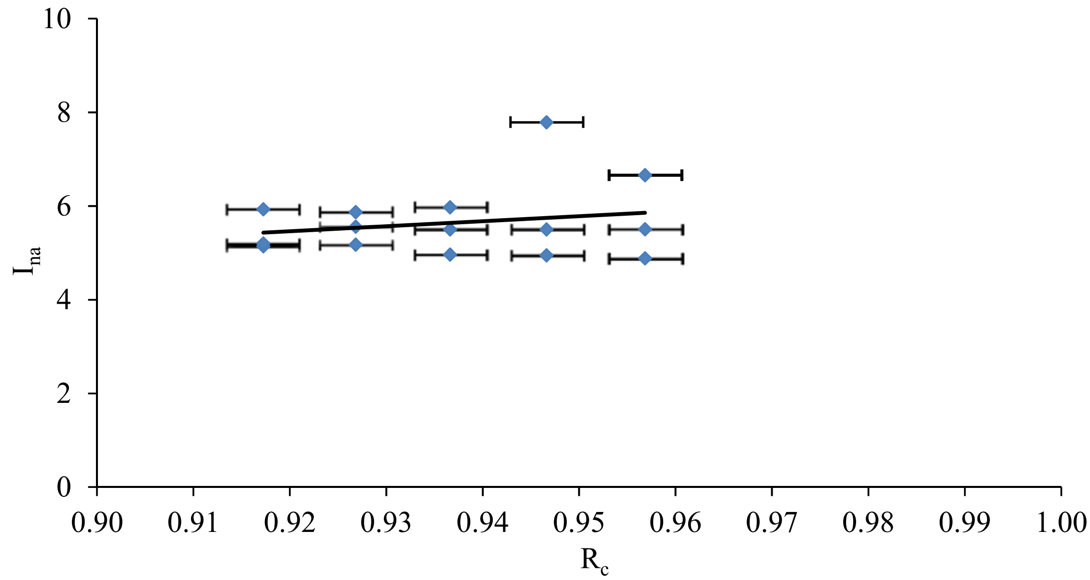


Fig. 5.4 (e) I_{na} v/s R_c for clean sand at 20% fines ($Dm^{0.213}$) with 5 percent error bars

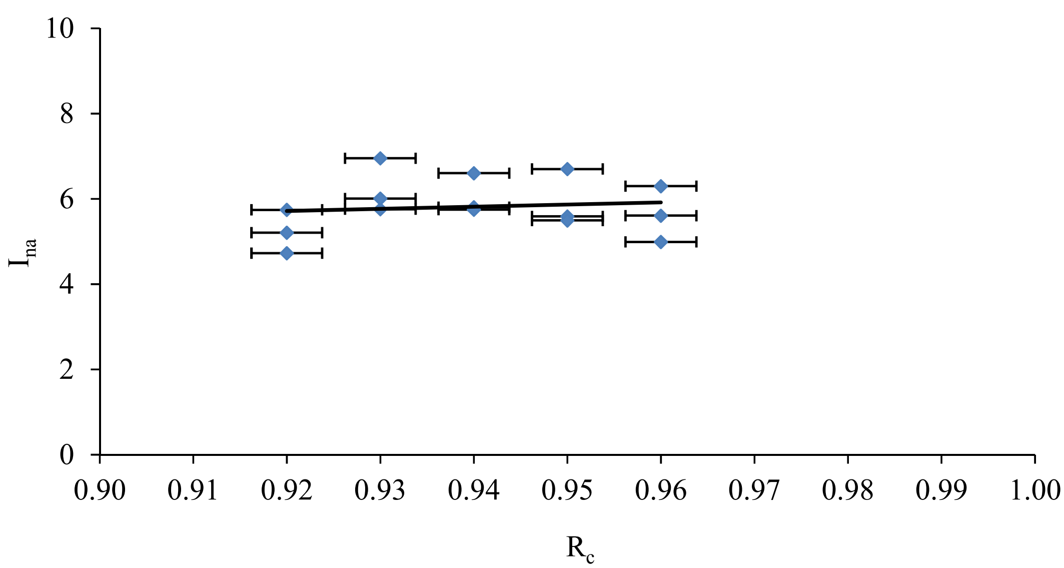
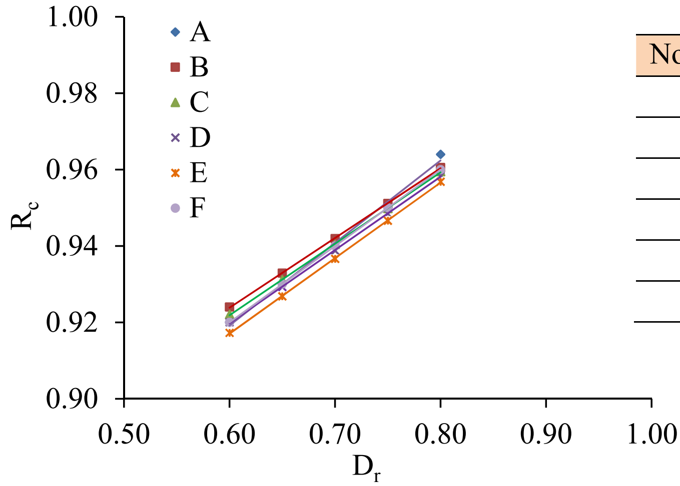


Fig. 5.4 (f) I_{na} v/s R_c for clean sand at 25% fines ($Dm^{0.208}$) with 5 percent error bars



Notation	Description	m	n
A	clean sand	0.216	0.789
B	clean sand + 5% fines	0.182	0.814
C	clean sand + 10% fines	0.186	0.809
D	clean sand + 15% fines	0.191	0.804
E	clean sand + 20% fines	0.198	0.798
F	clean sand + 25% fines	0.200	0.800

Fig. 5.5 Variation of relative compaction with relative density

5.4 Comparison of Non-Linear Engineering Behaviour of Yamuna Sand with Ottawa Sand

In this section the values of shear strength parameters Q_{af} and R_{af} obtained from tri-axial test results of Yamuna sand with varied proportion of fines has been compared with Ottawa sand as evaluated by Salgado et al. (2000). The findings of Salgado et al. (2000) have been re-evaluated by Ojha and Trivedi (2013) in terms of relative compaction to bring the results comparable with similar parameters. The sequential comparison with discussion of results is presented here. Plot between maximum and minimum void ratio for Yamuna sand and Ottawa sand has been plotted based on laboratory test as shown in Fig.5.6

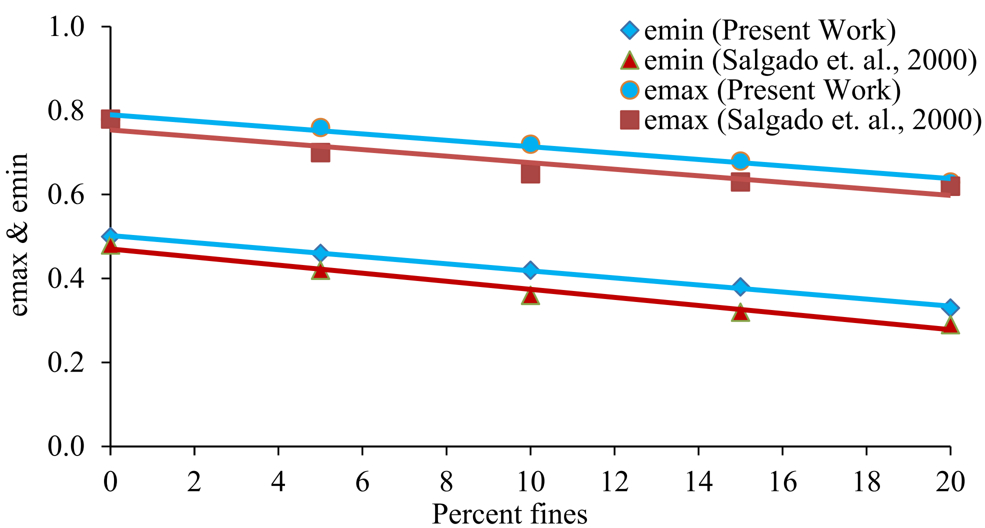


Fig. 5.6 Variation of maximum and minimum void ratio with percent fines

It is observed that both maximum and minimum void ratio decreases with increase in fines for both sand but the total voids for the same percent fines are more in Yamuna sand than for Ottawa sand. This is due to the fact that fines used in the analysis of Yamuna sand was plastic fines and for Ottawa sand non plastic fines were used.

The plot between critical state friction angle and percent fine were also drawn for Ottawa and Yamuna sand as shown in Fig.5.7. It has been found that the Yamuna sand reaches its peak much earlier than Ottawa sand for the same percent fines due to presence of plastic fines which slips easily under the same normal and confining stress. The plot shown ignores the scattered data falling beyond the range of twice the standard deviation

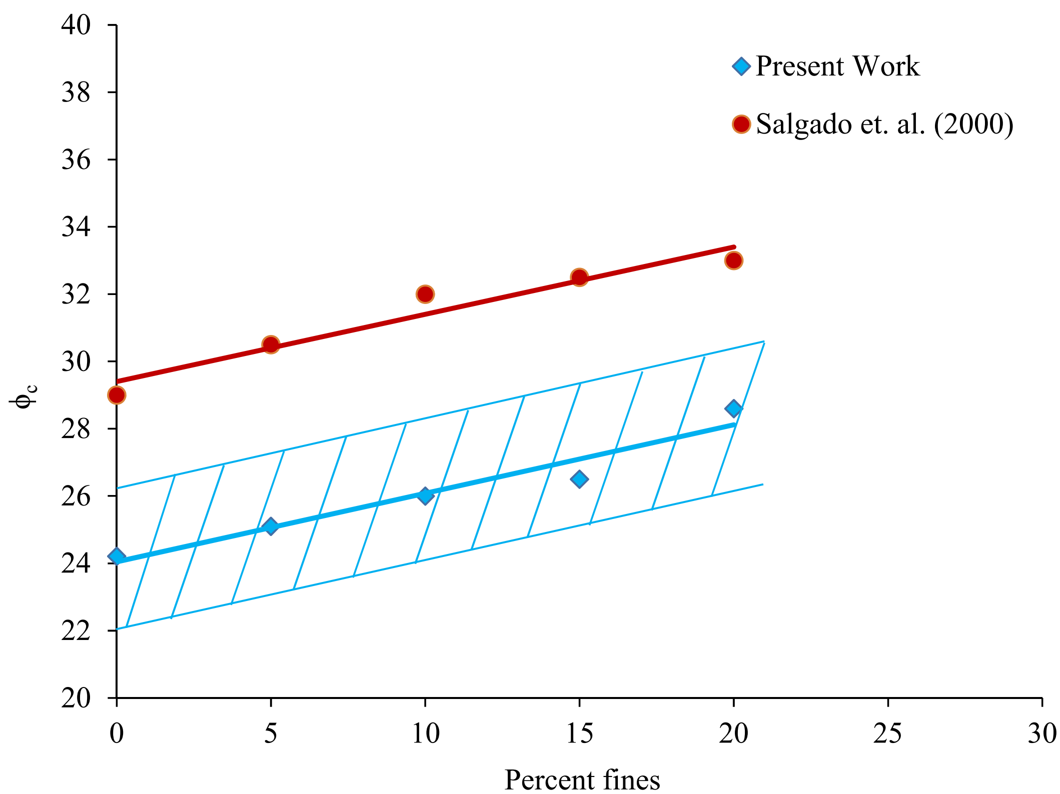


Fig. 5.7 Critical friction angle vs. percent fines

Based on the test results data obtained from triaxial test of Ottawa sand by Salgado (2000) for maximum and minimum proctor density, relative density has been converted into relative compaction and the plot between D_r and R_c at different percent fine has been drawn as shown in Fig. 5.8(a-e). The same plot for Yamuna sand in the present work was also superimposed with that of Ottawa sand and an empirical relation between R_c and R_d has been found which is validated theoretically by substituting in Bolton's equation. From the plot it is clear that

Ottawa sand is more compressible for fines percent up to 10% and the rate of compression decreases when percent fine increases above 10%.

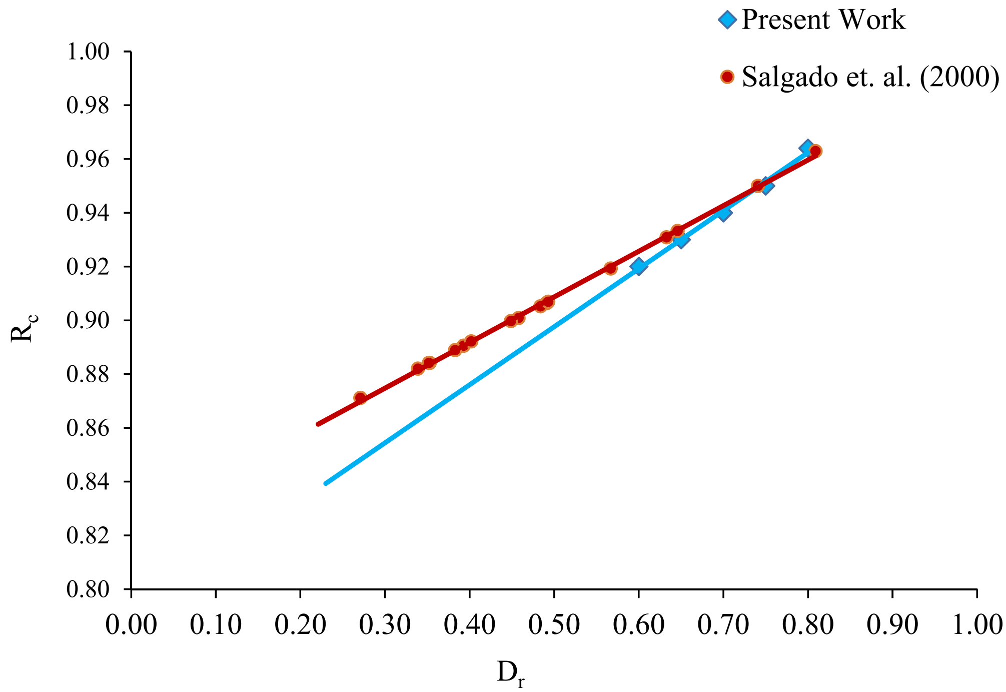


Fig. 5.8 (a) Variation of R_c vs. D_r without silt content

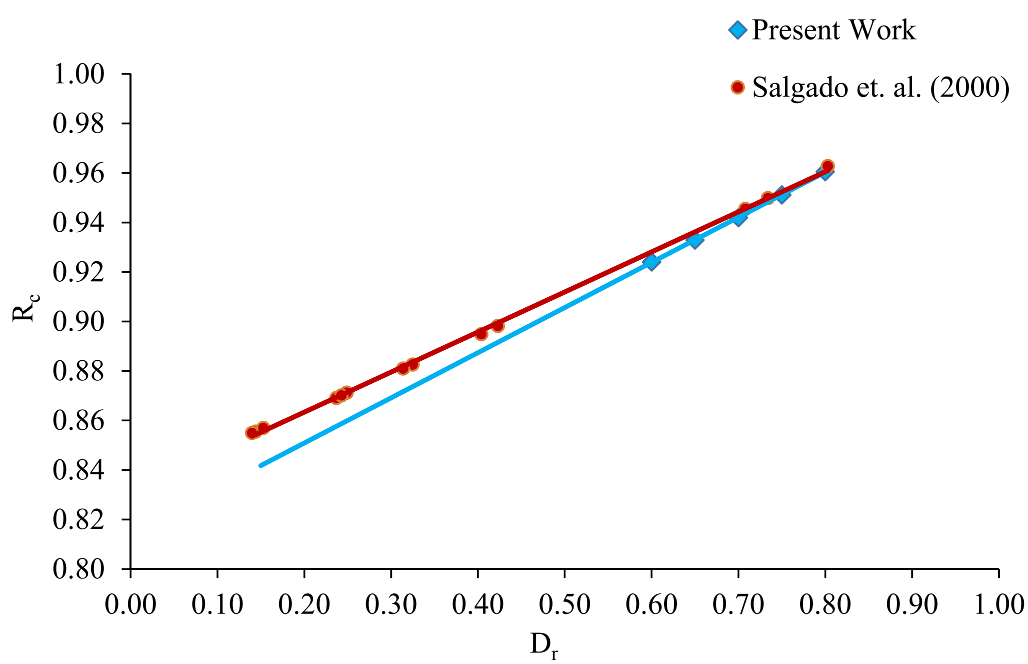


Fig. 5.8 (b) Variation of R_c vs. D_r at 5 percent silt content

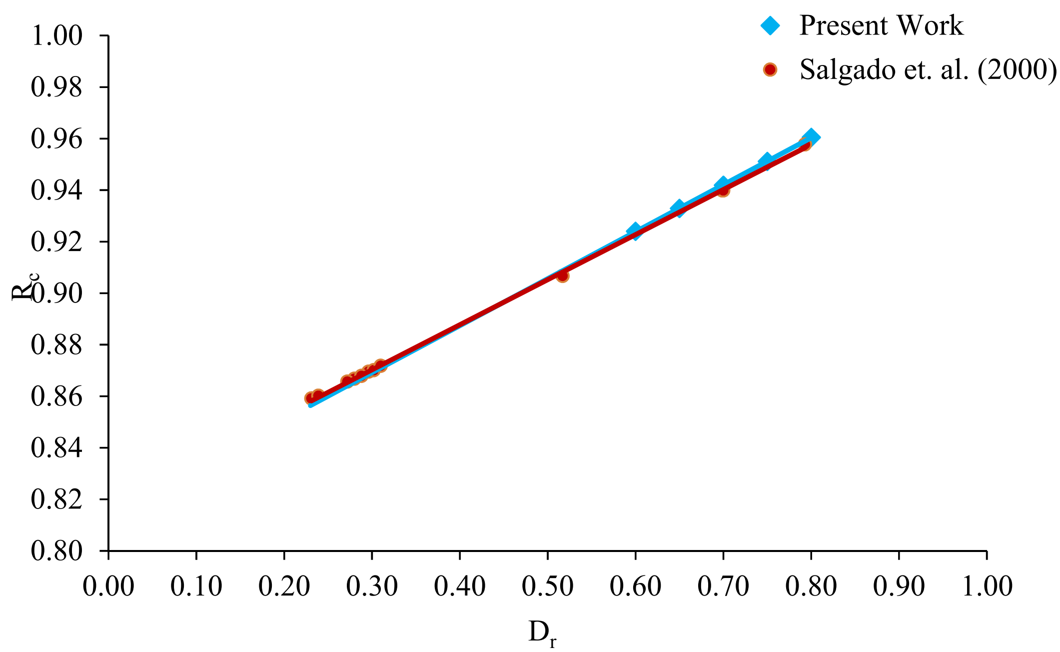


Fig. 5.8 (c) Variation of R_c vs. D_r at 10 percent silt content

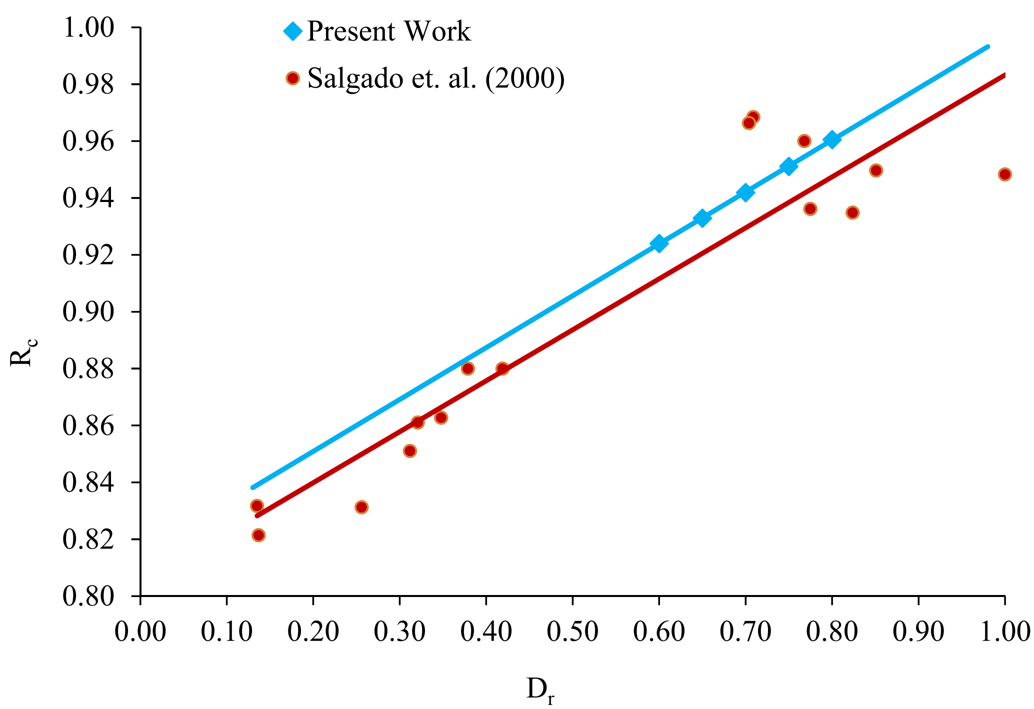


Fig. 5.8 (d) Variation of R_c vs. D_r at 15 percent silt content

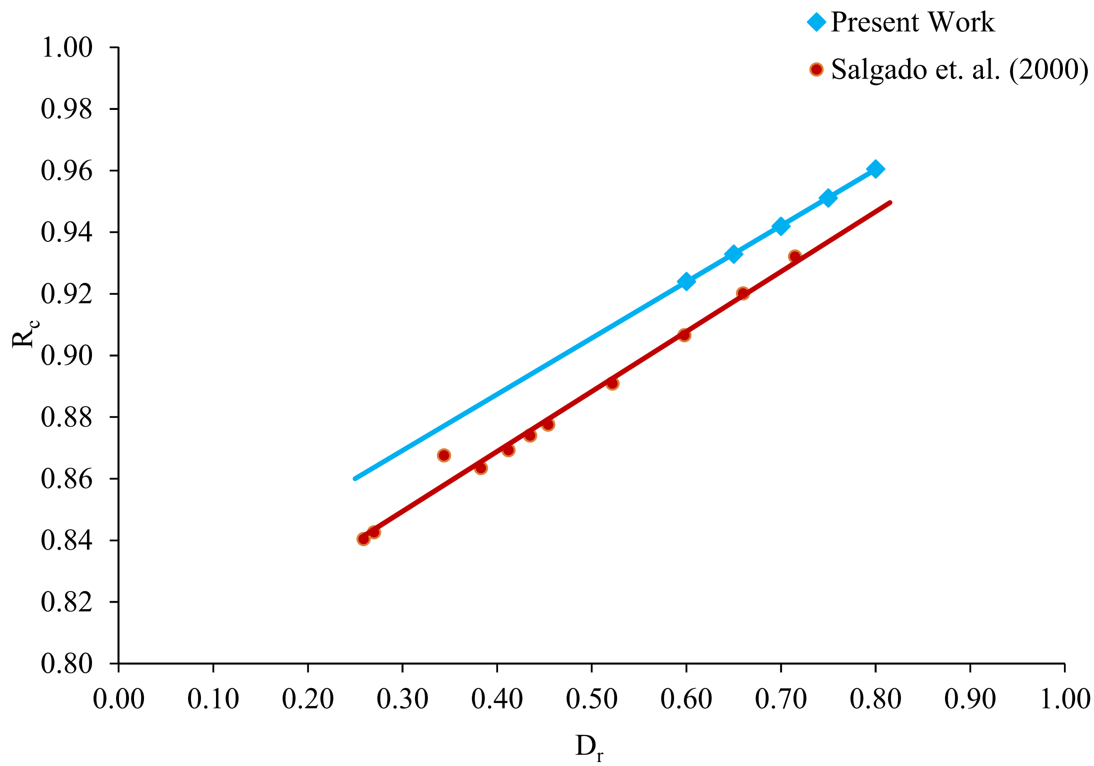


Fig. 5.8 (e) Variation of R_c vs. D_r at 20 percent silt content

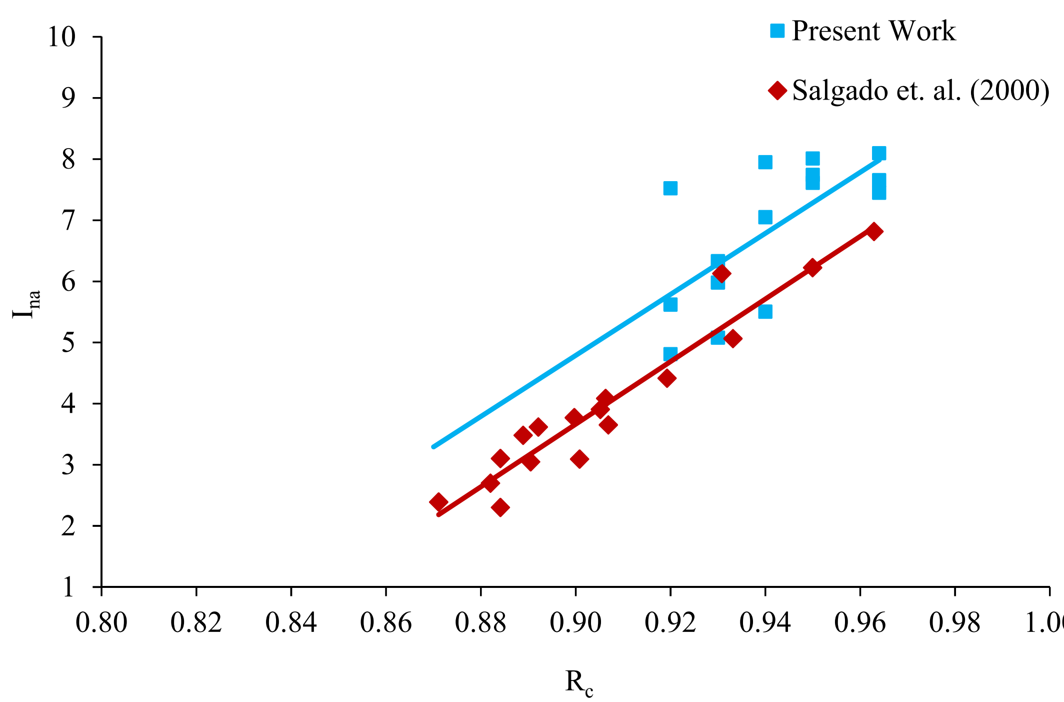


Fig. 5.9 (a) Variation of I_{na} vs. R_C without silt content

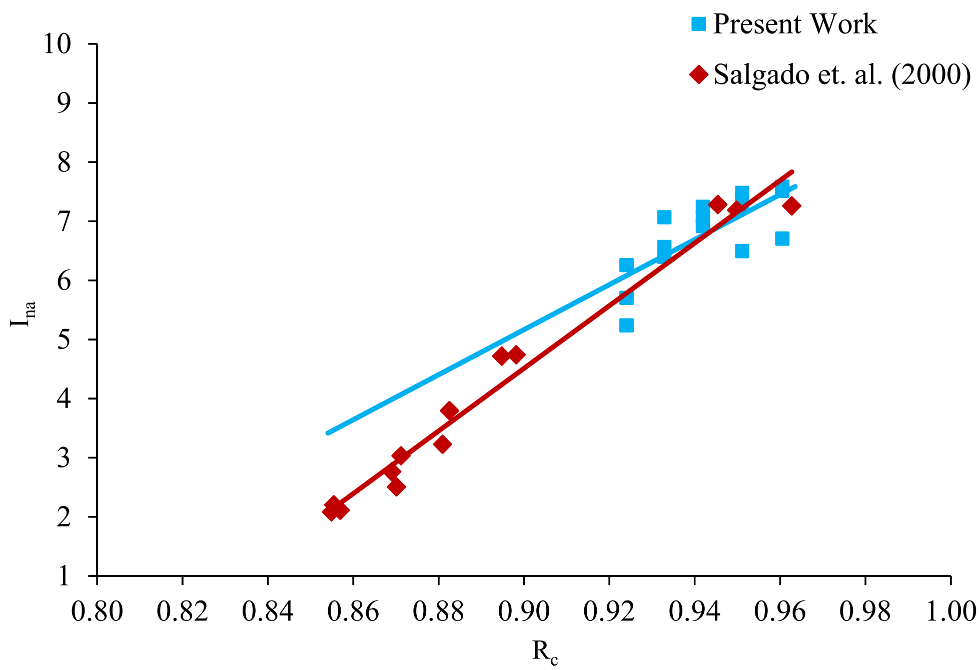


Fig. 5.9 (b) Variation of I_{na} vs. R_C at 5 percent silt content

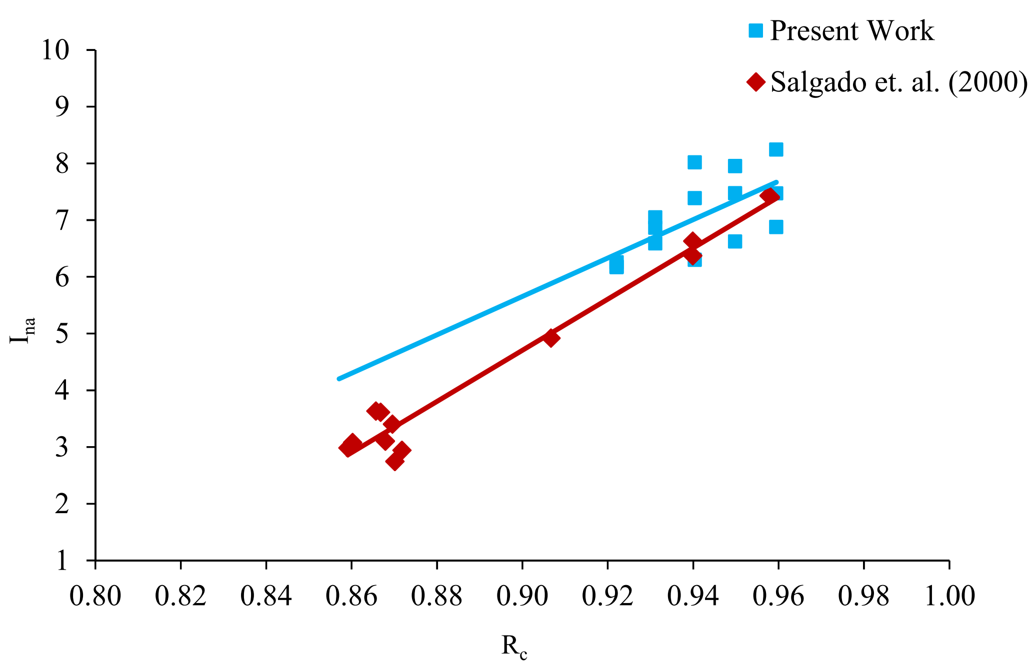


Fig. 5.9 (c) Variation of I_{na} vs. R_C at 10 percent silt content

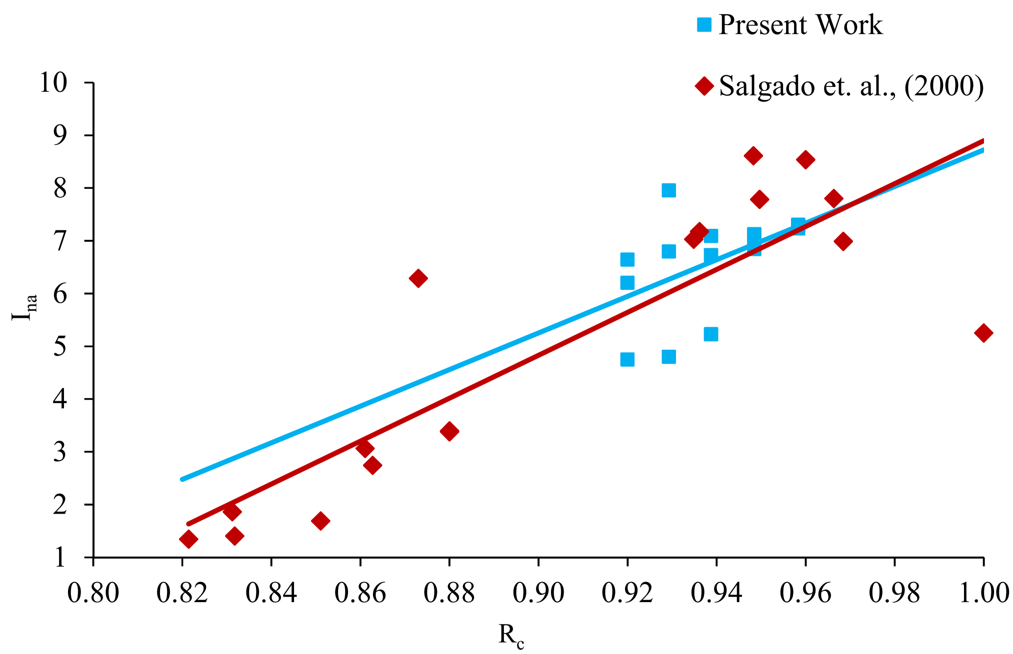


Fig. 5.9 (d) Variation of I_{na} vs. R_C at 15 percent silt content

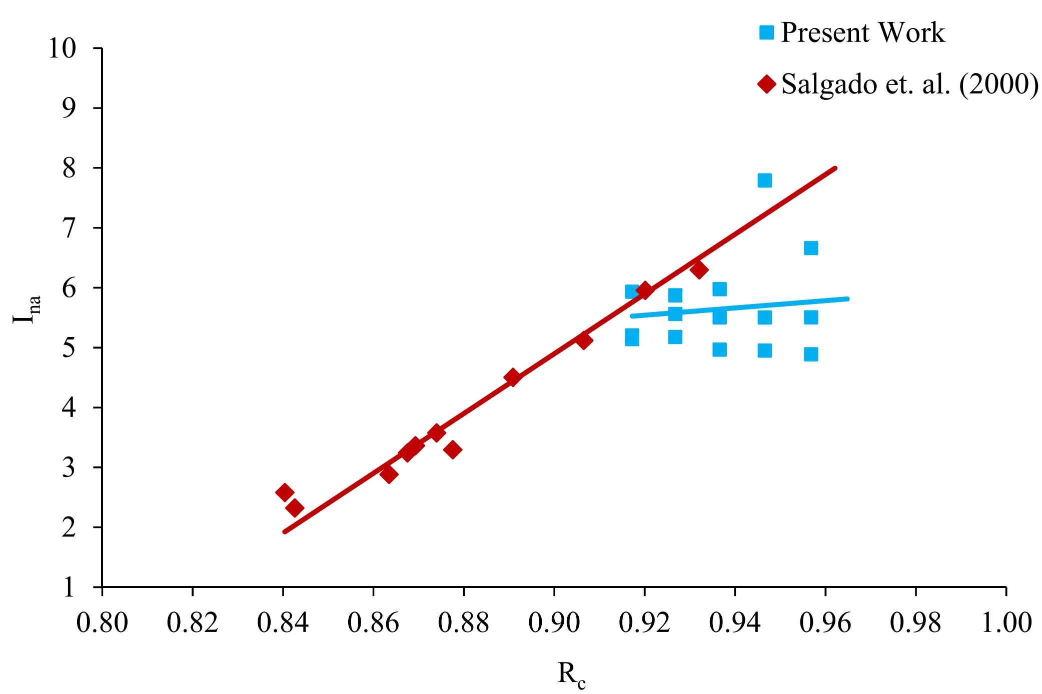


Fig. 5.9 (e) Variation of I_{na} vs. R_C at 20 percent silt content

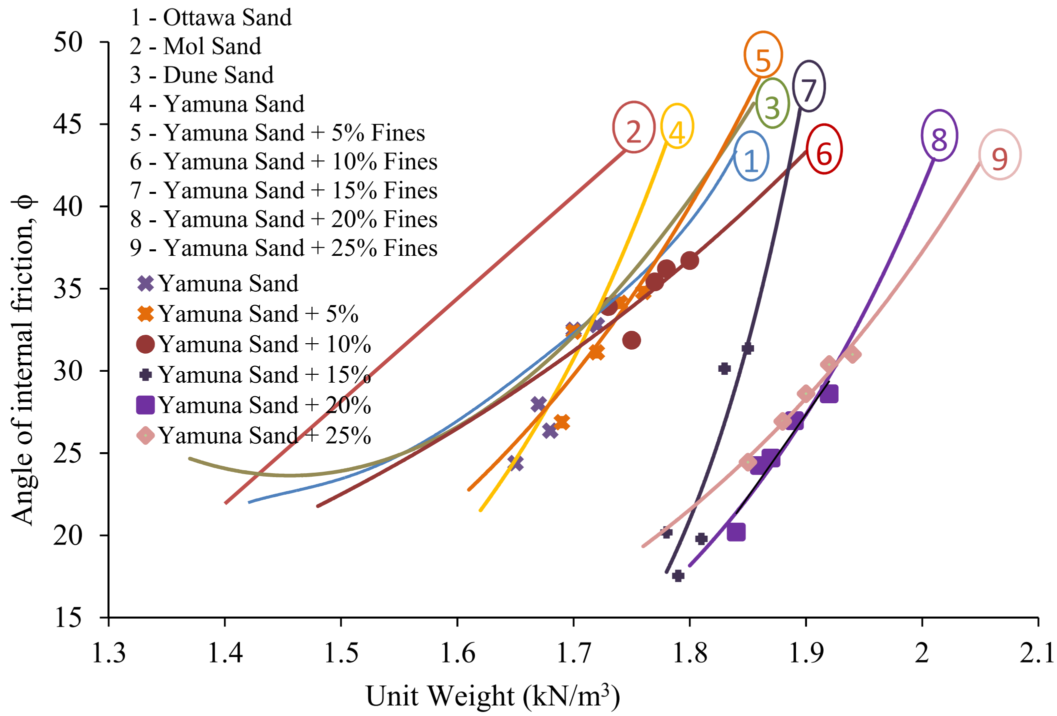


Fig. 5.10 Variation of unit weight with angle of internal friction for few sands

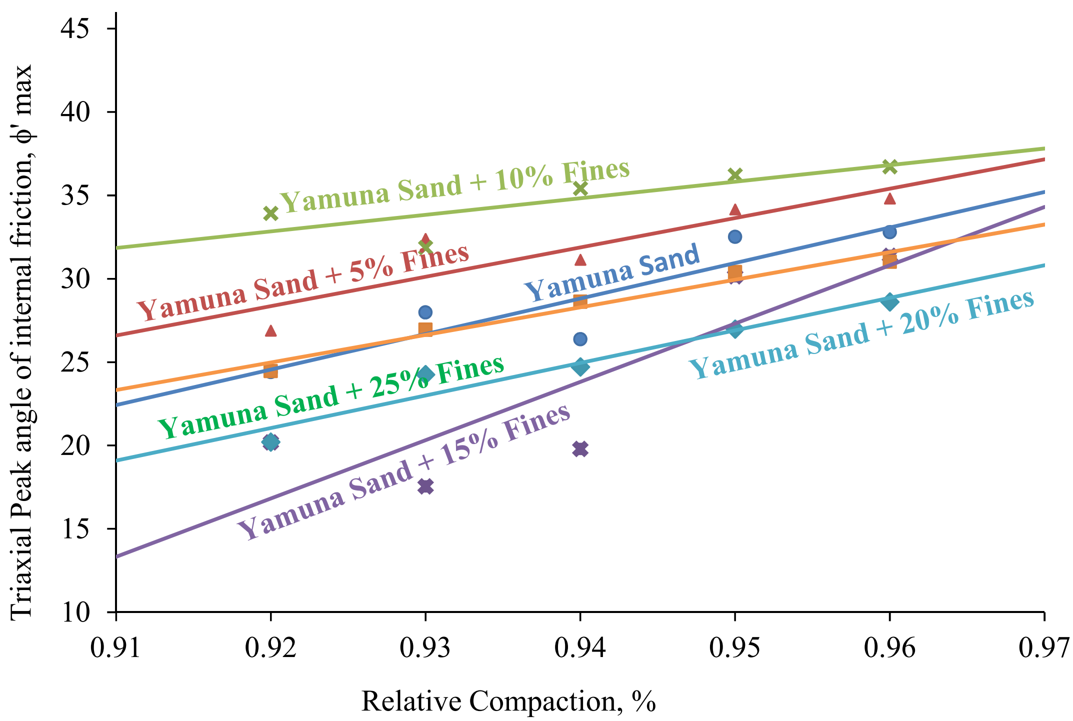


Fig. 5.11 Variation of peak friction angle with relative compaction of Yamuna sand with varied proportion of fines

Table 5.9 Dilatancy parameters with reference to relative compaction for silty Yamuna sand

Silt (%)	D_{50} (mm)	Best Fit		Tread line with R_{af} = 40	Tread line with R_{af} = 30	Tread line with R_{af} = 25
		Q_{af}	R_{af}	Q_{af}	Q_{af}	Q_{af}
0	0.225	49.883	40.105	49.77	39.14	33.83
5	0.224	38.091	29.115	49.64	39.03	33.72
10	0.222	33.863	24.821	49.99	39.36	34.05
15	0.215	34.700	25.977	49.63	38.98	33.66
20	0.213	10.689	4.372	48.71	38.03	32.70
25	0.208	5.023	1.094	48.73	38.09	32.77

Table 5.10 Compressibility classifier of Yamuna sand with fines based on volumetric strain

Sand (S)	Silt (M)	R_c	D_m	σ_3	σ_D (σ_1-σ_3)	Mean pressure (σ_1+2σ_3)/3	e_s	$\lvert e_v \rvert$	$\lvert e_v/e_s \rvert$	Classifier [a]
S	0	0.92	0.225	200	35.68	211.89	1	0.18	0.18	Hc
			0.225	200	123.25	241.08	2	0.40	0.20	Hc
			0.225	200	198.08	266.03	3	0.46	0.15	Hc
			0.225	200	236.58	278.86	4	0.20	0.05	Sd
S	0	0.93	0.225	200	42.81	214.27	1	0.22	0.22	Hc
			0.225	200	200.72	266.91	2	0.44	0.22	Hc
			0.225	200	264.11	288.04	3	0.42	0.14	Hc
			0.225	200	288.01	296.00	4	0.15	0.04	Sd
S	0	0.94	0.225	200	46.38	215.46	1	0.23	0.23	Hc
			0.225	200	179.59	259.86	2	0.44	0.22	Hc
			0.225	200	236.30	278.77	3	0.40	0.13	Hc
			0.225	200	267.44	289.15	4	0.20	0.05	Sd
S	0	0.95	0.225	200	64.22	221.41	1	0.22	0.22	Hc
			0.225	200	197.20	265.73	2	0.45	0.23	Hc
			0.225	200	236.30	278.77	3	0.45	0.15	Hc
			0.225	200	270.87	290.29	4	0.16	0.04	Sd
S	0	0.96	0.225	200	99.90	233.30	1	0.21	0.21	Hc
			0.225	200	204.24	268.08	2	0.42	0.21	Hc
			0.225	200	243.26	281.09	3	0.43	0.14	Hc
			0.225	200	260.58	286.86	4	0.16	0.04	Sd
S	5	0.92	0.224	200	53.52	217.84	1	0.17	0.17	Hc
			0.224	200	183.11	261.04	2	0.35	0.18	Hc
			0.224	200	250.21	283.40	3	0.20	0.07	Hc
			0.224	200	281.16	293.72	4	0.33	0.08	Sd
S	5	0.93	0.224	200	17.84	205.95	1	0.15	0.15	Hc

Sand (S)	Silt (M)	R_c	D_m	σ_3	σ_D $(\sigma_1-\sigma_3)$	Mean pressure $(\sigma_1+2\sigma_3)/3$	e_s	$\|e_v\|$	$\|e_v/e_s\|$	Classifier [a]
			0.224	200	109.16	236.39	2	0.42	0.21	Hc
			0.224	200	201.55	267.18	3	0.29	0.10	Sd
				200	246.87	282.29	-	-	-	
S	5	0.94	0.224	200	53.52	217.84	1	0.15	0.15	Hc
			0.224	200	183.11	261.04	2	0.43	0.22	Hc
			0.224	200	253.68	284.56	3	0.20	0.07	Sd
				200	284.59	294.86	-	-	-	-
S	5	0.95	0.224	200	74.92	224.97	1	0.15	0.15	Hc
			0.224	200	200.72	266.91	2	0.38	0.19	Hc
			0.224	200	264.11	288.04	3	0.55	0.18	Hc
			0.224	200	294.87	298.29	4	0.48	0.12	Sd
S	5	0.96	0.224	200	92.76	230.92	1	0.07	0.07	Hc
			0.224	200	211.28	270.43	2	0.24	0.12	Hc
			0.224	200	278.01	292.67	3	0.48	0.16	Hc
			0.224	200	294.87	298.29	4	0.47	0.12	Sd
S	10	0.92	0.222	200	35.68	211.89	1	0.15	0.15	Hc
			0.222	200	176.07	258.69	2	0.45	0.23	Hc
			0.222	200	250.21	283.40	3	0.20	0.07	Hc
				200	277.73	292.58	-	-	-	-
S	10	0.93	0.222	200	10.70	203.57	1	0.13	0.13	Hc
			0.222	200	98.60	232.87	2	0.35	0.18	Hc
			0.222	200	208.50	269.50	3	0.18	0.06	Sd
				200	257.16	285.72	-	-	-	
S	10	0.94	0.222	200	28.54	209.51	1	0.05	0.05	Hc
			0.222	200	158.46	252.82	2	0.24	0.12	Hc
			0.222	200	271.06	290.35	3	0.05	0.02	Sd
				200	325.73	308.58	-	-	-	
S	10	0.95	0.222	200	89.19	229.73	1	0.07	0.07	Hc
			0.222	200	211.28	270.43	2	0.18	0.09	Hc
			0.222	200	284.96	294.99	3	0.38	0.13	Hc
			0.222	200	305.16	301.72	4	0.37	0.09	Sd
S	10	0.96	0.222	200	107.03	235.68	1	0.08	0.08	Hc
			0.222	200	228.89	276.30	2	0.18	0.09	Hc
			0.222	200	298.86	299.62	3	0.37	0.12	Hc
			0.222	200	308.59	302.86	4	0.38	0.10	Sd
S	15	0.92	0.215	200	10.70	203.57	1	0.09	0.09	Hc
			0.215	200	116.21	238.74	2	0.36	0.18	Hc
			0.215	200	187.65	262.55	3	0.03	0.01	Sd
				200	226.30	275.43	-	-	-	Hc
S	15	0.93	0.215	200	24.97	208.32	1	0.15	0.15	Hc

Sand (S)	Silt (M)	R_c	D_m	σ_3	σ_D $(\sigma_1-\sigma_3)$	Mean pressure $(\sigma_1+2\sigma_3)/3$	e_s	$\lvert e_v \rvert$	$\lvert e_v/e_s \rvert$	Classifier [a]
			0.215	200	116.21	238.74	2	0.45	0.23	Hc
			0.215	200	191.13	263.71	3	0.10	0.03	Sd
				200	212.58	270.86	-	--	-	
S	15	0.94	0.215	200	49.95	216.65	1	0.16	0.16	Hc
			0.215	200	172.55	257.52	2	0.32	0.16	Hc
			0.215	200	222.40	274.13	3	0.10	0.03	Sd
				200	253.73	284.58	-	-	-	
S	15	0.95	0.215	200	82.06	227.35	1	0.05	0.05	Hc
			0.215	200	186.63	262.21	2	0.22	0.11	Hc
			0.215	200	239.78	279.93	3	0.45	0.15	Hc
			0.215	200	264.01	288.00	4	0.55	0.14	Hc
S	15	0.96	0.215	200	114.17	238.06	1	0.15	0.15	Hc
			0.215	200	207.76	269.25	2	0.39	0.20	Hc
			0.215	200	243.26	281.09	3	0.48	0.16	Hc
			0.215	200	270.87	290.29	4	0.28	0.07	Sd
S	20	0.92	0.213	200	21.41	207.14	1	0.16	0.16	Hc
			0.213	200	35.21	211.74	2	0.38	0.19	Hc
			0.213	200	62.55	220.85	3	0.04	0.01	Hc
				200	78.86	226.29	-	-	-	
S	20	0.93	0.213	200	14.27	204.76	1	0.08	0.08	Hc
			0.213	200	21.13	207.04	2	0.36	0.18	Hc
			0.213	200	90.35	230.12	3	0.00	0.00	Nc
				200	130.29	243.43	-	-	-	
S	20	0.94	0.213	200	32.11	210.70	1	0.05	0.05	Hc
			0.213	200	45.78	215.26	2	0.39	0.20	Hc
				200	90.35	230.12	-	-	-	
				200	130.29	243.43	-	-	-	
S	20	0.95	0.213	200	14.27	204.76	1	0.05	0.05	Hc
			0.213	200	56.34	218.78	2	0.22	0.11	Hc
			0.213	200	66.03	222.01	3	0.45	0.15	Hc
			0.213	200	68.57	222.86	4	0.53	0.13	Sd
S	20	0.96	0.213	200	28.54	209.51	1	0.18	0.18	Hc
			0.213	200	42.26	214.09	2	0.20	0.10	Hc
			0.213	200	79.93	226.64	3	0.70	0.23	Hc
			0.213	200	113.15	237.72	4	1.19	0.30	Sd
S	25	0.92	0.208	200	14.27	204.76	1	0.06	0.06	Hc
			0.208	200	28.17	209.39	2	0.35	0.18	Hc
			0.208	200	55.60	218.53	3	0.04	0.01	Sd
				200	102.86	234.29	-	-	-	
S	25	0.93	0.208	200	28.54	209.51	1	0.06	0.06	Hc

Sand (S)	Silt (M)	R_c	D_m	σ_3	σ_D $(\sigma_1-\sigma_3)$	Mean pressure $(\sigma_1+2\sigma_3)/3$	e_s	$\lvert e_v \rvert$	$\lvert e_v/e_s \rvert$	Classifier [a]
			0.208	200	42.26	214.09	2	0.39	0.20	Hc
				200	79.93	226.64	-	-	-	
				200	150.86	250.29	-	-	-	
S	25	0.94	0.208	200	28.54	209.51	1	0.08	0.08	Hc
			0.208	200	28.17	209.39	2	0.24	0.12	Hc
			0.208	200	97.30	232.43	3	0.00	0.00	Nc
				200	144.01	248.00				
S	25	0.95	0.208	200	32.11	210.70	1	0.06	0.06	Hc
			0.208	200	35.21	211.74	2	0.25	0.13	Hc
			0.208	200	69.50	223.17	3	0.50	0.17	Hc
			0.208	200	171.44	257.15	4	0.53	0.13	Hc
S	25	0.96	0.208	200	35.68	211.89	1	0.07	0.07	Hc
			0.208	200	35.21	211.74	2	0.23	0.12	Hc
			0.208	200	121.63	240.54	3	0.47	0.16	Hc
			0.208	200	164.58	254.86	4	0.50	0.13	Hc

[a][Hc-hardening contractile behaviour; Nc-non-contractile behaviour; Sd-softening dilative behaviour]

6.0 General

The present study considers the non-linearity in the strength behaviour of sand with varied proportions of fines. Based on the values of non-linear shear strength parameters ($Q_{af,}$ R_{af}) for Yamuna sand containing varied proportion of fines, engineering implication such as bearing capacity, the shear strength and the liquefaction potential are evaluated. The bearing capacity is the most important consideration which governs the design of foundation. Very soft silty sand and low density fills materials namely ash fills [Trivedi and Sud, 2005] and highly jointed rock masses containing silty clayey gouge material [Trivedi and Arora, 2007] are often unable to bear the load transferred from the super structure to the foundation. The method of improving the bearing capacity of silty clay soil with thin sand layer on top and placing geogrids at different depths were found by different investigators [Gassler, 1990; Gill et al., 2012; Kolay et al., 2013]. Civil engineering projects such as buildings, bridges dams and roadways require detailed sub-surface information as part of the design process [Burland et al., 1977; Cho et al., 2006]. Bearing capacity is affected by various factors namely strength parameters, internal friction, presence of fines, and shape and size of the footings which induce non-linearity in its strength and deformation behaviour.

6.1 Bearing capacity of soil

Bearing capacity is defined as the power of foundation soil to resist the forces from the superstructure without undergoing shear failure or excessive settlement. Foundation soil is that portion of ground which is subjected to additional stresses when foundation and superstructure are constructed on the ground. The following are some important terminologies related to bearing capacity of soil.

(a) Ultimate bearing capacity (q_{ult})**:** It is the maximum pressure that a foundation soil can withstand without undergoing shear failure.

(b) Net ultimate bearing capacity (q_n): It is the maximum extra pressure (in addition to initial overburden pressure) that a foundation soil can withstand without undergoing shear failure. It is represented by,

$q_n = q_{ult} - q_o$

Here, q_o represents the overburden pressure at foundation level and is equal to γD for level ground without surcharge where γ is the unit weight of soil and D is the depth to the foundation bottom from ground level.

(c) Safe bearing capacity (q_s): It is the safe extra load the foundation soil is subjected to in addition to initial overburden pressure.

$q_s = \frac{q_{ult}}{F} + q_o$ here 'F' denotes factor of safety.

(d) Allowable bearing Pressure (q_a): It is the maximum pressure the foundation soil is subjected, considering both shear failure and settlement criteria.

6.2 Evaluation of bearing capacity of Yamuna sand with fines

Methods of evaluation of bearing capacity of shallow foundation have been provided by many researchers in the past. The correct evaluation of ultimate bearing capacity of soil is a very important aspect for the overall stability and safety of any structure. The safe life of every structure depends on the accuracy and correctness of its foundation design which in turn is dependent on the soil bearing capacity. The initial contributions to evaluate the allowable bearing pressure of soil were made by Prandtl (1920) and Resissner (1924). They obtained analytical closed form solutions for ultimate bearing pressure for the case of a strip footing on weightless semi-infinite space. Terzaghi (1943) proposed the first comprehensive bearing capacity analysis for the case of strip footing with rough base for a frictional cohesive soil using limit equilibrium method. The experiments were performed on foundation of small-scale as developed by Meyerhof (1955, 1965); de Beer (1965) and Vesic (1973). In most of these experiments limit equilibrium method was used for the evaluation of ultimate bearing capacity of shallow foundation with rough base for a Mohr coulomb soil. Limit analysis approach was used for the evaluation of bearing capacity factors for rough and smooth footings [Feda, 1961; Chen, 1975]. The use of small sized plate in the field experiment as compared to large size of actual foundation resulted in the error in the estimation of actual bearing capacity and is called as scale effect. However when a large-scale tests were also performed, it also indicated the inability of these solutions to predict actual field behaviour [Muhs, 1965; Fellenius and Altaee, 1994]. This variation between the field tests and the actual field results is technically known as scale error. With the advancement of technology over the past many years and availability of latest art of the instruments and an advanced understanding of the problem, the fairly accurate solutions to predict the bearing capacity has been proposed by many eminent researcher [Yamaguchi et al., 1976; Kutter et al., 1988].

The majority of the solutions developed as a result of these studies have been difficult to implement in practice due to problems associated with obtaining required material strength parameters. Recent studies by [Perkins and Madson, 1996a] provided additional data for development of a design method for evaluation of bearing capacity of silty sand. The relationship between strength and dilation in granular soils proposed by Bolton (1986) has improved the evaluations of the bearing capacity. It is difficult to predict accurately the bearing capacity of shallow foundations on cohesion less soils due to scale effects. The scale effects are predominant due to the nonlinearity in the strength behaviour of the granular soil and the phenomenon of progressive failure. The effect of non-linear strength behaviour can be taken care of by using non-linear non-dimensional shear strength parameters ($Q_{af,}$ R_{af}) evaluated earlier in the present work using strength-dilatancy relationships. It is also observed that the effect of progressive failure on ultimate bearing capacity can be accounted in terms of the relative dilatancy index, first proposed by Perkins and Madson (2000) an**d** Trivedi and Sud (2005). A methodology to evaluate bearing capacity based on these considerations for Yamuna sand has been devised in the present work. This approach reduces the need for extensive laboratory testing and fairly accurate predictions of bearing capacity can be made as compared to conventional methods. The basic bearing capacity equation of soil is given by

$$q_{ult} = cN_c + q_o\,Nq + 0.5\,\gamma\,B\,N\gamma \qquad (6.1)$$

for a cohesion less soil Eq. (4.1) reduces to

$$q_{ult} = q_o\,Nq + 0.5\,\gamma\,B\,N\gamma \qquad (6.1a)$$

where $q_o = \gamma D$ is the surcharge existing at the footing base expressed in terms of an effective stress; γ = bulk density of soil; and B = footing width, D= the depth of bottom of foundation below ground level; N_c, Nq and Nγ are the bearing capacity factors estimated based on peak friction angle.

The bearing capacity of soil depends on many factors. Terzaghi's (1943) bearing capacity equation does not take in to consideration all the factors. Brinch Hansen (1970) and several other researchers have provided a comprehensive equation for the determination of bearing capacity called generalised bearing capacity equation considering the almost all the factors mentioned above. The equation for ultimate bearing capacity from the comprehensive theory is given by

$$q_{ult} = cN_c\,s_c\,d_c\,i_c + q_o\,s_q\,d_q\,i_q\,Nq + 0.5\,\gamma\,B\,N\gamma\,s_\gamma\,d_\gamma\,i_\gamma \qquad (6.2)$$

Equations are valid for shape factors (s_c, s_q, s_γ), depth factors (d_c, d_q, d_γ) and load inclination factors (i_c, i_q, i_γ). For cohesion less soil Eq. (6.2) reduces to

$$q_{ult} = q_o\, s_q\, d_q\, i_q\, Nq + 0.5\, \gamma\, B\, N\gamma\, s_\gamma\, d_\gamma\, i_\gamma \qquad (6.3)$$

It is observed that bearing capacity does not increase linearly with footing width as predicted in Eq. (6.1 to 6.3). This is due to the scale effect and was reviewed and discussed by de Beer (1965). The same observation has also been made by Yamaguchi et al. (1976); Kutter et al. (1988). The implication of scaling effect indicates that bearing capacity factors determined from small-scale laboratory tests will generally be on conservative side when extrapolated to field-size footings. The observed scale effect is partly due to the nonlinear shape of the soil failure envelope [de Beer, 1965], which results in a secant measure of the friction angle that decreases with increasing mean normal effective stress. The progressive failure effects based on the non-linear strength behaviour of the granular soil was applied in relation to the relative dilatancy of coal ash to evaluate the bearing capacity of granular fills [Trivedi and Sud, 2005]. Different methods were proposed to account for nonlinear strength behaviour within the framework of classical rigid-plasticity [Meyerhof, 1950; de Beer, 1965; Kutter et al., 1988; Perkins, 1995 a, b]. The correctness of these methods depends upon the accuracy of collection of undisturbed sample of sand for triaxial testing and plot the nonlinear material failure envelope correctly. Previous investigator [Bishop and Hankel, 1972] stressed the need for adjusting the angle of friction from axisymmetric tri axial tests to the proper strain condition. Bowles in his book suggested that the friction angle from triaxial tests should be increased by 5 degree from axisymmetric conditions to plane strain conditions. Strength and dilatancy concepts suggest that the difference between triaxial and plane strain friction angles is related to the dilatational characteristics of the sand. The rate at which a material dilates in shear is primarily dependent on observed peak friction angle. Bolton's (1986) stress-dilatancy equation in terms of peak and constant volume friction angles and an angle of dilatancy at peak strength can be expressed by a common term I_{af} called relative dilatancy.

$$\phi_{peak} - \phi_c = A\, I_{af} \qquad (6.4)$$

where

$$I_{af} = R_c\,(Q_{af} - \ln 100\, p' / P_A) - R_{af} \qquad (6.5)$$

A = empirical constant having a value of 3 for triaxial strain conditions and takes care of scaling effect, inclination factor and shape factor, R_c is relative compaction defined as a ratio of natural dry unit weight to maximum unit weight, expressed as a number between 0 and 1,

p' is mean effective stress at peak strength in kPa, P_A is reference stress (100 kPa) in the same units as p', Q_{af} and R_{af} are non-dimensional and non-linear shear strength parameters. It has been observed that the dilatancy angle at peak strength was a function of the relative compaction R_c, and mean normal effective stress confinement p' and strain condition. In Eq. (6.5), Q_{af} and R_{af} are non-linear shear strength parameters for Yamuna sand evaluated in the present work for varied proportion of fines. Depth factor if taken gives conservative results hence can be ignored. Eq. (6.4) and Eq. (6.5) indicate that the amount by which ϕ_{peak} exceeds ϕ_c and depends on the dilatational characteristics of the soil at peak strength represented by I_{af}. Eq. (6.5) shows that I_{af} increases as relative compaction increases and as mean normal confining stress decreases. The implication for the use of Eq.(6.4) and (6.5) in a bearing capacity evaluation require the correct estimation of the values of relative compaction and the constant volume friction angle since the values of Q_{af} and R_{af} have already been estimated in the present work. As such, the need for extensive triaxial testing of undisturbed samples is reduced. The determination of R_c can easily be done by knowing maximum and natural dry density of Yamuna sand, whereas ϕ_c may either be estimated by knowing the mineralogy of the sand and silt or by performing a single shear test on a loose, reconstituted sample. Step by step method to evaluate bearing capacity is illustrated below:

1. Assume suitable value of p'. Evaluate the relative dilatancy I_{af} using Eq. (6.5) knowing the values of Q_{af} and $R_{af.}$
2. Find the value of ϕ_{peak} from Eq. (6.4) since ϕ_c and I_{af} is known.
3. Knowing the value of ϕ_{peak} bearing capacity factor N_q and $N\gamma$ can be evaluated.
4. Find the ultimate bearing capacity (q_{ult}) from Eq. (6.1a).
5. Find the ratio of p'/q_{ult}
6. Find out the value of p'/q_{ult} from Fig. 6.2 to Fig. 6.5 (Present work) corresponding to ϕ_{peak} and R_c evaluated earlier.
7. If the value of $\frac{p'}{\text{qult}}$ is same, then calculate the value of ultimate bearing capacity , else repeat the step 1 to 6 by assuming another suitable value for p' till the values from step 5 and step 6 are compatible. Fig. 6.1 shows the plot between percentage fine and Q_{af} at different intercept enabling us to interpolate values of non-linear shear strength parameters Q_{af} at any given percentage fine for a suitable intercept directly without undergoing the detailed procedure of repeating the triaxial tests. Fig. 6.2 to 6.5 shows the plot between $p'/q_{ult\text{-}peak}$ and

peak friction angle for varied value of shear strength parameters Q_{af} and $R_{af.}$ at different relative compaction.

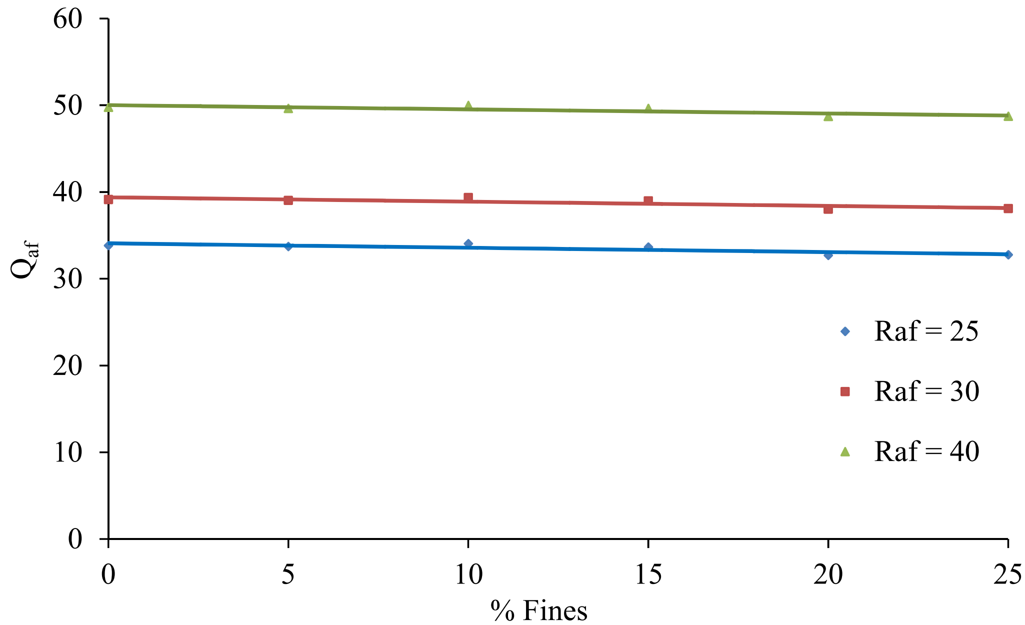


Fig. 6.1 Plot showing variation of Q_{af} with percent fine at varied R_{af}

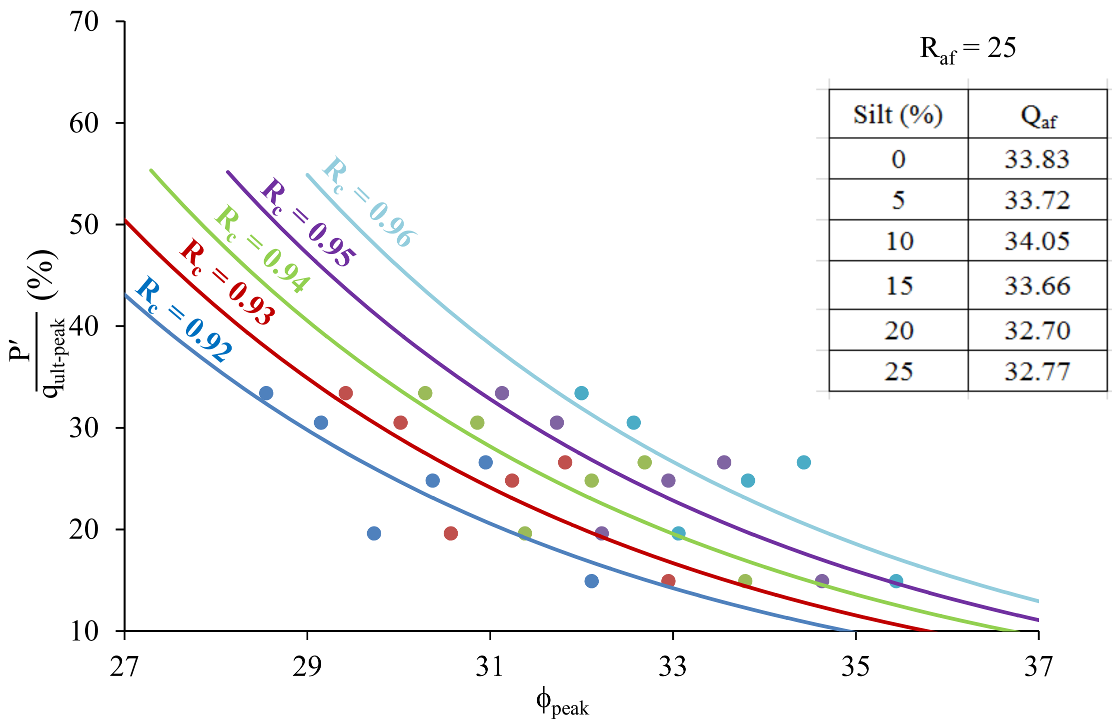


Fig. 6.2 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle p′ at varied relative compaction for $R_{af} = 25$

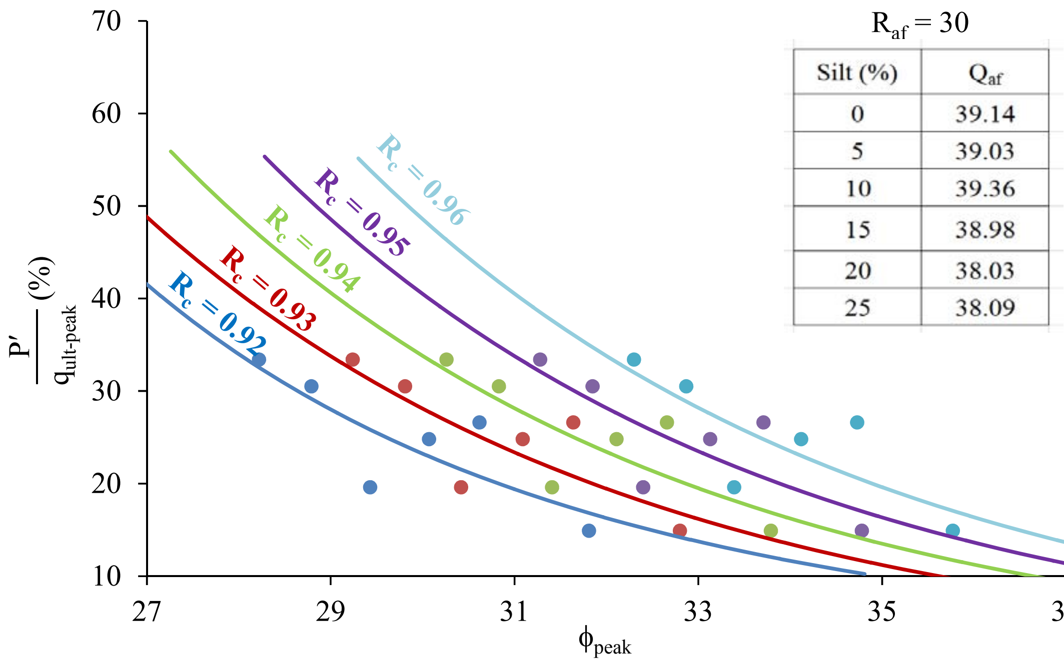


Fig. 6.3 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for Raf= 30

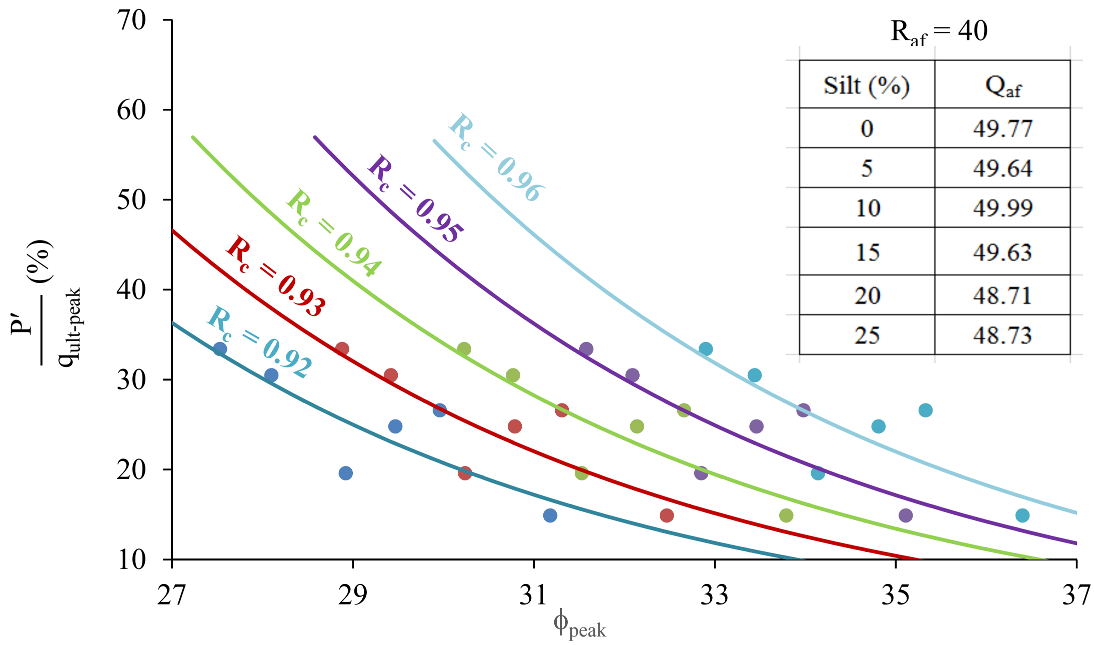
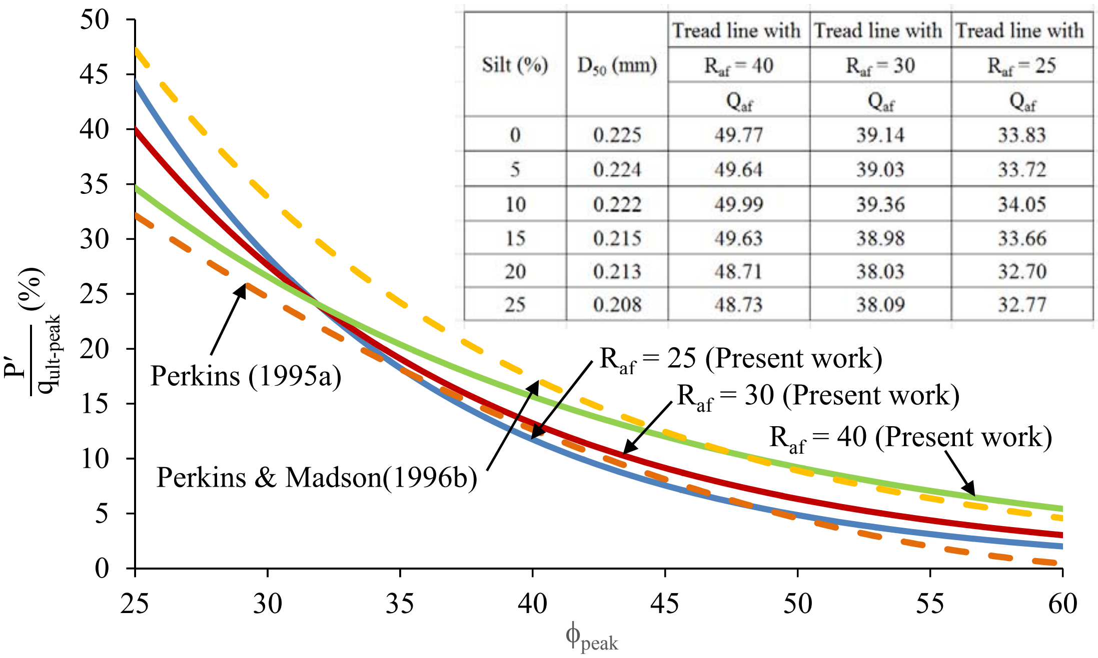


Fig. 6.4 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for R_{af} = 40



Silt (%)	D_{50} (mm)	Tread line with R_{af} = 40 Q_{af}	Tread line with R_{af} = 30 Q_{af}	Tread line with R_{af} = 25 Q_{af}
0	0.225	49.77	39.14	33.83
5	0.224	49.64	39.03	33.72
10	0.222	49.99	39.36	34.05
15	0.215	49.63	38.98	33.66
20	0.213	48.71	38.03	32.70
25	0.208	48.73	38.09	32.77

Fig. 6.5 Variation of $p'/q_{ult\text{-}peak}$ with peak friction angle (ϕ_{peak}) at varied relative compaction for Few Sands

6.3 Liquefaction

Liquefaction is defined as the sudden loss of strength which occurs during seismic loading. Liquefaction is normally associated with cohesion less soils and dynamic loadings but the case studies conducted have established that it may occur in other types of soils under dynamic loadings [Ishihara, 1984; El Hosri et al., 1984; Chang, 1990; Idriss, 1991; Carraro et al., 2003; Cubrinovski et al., 2010]. Passage of seismic waves during earthquake through saturated loose sand deposits results in development of cyclic shear stresses. These stresses cause progressive build-up of pore water pressure [Ladd et al., 1982; Robertson et al. 1995; Cho et al., 2006;]. Cohesion less soils of loose and medium density have a tendency to compact under vibrations leading to decrease in the inter-granular space [Roscoe et al., 1958; Erten and Maher, 1995]. This tendency for volume decrease gives rise to increase in pore water pressure which in turns increases the pressure on inter-particle soil grain contact [Rogers et al., 1991; Prakash and Puri 1998; 2010]. This progressive build-up of pore water pressure may eventually become large enough resulting in a considerable loss of shear strength resulting in large deformations and consequent failure. Liquefaction potential is defined as the capacity of soil to resist liquefaction. Hence the evaluation of liquefaction potential of soils at any site requires determination of two sets of parameters namely the cyclic stress ratio (CSR) due to seismic action and soil properties which describe the soil resistance under these loads termed as cyclic resistance ratio (CRR). The liquefaction of sands during earthquakes has occurred throughout in the recorded history, and even before that, however scientific research into the subject began in the early 1960. Since the 1964 Anchorage (Alaska) and Nigata (Japan) earthquakes, great efforts have been made by many researchers to understand the mechanisms behind liquefaction and the factors responsible that make soils susceptible to liquefaction [Pitman, et al., 1994]. There are two types of liquefaction phenomena [Finn, 1991, 1993; Figueroa et al., 1995].

(a) Flow liquefaction

(b) Cyclic mobility

Flow liquefaction occurs much less frequently than cyclic mobility but its effects are usually far more severe. Cyclic mobility, on the other hand, can occur under a much broader range of soil and site conditions than flow liquefaction. Flow liquefaction will occur when the static shear stress of a soil mass (shear stress required for static equilibrium) is greater than the static shear resistance of the soil in its liquefied state, which produces large permanent deformations. If the loading is monotonic, only the stress ratio matters in triggering flow liquefaction, whereas if the loading is cyclic, it is the cyclically induced excess pore pressure that may be sufficient to cause soil failure under the imposed loadings, producing flow failure. Flow failure is characterized by the sudden nature of its origin, the speed it develops and the movement of liquefied materials to large distances. However cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil but the cyclic shear stress is large enough that the steady-state strength is exceeded momentarily. The difference between static and cyclic induced liquefaction is the way plastic strains are generated. The case of cyclic induced liquefaction, the plastic volumetric strains arise through densification which tends to pack the soil particles close together. The densification thus induced affects any soil from loose to dense sands and even over consolidated clays. In contrast to static induced liquefaction, in cyclic mobility the zone of maximum excess pore pressure generation may not be the loosest soil, but rather the soil that was in the most stressed location. A special case of cyclic mobility is level-ground liquefaction that can occur when cyclic loading is enough to produce high excess pore pressures. Level ground liquefaction failures are caused by the upward flow of water that occurs when seismically induced excess pore pressure dissipate. Since there are no horizontal shear stresses that could drive lateral deformations, level-ground liquefaction can produce large movements known as ground oscillation (excessive vertical settlement and consequent flooding of low-lying land). Level-ground liquefaction may occur well after ground shaking has ceased depending on the time length required to reach hydraulic equilibrium which makes it highly unpredictable. The liquefaction phenomenon is also related to strain softening. The principal of strain softening occurs not only in soil mass but even in rocks. One-dimensional numerical solution has been presented to obtain the ground reaction curve (GRC) for circular tunnels excavated in strain-softening materials [Alonso and Alejano, 2003; Chu and Leong, 2001]

6.3.1 Liquefaction resistance of silty sands: Background

The cyclic behavior of silty sandy soils is at present moderately understood, yet these materials are commonly found in alluvial deposits and hydraulic fill, which have a history of liquefaction during earthquakes [Chillarige et al 1997; Chen and Liao, 1999; Andrews et al., 2000; Amini and Qi, 2000; Youd and Idriss, 2001]. They compared the behavior of stratified and homogeneous silty sands during seismic liquefaction conditions for various silt contents and confining pressures. The silt contents ranged from 10 to 50%, and confining pressures in the range of 50 to 250 KPa were considered. The results indicated that the liquefaction resistances of layered and uniform soils are not significantly different, despite the fact that the soil fabric produced by the two methods of sample preparation is totally different. With advancement of research on liquefaction resistance, it has been established that other factors besides initial density and stress conditions influence liquefaction resistance of a soil. These factors include soil fabric, history of prior seismic straining (a soil mass that has been subjected to prior seismic straining has greater liquefaction resistance than another soil mass, with the same density, without seismic straining) and length of time under sustained pressure (liquefaction resistance increases with the specified length of time). These soil characteristics are destroyed in the process of sampling, making it impossible to test important liquefaction resistance factors in specimens with laboratory testing. Because of these factors, the characterization of liquefaction resistance is now mainly based on in situ test results. Some SPT and CPT based liquefaction potential assessment methods used for sand containing fines have been proposed for various magnitudes of earthquakes. Comparing the two most common tests, SPT and CPT, the SPT is the most commonly used worldwide for liquefaction resistance characterization and has the largest case history database of any in situ test. However, the CPT is becoming a more common test for liquefaction resistance characterization since it is able to detect thin layers since it provides continuous record of penetration resistance (unlike the SPT) of potentially liquefiable soils that may exist. There is a known correlation between SPT and CPT resistances, by supplementing these data it is possible (within certain limits) to expand the database for both tests, especially for the CPT

[Tokimatsu and Yoshimi, 1983; Robertson and Campanella, 1985; Shibata and Teparaksa 1988; Zhou, 1981; Zlatovic and Ishihara, 1995]. The relationship between cyclic stress ratio (CSR) required to trigger liquefaction and the liquefaction resistance based on situ tests can be described graphically (most results are based on historical criteria), where earthquake magnitude and fine content play a very important part. Liquefaction resistance curves for sand containing fines have generally been referred with respect to mean grain size D_{50} and proportion of fines content. Whether liquefaction resistance goes up or down with fines content depends on total void ratio, sand skeleton void ratio and relative density or relative compaction. The liquefaction resistance charts for silty sand have been prepared by Robertson and Campanella (1985), Seed and De Alba (1986), Shibata and Teparaksa (1988), and Stark and Olson (1995). The "Modified Chinese Criteria" Wang (1979), Seed et. al. (1982, 1983, 1985), represent the most widely used criteria for defining potentially liquefiable soils over the last two decades.

These Chinese criteria consider that silty sands are potentially liquefiable type if:
Fraction finer than 0.005 mm $\leq$ 15%
Liquidity index $\leq$ 0.75
Liquid Limit, LL $\leq$ 35%
Natural water content $\geq$ 0.9 LL
Seed et al. (1983) identified two terms, one the cyclic stress ratio (CSR) and other the cyclic resistance ratio (CRR). The CSR is the ratio of the shear stress generated by the earthquake to the vertical effective stress $\sigma v'$ at the desired depth. The CRR is the ratio of the cyclic resistance to liquefaction to $\sigma v'$. Liquefaction at a given depth is expected to occur when CSR>CRR at that depth.

Kuerbis et al. (1988) also used the skeleton void ratio as the basis for comparison of liquefaction testing done on silty sand and observed the same increase in CRR with fines content as observed by Seed and De Alba (1986). They also found that liquefaction resistance progressively decreases as silt content increases up to 21%, for a given value of either void ratio or relative density.

Lade and Yamamuro (1997) studied the static liquefaction of two sands (Nevada and Ottawa), each with two different gradations, deposited with various percentages of non-plastic silt in a very loose state. They found evidence of ''unexpected'' soil behavior. While clean sands are usually increasingly dilatants with increasing relative density and decreasing effective confining stress, the silty sand specimens they tested showed increasing dilatancy with increasing effective confining stress.

Andrews and Martin (2000) suggested based on empirical observations from a few case histories and the relevance of various indices that (a) soils are susceptible to liquefaction if they have <10% finer than 2 μm and LL<32, (b) soils are not susceptible to liquefaction if they have ≥10% finer than 2 μm and LL>32, and (c) further study is required for soils that meet one, but not both, of these criteria.

Polito and Martin (2001) performed cyclic undrained triaxial tests on Yatesville and Monterey sands with non-plastic silt prepared by moist tamping. They found that the liquefaction resistance increases with increasing silt content for a given value of the skeleton void ratio for Yatesville sand; however, this trend was not observed for Monterey sand. Their specimens, prepared at constant void ratio, exhibited a decrease in liquefaction resistance with increasing silt content up to 35–50%, after which the specimens get stronger.

Yamamuro and Covert (2001) performed drained and undrained monotonic and undrained cyclic triaxial tests at different effective confining stresses on very loose Nevada sand with 40% silt content prepared by the dry deposition method. The sample when tested exhibited highly contractive behavior, even at large axial strains, due to the more compressible structure of this material compared with that of clean sand or sand with low silt contents.

Boulanger (2003) has evaluated the effect of overburden stress on liquefaction potential based on state parameters using a theoretical framework that provided consistency between the different components of the design process. Relations between CRR and state parameter for field conditions were subsequently derived from semi empirical liquefaction correlations, and these CRR– state parameter relations is used to calculate the effects of effective over burden pressure on predicted CRR.

To sum up the different interpretation of the majority of the available studies, it is concluded that the addition of non-plastic fines increases the CRR if the relative density is used as the basis for comparison. Addition of non-plastic fines reduces the CRR values at the same void ratio. It has been observed that if fines are present in the sands, the resistance to liquefaction decreases at the same void ratio. However if a sand-fines mixture has the same standard penetration value $(N_1)_{60}$, the addition of fines increases the liquefaction resistance (Seed et al., 1985). The liquefaction susceptibility of a soil with fines depends not only on the amount of fine but also on the nature of the fines [Ishihara, 1985, 1989 and 1993]. Seed and Idriss (1971); Seed et. al. (1983) presented a comprehensive method for evaluation of liquefaction potential of silty sand based on cyclic stress ratio and cyclic resistance ratio determined on the basis of SPT data and the empirical relations. Finally, comparison of these two stresses is used in the estimation of liquefaction susceptibility of the foundation strata.

The detailed procedure for evaluation of liquefaction potential based upon relative compaction is described as follows:

Step 1: Evaluation of CSR [Seed et. al., 1983, 1985]

Calculate Cyclic Stress Ratio (CSR) by Eq.

$$CSR = 0.65 * (\sigma_o / \sigma_o')* (a_{max} / g)* r_d \tag{6.6}$$

a_{max} = maximum surface acceleration proposed by Seed et. al. (1983, 1985) from the SPT data and peak ground acceleration likely to occur at the site for a horizontal ground surface.

r_d = Stress reduction factor

= [1.0 – 0.00765 * h] if h < 9.15 m

= [1.174 – 0.0267 * h] if h = 9.15 m to 23 m

= [0.744 – 0.008 * h] if h = 23.0 m to 30.0 m

= 0.50 if h > 30.0 m

σ_o = initial effective overburden pressure

σ_o' = effective over burden pressure at depth (h)

Step 2 : Evaluation of CRR [Seed et. al., 1983, 1985]

Find Cyclic Resistance Ratio (CRR). It is the capacity of soil to resist liquefaction. CRR is determined using correlation between corrected blow count $(N_1)_{60}$ and CRR for earthquake of magnitude 7.5. $(N_1)_{60}$ is the SPT blow count corrected to an effective overburden pressure of 100kPa and to hammer energy efficiency of 60%. Corrected blow count $(N_1)_{60}$ is determined as follows.

$$(N_1)_{60} = N_m\, C_N\, C_E\, C_B\, C_R\, C_S \tag{6.7}$$

where, N_m = Uncorrected SPT blow count

C_E = Correction factor for hammer energy ratio = 0.75

C_B =Correction factor for borehole dia = 1.05 for 150 mm dia borehole

C_R = Correction factor for rod length =0.75 for 3.0m to 4.0 m

=0.85 for 4.0 m to 6.0 m

=0.95 for 6.0 m to 10.0 m

=1.0 for 10.0 m to 30.0 m

C_S = Correction factor for standard sampler =1.0

Correction factor for effective overburden pressure (C_N) is given by the following relation.

$C_N = (P_a / \sigma_o')^{1/2}$ where P_a = Atmospheric pressure

The value of SPT blow count for soil with fines content (F_C) can be adjusted to the equivalent clean sand value of $(N_1)_{60CS}$ as follows:

$(N_1)_{60CS} = \alpha + \beta\ (N_1)_{60}$ (6.8)

where α and β can be determined as follows.

$\alpha = 0.0$ and $\beta = 1.0$ for FC <= 5.0 %

$\alpha = e_{xp}\ [(1.76 – (190/FC^2)]$ for 5.0 % < FC < 35.0 %

$\beta = [0.99+ (FC^{1.5}/1000)]$

$\alpha = 5.0$ and $\beta = 1.20$ for FC >= 35.0 %

$CRR_{M = 7.5}$ is given by the following equation.

$CRR_{M=7.5}=[1/(34-(N_1)_{60CS})] + [(N_1)_{60CS}/135]+[50/\{10*(N_1)_{60CS} + 45\}^2]–[1/200]$ (6.9)

Hence the CRR for a particular earthquake magnitude is determined as

$CRR = CRR_{M = 7.5} * MSF * K_\sigma$ (6.10)

The MSF value is 1.0 for earthquake of magnitude 7.5. K_σ is taken as 1.

Step 3

Find the factor of safety against liquefaction, FS_L,

$FS_L = CRR/CSR$

The value of CSR and CRR are computed at different depth and depth susceptible to liquefaction is determined. Liquefaction is probable when FS_L is less than 1.0.

6.3.2 Evaluation of liquefaction potential of Yamuna sand based on relative compaction

Liquefaction potential of Yamuna sand with varied proportion of fines have been evaluated on the basis of non-linear shear strength parameters Q_{af} and R_{af} estimated in the present work. Field test conducted for the evaluation of liquefaction potential, consist of determination of SPT blow count and taking out disturbed and un-disturbed sample for the purpose of measurement of density and the proportion of fine content at a specific overburden pressure. The error involved in the evaluation of relative density as discussed in the previous section necessitates the use of relative compaction for evaluation of liquefaction potential at varied mean confining pressure. In the present work on Yamuna sand a simplified approach based on critical value of relative compaction corresponding to critical relative dilatancy ($I_{af} = 0$) has been evaluated. The relative dilatancy of Yamuna sand can be expressed as per Eq. 4.5 of chapter 4 as

$$I_{af} = R_c\,(Q_{af} - \ln 100\,p'/P_A) - R_{af} \qquad (6.11)$$

for critical state putting I_{af} equal to zero Eq. (6.11) reduces to

$$R_{c\text{-critical}} = \frac{R_{af}}{Q_{af} - \ln p'} \qquad (6.12)$$

The I_{af} relation in Eq. (6.11) are used to derive a state parameter ξ_R which is the difference between the field relative compaction (R_c) and the critical state relative compaction ($R_{c\text{-critical}}$) for the field value of p'. The definition of ξ_R is shown in Fig. 6.8 to Fig. 6.11 along with the critical state line produced from the I_{af} relation with varied value of Q_{af} (i.e. critical state corresponds to $I_{af} = 0$). This plot of the critical state line between $R_{c\text{-critical}}$ and effective overburden pressure p' can be drawn for Yamuna sands containing varied proportion of fines. The slope of the critical state line is the limiting values of relative compaction above which there is no liquefaction. Thus the liquefaction potential of Yamuna sand is controlled by non-dimensional, non-linear shear strength parameters (Q_{af} and R_{af}). The state parameter ξ_R ($\xi_R = R_c - R_{c\text{-critical}}$) provides useful correlations to the shear behavior of Yamuna sand. A positive value of ξ_R indicates no liquefaction whereas negative value represents onset of liquefaction. The particular advantage of ξ_R for this study is that it provides a simple co-relation to the cyclic resistance ratio (CRR) of Seed et al. (1983) based on the field SPT values.

Based on results of twenty five reconstituted samples for Yamuna sand with varied proportion of fines at various residential sites located in the Yamuna basin near Delhi, cyclic resistance ratio (CRR) for Yamuna sand having varied proportion of fines is evaluated on the basis of critical value of relative compaction. Correction factor (η_c) for relative compaction is applied on the values of critical relative compaction through an empirical relation given by Eq. (6.13) below.

$$CRR = \eta_c\, R_{c\text{-critical}} \qquad (6.13)$$

where $\eta_c = A\,(p')^B$ (6.14)

is a Relative compaction correction factor. Constants A and B is given as below:

A = 0.0648 for clean Yamuna sand

A = 0.0469 $(F_c)^{0.1798}$ for $F_c \geq 5\%$ where F_c is proportion of fine in %

B = 0.1958 for clean Yamuna sand

B = 0.0001 $(F_c)^2$ – 0.003 (Fc) + 0.2082 for $F_c \geq 5\%$

Plots were drawn between p′ and CRR evaluated in the present work based on non-linear and non-dimensional shear strength parameters (Q_{af} and R_{af}). The findings have been validated using field data collected from nearby site using Seed et al. (1983) method and also using the method of relative compaction and are shown in Fig. 6.6(a-e) and combined results are plotted in Fig. 6.7. Procedure for Evaluation of liquefaction potential using non-linear strength parameters Q_{af} and R_{af}.is given in the next section.

6.3.3 Relative compaction (R_c) based method for evaluation of liquefaction potential using non-linear strength parameters Q_{af} and R_{af}

A step by step method is enumerated below:

(a) Find out the value of $R_{c\text{-}critical}$ knowing the values of Q_{af} corresponding to three different values of R_{af} for each proportion of fine contents ranging from 0 to 25% (0, 5, 10, 15, 20 and 25%) using Eq. (6.12) for different values of effective overburden pressure (p′).

(b) Calculate the values of CRR using Eq. (6.13)

(c) Plot the curve between p′ and CRR on log scale marking it as present work.

(d) The line in this plot represents the limiting value of CRR at and below which there is no liquefaction.

6.4 Validation of liquefaction potential plots

In this section the liquefaction potential of Yamuna sand with certain proportion of fines has been evaluated. Certain data and the same parameters have been assumed for evaluation of liquefaction potential using Seed et al. (1983) method and field value of relative compaction. The results obtained by this method have been compared to validate the present study. The details of validation methods are given below.

A. Validation using technique prepared by Seed et al. (1983) by evaluation of CRR

(a) Soil exploration has been carried out using SPT methods.

(b) SPT values are recorded at different depth.

(c) Water table is recorded.

(d) Undisturbed and disturbed samples are taken and index properties are evaluated in the laboratory.

(e) SPT values are corrected for overburden pressure and fine contents using equations given by Seed et al. (1983) as explained earlier.

(f) CRR values were evaluated corresponding to various effective overburden pressures.

(g) Plot is drawn on the same log scale along with the plot drawn as in present work using Q_{af} and R_{af} for varied proportion of fines.

(h) It is clear from the plots that CRR calculated by using equation developed in the present work using relative compaction and non-linear shear strength parameters shows similar trends. Values of CRR for fines up to 10% nearly matches with the values of CRR evaluated using Seed et al. (1983) method. The plot differ slightly at fine proportion greater than 10% due to the reason that in the present work non-linearity of soil due to presence of fines has been taken care of by considering dilatancy whereas the dilatancy aspects are missing in Seed et al. (1983) method.

B. Validation of present work by evaluating CRR using field relative compaction values.

1. Using $(N_1)_{60CS}$ values corrected from SPT test, relative density has been calculated using the equation given by Meyerhof in Joseph E. Bowles book titled "Foundation Analysis and Design" page 163 as

$$D_r = 25.\, p'_o{}^{-0.12}.\, N_{60}{}^{0.46} \tag{6.15}$$

2. Calculate relative compaction using equation developed in the present work and as reproduced below:

$R_c = m\,D_r + n$ where m and n are taken from Fig. 5.5

3. Find the field value of R_c using equation γ_d/γ_{max}

4. Find out CRR using equation (6.13) and using value of field R_c in place of $R_{c\text{-}critical}$

5. The findings are given in Fig. 6.6(a – e) and combined results are plotted in Fig. 6.7

From the plot it is clear that the plot drawn using relative compaction in the present work closely matches with the two validation plots drawn using Seed et al. (1983) method and relative compaction ($R_c = \gamma_d/\gamma_{max}$) values obtained from field. Hence the method proposed in the present work can be used in determination of liquefaction potential of silty Yamuna sand without going for the complexity of the equation proposed by Seed et al (1983). The present method is much simpler and requires only evaluation of relative compaction of soil on field. It has been observed that liquefaction potential obtained on the basis of relative compaction is more near to realistic value since the dilatancy of Yamuna sand has been taken care of in terms of non-linear non-dimensional strength parameters Q_{af} and R_{af}. The liquefaction resistance of Yamuna sand with varied proportion of fines increases with increasing shear strength parameter Q_{af} for the same relative compaction. Since the value of Q_{af} decreases with increase in fines, hence the resistance to liquefaction decreases with increase in fine content. Critical examination of relevant data suggests that there is a threshold or critical value of relative compaction for each fine content, above which the liquefaction occur and below which the liquefaction do not occur. The plot between mean effective confining pressure p′ and R_c as shown in Fig. 6.6(a – e) and Fig. 6.7 depicts that there is a critical line corresponding to a specific value of non-linear shear strength parameters (Q_{af} and R_{af}) below which there is no liquefaction.

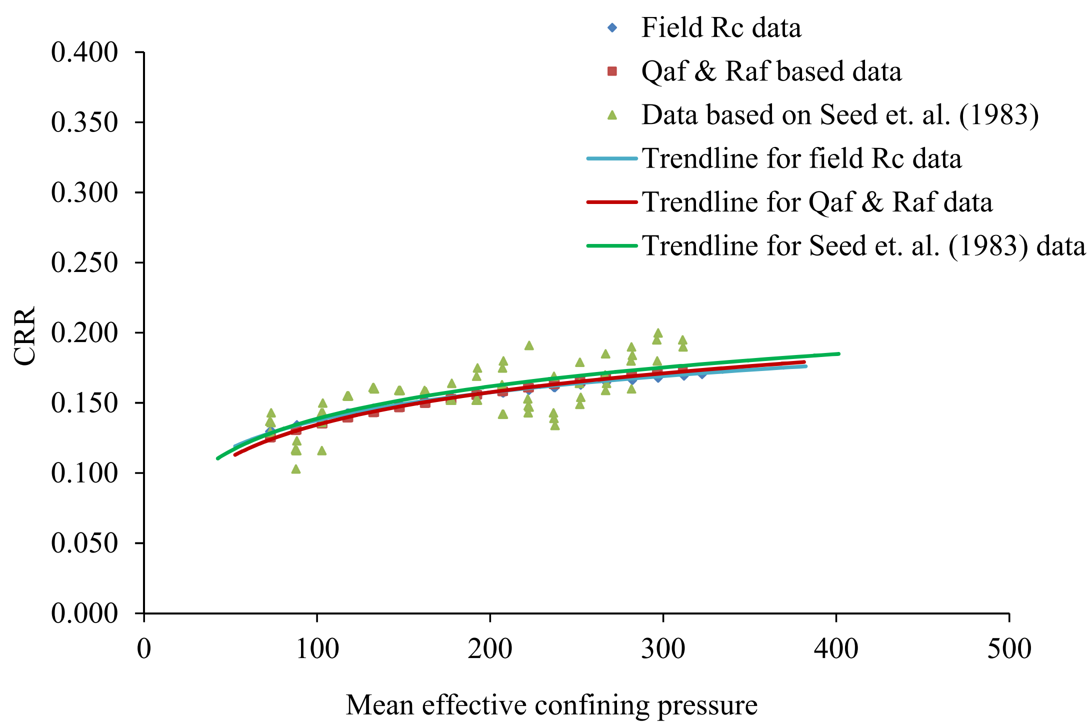


Fig. 6.6 (a) Variation of CRR with mean confining pressure less than equal to 5 percent fine content

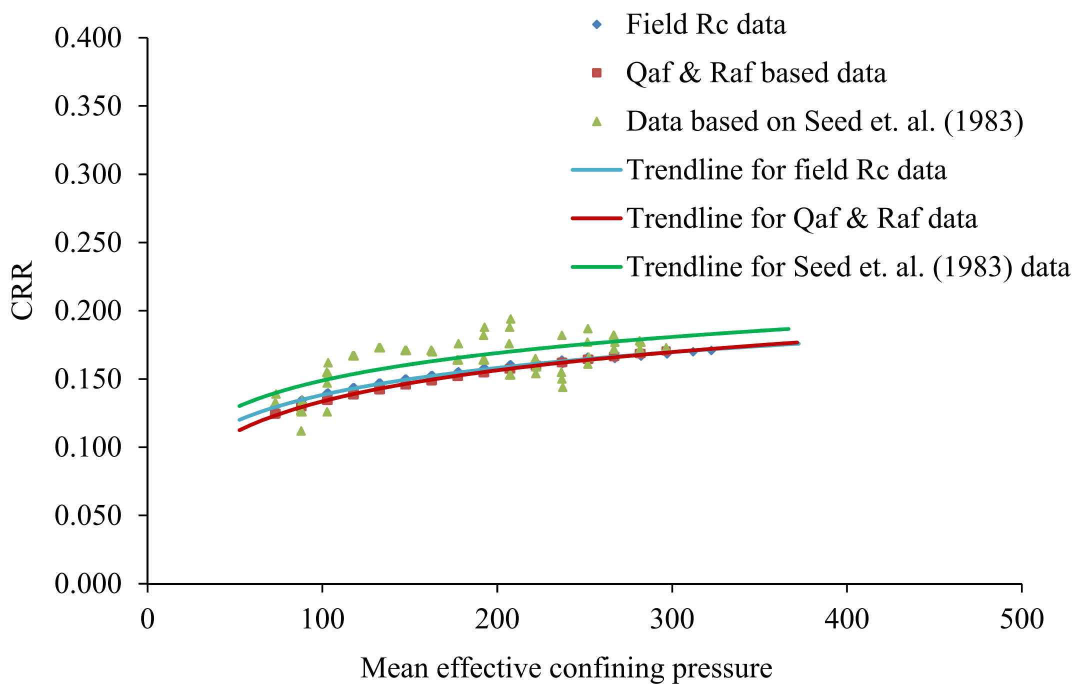


Fig. 6.6 (b) Variation of CRR with mean confining pressure less than equal to 10 percent fine content

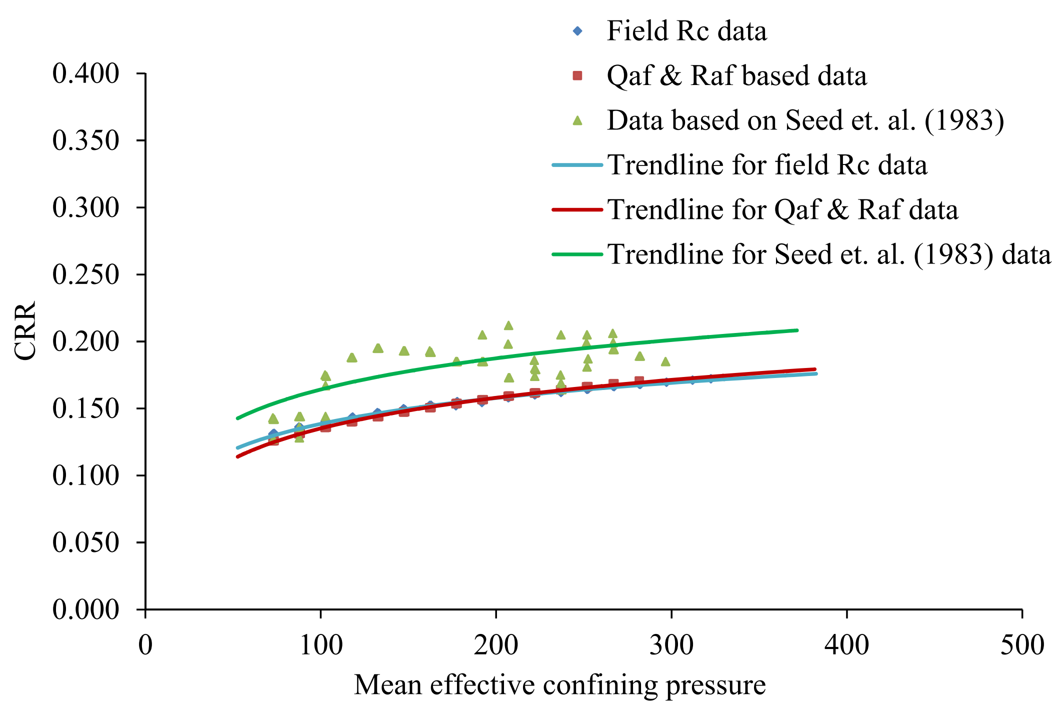


Fig. 6.6 (c) Variation of CRR with mean confining pressure less than equal to 15 percent fine content

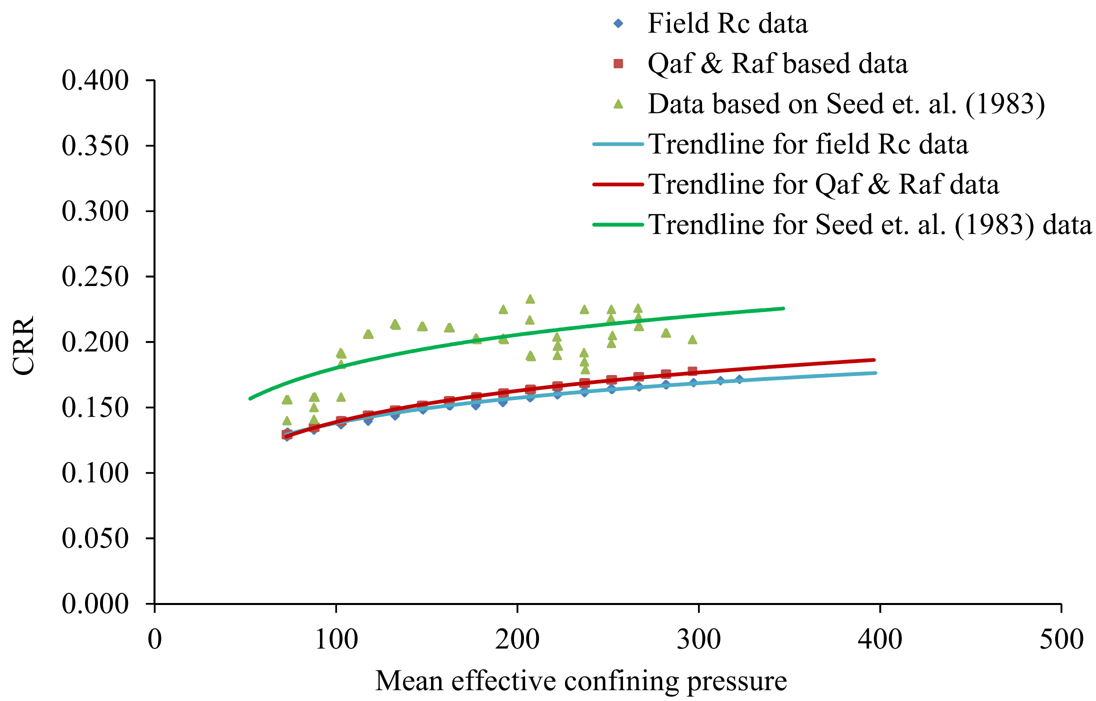


Fig. 6.6 (d) Variation of CRR with mean confining pressure less than equal to 20 percent fine content

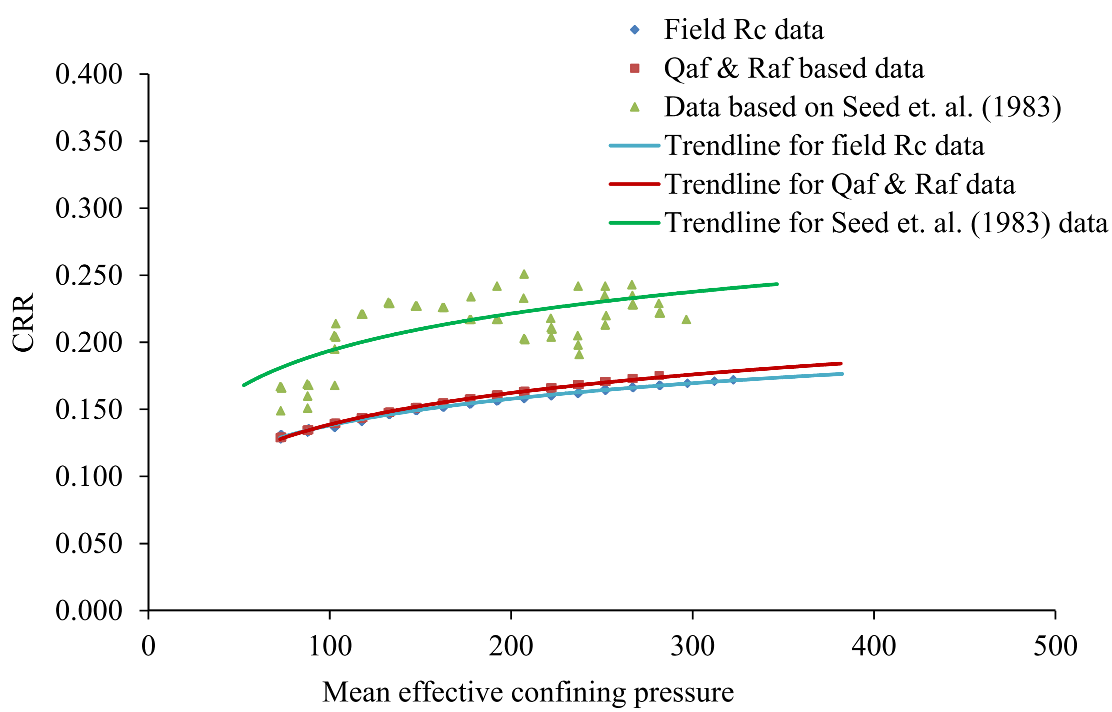


Fig. 6.6 (e) Variation of CRR with mean confining pressure less than equal to 25 percent fine content

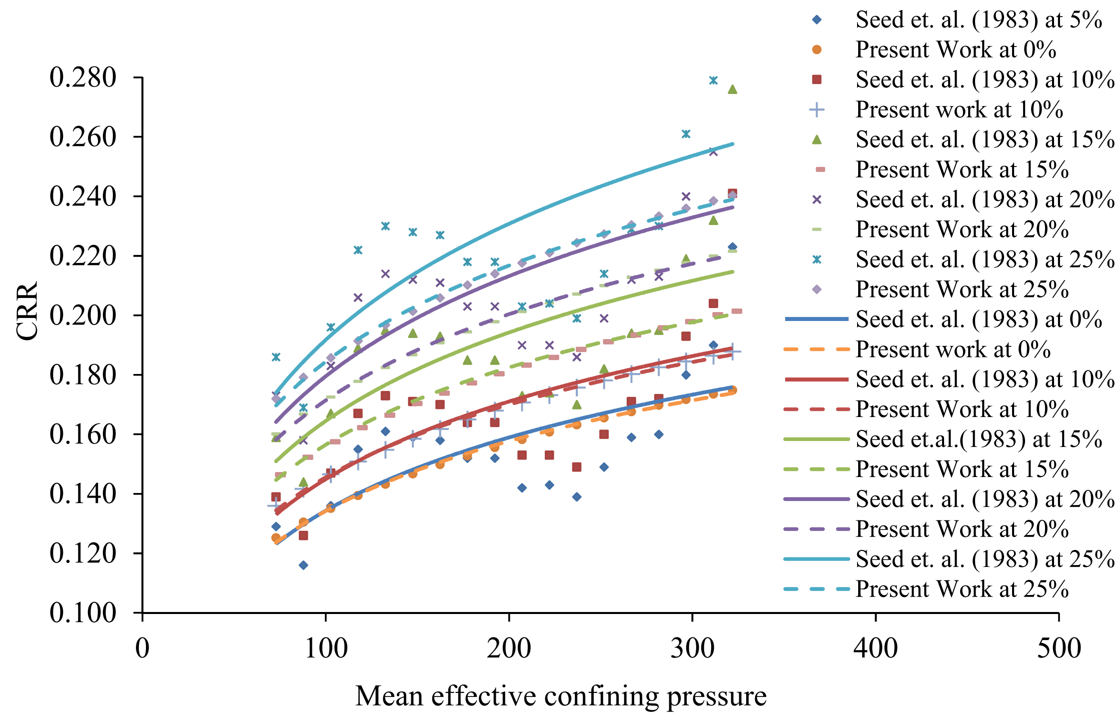


Fig. 6.7 Variation of CRR with mean confining pressure (For all types of sands)

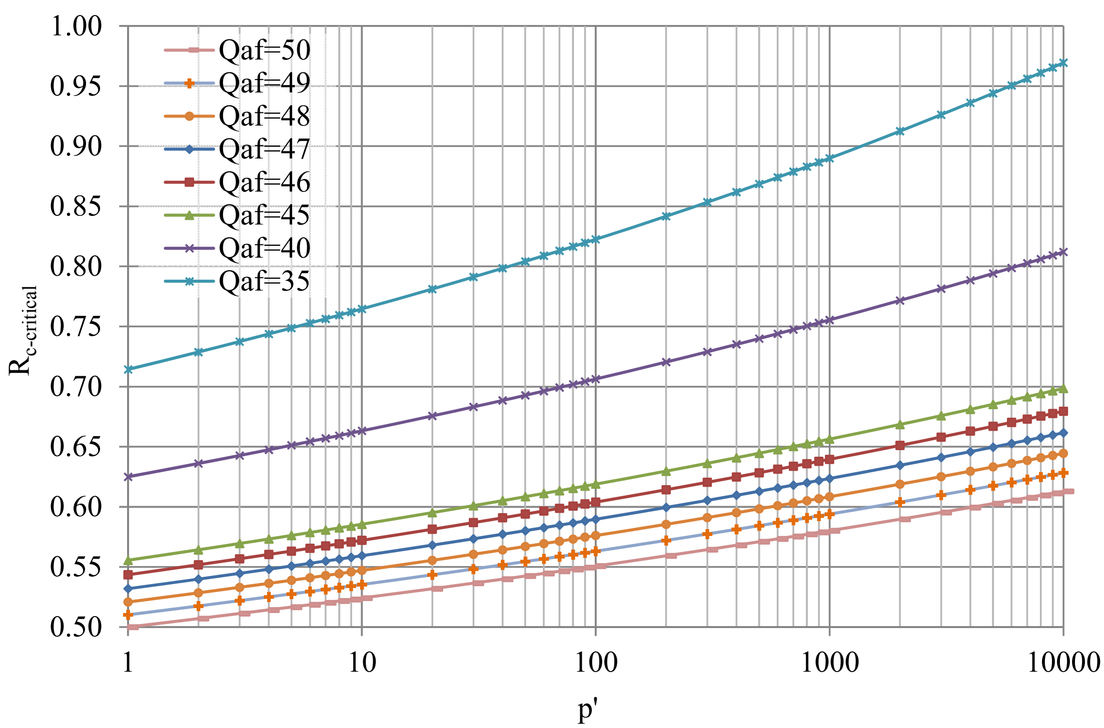


Fig. 6.8 Variation of R_c vs. p' at $R_{af} = 25$

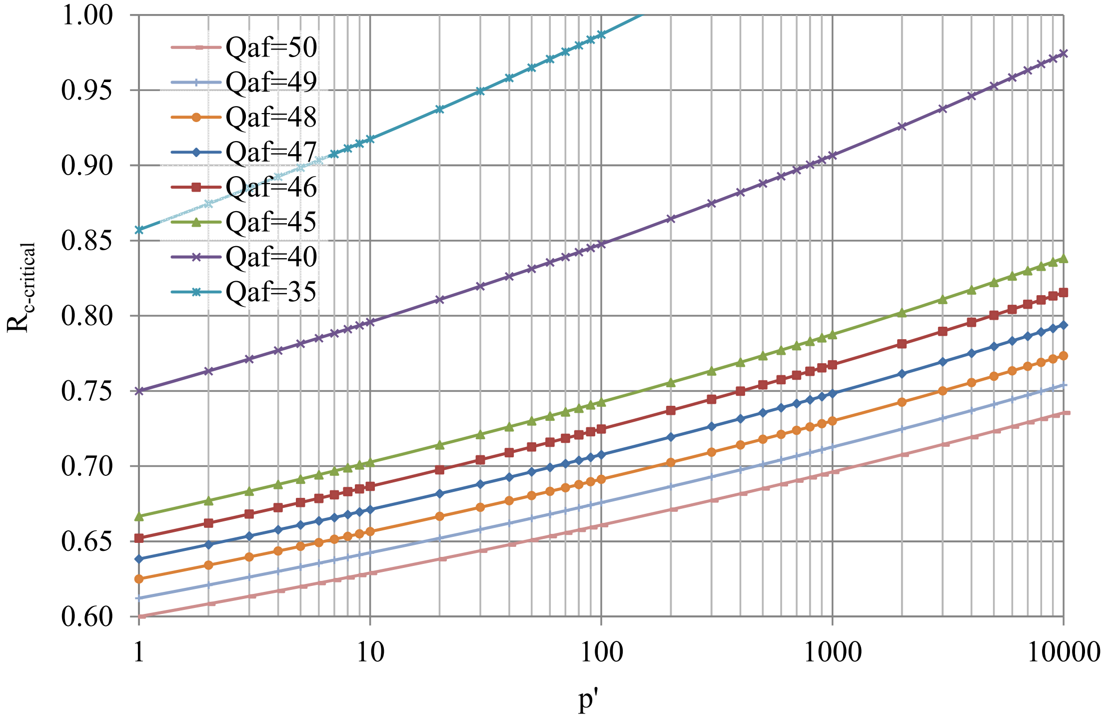


Fig. 6.9 Variation of R_c vs. p′ at R_{af} = 30

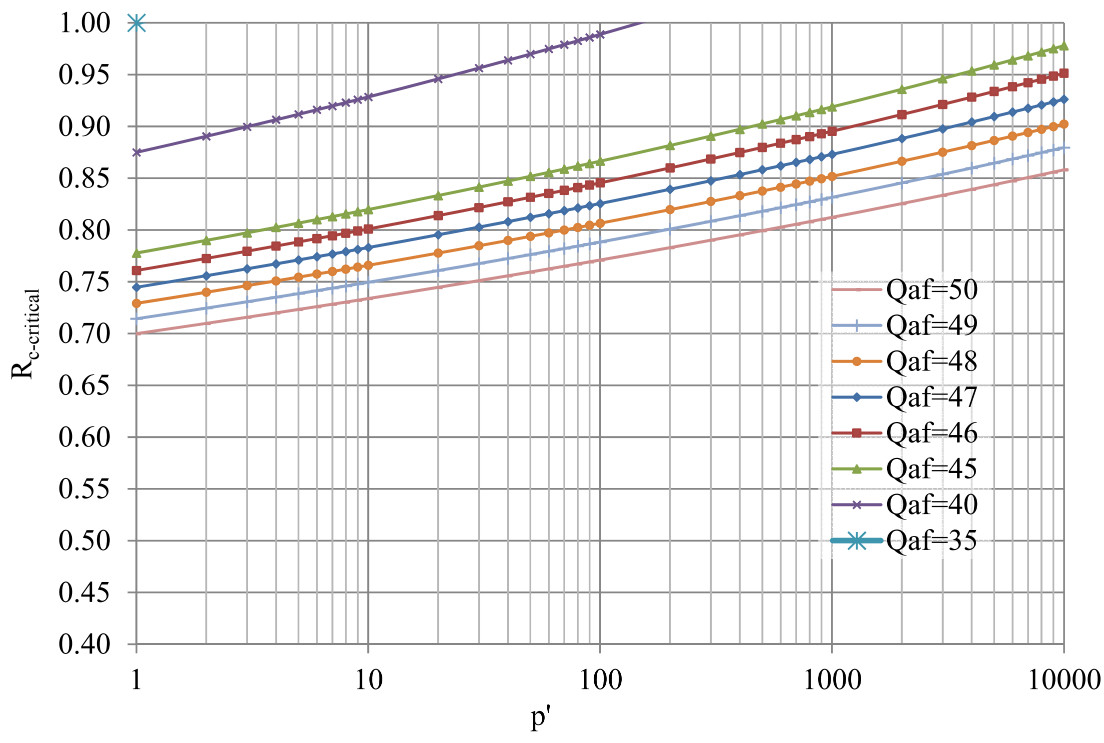


Fig. 6.10 Variation of R_c vs. p′ at R_{af} = 35

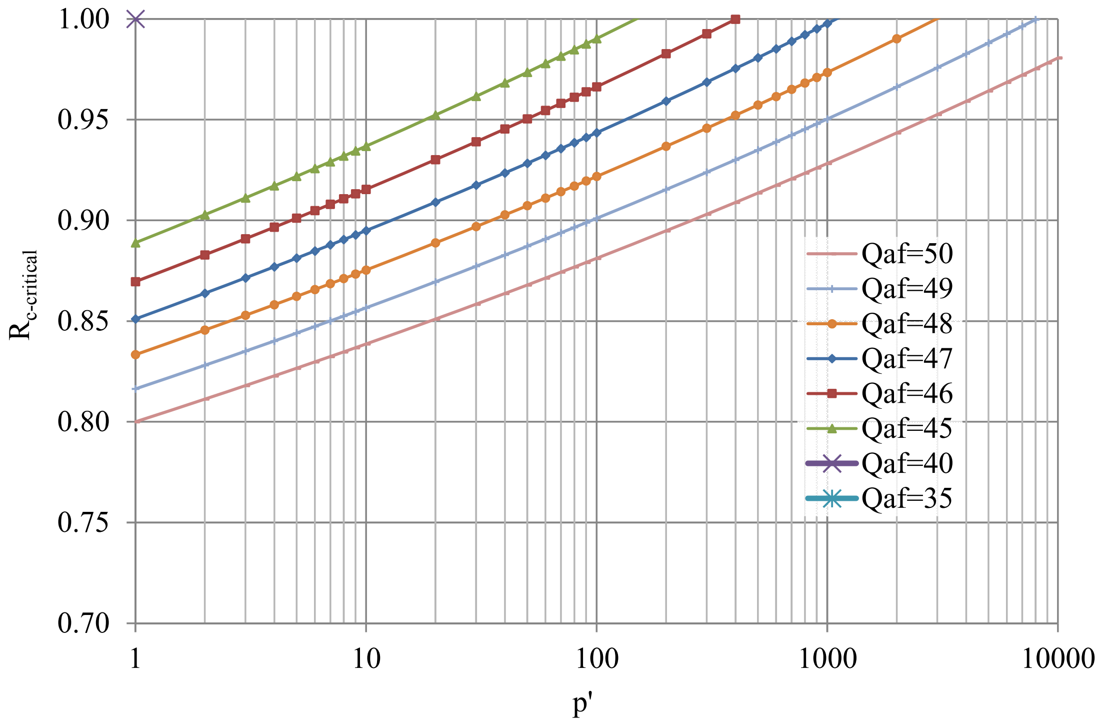


Fig. 6.11 Variation of R_c vs. p′ at R_{af} = 40

Liquefaction graph developed in Fig. 6.8 to Fig. 6.11 for different values of Q_{af} corresponding to values of R_{af} = 25, 30, 35 and 40 provides a simple and easy estimate of probability of liquefaction in silty sands. The state parameter ξ_R ($\xi_R = R_c - R_{c\text{-critical}}$) provides a simple correlations to the shear behavior of Yamuna sand. A positive value of ξ_R represents no liquefaction whereas negative value indicates liquefaction. The particular advantage of ξ_R for this study is that it just requires evaluation of relative compaction to assess the phenomena of occurrence of liquefaction.

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A. List Of Published Papers

1. Ojha, S., & Trivedi, A. (2013). "Comparison of shear strength of silty sand from Ottawa and Yamuna river basin using relative compaction." Electronic Journal of Geotechnical Engineering, Vol.18 (Bund. J), 2005–2019.
2. Ojha, S., Trivedi, A. (2013). "Shear strength parameters for silty sand using relative compaction." Electronic Journal of Geotechnical Engineering, Vol.18 (Bund. A), 81–99.
3. Ojha S., Goyal P., Trivedi A., (2012) "Non-linear behavior of silty sand from catchment area of Yamuna river" Proceeding of IGC Conference Dec 13-15, 277-280.

B. List Of Communications Under Review

1. Evaluation of liquefaction potential of Yamuna sand using relative compaction.
2. Estimation of bearing capacity of silty sand using shear strength parameters and relative compaction.

NON-LINEAR BEHAVIOR OF FEW SANDS AND ITS ENGINEERING IMPLICATION

A THESIS

SUBMITTED IN FULFILMENT OF THE REQUIREMENT

FOR THE

AWARD OF THE DEGREE OF

DOCTOR OF PHILOSPHY

IN

CIVIL ENGINEERING

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JUNE -2013

CHAPTER VII

CONCLUSIONS AND FUTURE SCOPE OF STUDY

7.0 General

7.1 Summary

The objective of the present study is to develop a relation which can bridge the gap between the linear and non-linear behavior of few sands for practical applications in a simple and realistic way. The objective of defining the non-linearity shown in soil behavior through strength parameters best fitted to Yamuna sand containing varied proportion of fines has been achieved in the present work. The non-dimensional parameters established in this study effectively represent the nonlinearity in the engineering behavior of few sands with varied fine contents investigated in the present work. To achieve the goal, a series of field tests combined with laboratory testing and theoretical analysis have been carried out during the course of research. A detailed literature review compliments the work carried out in the past twenty years by the eminent researchers in this field. Based on the past work and the present field and laboratory test analysis the nonlinear and non-dimensional strength parameters (Q_{af} and R_{af}) are evaluated for Yamuna sands containing varied proportions of fines. The effect of presence of varied proportion of fines, confining pressure and the extent of relative states (relative density/relative compaction) on the values of Q_{af} and R_{af} are established in the present work in detail. The engineering implication of these strength parameters (Q_{af} and R_{af}) thus obtained are applied to evaluate the bearing capacity of Yamuna sand taking into consideration the dilatancy factor in a simple and more convenient way. Also a simple approach for evaluation of liquefaction potential of soil was established on the basis of relative compaction. A relationship for estimation of cyclic resistance ratio (CRR) for Yamuna sand having varied proportion of fines is established based on the critical value of relative compaction. A correction factor (η_c) for relative compaction is applied on the values of critical relative compaction to get the values of CRR corresponding to varied level of effective overburden pressure. The findings were validated by using Seeds et al. (1983) methods by plotting the CRR values thus obtained against the effective overburden pressure along with the plots drawn on the basis of existing techniques for deriving liquefaction potential to validate the present approach. Also the results obtained have been verified through corresponding CRR plots on the basis of relative compaction values obtained from field data. Finally plots for liquefaction have been drawn to assess the liquefaction potential of sandy soil with varied proportions of fines without performing the rigorous analysis.This outcome allows a quick estimation of liquefaction potential of soil without going into the intricacy of numerous parameters proposed by Seeds et al. (1983). The main conclusions drawn from the present work are as follows:

7.2 Conclusions

Based on the research work performed in this study, the following conclusions are made.

1. Mean particle size of silty Yamuna sands considered in this study varies from 0.225 to 0.208 mm having silt content from 0 to 25%. (Refer Table 5.1)
2. The presence of plastic silt in the Yamuna sand increases peak friction angles up to a fine content of 10% and decreases as the fine contents increases beyond 10% for the same confining pressure and makes its axisymmetric compression response more dilative than that of clean sand. The peak friction angle decreases with increase in confining pressure at the same relative compaction. (Refer Table 5.3)
3. The presence of plastic silt in the Yamuna sand increases critical-state friction angles and makes its axisymmetric compression response more dilative than that of clean sand. (Refer Table 5.4)
4. Extreme void ratio e_{max} reduces from 0.78 to 0.62 with increase in silt content from 0 to 25% and e_{min} decreases from 0.50 to 0.31. (Refer Table 5.4)
5. The Yamuna sand reaches its peak much earlier (10-12% of strain) than other silty sands up to 10% of fines due to an anticipated slip under the same normal and confining stress [Refer Fig. 5.3(a) to Fig. 5.3(e)].
6. Yamuna sand with varied proportion of fines are less compressible than Ottawa sand for fines percent up to 10% and the compressibility increases when percent fine increases above 10%.
7. The strength properties of Yamuna sand with varied proportion of fines vary significantly with increase in proportion of fines as reflected by the changes in the values of Q_{af}. The value of parameter Q_{af} first increase with increase in fines up to 10% (Q_{af} = 49.99 for R_{af} = 40), and then reduces to 48.73 at a silt percentage of 25%. For best fit curve the value of Q_{af} = 49.883 for clean sand reduces to Q_{af} = 34.7 for fines percentage of 15%. With further increase in silt content (above 15 %.) the value of Q_{af}

drops to 5.0 (silt content of 25%) which shows that behavior of Yamuna sand is no longer similar to sand but nearly simulates the behavior of silt. (Ref. Table 5.5)

8. Values of Q_{af} decreases (Q_{af} = 49.883, D_{50} = 0.225) with decrease in the mean sizes (Q_{af} = 5.03, D_{50} = 0.208). (Ref. Table 5.5)

9. The values of r^2 (coefficient of determination) first increases from 0.797 to 0.856 with increase in silt content up to 10% and then decreases significantly (r^2 = 0.38) as the silt content increases to 20%. It indicates more uncertain behaviour of silty sand due to the increase in silt content above 10 %. (Ref. Table 5.8)

10. Coefficient of determination (for evaluation of Q_{af} and R_{af}) is higher (r^2 = 0.86 to 0.78) as calculated based on relative compaction than when calculated based on relative density (r^2 =0.72 to 0.55) due to the uncertainties involved in estimation of relative density for Yamuna sand with fine contents up to 15% [Ref. Table 5.8(a) and Table 5.8 (b)].

11. There is an empirical relation between relative density and relative compaction given by equation $Rc = mD_r+n$, where 'm' and 'n' are constants which depend on proportion of fines in the clean sand (Ref. Fig 5.5).

12. The behavior of Yamuna sand with the presence of silt up to 20% is non- plastic but for fines greater than 20 %, its behavior changes to plastic as shown in Table 5.2 corresponding to silt percentage of 25 %. Thus the shear strength of silty sands depends upon the index properties of fines. For plastic fines the shear strength is governed by the plasticity characteristics (LL and PI) and the physico-chemical interactions among finer grains influences the soil behavior whereas if fines are non-plastic then the frictional and interlocking resistance between individual soil particles influence the soil behavior.

13. Values of Q_{af} increases with increase in extent of confinement as shown in stress strain plots with shifting peaks for increasing value of mean confining stressess and hence increase in relative compaction. The rate of increase of Q_{af} decreases with increase in confinement.

14. The bearing capacity of Yamuna sand with fines can be easily evaluated on the basis of non-linear, non-dimensional shear strength parameters using the field value of relative compaction without performing the triaxial test. Bearing capacity of Yamuna sand with varied proportion of fines (0 to 25 %) first increases when fine contents are up to 10% then decreases with increase in fine content due to reduction in the value of Q_{af}.

15. The liquefaction potential of Yamuna sand with varied proportion of fines has been evaluated using non-linear, non-dimensional shear strength parameters and relative compaction. The limiting value of cyclic resistance ratio (CRR) has been drawn on the basis of this work. The method suggested is convenient and simple since it involves only the estimation of relative compaction and effective overburden pressure at any depth of soil strata.

16. A state parameter ξ_R ($\xi_R = R_c - R_{c\text{-crictical}}$) has been evaluated which provides unique solution and simple correlations to the liquefaction potential of Yamuna sand. A positive value of ξ_R indicates no liquefaction whereas negative value represents onset of liquefaction. The particular advantage of ξ_R for this study is that it provides a simple co-relation to the cyclic resistance ratio (CRR) of Seeds et al. (1983) based on the field SPT values.

7.3 Scope for further Study

This study can be further extended to the following areas of research namely,

1. The study can be extended further to the analysis of the effect of damage of grains on non-linearity of sandy soil containing varied proportion of fines at higher confining pressures.
2. The evaluation of CSR can be based upon attenuation of ground response due to varied density states (relative compaction) and state parameters of the soils with varied proportion of fines.