Developments in Geotechnical Engineering

K. Ilamparuthi · R. G. Robinson *Editors*

Geotechnical Design and Practice

Selected Topics

Springer

Developments in Geotechnical Engineering

Series editors

Braja M. Das, California State University, Henderson, CA, USA
Nagaratnam Sivakugan, James Cook University, Townsville, QLD, Australia

This series on Geotechnical Engineering has been initiated to provide an integrated platform for publishing in all areas of geotechnics and their applications to engineering problems. The series focuses on the topics that have garnered recent interest from a large research audience, including, but not limited to, saturated and unsaturated soils, expansive soils, geosynthetics, marine geotechnics, and soil improvement. This series includes monographs, textbooks, and references in the general area of Geotechnical Engineering. The books in this series will serve researchers, practitioners, and students alike.

More information about this series at http://www.springer.com/series/13410

K. Ilamparuthi · R. G. Robinson
Editors

Geotechnical Design and Practice

Selected Topics

Springer

Editors
K. Ilamparuthi
Department of Civil Engineering,
College of Engineering Guindy
Anna University
Chennai, Tamil Nadu
India

R. G. Robinson
Department of Civil Engineering
Indian Institute of Technology Madras
Chennai, Tamil Nadu
India

ISSN 2364-5156 ISSN 2364-5164 (electronic)
Developments in Geotechnical Engineering
ISBN 978-981-13-0504-7 ISBN 978-981-13-0505-4 (eBook)
https://doi.org/10.1007/978-981-13-0505-4

Library of Congress Control Number: 2018940875

Printed on acid-free paper

This Springer imprint is published by the registered company Springer Nature Singapore Pte Ltd.
part of Springer Nature
The registered company address is: 152 Beach Road, #21-01/04 Gateway East, Singapore 189721, Singapore

Foreword

The Indian Geotechnical Society (IGS) was started as the Indian National Society of Soil Mechanics and Foundation Engineering in the year 1948, soon after the Second International Conference on Soil Mechanics and Foundation Engineering held at Rotterdam. The Society was affiliated to the International Society in the same year, and since then, it has strived to fulfil and promote the objectives of the International Society. In December 1970, the name 'Indian Geotechnical Society (IGS)' was adopted.

The Society conducted several symposia and workshops in different parts of India since its inception in 1948. It was in the year of 1983, the Indian Geotechnical Society organized its first annual conference IGC 1983 in Indian Institute of Technology Madras.

Several local chapters of the Society were established over the years, and gradually, the annual conferences were held in different cities under the leadership of the respective local chapters. The conferences were well utilized as the venue for showcasing the research works and the case studies on geotechnical engineering and geo-environmental engineering, and the papers presented during the deliberations were being published as conference proceedings volume.

The responsibility of organizing the annual conference of 2016 was taken up by IGS Chennai Chapter ,and the conference was held during 15–17 December 2016. Eminent professors and practitioners were invited to present keynote and theme lectures during this conference. These keynote and theme lectures comprise this volume 'Geotechnical Design and Practice—Selected Topics' under the series 'Developments in Geotechnical Engineering'.

The Chennai Chapter of Indian Geotechnical Society placed on record its acknowledgement of the efforts put forth by Springer in bringing out this book for the benefit of the geotechnical engineering community.

Chennai, India

Prof. A. Boominathan
Chairman, Indian Geotechnical Conference 2016
Department of Civil Engineering
Indian Institute of Technology Madras

Preface

The Indian Geotechnical Society has been organizing national-level annual conference since 1962. The Indian Geotechnical Conference 2016 (IGC 2016) was conducted at IIT Madras in collaboration with the Chennai Chapter of Indian Geotechnical Society, Indian Institute of Technology Madras and College of Engineering, Guindy, Anna University, Chennai, from 15 to 19 December 2016. The theme of the conference was 'Geotechnology Towards Global Standards'. To fulfil the objectives of the conference, it was proposed to cover various disciplines of geotechnical engineering, and therefore, experts from various parts of the globe were identified and invited to share their knowledge by presenting keynote lectures and invited papers.

There were 12 keynotes and 17 invited speakers, and their presentation covered a wide spectrum of geotechnical engineering from fundamental properties of soils to remote sensing applications including large-scale test to understand passive response of skewed bridge abutments. The speakers are well-known researchers in the fields of soft ground improvement, seismic response of retaining structure using SSI principles, unsaturated soils, etc. The deliberations of eminent researchers generated useful discussions among the practising geotechnical engineers, consultants, researchers and academicians. It was decided to bring together an edited book based on the talks delivered by the keynote and invited speakers.

With this view, a book on 'Geotechnical Design and Practice—Selected Topics' was compiled which contains 22 chapters covering properties of soils, unsaturated soil mechanics, ground improvement, liquefaction and seismic studies, soil–structure interaction and stability analysis of man-made and natural slopes.

We believe that the contents of the book on the recent developments in the said areas of geotechnical engineering will be a useful source of reference for practising engineers and researchers apart from providing insights for future research. The editors are grateful to all the speakers of keynote and invited papers for sparing their valuable time in preparing the source material and sharing their knowledge with participants during the sessions of the conference. Finally, we the editors of this volume would like to express our sincere thanks to the organizers of IGC 2016 for the wonderful opportunity of editing the volume.

Chennai, India

Dr. K. Ilamparuthi
Dr. R. G. Robinson
General Editors

Contents

About the Editors

Dr. K. Ilamparuthi is a professor in the Department of Civil Engineering, College of Engineering Guindy, Anna University, Chennai, with more than three decades of teaching and research experience. He obtained his Ph.D. from the Indian Institute of Technology Madras, India. His areas of research include ground improvement, soil–structure interaction, anchor foundations, slope stability analysis and finite element analysis. Over the course of his career, he has published over 200 research papers in journals and conference proceedings and has guided 9 Ph.D. students and 145 M.E. students. He is actively involved in the administrative running of the College of Engineering, Guindy, and has held various posts including dean in-charge, chairman of the Faculty of Civil Engineering and head of department.

He is an active consultant involved in several major projects. He has received numerous prestigious awards for his research and his consultation work, as well as for the Commonwealth fellowship. He worked at the University of Liverpool, UK, for a period of 13 months from October 1999 to November 2000 and has visited several countries such as UK, Belgium, Australia, Hong Kong, USA, Middle East and Singapore on various assignments.

Dr. R. G. Robinson is a professor in the Geotechnical Engineering Division in the Department of Civil Engineering at IIT Madras, India. He obtained his Ph.D. from the Indian Institute of Science Bangalore, India, and has pursued postdoctoral research at the National University of Singapore, in addition to visiting University of Wollongong in Australia for a period of 6 months under the Endeavour India Fellowship. His research interests include physical modelling, soil characterization and ground improvement with special reference to soft clays and expansive soils. He has published over 100 research papers in journals and conference proceedings and has guided 8 Ph.D. and 15 Master's students. He and his students received the IGS-Mr H. C. Verma Diamond Jubilee Award for Innovative Instrument Design from IIT Madras.

Quality Assurance Studies for Ground Improvement Projects

Anand J. Puppala, Tejo V. Bheemasetti and Bhaskar C. S. Chittoori

1 Introduction

Soils are inherently heterogeneous in nature with wide distribution of their physical and mechanical properties (Phoon and Kulhawy 1999). When poor subsoil conditions are encountered, engineers often improve the soil properties by choosing suitable ground improvement techniques. Comprehensive state-of-the-art ground improvement techniques for different soil types were discussed in different papers and keynote lectures. (Mitchell 1981; Puppala and Perez 2009). Terashi and Miki (1999) have provided a detailed overview for the selection of ground improvement technique for a specific project based on the subsoil conditions. Broadly, the most commonly used ground improvement techniques can be classified into two categories: mechanical stabilization and chemical stabilization.

Mechanical stabilization such as deep dynamic compaction, vibro-compaction, stone columns, pre-loading, deep dynamic compaction are few prominent techniques which stabilize the soils by rearranging the soil particles (Mitchell and Zoltan 1984). Chemical stabilization technique refers to enhancing the soil properties by treating with additives such as lime, cement, fly ash, and other by-products (Bredenberg et al. 1999; Rathmayer 2000). The chemical stabilization technique became widely accepted ground improvement technique for treating highly expansive clayey soils which is the main focus of this paper.

A. J. Puppala (✉) · T. V. Bheemasetti
Civil Engineering Department, The University of Texas at Arlington, Arlington, USA
e-mail: anand@uta.edu

T. V. Bheemasetti
e-mail: tejovikash.bheemasetti@mavs.uta.edu

B. C. S. Chittoori
Civil Engineering Department, Boise State University, Boise, USA
e-mail: bhaskarchittoori@boisestate.edu

K. Ilamparuthi and R. G. Robinson (eds.), *Geotechnical Design and Practice*, Developments in Geotechnical Engineering,
https://doi.org/10.1007/978-981-13-0505-4_1

Since the twentieth century, new changes in the ground improvement techniques have taken place. This is mainly due to the innovations in geotechnical equipment and construction procedures. Along with new ground improvement techniques, considerable advancements have taken place in quality control and quality assurance methods. The bi-directional arrangement of osterberg cells (o-cell) in traditional load tests has improvised the quality assurance tests in deep foundations.

The pre- and post-standard penetration tests and cone penetration tests more accurately represents the variation in the strength properties before and after the implementation of ground improvement techniques (Sondermann and Weher 2004; Raju 2010). Researchers have also proposed and implemented new quality assurance and quality control methods for different ground improvement techniques (Puppala et al. 2004; Madhyannapu et al. 2010). In this paper, quality assurance studies performed on a pipeline bedding material and at a pavement infrastructure were discussed. In these studies, new design and construction procedures were implemented. The following subsequent sections present the details of these studies.

2 QA/QC in Pipeline Infrastructure

A 150-mile water pipeline was proposed to provide additional water supplies to Dallas/Fort Worth Metroplex area. The proposed pipeline is 2.74 m (9 ft.) in diameter and passes through the six different geological formations namely, Eagle Ford, Kemp, Wills, Neylandville, Ozan, and Wolfe.

Extensive borehole investigations were performed along the six (6) geological formations to determine the subsurface profile and soil properties. The subsurface profile mainly consists of clays with low to high plasticity characteristics. Of all, the Eagle Ford geological region depicted high plasticity characteristics with plasticity index of 37 and liquid limit of 62. The average swell pressure is observed to be 40.7 psi (280.6 kPa) and average volumetric strain is observed to be −19.7%. The native high plasticity soils possessed a serious threat to the pipeline project as the soils exhibited expansive swell/shrink behavior. In order to provide a proper bedding material to the pipeline material, the controlled low strength material (CLSM) was considered as a viable option.

CLSM is a self-compacted, cementitious material, which was widely known as flowable fill until American Concrete Institute Committee 229 documented its name as CLSM (ACI 1994). The CLSM is primarily used as a backfill material in lieu of compacted backfill and has become a popular material for projects such as void fill, foundation support, bridge approaches, and conduit bedding (Folliard et al. 2008). CLSM, with different additives such as cement and fly ash, has been demonstrated, by many researchers, to be an effective bedding material for pipelines due to the material's self-compacting behavior and strength performance (Rajah et al. 2012; Boschert and Butler 2013). In this study, the CLSM was prepared using native plasticity soils and Type I/II Portland cement. The utilization of native soils will provide sustainable engineering solution by minimizing the project cost and by reducing the negative impact on the environment (Puppala and Hanchanloet 1999; Chittoori et al. 2012; Puppala et al. 2012).

2.1 Pipeline Installation and CLSM Mix Design

In order to study the feasibility of CLSM prepared with native clayey soil as a pipeline bedding material, it was first tested on a 152.4 m (500 ft.) long test section. This test section was also referred to as prove-out section. Ten (10) pipeline sections of 15.2 m (50 ft.) were used to construct the full prove-out pipeline length of 152.4 m (500 ft.).

After excavating the soils and laying the pipeline, the native plasticity soils from test site were used in the place of aggregates in preparing the CLSM mix. The mix design of CLSM constitutes of cement, water and native plasticity soil. Based on laboratory studies conducted by Raavi (2012), 4% Type I/II Portland cement was selected to prepare the CLSM mix for the present test section. Table 1 provides the basic soil properties on native soils collected from the test site. Soil classification tests were performed based on Unified Soil Classification System (USCS). Sieve and hydrometer tests were conducted as per ASTM D 422 and Atterberg's Limit Tests (Liquid Limit and Plastic Limit) were conducted as per ASTM D 4318 standards. Specific gravity test was conducted in accordance with the ASTM D 854.

The CLSM prepared at the site was poured into the trench in two different layers. First, the CLSM was poured to 30% height of the pipeline. Due to its flowability nature the CLSM was spread around the pipeline and later it was finally poured up to 70% height of the pipeline. The stiffness monitoring studies were conducted at various elapsed curing periods to evaluate its material behavior. The quality assurance studies conducted were discussed in the subsequent sections.

2.2 Quality Assurance Studies

In this research study, an attempt was made to check the feasibility of CLSM prepared using native plasticity soils as a pipeline bedding material. Laboratory

Table 1 Soil properties and CLSM mix design

Basic properties	
Percent passing No. 200 sieve	73
USCS classification	CL
Liquid limit, LL	24.4%
Plastic limit, PL	14.3%
Plasticity index, PI	10.1%
Specific gravity, G_s	2.62
Cement used in CLSM	4%

studies were performed to check the strength enhancement of CLSM samples prepared using native plasticity soils. The test results and analysis demonstrated the enhancement in strength that satisfies the criteria for pipeline bedding material (Puppala et al. 2007; Raavi 2012, Chittoori et al. 2014). However, the assurance of strength and stiffness characteristics in the field is often challenging. The in situ stiffness of CLSM will be influenced by several factors such as construction methodology, environmental factors, and existing soil conditions. Past researchers have attributed several water pipeline failures due to lack of proper inspection (Kienow and Kienow 2009). In this research study, a quality assurance method, which is a combination of Spectral Analysis of Surface Waves (SASW) technique and Geostatistics, was proposed to ascertain the stiffness of pipeline bedding material.

2.2.1 Spectral Analysis of Surface Waves (SASW)

SASW technique was developed in early 1980s based on the dispersive characteristics of surface waves (Nazarian and Stokoe 1984; Stokoe et al. 1989). Based on the nature of investigation, different types of tools can be employed for generating the seismic waves and recording the wave forms. In this research study, SASW tests were performed at seventeen (17) test sections along the prove-out section, comprising of at least one (1) test section for every 9.1 m (30 ft.). Five (5) test points (South 2, South 1, Center, North 1, and North 2) were selected at each section to determine the variation in stiffness of CLSM bedding material across the pipe as shown in Fig. 1.

SASW tests were performed with a geophone spacing of 0.61 m (2 ft.). This distance was selected so that the stiffness profile of CLSM beneath the pipeline (0.45 m ~ 1.5 ft. thickness) can be obtained. The recorded waveforms were analyzed to determine the stiffness of the CLSM across the pipeline. Since, the thickness of pipeline is only 0.025 m (1 in.); the results obtained are direct indication of CLSM across the pipeline. However, a higher thickness of the pipeline must be carefully analyzed as the waves can directly pass through the pipeline.

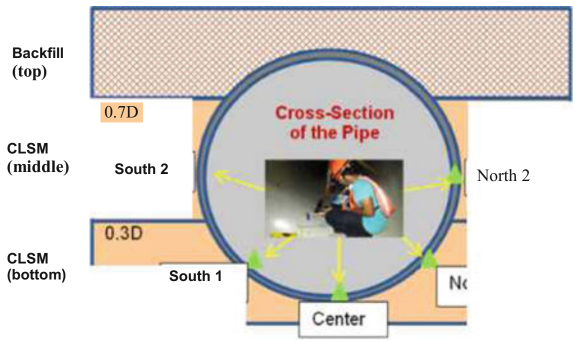
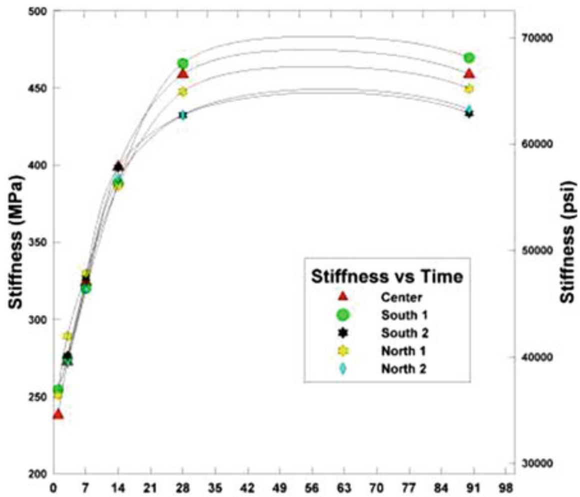


Fig. 1 Cross-section of test section inside pipeline

Fig. 2 Stiffness enhancement at section 1066-25



Figures 2 represent the stiffness profile of CLSM with respect to different curing periods in days. The five (5) test points at each section have depicted an increase in the stiffness values with an increase in curing period. A close observation of the results indicates that, there has been a consistent increase in stiffness values from day 1 to 28, but the rate of increase in stiffness of CLSM from day 28 to 90 was not significant. This suggests that CLSM had attained most of its strength by the end of 28 days curing period.

2.3 Geostatistical Studies

The geostatistical studies were employed in this research to analyze the spatial variability of stiffness measurements obtained from SASW tests. Geostatistics is a separate branch of statistics which deals with spatial analysis of the data sets (Isaaks and Srivastava 1989). The application of geostatistics was used mainly in the mining industry to predict the location of ore by describing the probability distribution of the existing ore locations (Krige 1951). In this study, geostatistical analysis was performed on SASW data obtained from the inside of the pipe as well as that obtained from the surface. The two main important steps that were performed in geostatistical analysis is modeling the spatial variability in stiffness values through variogram and performing predictions using kriging analysis.

2.3.1 Variogram Analysis

The variogram or semi-variogram γ (h) is a traditional geostatistical analysis tool used to model the spatial variability in the data sets. Mathematically, it is defined as one-half of the average squared differences between the *x* and *y* coordinates of each pair of points in the h-scatter plot (Isaaks and Srivastava 1989).

In this study, variogram analysis was performed to identify and model the spatial variability present in the stiffness measurements. This was performed by constructing the variogram plots, where variogram values were calculated and plotted against the corresponding lag distance values. Figure 3 represents the experimental variogram plots for the stiffness measurements obtained after 28 days curing period. The variogram values are plotted on the *y*-axis which is dependent on the lag distance values that are plotted on the *x*-axis.

The range: 24; sill: 600 and nugget: 200 are three important characteristics that are modeled in the variogram plot. The range represents the spatial continuity of the data sets and sill represents the level at which the variogram reached asymptotic value, whereas the nugget represents the variability in the small distances. In this study, the nugget behavior is due to the variability in the five tests points across the section. The blue solid exponential line in the variogram plot represents the spatial variability model developed for stiffness measurements. Similar analysis is conducted for all the stiffness measurements obtained at different curing periods. The spatial variability in the stiffness measurements obtained at different curing periods is modeled using the nugget effect and exponential model. The developed spatial variability models were incorporated in the kriging analysis for predicting the stiffness measurements of CLSM at untested locations.

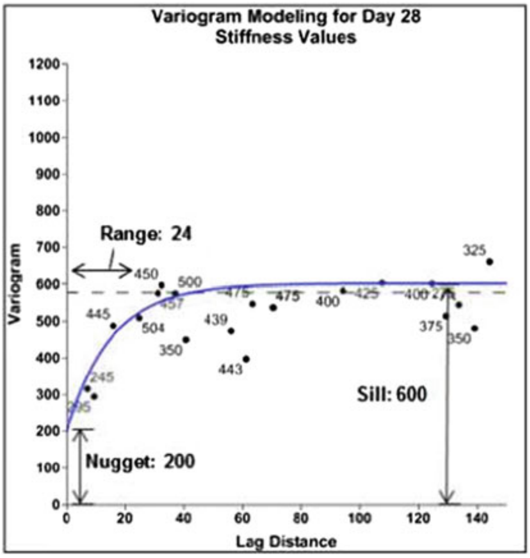


Fig. 3 Variogram plot for stiffness measurements

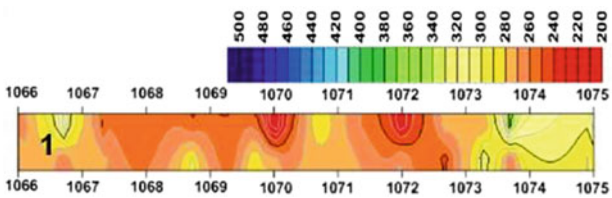


Fig. 4 Variation of stiffness measurements using kriging analysis

2.3.2 Kriging Analysis

The kriging analysis was used in this study to predict the stiffness measurement along the entire pipeline using the known measurements and spatial variability models. Kriging provides the best linear unbiased estimates to predict the values at unknown locations (Armstrong 1994). This is because of its ability to reduce the error variance of the predicted values. The spatial variability models developed earlier for the stiffness values were used along with the kriging algorithms to obtain the weights of the neighboring values around the untested location. Figure 4 represents the day 1 and day 28 stiffness variation of the CLSM along the pipeline in the plan view. The horizontal axis represents the 10 sections 1066 to 1075 of 152.4 m (500 ft.) length.

The color index presented at the top of Fig. 4 represents the stiffness values of CLSM in MPa. It can be observed that after 1 day of curing period, the CLSM depicted a stiffness of 200–280 MPa and on day 28 the stiffness has significantly increased to more than 360 and 400 MPa. Also, it can be observed that the stiffness development after each curing period is uniform along the prove-out section. This ascertains the quality of the CLSM prepared in the field using the native soil and self-compaction effort the CLSM.

2.4 Surface Testing for Quality of Stiffness Assessments

The main objective of this task is to determine the variation of stiffness of the subsurface layers including CLSM and backfill across and beneath the pipeline. This analysis was performed by conducting both SASW tests from the surface and geostatistical analysis. Several trial and error procedures were adopted for the surface testing as the wave interaction with the pipeline creates an erroneous wave forms which could not be analyzed. After repeated trails and with interpolations a final design test layout as shown in Fig. 5 was implemented. It should be noted that 0.91 m (3 ft.) spacing in the geophone provides a seismic wave to a depth of 1.83 m (6 ft.). In view of the above perspective the geophone spacing are kept at 0.91 m (3 ft.), 1.22 m (4 ft.), 1.52 m (5 ft.) and 2.44 m (8 ft.) to avoid impact of pipeline.

SASW tests were conducted from the surface using geophones as receivers. Tests were conducted at three locations within 30.5 m (100 ft.) stretch with different distant

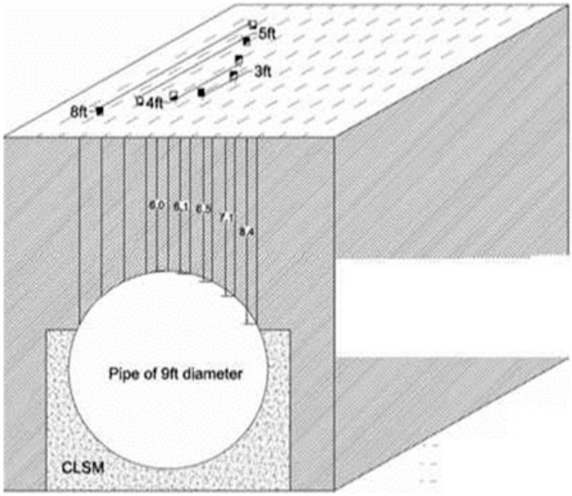


Fig. 5 SASW test grid layout from surface

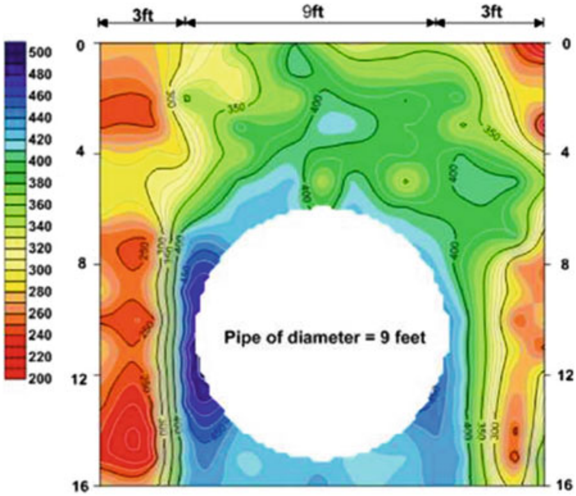


Fig. 6 Subsurface stiffness variation

spacing. Necessary precautions were taken to ensure the wave forms generated are not from any field construction activity by adjusting the sensitivity of the equipment. Geostatistical analysis was conducted on the stiffness measurements obtained from the surface tests. The spatial variability model developed for the stiffness measurements determined after 28 days curing period was incorporated with the kriging analysis to predict the variation of stiffness measurements along the subsurface layers.

Figure 6 presents the variation of stiffness layers from the surface. The vertical axis in Fig. 6 represents the depth of the subsurface profile. The hollow circle in the figure represents the 2.74 m (9 ft.) diameter pipeline.

From Fig. 6, it can be observed that the material around the pipeline indicates high stiffness value, corresponding to CLSM. The stiffness values decreases with an increase in the distance from the pipeline depicting the transition from CLSM to soil. The soil above the CLSM depicts high stiffness values which can be due to continuous compaction effort performed in the field. Also, a clear pattern of high stiffness values are observed on the left side of the pipeline. This can be attributed to the side where the CLSM was poured into the trench.

2.5 Summary and Conclusions

In the current study, the CLSM prepared using native plasticity soil was used as a bedding material to the pipeline of length 152.4 m (500 ft.). An attempt was made to ascertain the stiffness enhancement of CLSM using both seismic non-destructive test method and Geostatistics. Seventeen test sections were chosen along the prove-out section to monitor the stiffness behavior. The test results and analysis depicted that the stiffness of CLSM has increased significantly with increase in curing period. After 28 days curing period, the stiffness has reached to a constant value. The SASW test was successfully utilized to determine the stiffness of CLSM and geostatistics could predict the stiffness variation of the CLSM along the prove-out pipeline section.

3 QA/QC Studies in Deep Soil Mixing Technique (DSM)

Deep soil mixing (DSM) is a ground improvement technique mainly used to enhance strength and stiffness properties of soft clays, loose sands, peaty soils, and problematic soils such as expansive clays. Often the stabilization is performed using the lime or cement, of which the former is more specifically used to reduce the hydraulic conductivity of treated soils due to flocculation (Rathmayer 1996; EuroSoilStab 2002). Over the years, researchers and practitioners have demonstrated the successful use of DSM technique over wide variety of projects (Rathmayer 1996; Porbaha 1998; Bruce 2001). In this research study, the DSM technique is implemented to stabilize the expansive soils and mitigate the shrink and swell behavior. One of the key parameter that influences the overall performance in this technique is quality assurance studies.

Several factors such as soil type, binder type and concentration, binder–water ratio, curing conditions, mixing methods and construction practice play a key role in influencing the performance of a DSM project (Madhyannapu et al. 2010). Especially, it is highly necessary to ensure that the design strength parameters achieved in the laboratory and field conditions are same. This paper presents the quality assurance studies performed on expansive soils treated using DSM

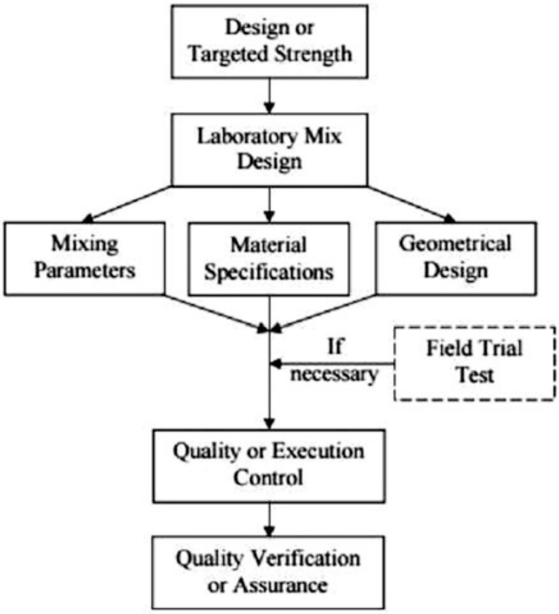


Fig. 7 Typical QA/QC procedure (modified after Coastal Development Institute of Technology 2002 and Usui 2005)

technique. Figure 7 presents the modified version of typical QA/QC procedure adopted in DSM technique.

In order to evaluate the DSM technique for expansive subgrade soils, two field test sites located in Fort Worth, Texas were selected. QA/QC studies were performed to evaluate the effectiveness of DSM technique in mitigating the swell-shrink behavior of expansive soils. Both test sites comprises of expansive clay subsoils with medium to high plasticity characteristics. Soils from two test sites were treated using 25% lime and 75% cement with a binder dosage of 200 kg/m^3 and w/b ratio of 1.0. During the field construction of DSM columns, the wet samples were grabbed at different depths to assess the strength and stiffness parameters of the soils. Similar field composition of DSM samples was also prepared in the laboratory with a unit weight close to the field samples. Both laboratory and field samples were subjected to a curing period of 14 days.

Stiffness measurements were performed using the bender element tests and strength tests were performed using unconfined compressive strength tests. Also, to evaluate the effectiveness of this treatment technique for expansive behavior, free swell tests and linear shrinkage bar tests were performed on both laboratory and field specimens. The test results depicted that the shear wave velocity for moderate and high PI treated soils at both curing periods of 7 and 14 days varied from 24.6 (170 MPa) to 43.6 psi (301 MPa) and 25.5 (176 MPa) to 46.7 psi (322 MPa), respectively. The improvement in stiffness of treated soils, when compared to the control soils, was approximately 4–7 times for Site 1 and 5–9 times for Site 2. Table 2 presents the field to laboratory strength and stiffness ratios.

Table 2 Strength and stiffness ratios

Site	$G_{\max,\text{field}}/G_{\max,\text{ lab}}$	$q_{u,\text{field}}/q_{u,\text{lab}}$
1	0.43–067	0.67–0.70
2	0.56–0.65	0.83–0.86

From Table 2, it can be inferred that the field strength values were 20–30% lower than the laboratory measurements, whereas the field stiffness measurements were 40% lower than the laboratory measurements. Also, in order to evaluate the in situ stiffness measurements, field tests were performed using natural gamma logging, down hole P-wave velocity and the SASW testing. Natural gamma-ray measurements and Downhole P-wave velocity tests were performed in the cased boreholes at each site. SASW tests were performed along two parallel lines (to balance the effect of wave paths relative to the DSM columns for shallow depths) in the treated area and one line in the untreated area (outside the treated area) at each site.

The natural gamma logging tests were performed to detect the variations in the natural radioactivity originating from changes in concentrations of the trace elements Uranium (U) and thorium (Th) as well as changes in concentration of the major rock forming element potassium (K). The downhole tests were performed to estimate the seismic wave velocity profile. In the DSM column at two test sites, P-wave velocity ranges from 1080 to 1140 m/s, whereas in the untreated areas the P-wave velocity averages around 780 m/s which is significantly lower than the global average value in the treated area. The SASW tests were performed to evaluate the shear wave velocity profile in the treated and untreated areas.

In order to evaluate the performance of the DSM technique in expansive clays at in situ condition, periodic measurements were taken in the field for two years. The Gro-Point moisture probes were installed in field to measure the moisture content. The average moisture content in site 1 varied from 13 to 30% over a depth of 14 ft. (4.3 m), whereas in site 2 the moisture content varied from 24 to 30% over a depth of 14 ft. (4.3 m). In order to measure the swell and shrink movements, the horizontal and vertical inclinometer casings were installed in both treated and untreated sections of sites 1 and 2. It was observed that the range of surface movements of Site 1 is 0.07–0.74 in. (0.17–1.87 cm) and 0.12–0.63 in. (0.3 to 1.6 cm), respectively, and of Site 2 for Phases I and II are 0.06–0.12 in. (0.15–0.3 cm) and 0.01–0.25 in. (0.025–0.63 cm), respectively. The site 2 depicted less movement due to increase in the treatment area ratio when compared to site 1. Also, periodic measurements were obtained using both downhole testing and SASW testing. It was observed that in both the tests, the initial measurements taken in the first year is considerably higher than the measurements taken over the next two year period. However, the treated sections depicted considerably higher values when compared to the untreated sections.

Overall, the comparisons between the field and laboratory test results indicated that the stiffness ratio $G_{\max,\text{field}}/G_{\max,\text{lab}}$ for Site 1 and Site 2 specimens varied between 0.43–0.67 and 0.56–0.65, respectively. The strength ratios ($q_{\text{ucs,field}}/q_{\text{ucs,lab}}$)

for Site 1 and Site 2 varied from 0.67 to 0.70 and 0.83 to 0.86, respectively. Both stiffness and strength ratios indicate that the field stiffness and strength values are 40 and 20–30% lower, respectively, when compared to the laboratory treatments. The P-wave velocities of the treated soil column zones exhibited higher values than those recorded in the untreated soils. Also, the same measurements for the three consecutive yet different years showed a decrease in the P-wave velocities.

4 Conclusions

Quality assurance studies play a key role in evaluating the new techniques that are implemented in geotechnical engineering projects. In this research paper, two quality assurance techniques performed in two different case studies were discussed. In both the studies, the quality assurance studies are highly challenging demanding for new techniques. In the first case study, the implementation of SASW testing along with geostatistical theories provided a real time assessment of the stiffness of CLSM with respect to curing period. In the second study, the quality assurance procedures along with advanced statistical analysis have provided the actual performance of the DSM technique.

Acknowledgements The authors would like to acknowledge Mr. David Marshall, Ms. Shelly Hattan, and Mr. Richard S. Williammee for their help and excellent cooperation throughout the research.

References

American Concrete Institute, Committee 229, Controlled Low-Strength Materials (CLSM). 1994. *ACI 229R-94 Report*.

Armstrong, M. 1994. Is research in mining geostats as dead as a dodo? In *Geostatistics for the next century*, ed. R. Dimitrakopoulos, 303–312. Dordrecht: Kluwer Academic.

Boschert, J., and J. Butler. 2013. CLSM as a pipe bedding: Computing predicted load using the modified Marston equation. *ASCE Pipelines* 2013: 1201–1212.

Bredenberg, H., G. Holm, and B.B. Broms (eds.). 1999. Dry mix methods for deep soil stabilization. In *Proceedings of the international conference on dry methods for deep soil stabilization*, Stockholm, 358 p.

Bruce, D. 2001. *An introduction to the deep mixing methods as used in geotechnical applications. Volume III. The verification and properties of treated ground.* Report No. FHWA-RD-99-167, US Department of Transportation, Federal Highway Administration.

Chittoori, B., A.J. Puppala, R. Reddy, and D. Marshall. 2012. Sustainable reutilization of excavated trench material. In *Geocongress 2012*, ASCE, Reston, VA, 4280–4289.

Chittoori, B., A. Puppala, and A. Raavi. 2014. Strength and stiffness characterization of controlled low-strength material using native high-plasticity clay. *Journal of Materials in Civil Engineering* 26 (6): 0401400.

EuroSoilStab. 2002. *Design guide soft soil stabilization*. Project No. BE 96-3177, Ministry of Transport Public Works and Management.

Folliard, K.J., David, Trejo, S.A. Sabol, and D. Leshchinsky. 2008. *Development of a recommended practice for use of controlled low-strength material in highway construction*, NCHRP 597 Report, 5–39.

Isaaks, E.H., and R.M. Srivastava. 1989. *An introduction to applied geostatistics*. New York: Oxford University Press.

Kienow, K., and K. Kienow. 2009. Most pipeline failures can be prevented by proper inspection. *Pipelines* 2009: 182–197. https://doi.org/10.1061/41069(360)18.

Krige, Danie G. 1951. A statistical approach to some basic mine valuation problems on the Witwatersrand. *Journal of the Southern African Institute of Mining and Metallurgy* 52 (6): 119–139.

Madhyannapu, R.S., A.J. Puppala, S. Nazarian, and D. Yuan. 2010. Quality assessment and quality control of deep soil mixing construction for stabilizing expansive subsoils. *ASCE, Journal of Geotechnical and Geoenvironmental Engineering* 136 (January, 1), 119–128.

Mitchell, J.K. 1981. State of the art—Soil improvement. In *Proceedings of the 10th ICSMFE*, Stockholm, vol. 4, 509–565.

Mitchell, J.K., and V.S. Zoltan. 1984. Time dependent strength gain in freshly deposited or densified soil. *Journal of Geotechnical Engineering, ASCE* 110 (11): 1559–1576.

Nazarian, S., and K.H. Stokoe II. 1984. In situ shear wave velocities from spectral analysis of surface waves. In *Proceedings of the 8th world conference on earthquake engineering,* vol. III, 31–38. Englewood Cliffs, New Jersey: Prentice-Hall, Inc.

Phoon, K., and F.H. Kulhawy. 1999. Characterization of geotechnical variability. *Canadian Geotechnical Journal* 36: 625–639.

Porbaha, A. 1998. State-of-the-art in deep mixing technology, Part I: Basic concepts and overview of technology. *Ground Improvement* 2 (2), 81–92.

Puppala, A, J., and S. Hanchanloet. 1999. Evaluation of chemical treatment method (sulphuric acid and lignin mixture) on strength and resilient properties of cohesive soils. In *Proceedings of the 78th Transportation Research Board annual meeting*. Washington, DC: Transportation Research Board.

Puppala, A.J., A. Porbaha, V. Bhadriraju, and E. Wattanasanthicharoen. 2004. In situ test protocols for quality assessments of deep mixing columns. In *Geo-Trans 2004*, ASCE Geotechnical Special Publication No. 126, Los Angeles, 2004, 1429–1438.

Puppala, A.J., P. Balasubramanyam, and R.S. Madhyannapu. 2007. Experimental investigations on properties of controlled low-strength material. *Ground Improvement Journal* 11 (3): 171–178.

Puppala, A.J., and D. Perez. 2009. New ground improvement techniques for expansive soils. In *12th geotechnical conference*, Bogota, Colombia, Keynote Lecture, October 22–25, 2009.

Puppala, A.J., S. Saride, and R. Williammee. 2012. Sustainable reuse of limestone quarry fines and RAP in pavement base/subbase layers. *Journal of Materials in Civil Engineering,* 418–429. https://doi.org/10.1061/(asce)mt.1943-5533.0000404.

Raavi, A. 2012. *Design of controlled low strength material for bedding and backfilling using high plasticity clay.* M.S. thesis, University of Texas, Arlington, TX, 118.

Rajah S., M. McCabe, and J. Plattsmier. 2012. Classification and specification of bedding and backfill for buried pipelines. ASCE Pipelines 2012: Innovations in Design, Construction and Maintenance, 940–951.

Raju, V.R. (2010) Ground improvement—Applications and quality control. In *Proceedings of Indian geotechnical conference*, 121–131.

Rathmayer, H. 1996. Deep mixing methods for soft soil improvement in the Nordic countries. In *Proceedings of the 2nd international conference on ground improvement geosystems, grouting and deep mixing*, 14–17 May, Tokyo, 2, 869–877.

Rathmayer, H. (ed.). 2000. Grouting, soil improvement including reinforcement. In *Proceeding for the 4th international conference on ground improvement geosystems*, Helsinki, 570 p.

Sondermann, W., and W. Weher. 2004. Round Improvement, 2nd ed, 57–92. Spon Press, Taylor and Francis Group.

Stokoe II, K.H., G.J. Rix, and S. Nazarian. 1989. In situ seismic testing with surface waves. In *Proceedings of the 12th international conference on soil mechanics and foundation engineering*, Rio De Janiero, 331–334.

Terashi, M., and H. Miki. 1999. Importance of prediction in ground improvement. In Prediction and performance of ground improvement, 1–10. Practitioners Series No. 11. Japanese Geotechnical Society.

Liquefaction Screening—Non-plastic Silty Sands and Sands

S. Thevanayagam, U. Sivaratnarajah, V. Veluchamy and Q. Huang

1 Introduction

1.1 Soil Liquefaction and Screening

Current liquefaction screening techniques rely on knowledge from extensive laboratory research conducted on liquefaction resistance of clean sands, and extrapolations of observed field performances during past earthquakes (NCEER 1997; Youd et al. 2001). Such observations have been documented in the form of normalized penetration resistances (SPT $(N_1)_{60}$, CPT q_{c1N}) (Seed et al. 1983; Robertson and Wride 1998), and shear wave velocity (v_{s1}) (Andrus and Stokoe 2000) versus cyclic stress ratio (CSR) induced by the earthquakes, corrected for a standard earthquake magnitude of 7.5, for many soil deposits where occurrence or non-occurrence of liquefaction were recorded during the earthquakes (Fig. 1). For liquefaction screening applications, the cyclic resistance ratio (CRR) of a soil deposit, applicable for number of cycles and frequency content relevant for a standard earthquake magnitude of 7.5, with a known value of $(N_1)_{60}$, q_{c1N} or v_{s1} for the site is obtained from a demarcation line drawn between the past field-observation-based data points which correspond to liquefied deposits and those that did not liquefy in Fig. 1. This is denoted as $CRR_{7.5}$. This $CRR_{7.5}$ value is

S. Thevanayagam (✉) · U. Sivaratnarajah · V. Veluchamy · Q. Huang
Department of Civil Engineering, University at Buffalo, Buffalo, USA
e-mail: theva@buffalo.edu

U. Sivaratnarajah
e-mail: umaipala@buffalo.edu

V. Veluchamy
e-mail: vijayakr@buffalo.edu

Q. Huang
e-mail: qhuang4@buffalo.edu

K. Ilamparuthi and R. G. Robinson (eds.), *Geotechnical Design and Practice*,
Developments in Geotechnical Engineering,
https://doi.org/10.1007/978-981-13-0505-4_2

Fig. 1 Silt content-dependent liquefaction screening charts