LANDSLIDE INVENTORY IN AN URBAN SETTING IN THE CONTEXT OF CHITTAGONG METROPOLITAN AREA, BANGLADESH

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ABSTRACT
Chittagong Metropolitan Area (CMA) is vulnerable to landslide hazards with an increasing trend infrequency and damage. Devastating landslides have hit CMA repeatedly in recent years. Under a project financed through ICIMOD of Nepal, detail survey has been conducted to prepare an inventory of landslides in CMA. Landslide events have been observed to occurattimes of much higher rainfall amount compared to the monthly average. Moreover, rapid urbanization, increased population density, improper land-use, cutting of hills, indiscriminate deforestation and agricultural practices are aggravating the landslide vulnerability in CMA. A landslide inventory is helpful for landslide modelling, runoff modelling and urban land-use planning.A three step methodology, identifying past landslide events from news archive, information from local people and satellite image interpretation has been followed. An inventory of 57 landslide events has been prepared which includes landslide locations, types, dimension, activity, potential causes of landslides, triggering mechanism and slope stabilizing mechanism. This inventory is expected to be a useful resource for future landslide studies in this port city of Bangladesh.

Keywords: Landslide; inventory map; Chittagong city; urban setting; remote sensing.

INTRODUCTION
In Bangladesh landslide events mainly occur in the hilly parts ofChittagong Division. Chittagong is the largest port and second largest city of Bangladesh andplays asubstantial role in the economic development of Bangladesh. Locational advantages and opportunities lead to rapid urbanization and compact urban form(Rahman, 2012). Landslides have occurred frequently in Chittagong city due to extreme rainfall. The devastation aggravates with weak geological structure, unplanned and erratic use of hills and settlement development. The unplanned and haphazard urbanization (Rahman et al., 2012), land-cover change, coupled with the increased intensity and frequency of heavy rainfall, is causing landslides in Bangladesh. Land cover changes (e.g. urbanization, deforestation) cause large variations in the hydro-morphological functioning of hill-slopes, affecting rainfall partitioning, infiltration characteristics and runoff production(Chau et al., 2004).Many urban dwellers and their livelihoods, quality of life, property and future prosperity are being continuously threatened by the risks of rainfall triggered landslides(Ahmed et al., 2014a). The development authorityof Chittagong hasidentified 30 risky hills (Chakraborty and Uddin, 2014) among 88 hills of 18304 acre(Chisty, 2014). More than 10,000 people are currently living in such vulnerable areas. People are living at the toe and on the slopes of hills with high risk of landslides and associated damage(Mia et al., 2016). Many devastating landslides occurred in these hills in recent past. Landslides triggered by heavy rains in Chittagong claimed at least 185 lives in the last seven years.
Landslide is a term generally used to describe the downward movement of a portion of a hill slope containing soil, rock, and organic materials under the effects of gravity and also the landform that results from such movement (Highland et al., 2008). Landslide inventory can be seen as datasets of multiple events which may include but are not limited to landslide locations, date of landslide, type of the landslide, potential causes, and damage information(Hervás, 2013). A past disaster event can be

seen as an opportunity to learn the lesson to enhance future disaster mitigation capacity (Rahman and Kausel, 2012). Thus, it is important to have a critical evaluation of past landslides to understand the causes and issues related to these events. Landslide inventory is one of the most important data for many landslide studies such as susceptibility mapping(Ahmed, 2015; Cardinali et al., 2006), landslide hazard zonation(Anbalagan et al., 2015), slope instability recognition (Soeters and van Westen, 1996), spatial distribution of mass movement. This kind of information can also be useful for urban land-use planning (Rahman and Islam, 2013). A detail landslide inventory for Cox′s Bazaar and Teknaf Municipalities has been prepared by Comprehensive Disaster Management Program (CDMP II, 2012) which can be considered as first attempt for this kind of study in Bangladesh. This attempt mentioned many challenges to prepare landslide inventory in small and medium towns. Preparation of landslide inventory is much more challenging in a fast growing large urban area.

It is important to understand the process and pattern of all landslides of a particular area from publishedrecords of landslide events of previous years. But there is no such record for Chittagong Metropolitan Area (CMA). Only the landslide events with casualty receivethe attention through newspaper and media(Ahmed et al., 2014b). Therefore, the aim has been to prepare a detailed landslide inventory ofCMA by incorporating the information from published media, information from local witness of the event, and from satellite image. This is probably the first attempt toprepare a landslide inventory for the second largest metropolitancity ofBangladesh. This paper presents the methodology, challenges and key findings of the landslide inventory work carried out in CMA under a project financed through International Center for Integrated Mountain development (ICIMOD), Nepal.

METHODOLOGY

Any study related to landslide hazard and risk begins with the investigation where previous landslide occurred. It is difficult to identify and characterize past events for variety of reasons. People might forget the event; an event may not be documented. Small landslide may not catch the attention to be documented properly, if this event did not involve casualty. Finding old document is much more challenging in developing countries where detailed documentation is overlooked in many cases. Furthermore, landslide inventory is challenging in an urban setting because rapid urbanization takes place on the site of landslide. The hills in Chittagong are small and in between urban area. Thus the signature of these landslides disappear quickly.

There are four fundamental assumptions summarized by Guzzetti et al., (2012) on which landslide inventory relies on. These assumptions are (i) landslide leave discernible signs, most of which can be recognized, classified and mapped in the field by image interpretations techniques. (ii) Landslides can be identified and mapped using a variety of techniques and tools, including the morphological signature depend on the type and the rate of mass movement. (iii) Landslide do not occur randomly or by chance. (iv) For landslides, geomorphologists adopt the principle that “the past and present are keys to the future”. Many techniques for landslide inventory are available depending upon location, scope and purpose, scale of the map, and available resource for inventory preparation. Among these available tools and techniques, someof these techniques are popular and applied by many scientists around the world. These approaches are geomorphological field mapping (Brunsden, 1993), information collection on landslide from published documents (Guzzetti et al., 1994), visual image interpretation and stereo vision (Tsai et al., 2010; Cheng et al., 2004), surface or digital elevation monitoring (Tarchi et al., 2003; Farina et al., 2006).With the improvement of GIS and GPS technology, landslide inventory mapping is being facilitated by the location accuracy and mapping capacity of these advance technologies. Digital landslide inventory can be built using satellite based GPS system and GIS software. These mapping techniques have multiple benefits such as it can easily be incorporated with other data set, landslide can be visualized by type or other parameters, and the inventory data can also be used for data driven hazard and risk analysis.Guzzetti et al., 2012 suggest different techniques based on the scale of the map. Small scale inventory can be completed using data from literature, inquiries from public and from private organizations, searching journal, newspaper as well as from interview of experts and local people. Medium scale inventories can be done by systematic interpretation of images, photo interpretation, digital elevation monitoring and extensive field investigations.

This study is aiming for large scale inventory in an urban setting. The limited budget, limited access to aerial photograph and satellite image,as well as limited documentation has made this inventory mapping complex. This study integrates all of the techniques mentioned aboveexcept digital stereo interpretation.

Landslide Information Searching

The information on landslideshave been searched in documents, reports, newspaper, digital archives etc. The information on date and casualty are availablein news media for those landslideswhich have casualty. Therefore, the information on these events such as location, intensity, damage have been listed for further crosscheck and additional information.An initial field visit has been conducted to collect available information on past landslide events.During this initial visit, documents on landslide events and management have been collected from different government authorities, organizations and stakeholders. A list of vulnerable hills in CMA has been made with the help of the information from Department of Environment (DoE) and Chittagong Development Authority (CDA). Following a major landslide event, a committee comprising members from different stakeholders has always been formed to investigate that event. The investigation reports on such landslide eventswere also helpful for the documentation. A questionnaire was prepared to gather landslide information from local people and witness of the landslide events.

Field Survey

During the field survey, it was difficult to identify the exact landslide occurrence locations. Landcoveralteration process remove the signature of landslides. In this field visit, information on location name, coordinates (latitude, longitude), area of displacement mass, rainfall information, landslide mechanism (type of movement, state, distribution, style, water content, material), existing land cover/ use type, causes of movement, landslide history (date of occurrence, duration of rainfall), consequences (casualties, injuries, damages, impacts) and future risk of landslide were collected as much as was available.Location name was collected by interviewing people. Coordinate values of landslide locations was determinedby using GPS [Fig.1-a]. In case of some restricted/unreachable places, information was collected through interpreting Google earth image. The displacement of mass has been measured where possible [Fig.1-b]. Ground photographs of landslide site taken during field visit has also been added to the inventory [Fig.1-c]. Besides, landslide mechanism, causes of movement, landslide history and consequences have been collected from the local people as the collected documents could not provide sufficient information. The daily rainfall during and before the event from Bangladesh Meteorological Department is added to the inventory. However, for some landslide events, the exact date could not be found, as a result the rainfall data could not be provided for those landslide events in the inventory.

Figure 1: Field Survey (a) Taking GPS measurement (Latitude, longitude, and elevation) (b) Measuring displacement of mass, (c) Taking Photographs; Source: Field Survey, August, 2014

Visual Image Interpretation

Many landslides have been identified, however some of the information was difficult to find, for instance the surface area of displaced mass. The signatures of past landslide have been removed through land cover alteration and morphological change [Fig.2]. The goal is to draw the polygon of displacement mass to support landslide studies based on landslide density (e.g. empirical modelling such as information value, weights of evidence). The Google Earth time slider tool is providing free access to fine resolution(pan-sharpened submitter originally 2.5m DigitalGlobe image) historic image.

The visual image interpretations technique has been applied to draw the polygon around the displacement mass. The figure 2 shows Kaichaghona area near Chittagong cantonment where a devastating landslide in June 2007 took many lives from foothill slum. Documentation of such landslide events in these restricted area is much more challenging. The remote sensing imageriesare useful to deal with these challenges.Landslide signature extraction from visual or digital interpretation of remote sensing time series imagery can help landslide mapping.

Figure2: Image shows the signature of October 29,2007 landslide (left) and the current situation during field visit in October 19,2014 (right);Source: Google Earth

Preparation of Inventory Map

The final step is to prepare landslide inventory. The coordinate pairs from field survey collected by GPS have been plotted using geographic coordinate system WGS 1984. The Google Earth historical images have also been downloaded. Polygons around displacement mass have been drawn based on the signature on satellite image captured after the event. The polygons are then projected to UTM 46 N system to map with other database such as Digital Elevation Model (DEM) and surface geology. Other attributes such as landslide types, activity area of displacement mass have been built with IDs related to spatial locations. The inventory is a GIS data layer shown as inventory map [Fig.3] which is handy and useful for landslide related studies.

RESULTS

Theoutcome of this study is an inventory of total 57 past landslide events in CMA.Majority of the area in this metropolitan are flat plains with no landslide activity. Hills are located in only central and north-west parts of the city. Landslides are accrued mainly between 30 to 60 m elevation above mean sea level. The natural slope of the area is gentle slope, however hill cutting activity makes some slopes steeper. Based on the slope calculated from Aster Global DEM (GDEM) data, landslides have mainly occurred between 30-40-degree slope. Because of the course resolution (30m) of ASTER GDEM, slope value may be showing lower slope than the actual. Thus these landslide locations might be actually steeperslope condition. During the field survey it has been observed that there are many landslide locations whose slope is near vertical.Most of the landslides are located in Dupitila formation orTipam sandstone geological class [Fig. 3]. It is difficult to judge the landslide state, style and types of movement after several years of occurrence where most of the signs of landslide have been removed. Therefore, most of the judgements are subjective and based on the description of the witness of the event. These witness are not the landslide expert, therefore the confidence of these judgement is not very high. From these subjective judgements, the types of the mass movement are mainly slide, fall and topple. Many of the landslidesare still active because mud is falling down every year from these locations. Some of the sites are stabilized by the structural measures. Few of the landslide location became dormant because no activity took place after the event. The style of the landslidesmovement is single in most of the cases. The possibility of multiple landslide is very lowdue to the small hill area in urban setting. The landslide map [Fig. 3] and attribute table [AppendixA] show the detail information of each event.

Factors Affecting Landslide Distribution

The rainfall characteristic in Chittagong district is different from the other districts in Bangladesh because of its geographical location and orographic effect. Usually in the monsoon period the rainfall shows highest precipitation. Most of the landslides in this region occurred during incessant and prolonged rainfall. A heavy rainfall,70mm or more over a shorter period of time may lead to a fatal landslide event in this hilly area. Rainfall less than threshold value but over many consecutive days may also lead to landslide. The months of June and July show most landslide occurrencesin Chittagong and show strong correlation between heavy rainfall and landslides in the country.

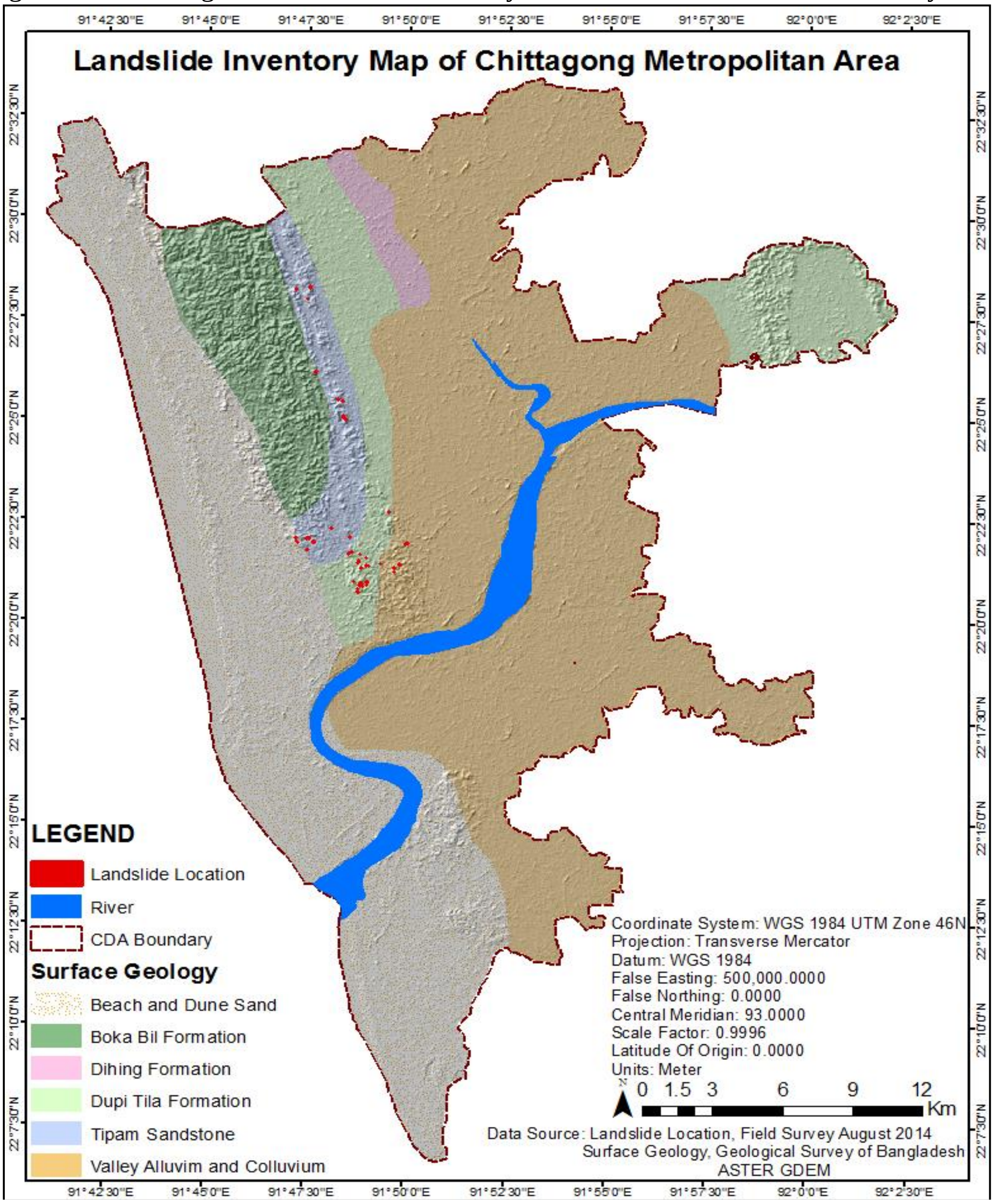

Figure 3: Land slide Inventory Map of Chittagong Metropolitan Area

Some Hillsin CMA are indiscriminately being flattened and are diminishing due tobuilding construction, residential development, road network development, soil mining. Some of the hill slopes are being steepened day by day and where hill cutting exists the slopes varies from 70-80 degrees which makes slopes unstable. The recent landslides in CMA were the result of hill cutting and steep slopes of the hills. The most affected areas are Khulshi, Panchlaish, BaizidBostami, Kotowali, Lalkhan Bazar, Sholoshahar, Foy's Lake, Pahartali and Polytechnic area. Most of the landslide have occured in the heart of the city becauseinformal settlements, high class residential area and commercial buildings are being established in the hill cutting context.The soil is mainly composed of silt and sand content. During the monsoon, heavy rainfall water cancreatesignificantpore water pressure in cracks which can destabilize the hill slope. Deforested areas of hills are easily exposed and top soils are eroded by surface run-off or by wind erosion. The vegetation covered areas are less vulnerable to landslide than deforested area. Seismic activities can also destabilize the hill slope triggering landslides. Besides hill in the Chittagong city inone third area are in low land whichexperiences abnormal tidal flow during monsoon period affecting its bottom.

CONCLUSIONS

A detail landslide inventory has been developed for the Chittagong Metropolitan Area. Landslides have appeared to beone of the most significant natural damaging disasters in CMA. The rapid hill cutting coupled with land cover change and increasedintensity of rainfallhas beencausing devastating landslidesin the port city. The landslide risk is very high in the city due to the vulnerability of homes built at the bottom of risky hill slopes.The landslide inventory is expected to be useful for future studies such as susceptibility modelling, slope stability modelling as well as urban land use planning. It is useful either for data driven modelling or for the validation of the result from deterministic or heuristic modelling. This study may be considered as first attempt for the documentation of landslides in Chittagong city from 1990up to 2014. Dates of some landslides are not known.Atotal of 57 landslides are documented and mapped in this study. This inventory shows all landslide prone locations, historical landslides, landslide mechanism, elevation,area of displaced mass, casualty and future risk. Documentation of past event is always challenging. Rapid change in land cover, hill cutting activity in a fast growing urban context, poor documentation in a developing country like ours have made the inventory preparation achallenging task. Considering all challenges, it was a formidable job to prepare the first landslide inventory for the port city of Chittagong.

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Appendix A: Detail Landslide Inventory of Chittagong Metropolitan Area

ID	Latitude	Longitude	Elevation (m)	Date (d-m-y)	Hill Name	Area of mass (m2)	Types	State	Style	Rainfall (mm)	Casualty
1	22.3476	91.8138	55.95	28/07/2013	Motijhorna	89.91	Fall	Stabilized	Single	26	2
2	22.3473	91.8147	51.79	28/07/2013	Motijhorna	116.32	Fall	Stabilized	Single	26	2
3	22.3480	91.8165	44.46	11/6/2007	TankirPahar	11.02	Topple	Active	Single	88	0
4	22.3493	91.8166	32.56	22/06/2014	Chanmari Bi Lane	15.03	Topple	Stabilized	Single	54	0
5	22.3484	91.8143	41.18	1998, 2013	TankirPahar	331.84	Slide	Reactivated	Single	26	22
6	22.3488	91.8139	47.04	15/06/2010	TankirPahar	211.06	Slide	Reactivated	Single	77	11
7	22.3479	91.8131	44.26	23/8/2008	Batali Hill	59.1	Fall	Active	Single	50	0
8	22.3464	91.8127	55.03	1/7/2011	Batali Hill	126.7	Fall	Active	Successive	55	0
9	22.3451	91.8128	31.66	1/7/2011	Tiger Pass	427.04	Topple	Active	Single	55	17
10	22.4174	91.8064	30.82	11/6/2007	Lebu Bagan	757.61	Slide	Dormant	Single	88	30
11	22.4179	91.8062	41.22	11/6/2007	Lebu Bagan	84.56	Slide	Dormant	Single	88	5
12	22.4180	91.8062	40.19	11/6/2007	Lebu Bagan	50.17	Slide	Dormant	Single	88	2
13	22.4246	91.8041	37.64	11/6/2007	Kaichaghona	145.5	Slide	Stabilized	Single	88	2
14	22.4239	91.8058	24.71	11/6/2007	Kaichaghona	582.27	Slide	Stabilized	Single	88	7
15	22.4330	91.8073	46.07	11/6/2007	Lebu Bagan	1359.5	Slide	Dormant	Single	88	40
16	22.4360	91.7949	40.68	11/6/2007	Sekandar Para	181.7	Slide	Stabilized	Single	88	3
17	22.4358	91.7946	45.36	11/6/2007	Sekandar Para	198.89	Slide	Stabilized	Single	88	2
18	22.4362	91.7948	48.51	11/6/2007	Sekandar Para	211.61	Slide	Stabilized	Single	88	4

Basic Information

ID	Latitude	Longitude	Elevation (m)	Date (d-m-y)	Hill Name	Area of mass (m2)	Types	State	Style	Rainfall (mm)	Casualty
19	22.3551	91.8141	32.39	-	Espahani Hill	233.06	Fall	Active	Successive	-	0
20	22.3559	91.8172	26.98	-	Kusumbagh	152.79	Fall	Active	Successive	-	0
21	22.3560	91.8167	36.68	-	Goribullal Shah	241.79	Topple	Active	Successive	-	0
22	22.4711	91.7920	39.81	11/6/2007	Ctg. University	390.34	Topple	Active	Single	88	0
23	22.4702	91.7861	38.64	11/6/2007	Ctg. University	1134.77	Slide	Active	Single	88	0
24	22.4706	91.7921	35.18	11/6/2007	Ctg. University	313.42	Topple	Active	Single	88	0
25	22.4660	91.7910	37.92	24/06/2000	Ctg. University	212.7	Topple	Active	Single	46	0
26	22.3672	91.7869	58.72	1990	GolPahar	105.38	Fall	Active	Successive	-	0
27	22.3653	91.7876	45.69	1990	GolPahar	157.07	Fall	Active	Successive	-	0
28	22.3657	91.7872	45.12	1990	GolPahar	77.81	Slide	Active	Successive	-	0
29	22.3624	91.7917	28.41	11/6/2007	Akbar Shah Hill	213.26	Fall	Active	Single	88	7
30	22.3658	91.7943	38.51		Lal Pahar	153.55	Topple	Active	Successive	-	0
31	22.3672	91.7924	45.42	26/06/2012	Foy'z lake Obser.	456.7	Slide	Active	Successive	111	0
32	22.3673	91.7918	46.51	26/06/2012	Foy’z Lake Zoo hill	209.12	Slide	Active	Successive	111	0
33	22.3667	91.7914	34.63	26/06/2012	Foy’z Lake Zoo hill	45.86	Slide	Active	Successive	111	0
34	22.3672	91.7912	48.67	26/06/2012	Foy’z Lake Zoo hill	75.88	Slide	Active	Single	111	0
35	22.3675	91.7915	56.36	26/06/2012	Foy’z Lake Zoo hill	232.52	Slide	Active	Single	111	0
36	22.3575	91.8129	19.84	3/8/2005	Nasirabad Housing	208.57	Slide	Active	Single	25	1
37	22.3592	91.8160	23.5	3/8/2005	Nasirabad Housing	242.53	Slide	Active	Single	25	1
38	22.3609	91.8137	35	28/07/2013	Jakir Hossain Road	56.05	Fall	Active	Single	26	0
39	22.3617	91.8102	26.57	28/07/2013	Holy Crescent	52.3	Slide	Active	Single	26	0
40	22.3679	91.8092	13.93	-	AKS Brick Field	301.06	Slide	Active	Single	-	0
41	22.3715	91.8016	15.93	-	Krishnochura	145.06	Slide	Active	Single	-	0
42	22.3716	91.8013	15.11	-	Krishnochura	76.43	Topple	Active	Single	-	0
43	22.3583	91.8131	32.44	-	Nasirabad Housing	175.81	Topple	Active	Single	-	0
44	22.3610	91.8090	19.33	28/07/2013	Holy Crescent	50.26	Topple	Stabilized	Single	26	0
45	22.3551	91.8278	46.4	-	Finley Hill	130.32	Slide	Stabilized	Successive	-	0
46	22.3556	91.8277	50.12	-	Finley Hill	118.34	Slide	Stabilized	Single	-	0
47	22.3536	91.8281	29.28	-	Dolphin Hill	47.04	Slide	Active	Single	-	0
48	22.3564	91.8303	27	May-14	Medical Hill	16.5	Fall	Active	Single	-	0
49	22.3564	91.8302	23.12	May-14	Medical Hill	31.67	Fall	Active	Single	-	0
50	22.3650	91.8333	18.1	28/07/2013	The King of Ctg.	184.13	Slide	Active	Single	26	0
51	22.3651	91.8326	18.1	28/07/2013	The King of Ctg.	191.64	Slide	Active	Single	26	0
52	22.3566	91.8299	21.59	01//07/2011	Medical Hill	226.23	Fall	Active	Single	55	0
53	22.3566	91.8301	22.64	01//07/2011	Medical Hill	136	Fall	Active	Single	55	0
54	22.3493	91.8113	48.36	28/07/2013	AK Khan hill	33	Fall	Active	Single	26	0
55	22.3782	91.8255	37.54	-	Amin Textile	71.93	Topple	Active	Single	-	0
56	22.3782	91.8256	34.21	-	Amin Textile	71.93	Topple	Active	Single	-	0
57	22.3568	91.8256	21.31	28/07/2013	Blossom Garden	188.59	Slide	Stabilized	Single	26	0

***Note: GIS database of this inventory can be released on request for any research purpose. Please contact with corresponding author for the GIS database.*

Appendix B: A Sample of Detail Inventory

Landslide ID :05
Landslide Location: Tanker Pahar, Motijhorna
Coordinates: 22˚ 20’54.27’’N, 91˚ 48’51.60’’E

Datum: WGS 1984
Elevation (m): 41.18
Area of Displaced Mass (sqm): 331.84
Rainfall: Unknown

Source: *Field survey, August 2014*

Source: *Field survey, August 2014*

Landslide Mechanism

Type of Movement: Slide
State: Active, Reactivated, Suspended
Distribution: Advancing

Style: Single
Water Content: Moist
Material: Soil/Earth

Land Cover/Use Type (%):

Herbaceous vegetation is the Primary land cover of Tanker Pahar. Forest/ woodland type is also visible in this hill.

Causes of Movement:

Hill cutting is the major issue that caused landslide in this area and intense rainfall acted as a triggering factor for landslide.

Land Slide History and Future Risk of Landslide

Landslide in this site occurred in 1982, 1989,1991,1994,1996 and 2013. 10 houses got damaged and almost 22 people died due to landslide at different periods. Utility facilities were highly damaged in this incident. Economic activities were hampered so does the social life of people. Environment has been found to be severely damaged. Still there are many houses located at the down slope of the hill. Soil of this site has been found to be sandy. The escapement slope is found to be near vertical. The failed mass is a part of upper portion. Vertical Slope characteristics can be considered as a contributing factor to future landslide for this hill. Settlements located at the down slope of this hill are at a huge risk of massive landslide. The risk is high (Field survey, August 2014).

*** Note: For more details please see the technical report of the project(Ahmed et al., 2014b)*

GROUND INVESTIGATION AND RESPONSE OF JHILMIL RESIDENTIAL TOWN PROJECT

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ABSTRACT
In Bangladesh many low laying areas which remained under water are developing for residential and commercial purposes by sand filling. These areas are developing by different government and non-government organization by naming different projects. Such a project is Jhilmil Project of RAJUK (RajdhaniUnnayanKartipalha) located near the Dhaka city. Subsoil investigation was carried out in sixteen locations of the project. This paper represents the subsoil investigation reports and the ground response analysis of subsoil of Jhilmil project. SPT N values of sixteen boreholes were collected from Standard Penetration Test. Shear wavevelocity was determined by using universal correlation. The site response analysis was performed using DEEPSOIL (Hashash et al., 2011) V5.1. Equivalentlinear analysis was performed and the response spectrum, the PGA andthe amplification factor was determined and represented in this paper.

Keywords: SPT N; shear wave velocity; DEEPSOIL; PSA; PGA; amplification factor.

INTRODUCTION
Many historical earthquakes like Mexico earthquake (1985), Edgecumbe Earthquake (1987), San Francisco earthquake (1989), LosAngeles earthquake (1995) have established that local site conditions has significant role in the amplification of ground motion. In case of Bangladesh,April 2015 Nepal earthquake with a magnitude of 7.8, which is one of the strongest earthquake in the world killed 8857 in Nepal and about 600 in the region of Bangladesh and India and affected almost the whole of Bangladesh. For this reason accurate and proper soil investigation of a site has become an essential concern to grasp precise knowledge about site response and as well as seismic hazard. A geotechnical investigation was carried out at Jhilmil residential area, Keranigonj, Dhaka by the detailed sub-surface investigation program which includes sixteen (16) borings, execution of standard penetration test (SPT). Using the SPT N value, shear wave velocity was determined from empirical correlation equation and Equivalent linear site response analysis of the investigated area under a given earthquake motion was performed using program DEEPSOIL.

SITE INFORMATION
The study area “JhilmilResidential Town Project” is a ongoing project of RAJUK (RajdhaniUnnayanKartipakha) located near to the Dhaka city having latitude 23°40′N and longitude 90° 23′ E (Fig. 1). The soil profile of Jhilmil residential area is consists of an upper non-cohesive deposit of very loose sandy silt and silty fine sand. Occasional deposit of soft to medium stiff clay and clayey silt mixed with varying amount of fine sand upto the maximum depth of about 14.0 m from the existing ground surface. The deposit below upto the depth of exploration consists of non-cohesive deposit of medium dense to very dense silty fine sand mixed with trace amount of mica.

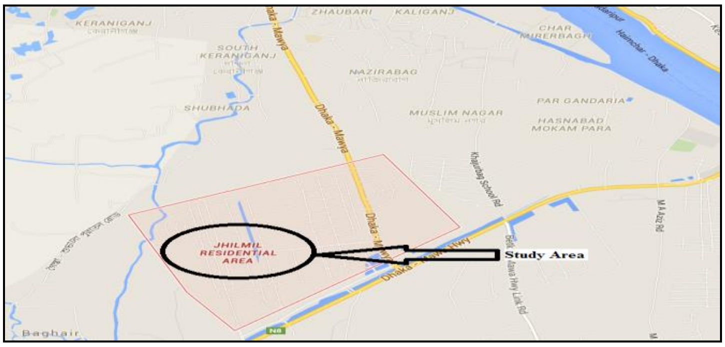

Fig. 1 Location map of the study area (Google map)

SUB SOIL INVESTIGATION

The field investigation was carried out by Dhaka Soil and the execution of total of sixteen borings which is up to maximum depth of 21 m from existing ground surface. Fig. 2 shows points of SPT Tests. Holes were made by driving the casing of 10cm (4”) diameter up to 1.83 m (6’-0”) depth. The distributed samples were collected at an interval of 1.5 m (5’) depth. Besides, The samples were collected by driving split spoon sampler which is of 3.15 cm (1-3/8”) inner diameter with a hammer of 63.5 kg (140 lbs.) weight falling freely at a height of about 76.2 cm (30”) in average and on the other hand, the number of blows required to drive the sampler for every 0.15 m (6”) penetration over 0.45 m 1.5 ft) depth was recorded to measure the standard penetration resistance-N per 0.30 m. Moreover, Shelby tubes are used for collecting the undisturbed samples having 7.62 cm (3”) diameter. In this case, the ground water table was recorded 24 hours after completion of each hole. The SPT N values with respect to depth of sixteen sites is shown in Fig. 3.

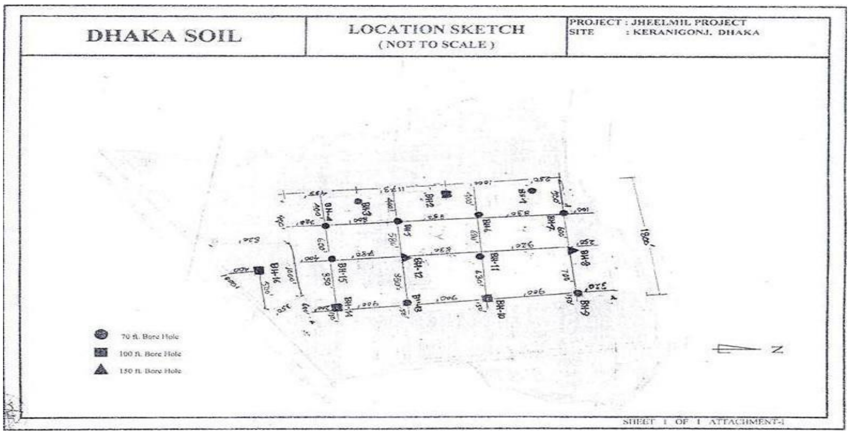

Fig. 2: Location Map of SPT Test points

Various samples of different depths were collected from different sites. They are visually examined and all undisturbed and representative disturbed samples are being selected for necessary testing. The following tests were performed on the selected samples.

- Natural Moisture Content.
- Liquid & Plastic Limit.
- Specific Gravity.
- Grain Size Analysis.
- Wet & Dry Density Test.
- Unconfined Compression Test.
- Consolidation Test.
- Direct Shear Test

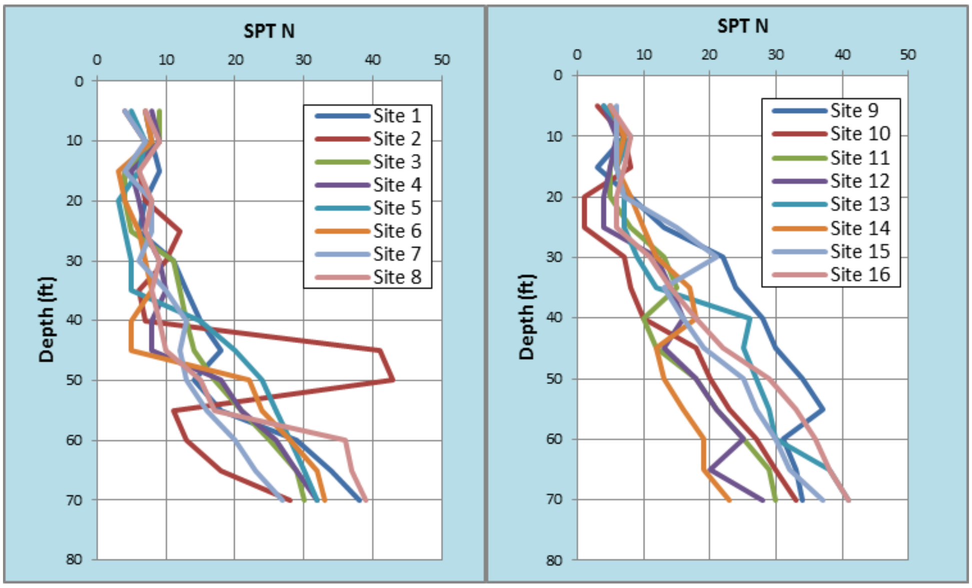

Fig. 3: SPT N Values with respect to depth

The results for different sites for different tests are represented in table 1 and table 2:

Table 1 Summary of laboratory test results (BH-1 to BH-7)

Bore Hole no.		1			2		3			4		5			6		7		
Depth (m)		1.50 to 1.95	7.50 to 7.95	12.00 to 12.95	9.00 to 9.45	21.00 to 21.45	4.50 to 4.95	5.55 to 6.00	15.00 to 15.45	6.00 to 6.45	16.50 to 16.95	5.55 to 6.00	7.50 to 7.95	12.00 to 12.45	4.50 to 4.95	18.00 to 18.45	5.55 to 6.00	6.00 to 6.45	21.00 to 21.45
Natural Moisture Content (%)							27.6	28.8				30.8	27.8				25.7	25.2	
Specific Gravity								2.670											
Atterberg Limits	Liquid Limit (LL)						30					41					38		
	Plastic Limit (PL)						26					23					27		
Density	Wet (P.C.F)							18.12			17.84						18.57		
	Dry (P.C.F)							14.07			13.64						14.77		
	Sand (%)	72	45	82	77	88	21		85	73	87		8	83	67	86		15	88
	Silt (%)	28	52	18	23	12	74		15	27	13		72	17	33	14		77	12
	Clay (%)	0	3	0	0	0	5		0	0	0		20	0	0	0		8	0
Consolidation Tests	Natural Void Ratio, Co							0.868											
	Compression Index, Cc							0.250											
Unconfined Compression Tests	Strain at failure (%)							14.0				18.0					12.0		
	Stress undist (P.S.I)							54.6				42.6					98.2		
Direct Shear Tests	(I) Degree			30.0										32.0					
	C (PSI)			0										0					

Table 2 Summary of laboratory test results (BH-8 to BH-16)

Bore Hole no.		8			9		10		11		12			13		14		15		16	
Depth in meter		3.00 to 3.45	15.00 to 15.45	30.00 to 30.45	9.00 to 9.45	18.00 to 18.45	6.00 to 6.45	24.00 to 24.45	12.00 to 12.45	19.50 to 19.95	6.00 to 6.45	5.55 to 6.00	15.00 to 15.45	12.00 to 12.45	18.00 to 18.45	9.00 to 9.45	21.00 to 21.45	4.50 to 4.95	10.50 to 10.95	7.05 to 7.50	21.00 to 21.45
Natural Moisture Content (%)											28.2	28.7									
Specific Gravity												2.675									
Atterberg Limits	Liquid Limit (LL)										42										
	Plastic Limit (PL)										23										
Density	Wet (P.C.F)											18.17								18.30	
	Dry (P.C.F)											14.11								14.43	
	Sand (%)	72	85	92	87	89	47	93	78	86	8		88	82	88	81	91	83	84		84
	Silt (%)	28	18	8	13	11	51	7	22	14	75		12	18	12	19	9	17	16		16
	Clay (%)	0	0	0	0	0	2	0	0	0	17		0	0	0	0	0	0	0		0
Consolidation Tests	Natural Void Ratio, Co											0.855									
	Compression Index, Cc											0.215									
Unconfined Compression Tests	Strain at failure (%)											16.0								14.0	
	Stress undist (P.S.I)											53.2								78.5	
Direct Shear Tests	(I) Degree		30.0					35.0					31.0								36.0
	C (PSI)		0					0					0								0

GROUND RESPONSE ANALYSIS

Equivalent Linear Site amplification was performed using the DEEPSOIL (Hashash, Y.M.A. et al., 2011). As input parameter in the deep soil, soil type, unit weight and the shear wave velocity according to depth were given. Shear wave velocity V_S was calculated from the SPT-N value using the following universal correlation equation (Ohta and Goto, 1978).

$$\mathbf{V_S = 85.35\ N^{0.348}} \quad (1)$$

The equation was chosen because it can be used for all types of soil (clay, fine sand, medium sand, coarse sand, sand and gravel, and gravel) and easy to use. The shear wave velocities of different sites along with depth are shown in table 3 below.

Table 3 Shear wave velocity with respect to depth

Depth (ft)	**Shear wave velocities (ft/S)**															
	Site 1	**Site 2**	**Site 3**	**Site 4**	**Site 5**	**Site 6**	**Site 7**	**Site 8**	**Site 9**	**Site 10**	**Site 11**	**Site 12**	**Site 13**	**Site 14**	**Site 15**	**Site 16**
5	168	135	179	172	146	164	135	168	149	125	149	138	138	149	159	149
10	176	164	179	179	164	172	164	183	168	168	168	159	176	168	159	176
15	183	155	135	146	155	122	135	159	125	176	149	149	159	159	159	168
20	168	164	135	155	122	135	172	176	176	85	149	138	168	176	168	159
25	159	198	146	164	135	155	172	168	208	85	176	138	168	190	219	159
30	197	186	192	179	146	164	155	183	250	168	208	203	183	203	246	197
35	208	155	198	186	146	172	186	176	258	176	219	214	203	229	208	214
40	219	164	203	172	214	146	203	183	272	190	190	224	265	233	224	233
45	233	303	209	172	236	146	198	190	279	233	203	208	262	203	238	250
50	214	309	223	228	252	244	203	219	291	242	233	233	269	208	262	275
55	233	192	240	240	259	252	219	229	300	254	246	246	275	224	269	288
60	275	203	255	259	266	266	236	297	282	269	262	262	279	238	279	297
65	291	228	269	269	272	278	248	300	288	279	275	242	303	238	285	303

70	303	266	272	278	278	281	262	305	291	288	279	272	311	254	300	311

As input motion, Kobe Earthquake was selected.Kobe Earthquake occurred on Tuesday, January 17th 1995 an earthquake of magnitude 7.2 on the Richter Scale struck the Kobe region of south-central Japan. The ground shook for only about 20 seconds but in that short time, over 5,000 people died, over 300,000 people became homeless and damage worth an estimated £100 billion was caused to roads, houses, factories and infrastructure. The time history of Kobe earthquake is shown in the fig. 4.

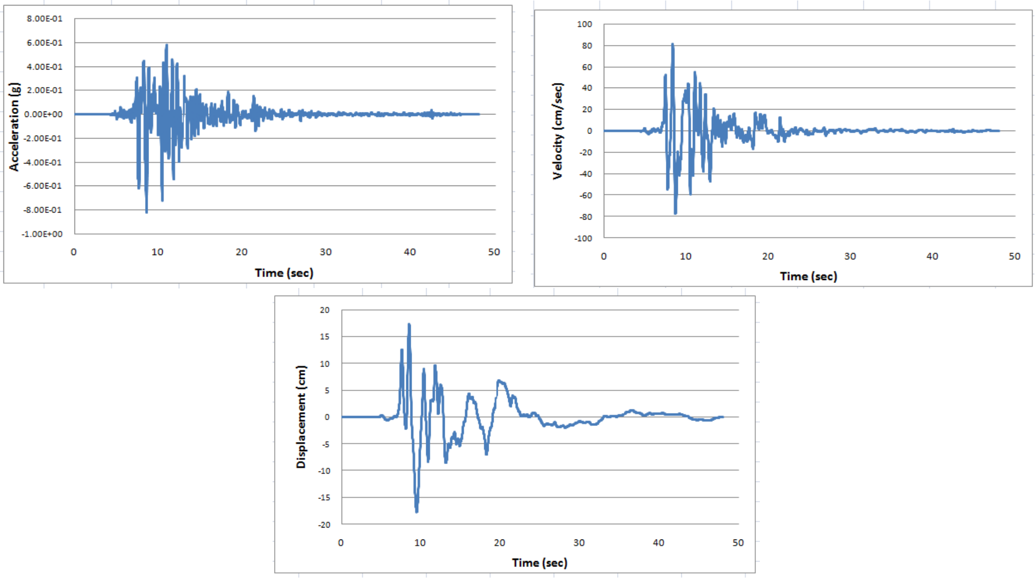

Fig. 4 Time history of Kobe earthquake

GROUND RESPONSE RESULTS

Response Spectrum of input motion and 16 bore holes are shown in fig.5. From the 16sites,site 9 and site 15 produce highest (0.28g) peak spectral acceleration (PSA) and site 10 producesthelowest (0.06g) peak spectral acceleration (PSA). It was observed that surface response in all locations were less than the response of Kobe.

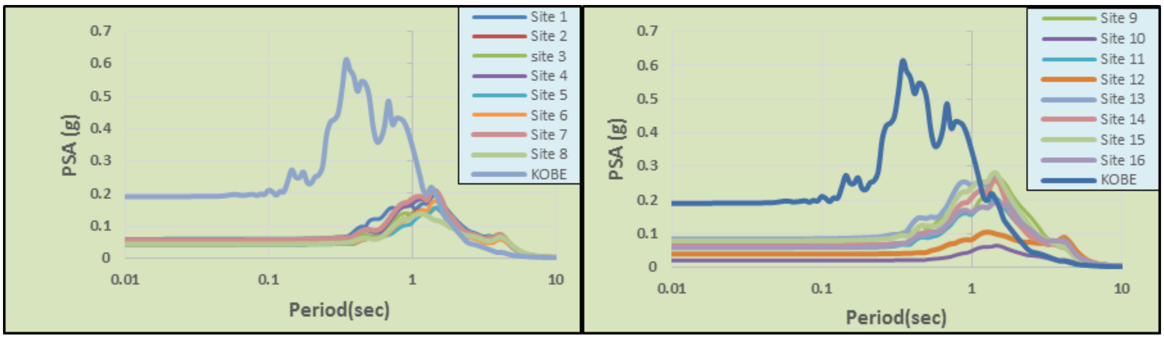

Fig. 5 Response Spectrum of different Sites

Peak Ground Acceleration (PGA) of different sites is represented in fig.6. PGA at surface and that at bedrock is obtained from the analysis. The peak ground acceleration values at surface are observed to be in the range of 0.002g (Site 10) to 0.008g (Site 13) and that of the bed rock were observed to be in the range of 0.08g (Site 12) to 0.17g (Site 15). The values were within the value of zone co efficient 0.15g of Dhaka city.

Site amplification factors at sub surface layers are used to measure the ground response. The amplification factor is the ratio of peak ground acceleration at surface to that of acceleration at hard rock.

Amplification Factor = PGA recorded at ground surface / PGA recorded at hard rock

The amplification factors of different sites are represented in the bar chart in fig. 7.The amplification factor ranges from 0.12 (site 10) to 0.52 (site 9).

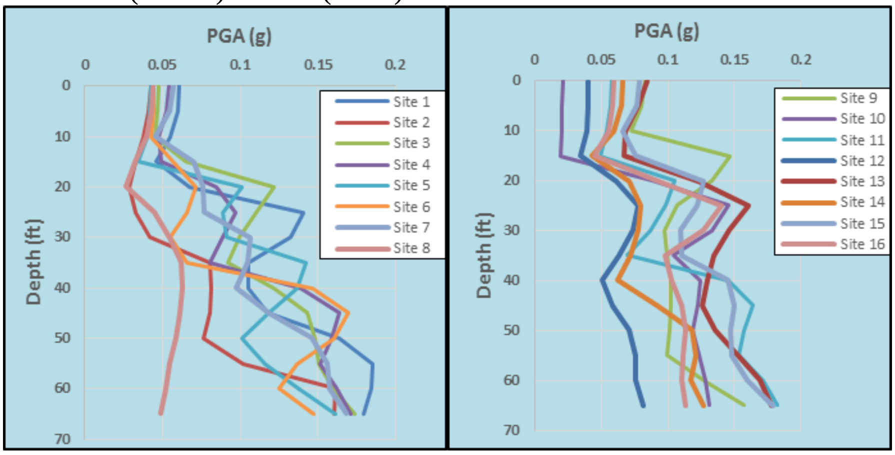

Fig. 6 Peak Ground Acceleration of different locations

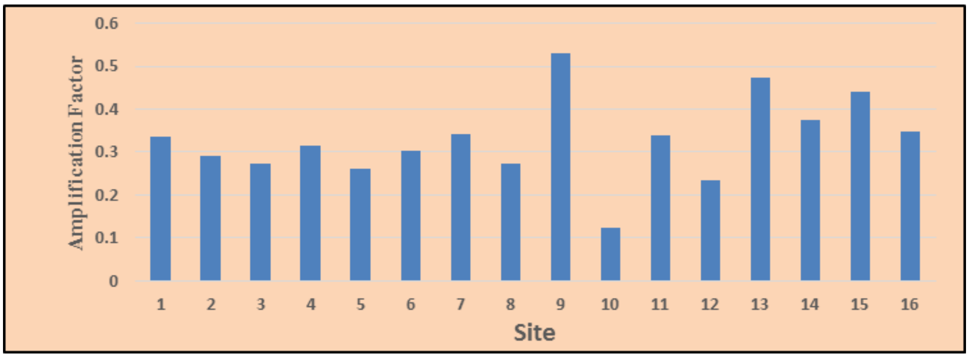

Fig. 7Amplification Factor of different locations

CONCLUSION

Jhilmil residential area covers 381.11 acres land is a new project which is taken by RAJUK. It will become an important place in Dhaka city as it was proposed to development with various infrastructures. Therefore the ground response analysis was performed for the area. The surface soil response was less than the input motion of the area. The PGA values were not too much high. The amplification factor of all locations was less than one. So the surface soil is not much vulnerable for earthquake like Kobe. It can be predicted that damage in this area will not so strong but anything can be happened. Thus deeper analysis is needed. Shear wave velocity should be measured using geophysical tests and using the values, site response analysis should be estimated.

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ANALYSIS OF SHORE PILE FOR DEEP EXCAVATIONS

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ABSTRACT
Deep excavations are essential for underground construction, but they also alter the ground conditions and induce ground movements which might cause risks to adjacent infrastructure. In this paper, a parametric study is carried out for the analysis of shore piles both in supported and unsupported conditions in cohesionless type of soil strata with theoretical approaches and developing models in ETABS based on Winkler spring model or subgrade reaction approach of modeling soil behavior. The study reveals the significant effect of modulus of subgrade reaction of soil and embedded length of shore pile in both unsupported and supported conditions. For different soil profiles based on modulus of subgrade reaction, comparisons are made for the variation of moments and lateral displacements with length of shore pile for a given depth of excavation and embedded length of pile in unsupported and supported conditions. Again, with varying embedded lengths of pile, variations of moments and lateral displacements are also observed for fixed values of modulus of subgrade reaction. The results obtained from the developed models are quite closer to that of theoretical approaches and thus the models work effectively in predicting results which can be improved by further studies to study more complicated cases.

Keywords: Deep excavation; shore pile; subgrade reaction; ETABS

INTRODUCTION
With economic development and urbanization, excavations go deeper and become larger in scale and structures in the immediate vicinity of excavations, dense traffic scenario, presence of underground obstructions and utilities have made excavations a difficult task to execute. In this context, analysis and design of proper deep excavations and their supporting systems are essential. When the excavation depth exceeds about 5 to 6 m, then steel sheet pile or rows of concrete piles are used around the boundary of the excavation. Diameter and spacing of the piles is decided based on soil type, ground water level and magnitude of design pressures. Very often, in order to reduce the lateral displacement and depth of embedment of piles, lateral supports or struts are provided. For different soil profiles and layers, theoretical analysis and design of shore piles and struts by conventional approach is a bit tedious process. Therefore, in this study, some models are successfully developed in ETABS for detail and quicker analysis of shore piles to compare the values of moments and lateral displacements with length of pile for different soil parameters below dredge line such as loose, medium and dense sand classified on the basis of modulus of subgrade reaction values in unsupported and supported state. This study also shows that after providing supports, lateral displacements and the depth of embedment of piles is also reduced. Once the analysis is complete, then design of the overall support system can also be readily done using the software and thus saving time.

DATA COLLECTION
In this study, data obtained from a soil report in construction site in Dhanmondi, Dhaka are used in analysis and building models in ETABS. In Table 2, the granular soils are classified into dense, medium and loose sand based on the values of unit weight of soil and friction angle according to the

values of Table 1 proposed by Terzaghi (Teng, 1988). Using these values, with the help of active and passive pressure distribution diagrams, plane of zero shear is found and thus maximum moment is calculated by conventional approach. (Murthy, 2003)

Table 1: Unit Weights of Granular Soil and Coefficient of Active Earth Pressure

Type of soil	Unit wt. of moist soil, ɣ (lb/ft^3)	Submerged unit wt, ɣ' (lb/ft^3)	Coefficient of active earth pressure, K_a			
			For backfill	For soils in place	Friction angles	
					ϕ	δ
Dense sand	110-140	65-78		0.2	38	25
Medium sand	110-130	60-68	0.35	0.25	34	23
Loose sand	90-125	56-63		0.3	30	20

After Terzaghi, 1954.

Table 2: Soil Parameters Used for the Analysis.

Soil type above dredge line	Below dredge line
Sand ɣ= 100pcf ɣ'=65pcf k_a=0.35, k_p=2.86 Φ=30^o	Dense sand ɣ= 115pcf, ɣ'=70 pcf k_a=0.2, k_p=5, Φ=38^o
	Medium Sand ɣ= 105pcf, ɣ'=65 pcf k_a=0.25, k_p=4, Φ=34^o
	Loose sand ɣ= 100pcf, ɣ'=60 pcf k_a=0.3, k_p=3.33,Φ=30^o

Following data are also used to make models in ETABS.
Width of excavation = 30ft, Depth of excavation = 25ft, Pile diameter= 2ft Pile face to face distance= 30 inch, Support is provided at 12.5 ft from top. Location of Ground water = 4ft from top.
Embedded pile length, L_1= 20ft Embedded pile length, L_2= 15ft Embedded pile length, L_3= 25ftMODELS IN ETABS

In ETABS, models are developed based on Winkler's subgrade reaction approach in which soil is modeled as a set of independent elastic springs. The spring coefficients of these springs reflect the material properties of soil and are known as coefficient of subgrade reaction. In ETABS, concrete shore piles are driven to the pre-determined depth. After setting the given parameters, the lateral pressure distribution above the dredge line is formed by using the equation proposed by Peck (Murthy, 2003). The expression is

$$P_a = 0.65 \gamma H K_a \qquad (1)$$

The pressure p_a is uniform with respect to depth. Fig. (1) gives the apparent pressure distribution diagrams as proposed by Peck.

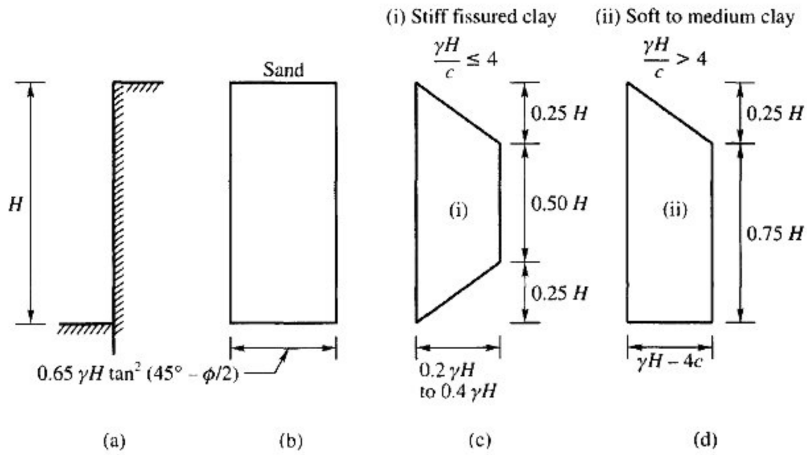


Fig. 1: Earth Pressure Distribution Diagrams for Braced Cuts (Peck, 1974)

Below the dredge line, Winkler's subgrade reaction concept is used. Springs are placed at one feet depth down to the depth of embedment. For sandy soil the coefficient of horizontal subgrade reaction is computed fromk= n_h
(2)

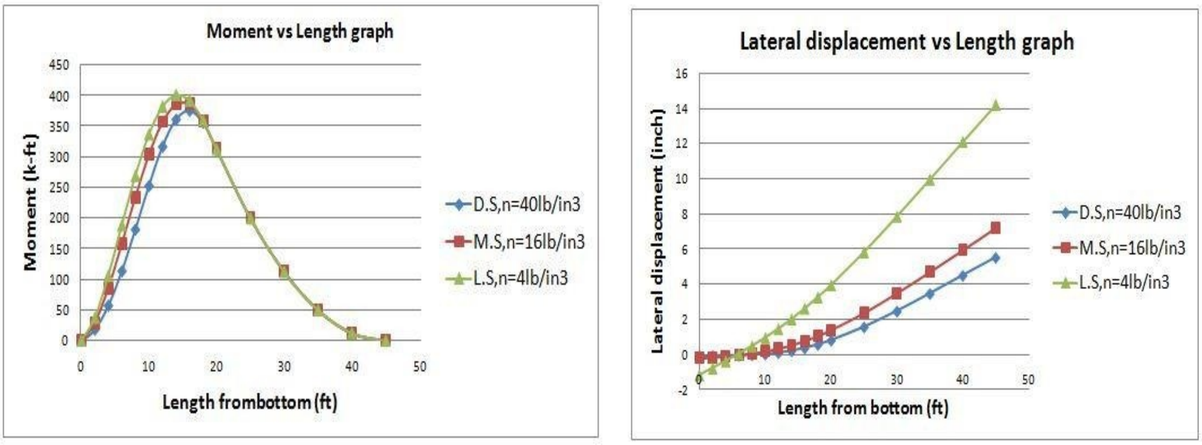
Where n_h= coefficient of horizontal subgrade reaction for a one feet wide pile at one feet depth. Z= depth in ft, B= width of pile in ft.
The values of n_h are used from a Table 2 given by Terzaghi. (Bowles, 1997)

Table 2: Values of Coefficient of Horizontal Subgrade Reaction (lb/in^3) for a one Feet Wide Pile at one Feet Depth Given By Terzaghi

	Loose sand	Medium sand	Dense sand
Dry or moist	8	24	65
Submerged	4	16	40

The values of coefficient of horizontal subgrade reaction thus computed are put in each feet depth down to the bottom. For each n_h values, separate models are developed and analysis is run on ETABS and thus maximum moments and corresponding deflections for a particular soil profile and loading conditions are obtained before and after providing supports. Analysis is also carried out in a similar way by changing the depth of embedment of pile.

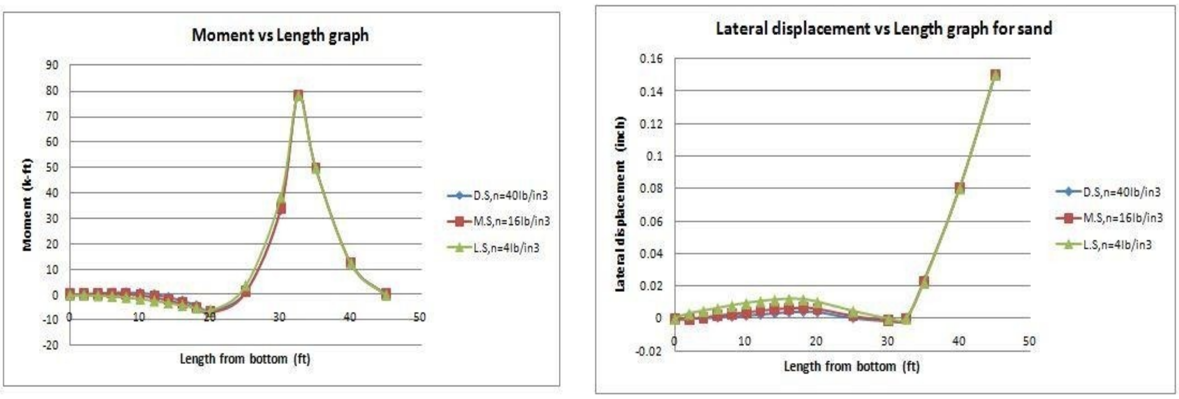
LOAD CALCULATION
P_a = 0.65ɣHK_a = 0.65*0.35*70.6*25= 0.4ksf
Considering each pile takes load from each side.
Load per ft of depth= 0.40*(pile diameter + pile face to face distance)
= 0.40*(2+0.5) = 1k/ft
This load and corresponding moments and lateral displacements are for 2.5 ft width. Coefficient of subgrade reaction, k_s= n_hz/B*1ft* pile diameter (k/ft)
This values obtained are placed after 1ft depth successively.RESULTS AND DISCUSSIONS



(a) (b)

Fig. 2: (a) Moment vs Length graph (b) Lateral Displacement vs Length graph in unsupported state for different n values

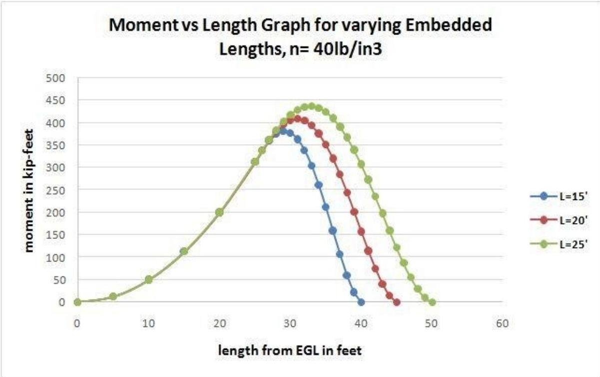
In this research, variations of moments and lateral displacements with length of pile for different n values are observed. In unsupported state, for a fixed embedded length of pile, from Fig.2 (a), the values of maximum moment are found to be 376.4 k-ft, 386.2 k-ft and 401.09 k-ft respectively and maximum lateral displacements are found to be 5.5 inch, 7.19 inch and 14.2 inch respectively from Fig.2 (b) for corresponding modulus of subgrade reaction values of 40lb/in^3, 16lb/in^3 and 4 lb/in^3 as proposed by Terzaghi (Teng, 1998). As can be seen from figure, up to dredge line the values of moment are same for different n values but after dredge line the curve splits as per n value. For lower n value moments are larger. As modulus of subgrade reaction represents the stiffness of the soils, greater values of it will yield comparatively less moment and deflection. Moreover, moment values are larger for loose sand, then for medium sand followed by dense sand in unsupported condition which is satisfactory. Moment values are also close to each other. In unsupported condition, deflection increases with length and becomes maximum at the top of the pile. The lateral displacement is quite higher for loose sand in comparison with medium and dense sand.

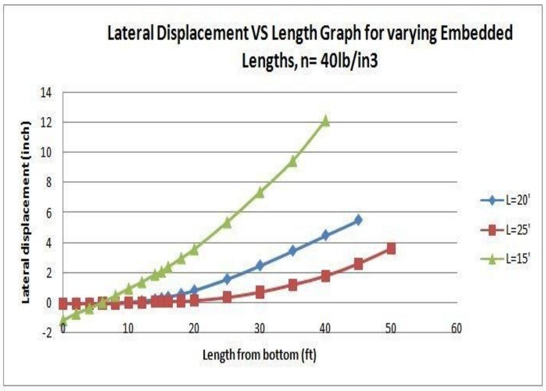


(c) (d)

Fig. 3: (c) Moment vs Length graph (d) Lateral Displacement vs Length graph in supported state for different n values

Afterwards, a support is provided at 12.5 ft from top and in this state, for subgrade reaction values 40lb/in^3, 16lb/in^3 and 4 lb/in^3, values of maximum moment are observed as 78.12 k-ft. 78.13 k- ft, 78.13 k-ft in Fig. 3(c) and maximum lateral deflections are reduced and found as 0.15 inch for each type of soil in Fig. 3(d). It is clear that after providing support lateral displacements and moments are reduced and assume very close values for dense, medium and loose sand as per n values.





(e) (f)

Fig. 4: (e) Moment vs Length graph (f) Lateral Displacement vs Length graph for varying embedded lengths

Again, for a fixed n value embedded length is changed (15ft, 20ft and 25ft) and the variations in moments and lateral displacements are also observed. Figures above are drawn for the same n value 40lb/in^3 and it is seen that the more the embedded length of pile is, the more the resisting moment is and also larger embedded length results in lower value of lateral displacement. For other n values, similar types of results are obtained.

These signs and magnitudes of the parameters are according to expectations.

CONCLUSION

This research was conducted to study the variations in moments and lateral displacements of shore pile with varying embedded lengths in supported and unsupported conditions using Subgrade Reaction approach of modeling soil behavior. From the developed models, one can readily find out the moments and lateral displacements of each point on the shore pile for the given loading conditions and soil profiles and thus providing lateral support at a suitable position, entire lateral support system can be designed by ETABS.

For different soil profiles, lateral displacements due to earth pressure can be reduced by providing supports or by increasing the embedded pile length. Lateral supports also reduce the embedded pile length. However, the models are developed only for granular soils which can be extended further to study the cohesive and layered type of soils as well.

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EFFECT OF FINE CONTENT ON SHEAR STRENGTH OF SAND

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ABSTRACT

The cumulative damage caused by landslides is far more widespread and poses greater total financial loss than any other geological calamity. Intrinsic soil properties such as apparent cohesion, pore - pressure and soil friction angle all of which are known to be highly influential on slope stability. The main objective of this study is to investigate the effect of fine content on shear strength of sand mainly on angle of internal friction. In order to achieve this objective, a number of experimental tests were performed and variation of angle of internal friction was observed with percent fine content and density. The sand samples were collected from different location of Bangladesh namely, Gabtoli (Gojaria sand), Sylhet (Sylhet sand) and Turag (Turag sand) which were used for the test purpose. Fines was derived by sieving non-plastic soil through No. 200 sieve. Tests were done by varying the percentage of fines in soil sample. The experimental result reveals that in general angle of internal friction decrease with the increase of fine content. Density also changes significantly with the change of % fine content.

Keywords: friction angle; shear strength; fine content; density; non-plastic soil

INTRODUCTION

Angle of internal friction, φ is one of the important parameters considered for reconnaissance of granular soils. Soil friction angle, unlike pore pressure and apparent cohesion, is not temporally variable and is a derivative of the measurement of soil shear strength. Again the fines contents in coarse soils are carefully considered because they determine the composition and type of soil and affect certain soil properties such as permeability, particle friction and cohesion. Fines have also been found to affect the liquefaction potential, compressional characteristics and stress-strain behavior of soil (Georgiannou, et al., 1990).

According to Wang et al. (2009), fines content could affect the dynamic response of soils significantly. The fines content in soil also plays an important role in phase problems including minimum and maximum void ratios and porosity (Lade et al. 1998). Ayodele (Ayodele, 2008) studied the effect of fines content on the performance of soil as sub-base material for road construction and found out that the engineering properties of the studied soil samples generally reduced with increase in fines content. Bayolu et al. (1995), made an experimental study. In this study, effects of the fine particles (diameter <0.074 mm.) on the shear strength and compressibility properties of the soil mixtures were investigated. Soil mixtures having wide range of grain size from sand to silt-clay mixtures were studied. Drained shear box and consolidated undrained triaxial tests were performed on normally consolidated clay-sand mixtures to obtain strength and compressibility parameters. According to the results of drained direct shear tests containing 5 %, 15 %, 35 %, 50 %, 75 %, and 100 % fines, the internal friction angles varied between 30-38 degrees until 50 % fines and a slight decrease existed in the friction angle with increasing fine content. At fine contents higher than 50%, the reduction in the friction angle was significant and decreased to about 10 degrees. Pitman et al. (1994) have carried out a study to investigate the influence of fines and gradation on the behaviour of loosely prepared sand samples. Vu To-Anh Phan et al. (2016), found the effect of fine content on engineering properties of sand-fines mixtures. From his experiments he found that as fines content increased, the internal friction angle decreased.

The specific relationship between fines content and angle of internal friction of soil is not clear. So, there is a need to advance the effect of fines on angle of internal friction of soil (φ). The present research aims to investigate the variation of angle of internal friction of Gojaria, Sylhet, Turag sand-collected from three selected location of Bangladesh- with the fines content and density.

METHODOLOGY

A detailed laboratory experiment was carried out to determine the physical and index properties of soil samples collected from Gabtoli, Sylhet and Turag riverside. Standard physical characteristics (Specific gravity of soils ASTM D 854) and classification (Grain size analysis of soil ASTM D 422) tests were performed as per ASTM standard procedure. Fines was obtained by sieving non-plastic soil (Gojaria sand) through #200 sieve. Obtained fines was added in varying amount (usually 0, 5, 10, 15%) with the soil samples. The experimental programs were performed to estimate the shear strength and angle of internal friction of the proposed sample. For this purpose, a shear devise was used in Consolidated Drained (CD) condition and the test was performed in the BUET lab as per ASTM D 3080 standard test procedures. Normally tests were conducted under one normal pressure (50kPa). To ensure the linearity some extra tests were done under two normal pressures. For both case zero normal stress and shear stress was used (as sand has no cohesion).

RESULTS AND DISCUSSIONS

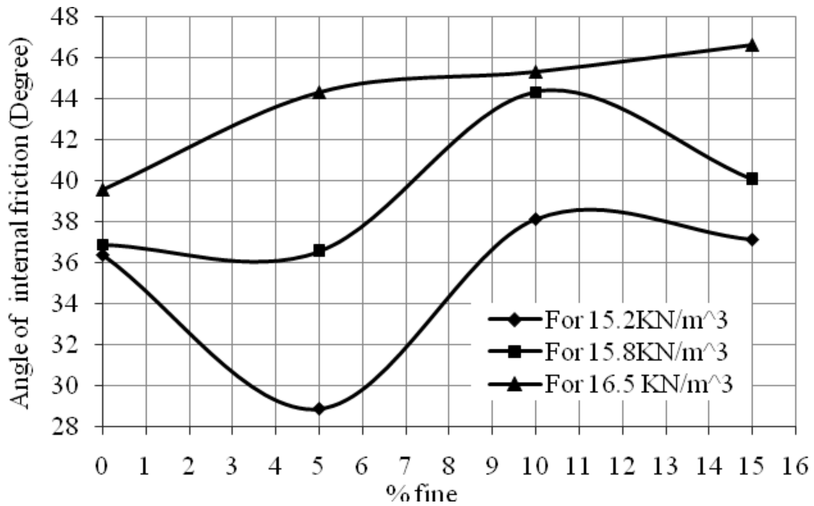
A series of experimental results have been presented herein to evaluate the effect of fines on the angle of internal friction and density. Based on these results detailed analyses and discussions are presented in the following subsections. The properties of Gojaria, Sylhet and Turag sands obtained by laboratory investigation are presented in table 1.

Table 1: Soil's physical properties

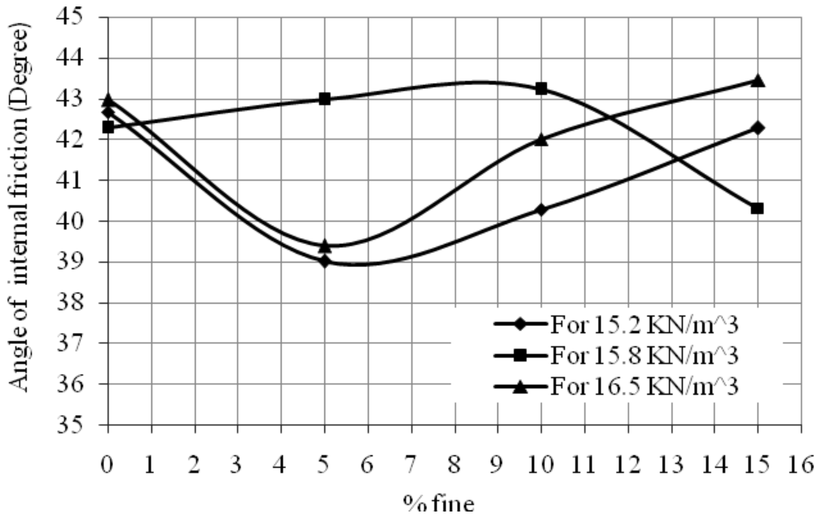
Soil Sample	Gojaria	Sylhet	Turag sand
Specific Gravity, G_s	2.69	2.68	2.74
EffectiveSize,D_{10} (mm)	0.025	0.25	0.0784
D_{30} (mm)	0.132	0.53	0.166
D_{60}(mm)	0.219	0.99	0.238
Uniformity Coefficient, C_u	8.75	3.96	3.04
Coefficient of Curvature, C_z	3.18	1.13	1.48
Fineness Modulus, F.M.	0.87	2.87	1

Effect of Percent fine

The variations of angle of internal friction (φ) with % fine are shown in Figure 1. It is seen that, for Gojaria sand, φ decreases with the increase of fine content upto 5%. Then φ increases with the increase of fine content. This occurs for loose and highly dense conditions. For medium dense condition, φ firstly increases then decreases after 10% fine content. For Sylhet sand, initially φ decreases with the increase of fine content upto 5% fine content then increases with the increase of fine content and decreases again after 10% fine content. This occurs for loose and medium dense conditions. For highly dense condition, φ increases with the increase of fine content. For Turag sand, the behavior of φ with fine content is similar to that of Gojaria sand but only for loose and medium dense condition. For highly dense condition, φ increases with the increase of fine content. But for loose condition, all sand follows the same behavior. Initially φ increases with the increase of fine content then decreases.



(a)



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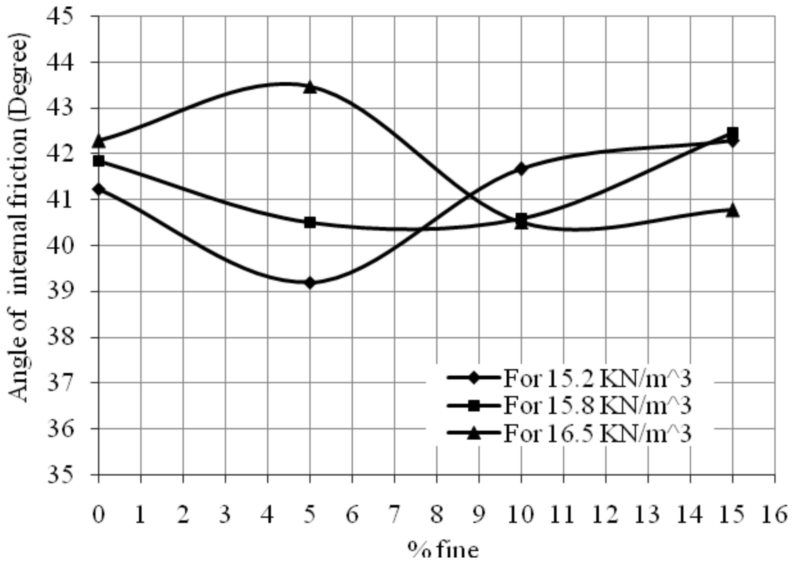

(c)

Fig. 1: Angle of internal friction (degree) vs % fine content (a) Sylhet (b) Gojaria (c) Turag sand

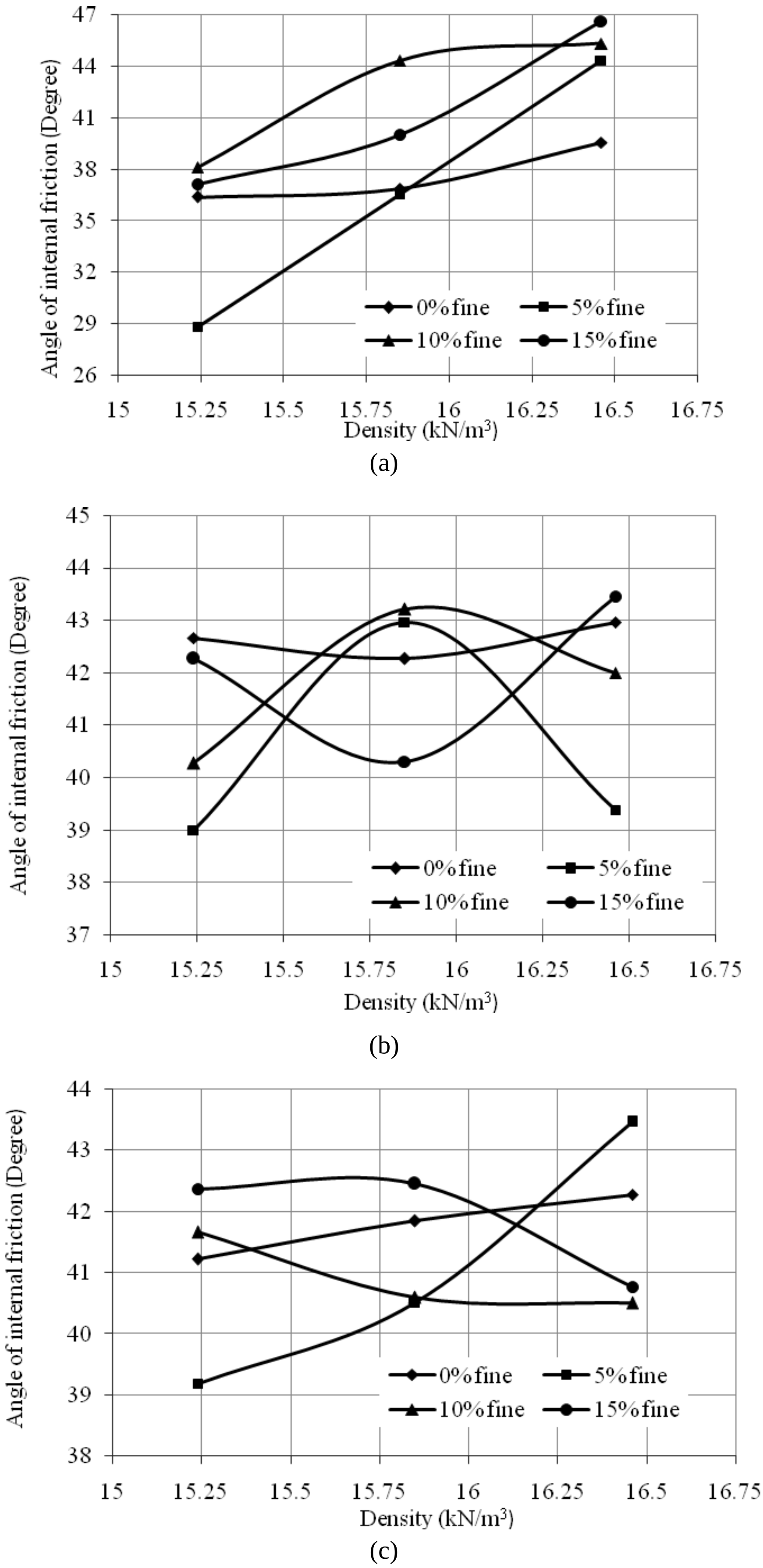

Fig. 2: Angle of internal friction (degree) vs density (kN/m^3) (a) Sylhet (b) Gojaria (c) Turag sand

Effect of Density

The behavior of the angle of internal friction (φ) with density is represented graphically in figure 2. For Gojaria sand the relationship is ambiguous. With the increase of density, φ initially increases then decreases and this occurs upto 10% fine content. At 15% fine content, φ increases with the increase of density. But for Sylhet sand, the relationship is almost linear. With the increase of density and fine content, φ also increases. For Turag sand, at small percentage of fines the relationship is almost linear. With the increase of percentage of fines the linear relation tends to deviate from the linearity.

CONCLUSIONS

It is found that fine content and density significantly affect the shear strength parameters of sand but it is ambiguous to make a specific conclusion from the test result. Therefore an attempt has been made characterizing some correlation among the friction angle, density and percent fine content. The main findings are:

(1) The angle of internal friction decreases with the increase of percent fines upto 5% fine content at loose and highly dense conditions for Gojaria sand and at loose and medium dense conditions for Turag and Sylhet sand.

(2) With the increase of density angle of internal friction increase upto 10% fine content for Gojaria sand. For Sylhet sand, this relationship almost linear. For Turag sand, upto 5% fine content this relationship all most linear and with the increase of fine this linear relation tends to deviate.

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CONSIDERATION OF SOIL PROPERTIES FOR STABILITY ANALYSES OFPADMA AND JAMUNA RIVERBANK

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ABSTRACT
This study deals with the characterization of Padma and Jamuna river bank soil and evaluation of river bank stability corresponding to those soil characteristics. It has been found that in some cases Padma and Jamuna river bank show distinct soil characteristics. A typical river bank section has been taken considering the geometric pattern of the rivers of Bangladesh. Stability analyses were conducted for three slopes, 33°, 45° and 56°, for different conditions using Geo-Studio 2012. For Padma and Jamuna river bank soils, specific gravity and liquid limits are quite similar whereas plastic limits are slightly higher for Jamuna soil. Again permeability of these soils is quite low leading to greater hydrostatic pressure. Cohesion and friction angle of Padma and Jamuna river bank soils are also comparable but their variation with change in water content and depth needs further study. In addition, from embankment section analysis, it is found that the factor of safety (FS) is overestimated about 25 to 30% if seepage is not considered in designing embankment. For river banks, it has been estimated that a slope of 1V:1.5H is safe against stability, providing a factor of safety greater than 1.2.

Keywords: Riverbank, embankment, slope stability, seepage.

INTRODUCTION

Rivers in Bangladesh are highly dynamic in terms of morphology and erosion processes are quite unpredictable here resulting in dramatic consequences in the lives of people living in the erosion prone areas (Rahman, 2010). Bangladesh is an agricultural based country with most of its agricultural lands near the riverbanks of Padma and Jamuna. Every year this erosion phenomenon has been taking away those lands from the farmers resulting in a great loss in national economy. Studies have shown that the most common factors that are responsible for riverbank erosion are the properties of soil, the geology and climatic condition of the surrounding area and most importantly the geometry of the river channel (Zomorodian, 2010). The main objectives of this study are (i) to characterize the soil of Jamuna and Padma riverbank, (ii) to determine the stability and failure pattern of those riverbank areas and (iii) to compare different methods of slope protection of riverbank and to provide suggestions for preventive measures.

REVIEW ON BWDB DESIGN

According to Bangladesh Water Development Board (BWDB), the slope mostly fails due to the drag forces generated by the velocity and this local velocity is mostly influenced by the boundary geometry in the immediate neighbourhood. Additionally, during thunderstorm wind velocity and direction have a major effect stimulating the water waves to cause slope instability. Due to high flow turbulence scour occurs frequently beneath the revetment to damage the protection measures. Standard code of practice recommends stable slope to be designed for a minimum safety factor of 1.5. Permeable as well as impermeable groyne, Guide Bundh, CC block revetments etc. are very common practices that have been taken by BWDB. n addition, biological treatment to slope protection is getting popularity recently because of its environment friendly nature (Islam et al., 2013). However, very few cases consider the soil properties in river bank design (BRTC, 2003).

COLLECTION AND TESTING OF SOILS

The soil samples were collected from the broken part of the Padma riverbank near Mawa ferrighat and also from the Jamuna riverbank near Sariakandi Upazila as shown in Figure 1. Soil was collected from approximately 0.5 m depth using Shelby tube as well as PVC pipe from each location shown in Figure 2. Thus, the collected soil samples were slightly disturbed. The testing procedures were in accordance with ASTM Standards and the soils have been classified according to USCS. The constant head method was followed to determine the coefficient of permeability. Unconfined compression test was carried out on samples having different water content.

Fig. 1: Soil sampling locations from Jamuna and Padma river banks.

(a) (b)

Fig. 2: (a) Pushing the PVC pipe into soil using a wooden plank, (b) soil sample collection by PVC pipe

RESULTS AND DISCUSSIONS

Soil Properties

Index Properties

From laboratory analyses, it has been found that liquid limit and specific gravity of those soils are quite similar but plastic limit is reasonably higher for Jamuna riverbank soil. Maximum dry density for Padma riverbank is found to be 1.54 gm/cc which is greater than the corresponding 1.28 gm/cc for Jamuna riverbank. From particle size distribution curve, it can be inferred that silts and clay governs the majority and this relation is shown in Fig. 3. Both the soils are low plastic clay (CL). Optimum water content has been determined for both soils which can be significantly useful in riverbank construction. Relationship between dry bulk density and water content has been shown in Fig. 4.

Shear strength

Cohesion and friction angle have been determined at 29% water content for Padma riverbank and found to be 31 kPa and 13°, respectively. For Jamuna riverbank, the corresponding values are found to be 28 kPa and 10° respectively at 26% water content. As water level fluctuation is very frequent in this region, shear strength parameter variation with change in water content and depth needs to be considered.

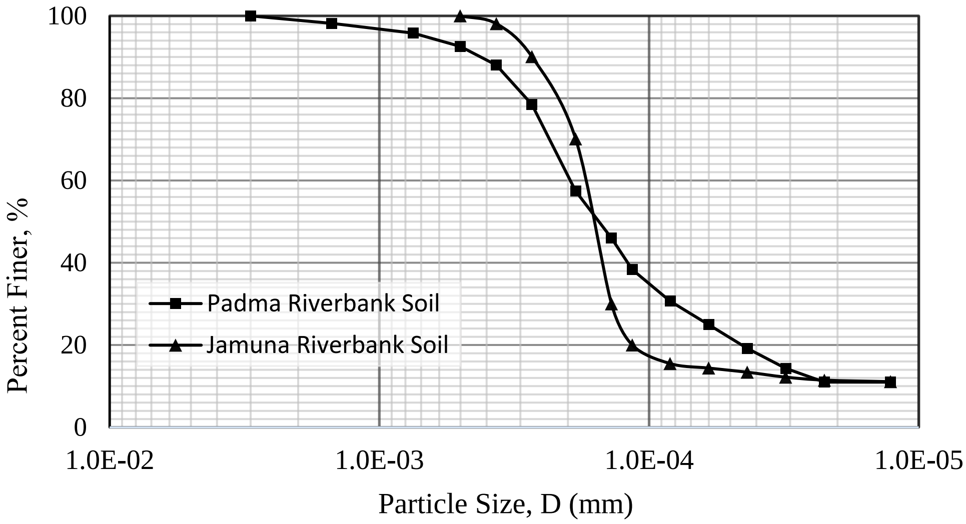

Fig. 1: Particle size distribution of the riverbank soils

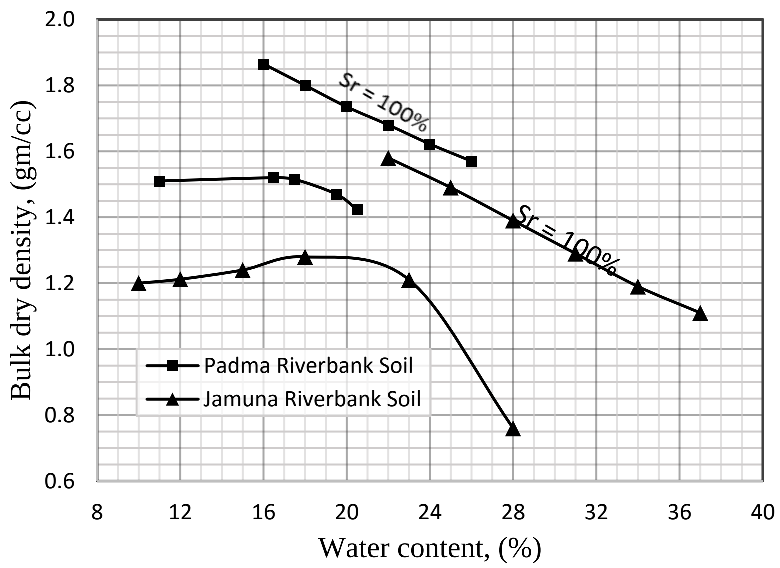

Fig. 2: Bulk dry density vs water content

Table 1: Properties of soils collected from Padma and Jamuna river bank

Soil Properties		Padma river bank: Current study (23º28´17″N,90º15´16″E)	Padma river bank: Hossain et al. (2010)	Jamuna river bank (24º53´32″N,89º34´43″E)
Index Properties	Liquid limit (%)	33	32	32
	Plastic limit (%)	21	27	27
	Optimum water content (%)	17.2	21	21
	Maximum dry density (gm/cc)	1.54	1.56	1.28
	Specific gravity (Gs)	2.66	2.66	2.65
	Sand (%) (USCS)	4.2	6.0	5.6
	Silt and clay (%) (USCS)	95.8	94	94.4
	Soil type	CL	CL	CL
Shear Strength	Cohesion, cu (kPa)	31 (at 29% Wn)	--	28 (at 26% of W_n)
	Frictional angle (ϕ)	13	10	22
Permeability	Permeability (cm/s)	2.73×10^{-3}	2.43×10^{-3}	1.29×10^{-4}

Permeability

Constant head permeability method has been adopted in this case to determine the permeability. The permeability found in this research are considerably low causing a greater hydrostatic pressure in case of sudden drawdown of water, which results in frequent tension cracks in the upper layer and these cracks are the prime reason for riverbank failure. Henceforth, this parameter should be carefully used in seepage analysis to predict the failure precisely.

Stability Analyses

SLOPE/W is formulated in terms of moment and force equilibrium factor of safety equations. In this program, stability can be analyzed by several Limit Equilibrium Methods including Ordinary, Bishop, Spencer, Janbu, Morgenstern-Price methods etc. This program allows integration with other applications, for example, finite element computed stresses from SIGMA/W or QUAKE/W can be used to calculate a stability factor by computing total shear resistance and mobilized shear stress along the entire slip surface. Then, a local stability factor for each slice is obtained. Using a Monte Carlo approach, program computes the probability of failure in addition to the conventional factor of safety.

Soil Parameters Used

For riverbank section the soil that has been taken into consideration is a c-ϕ soil with cohesion taken as 30 kPa, friction angle 10°, water content 29% and dry density to be 18 kN/m^2. For embankment section, permeability is taken as 2.43×10-3 cm/s with dry density of 18 kN/m^2. Cohesion and friction angle remain the same as riverbank section.

Water Level and Slope Conditions

For the typical riverbank section shown in Fig. 5, a water level of 8m has been assumed and three different slopes of 33°, 45° and 56° at different conditions has been taken to predict the safest slope. Factor of safety in each method has been shown in Table 2. Fatema and Ansary (2014) recommends a factor of safety greater than 1.2 to ensure a riverbank to be safe. So from this analysis, a slope of 1V:1.5H can be safe.

Again a typical embankment section shown in Fig. 6, has been taken to know the effect of seepage in designing embankment. Three different water tables have been taken and their corresponding factor of safety has been shown in Fig. 7. The study has found out that the factor of safety (FS) is overestimated about 25 to 30% if seepage is not considered in designing embankment which is similar to the results of Hossain et al. (2010).

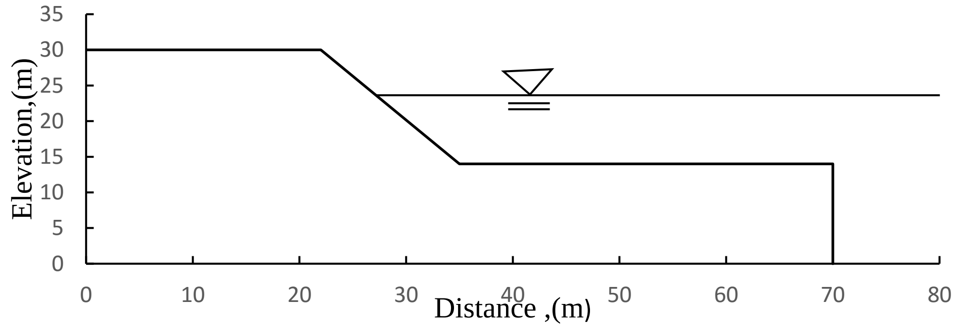

Fig. 3: Typical riverbank section

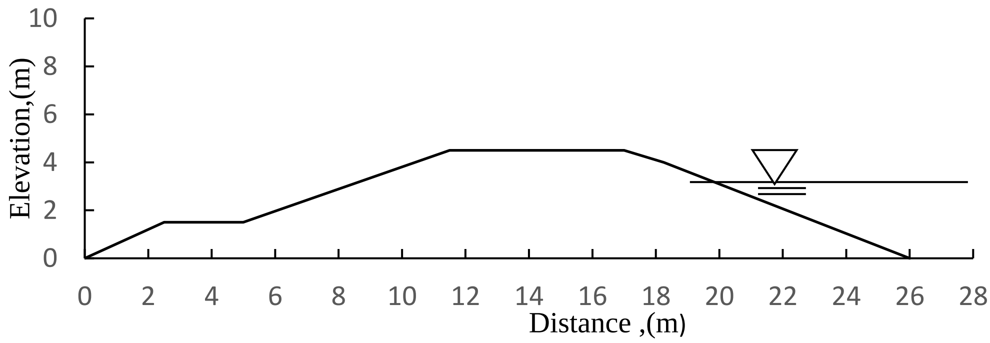

Fig. 4: Typical embankment section

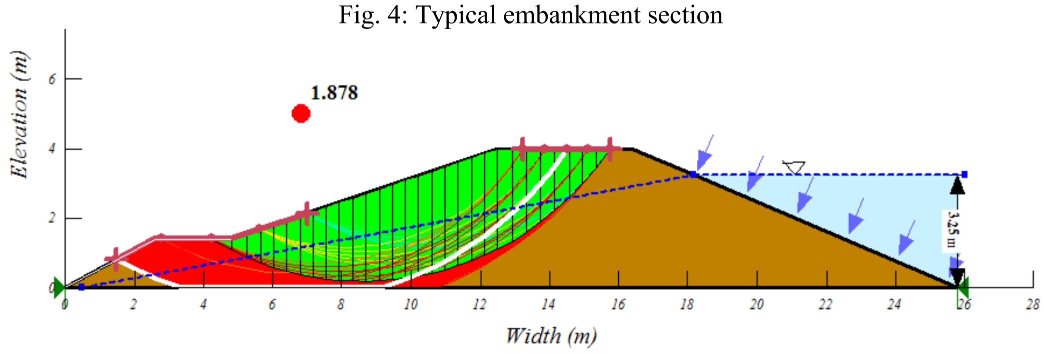

Fig. 7: FS against seepage failure for embankment with 3.25m water level from riverbed

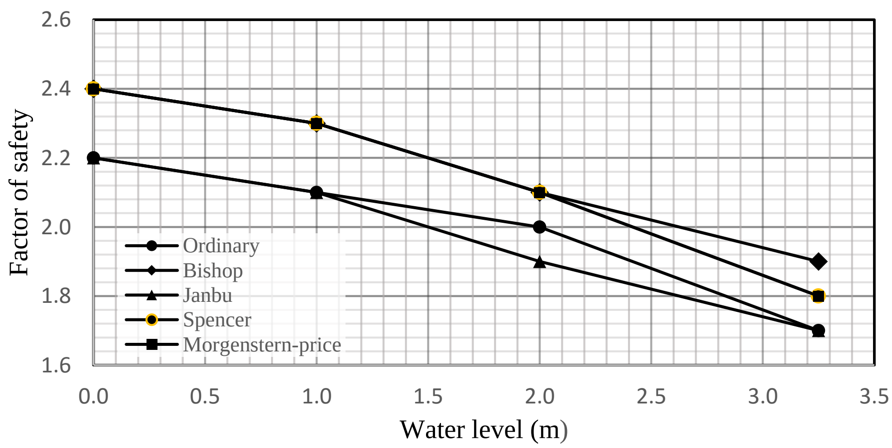

Fig. 8: Water level vs Factor of safety

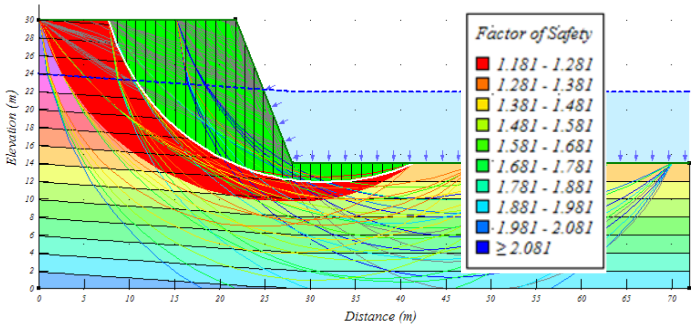

Fig. 9: FS for typical riverbank section by Bishop Method for slope of 1V:0.667H

Table 2: Factor of safety for typical riverbank section with water level of 8m

Slope)V:H(	Ordinary method (1927)	Bishop method (1955)	Janbu method (1973)	Spencer Method (1967)	Morgenstern- price (1963)
1:1.5 (33°)	1.24	1.38	1.24	1.34	1.34
1:1 (45°)	1.20	1.24	1.17	1.24	1.24
1:0.667 (56°)	1.11	1.18	0.96	1.11	1.12

CONCLUSIONS

Riverbank failure is a major devastating issue in Bangladesh. Preventive measures must be taken as early as possible to check this issue. Soil conditions must be considered in designing those measures. The following conclusion can be drawn from this study:

(1) Padma and Jamuna riverbank have distinct soil properties in terms of soil strength parameters.
(2) Seepage is a major issue for riverbank stability causing scour hole and different soils behave differently in case of rising water level and drawdown.
(3) A slope of 1V:1.5H satisfied all the criteria in different methods of slope stability analysis. Thus it is safe against stability failure. It agrees with the guidelines of BWDB.
(4) If seepage is not considered, the factor of safety is overestimated by about 25~30%.

Further study is being conducted to investigate and categorize the major riverbank soil so that proper and durable protection can be provided.

ACKNOWLEDGMENTS

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CHARACTERIZATION OF SOIL-ROOT SYSTEM TO DETERMINE STABILITY OF VEGETATED SLOPES

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ABSTRACT
Protection of embankments is of paramount importance from Bangladesh's context. Every year, slope instability related damages upset people's lives in many parts of the country. The conventional methods used for slope protection are generally temporary solutions and require large, long-term investment. Alternatively, plantation of vetiver grass can be a very cost-effective and innovative solution for the protection of banks/slopes. In this study, to evaluate the performance of vetiver grass in slope protection, direct shear tests were conducted on both bare and rooted soil samples. Samples were prepared with different root lengths varying from 1.25-5 cm with varying water content of 15% - 30%. Root content was 3% of the dry weight of soil. It has been found that both shear stress and strain largely depend on root length. From slope stability analyses, it has been found that vetiver grass is able to increase factor of safety of embankment slope up to 2.66 times to that of bare soil.

Keywords: Bio-engineering, bare soil, slope stability, rooted soil, vetiver

INTRODUCTION
Bangladesh is a low-lying deltaic country. The geographical characteristics have made the country vulnerable to different natural hazards. Due to lack of adequate mangrove forests and absence of coastal embankment, Bangladesh is the fifth most natural disaster prone country in the world, according to the World Risk Report (FE Report. 2012). The major disasters that are generally encountered are flood, cyclone and storm surge, flash flood, drought, tornado, riverbank erosion and landslide. These are recurrent natural hazards, causing loss of lands, agriculture and houses. It also destroys embankments, other hydraulic structures and livelihood along coastlines and estuaries. Often, landslides cause long-term economic disruption, population displacement, and negative effects on the natural environment. Each year, slope failures produce extensive property damage and result in loss of life. These numerous issues elucidates the vitality of earth slopes' protection to minimize the losses that occur every year.

There are different approaches to deal with slope instability, depending on needs, risks and available funds. Stabilization measures to fully remediate landslides according to the standard of practice often take time to investigate, design and construct and above all become expensive, particularly for developing and least developed countries like Bangladesh. Significant stabilization measures might be required to protect critical facilities such as dams, expensive structures and primary highway routes. However, there are situations where full stabilization is impractical due to size of landslide, excessive cost, and highly dense foothill settlement, environmental and ownership restrictions. Hence, the challenge is to develop an optimal treatment that is cost-effective.

Plantation of vetiver system along the slopes is a very effective and low-cost solution for slope protection (Verhagen et al., 2008, Islam et al., 2013b, Badhon, 2015). Vetiver not only serves the purpose of slope protection but also add 'green environment', reducing pollution. Vetiver grass is native to tropical and subtropical India. It has a dense structured deep root system capable of reaching 2.5–4 m (Islam et al., 2013a, 2013b) and its roots are very strong with high tensile strength of 75 MPa. In addition, it is highly tolerant to extreme soil conditions including prolonged drought, flood, submergence, extreme temperature (-10 to 48^{o}C), and a wide range of soil acidity and alkalinity (pH: 3 to 10.5). It has also been reported to be highly tolerant to soil salinity, sodicity, acidity, and presence of Al, Mn and heavy metal ions in the soil (Islam et al., 2013b). Bangladesh Water Development

Board, BWDB (2000) found vetiver grass to be very common in about 40% of the total land area of Bangladesh. Vetiver grass has wider applications due to its unique morphological, physiological and ecological characteristics that highlight its adaptability to a wide range of environmental and soil conditions. The most impressive characteristic of the vetiver grass is its root system. Islam et al. (2013a) conducted in-situ shear tests on vetiver rooted soil system and found that the shear strength and effective soil cohesion of vetiver rooted soil matrix are respectively 2.0 and 2.1 times that of the bare soil. To evaluate the performance of vetiver grass in slope protection, several laboratory tests were conducted on vetiver rooted soil. The objective was to determine the strength of vetiver rooted soil and to observe the change in shear strength, deformation, cohesion and angle of internal friction in comparison to bare soil.

METHODOLOGY

Laboratory direct shear tests were conducted to determine the shear strength and failure strain of vetiver rooted soil matrix and soil without root. Vetiver grasses were collected from a naturally grown source in Pubail, Gazipur and then roots were obtained from uprooted vetiver grass. Soil for making samples was also collected from the same place.

Laboratory Tests

A detailed laboratory investigation was carried out to determine the physical and index properties of the soil samples collected from Pubail. The laboratory testing program consisted of carrying out specific gravity, moisture content and particle size analysis. All the tests were conducted according to ASTM standards. Direct shear tests were conducted in Consolidated Undrained (CU) condition on reconstituted soil samples. Tests were conducted on both bare and root mixed composite soil samples collected from the Pubail region of Bangladesh. The tests were performed on samples prepared with different root lengths (1.25 cm, 2.54 cm and 5 cm) with same root content (3% of dry weight of soil). Samples were prepared at two different moisture contents (15% and 30%). Normal stresses were arbitrary selected in the range between 10.83 kPa and 20.12 kPa.

Preparation of Reconstituted Soil Samples

At first, roots were collected and then preserved in the refrigerator (at 4ºC) with arbitrary moisture content to keep the roots fresh. Then, roots were chopped to the desired length (1.25 cm, 2.54 cm, 5.0 cm). Collected soil samples were air dried and crushed to powder by wooden hammer. Next, water (amount corresponding to 15% and 30% moisture content respectively) was added to the dry soil. Chopped vetiver roots were randomly mixed with the wet soil. The soil was then compacted by a wooden rod inside a probing ring (size of 63.5 mm diameter and 25.4 mm height) from a falling height of 100 mm. The compaction was done in three layers with 25 blows being applied to each layer. The prepared samples were kept in a desiccator to keep the moisture content constant. Direct

(a) (b) (c)

Fig. 1: (a) 5 cm long chopped vetiver root, (b) root mixed soil, (c) soil sample inside the shear ring.

shear test was conducted on those prepared specimens according to the ASTM standards. Fig. 1 shows the photographs of different stages of reconstituted soil sample preparation.

Test Set-up

The remolded soil sample was placed in the shear box from the ring. Then, the desired normal load was applied. Normal stresses were arbitrarily selected in the range from 10.87 kPa to 20.12 kPa. Vertical displacement dial gauge was attached to record the vertical deformation with respect to time. Enough time (about 1 hour) was allowed for consolidation before applying the shear force. When two consecutive vertical deformation dial readings were the same, shear force was applied to the soil sample with a constant strain rate of 0.75 to 1.25 mm/min. The lateral deformation was recorded by a lateral constant strain rate of 0.75 to 1.25 mm/min via the means of a lateral displacement dial gauge of 25 mm capacity. The applied shear force was recorded by a load dial gauge of 2.22 kN capacity.

RESULTS AND DISCUSSIONS

Index Properties of Soils

The specific gravity of the soil sample collected from Pubail region is 2.68. Natural moisture content is 10%. Dry unit weight of the soil samples varies from 15.67-17.37 kN/m^3 at 15% moisture content and 17.23-17.87 kN/m^3 at 30% moisture content. Clay, silt and sand fractions of the soils have been determined according to ASTM D 422. Fig. 2 shows grain size distribution of Pubail soil samples. From Fig. 2, it is seen that clay, silt and sand content of the soil are respectively 24%, 60%, 16%. Liquid limit is 44%, plastic limit is 21% and plasticity index is 23%. It is found that Pubail soil is Lean clay and the designated group symbol according to ASTM D 2487 is CL (inorganic clays of low to medium plasticity or silty clay).

Strength Properties of Soils

Test result of the soil samples in different root length is presented in the Table 1 and Table 2. Typical shear stress vs shear strain and shear stress vs normal stress graphs are presented in Fig. 3a and Fig. 3b, respectively. Variation of angle of internal friction and cohesion are presented in Fig. 4 and Fig. 5. From the test results, it is observed that for 15% moisture content, the peak shear stress of vetiver rooted soil matrix varies from 33.57 kPa to 69.63 kPa and peak shear strain of vetiver rooted soil matrix varies from 1.65 to 5.66 mm.

For 30% moisture content, it is observed that the peak shear stress of vetiver rooted soil matrix varies from 15.78 kPa to 34.74 kPa and peak shear strain varies from 6.48 mm to 10.03 mm. In case of 30% moisture content, though, the peak shear stress is lower than that of soil samples for 15% moisture content but peak shear strain is quite higher. This means, for higher moisture content, soil sample tends to display more ductility.

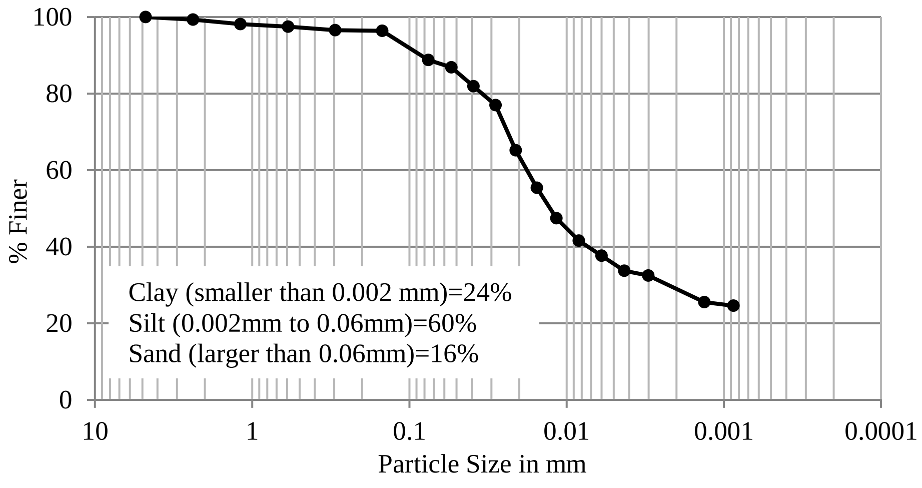


Fig. 2: Particle size distribution of soil sample collected from Pubail region

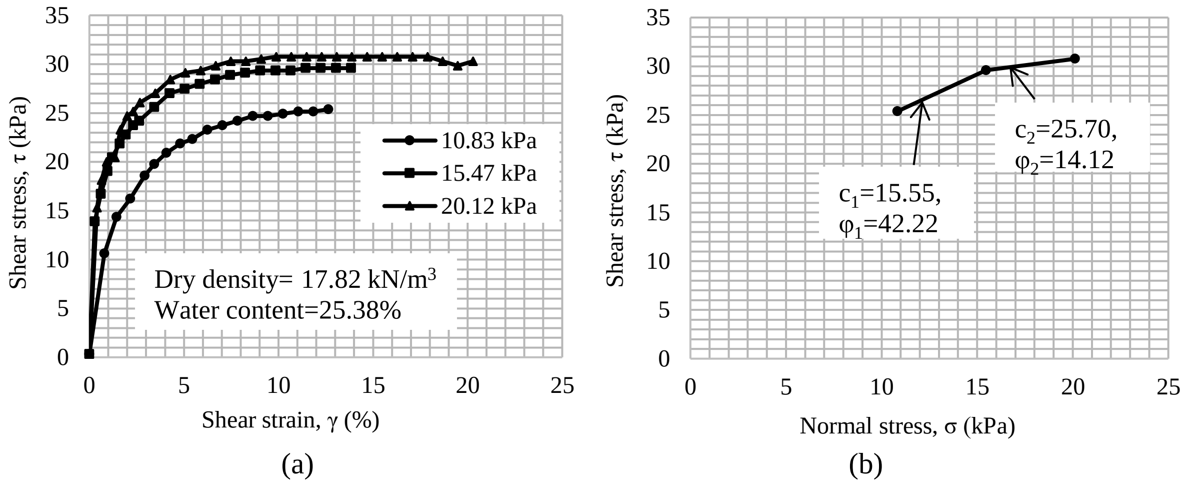


Fig. 3: (a) Shear stress vs shear strain for specimens prepared with 1.25 cm root length; (b) shear stress vs normal stress for specimens prepared with 1.25 cm root length

Table 1: Comparison of shear strength and shear deformation of reconstituted bare and rooted soil

σ_n	**Moisture content= 15%**								**Moisture content=30%**							
	Bare soil		1.25 cm root length		2.54 cm root length		5 cm root length		Bare soil		1.25 cm root length		2.54 cm root length		5 cm root length	
	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}	τ_{max}	δ_{hf}
10.8	42.9	1.7	38.3	9.7	33.6	4.7	51.8	3.4	24.2	9.6	25.4	8.0	28.0	6.9	15.8	10.0
15.5	46.2	2.6	49.0	2.6	52.8	5.7	66.8	2.9	29.1	9.7	29.6	7.3	34.7	6.5	20.2	9.8
20.1	56.5	1.9	53.7	1.9	57.9	3.6	69.6	3.7	34.3	9.7	30.8	6.3	31.2	6.5	21.6	9.8

Note: σ_n =Normal Stress in kPa; τ_{max} = Peak Shear stress in kPa; δ_{hf} = Horizontal Failure Deformation in mm

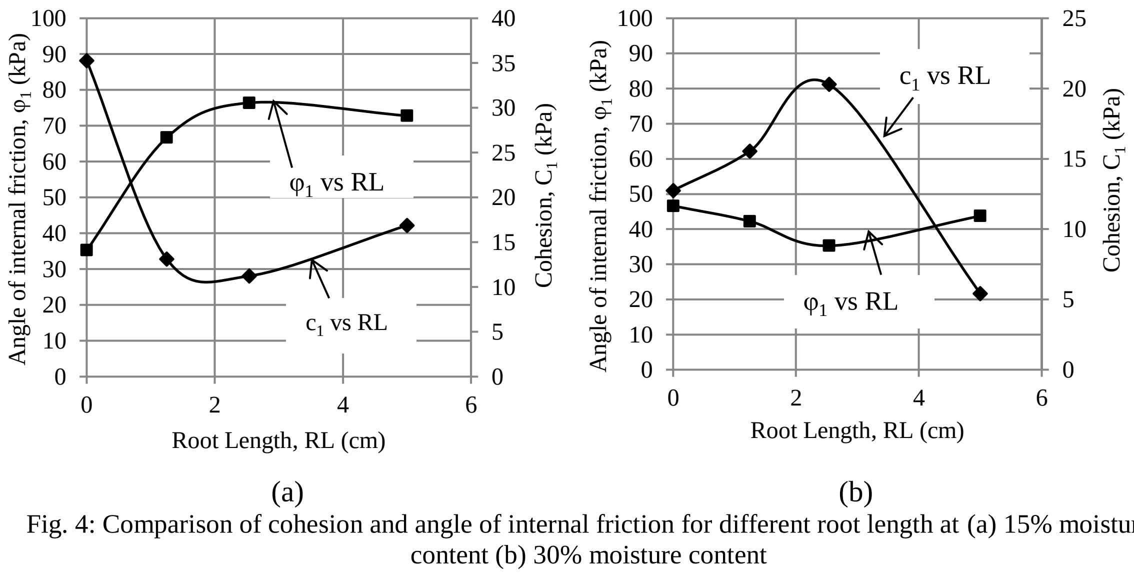
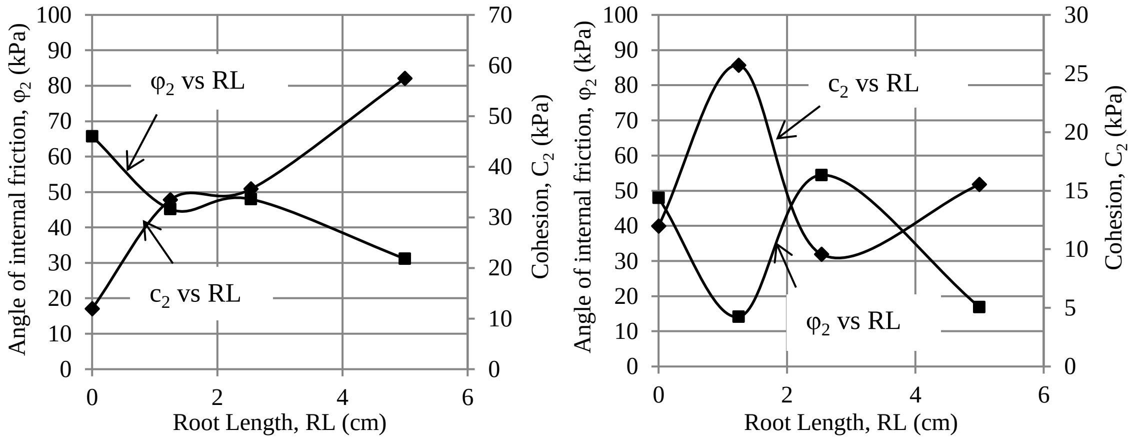


Fig. 4: Comparison of cohesion and angle of internal friction for different root length at (a) 15% moisture content (b) 30% moisture content

It is seen that for 30% water content, maximum shear stress almost increases with the increase in root length for all three normal loads except for the case of normal load of 10.83 kPa and for 2.54 cm root length, maximum shear stress decreases in comparison to bare soil.

However, for 15% water content maximum shear stress increases up to root length 2.54 cm but decreases for root length 5 cm. This may occur due to the failure of soil samples in a predetermined failure plane. As root is arbitrarily placed in soil sample, root may be positioned in vertical, horizontal or inclined direction in failure plane. Also, there is a possibility that there is no root in the failure plane. The above mentioned reasons might explain why shear strength varies.



(a) (b)

Fig. 5: Comparison of cohesion and angle of internal friction for specimens prepared with different root length at (a) 15% moisture content (b) 30% moisture content

Therefore, it is clear that the presence of root content significantly affects the strength of soil. Shear stress, deformation, cohesion and angle of internal friction of a soil matrix varies with the variation in root length. The shear strength of a rooted soil matrix depends on the interaction between the soil and the roots and the distribution of roots on the soil matrix. At a certain plane in a rooted soil matrix, the shear force is resisted by the soil and the roots. The performance of the rooted soil matrix depends on the critical combination of soil and roots on the failure plane. The critical combination also varies with root content, root ratio and root position. This is the reason the shear stress, deformation, cohesion and angle of internal friction of a soil matrix varies. It expounds on the fact that the root content and the length of roots in the soil mix have a major effect on the shear strength of soil.

Estimation of Slope Stability

Slope stability for a typical rural road section has been analyzed using Coppin and Richards's method (Coppin et al. 1990) for obtained by laboratory direct shear tests on reconstituted soil samples.

For bare soil, $FS = \frac{c' + (\gamma z - \gamma_w h_w)\cos^2\beta \tan\varphi'}{\gamma z \sin\beta \cos\beta}$

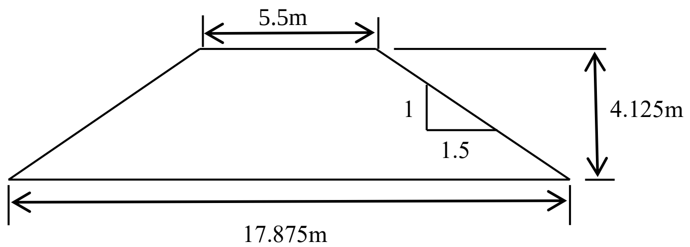


Fig. 6: LGED rural road section through hills, Road Type B (LGED, 2005)

Table 2: Parameters used for stability analysis

Parameter	**Unit**	**Value**
Vertical height of soil above slip plane	m	1
Slope angle, β	degree	34
Unit weight of water, γ	kN/m^3	9.8
Vertical height of ground water table above slip plane, h_w	kN/m^2	–
Surcharge due to weight of vegetation, W	kN/m^2	0.1
Vertical height of groundwater table above the slip plane with the vegetation, h_v	m	0
Tensile root force acting at the base of the slip plane, T	kN/m	0.4
Angle between roots and slip plane, θ	degree	–
Wind loading force parallel to the slope, D	kN/m	0.1

For rooted soil, $FS = \frac{(c' + c_R') + [\{(\gamma z - \gamma_w h_v) + W\}\cos^2\beta + T\sin\theta]\tan\varphi' + T\cos\theta}{\{(\gamma z + W)\sin\beta + D\}\cos\beta}$

Table 3: Factor of safety of embankment slope on the basis of reconstituted soil samples test results

Moisture content=15%			**Moisture content=30%**		
Root Length (cm)	Factor of Safety	Times higher	Root Length (cm)	Factor of Safety	Times higher
---	4.8	---	---	3.3	---
1.25	6.5	1.35	1.25	4.0	1.21
2.54	6.7	1.39	2.54	2.5	0.76
5.00	12.8	2.66	5.00	6.1	1.85

CONCLUSIONS

Slope failure due to erosion is a common problem in Bangladesh. Plantation of vetiver system along the slope is an alternative green solution to the problem. In order to characterize the soil-root system to determine the stability of vegetated earth slopes, several laboratory investigations has been carried out in this study. The main findings of the study are as follows:

(1) Direct shear tests were conducted on reconstituted samples for 3% root content for varying root length and varying water content under different normal stresses. Test results show that shear strength and deformation of rooted soil changes with root length significantly in comparison to that of bare soil.

(2) From the obtained cohesion and angle of internal friction, the stability of embankment slopes was estimated. From the analyses, it has been found that the factor of safety of embankment slope increases with the increase of root length.

Therefore, it can be said that the vetiver grass plantation is a very effective and low cost green solution to slope stability related problems.

ACKNOWLEDGMENTS

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DETERMINATION OF PLASTIC LIMIT USING CONE PENETROMETER

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ABSTRACT

The plastic limit is the minimum water content at which soil starts to show plastic behaviour. Any fine grained soil having natural water content close to its plastic limit is expected to have high shear strength and low compressibility. The standard thread-rolling method for determining the plastic limit has long been criticized for requiring considerable judgments from the operator. Cone penetrometer is usually used for determining the liquid limit of a soil. In this study, effort has been given towards using Cone Penetrometer to determine the plastic limit in a way to overcome the inconsistent results. Two approaches have been suggested. Firstly, tests have been done on soils of eight locations to find a specific penetration value using standard Cone Penetrometer whose corresponding water content in the penetration vs water content curve can be marked as the plastic limit of that soil. Secondly, a modified load is used in Cone Penetrometer to determine Plastic Limit. The advantages of such new procedure is that the test is more closely related to soil behaviour, less subjective, at least as reproducible as the Casagrande test and can be carried out in the same manner as determining liquid limit.

Keywords: Plastic limit; cone penetrometer; penetration value

INTRODUCTION

Plastic limit is defined as the moisture content at which the soil crumbles when rolled into threads of 3.2 mm (about 1/8 inch) in diameter by hand on a ground glass plate. However, there are several criticisms on this test since the operator is required to judge the state of crumbling and the 3-mm diameter of the thread. If full saturation and incompressibility are assumed, plasticity theory indicates that the soil yield stress will be a function of a number of parameters such as-

(1) The pressure applied to the soil thread,
(2) The geometry, i.e. the contact area between hand and thread,
(3) The friction between the soil, hand and base plate,
(4) The rate of rolling etc.

None of these variables is controlled easily and consequently the traditional plastic limit test does not provide a direct measurement of soil strength. For this reason our research was done. There are many advantages of this work. Some are mentioned below:

(1) It is not dependent on user’s sensitivity,
(2) Can be done simultaneously with liquid limit test,
(3) Since the result is obtained by interpolating from water content vs penetration curve, the result is more precise and accurate.

In this research, it has been tried to take an initiative to determine the plastic limit using “Cone Penetrometer” which is normally used to determine the Liquid Limit of soil. The objectives of this research are stated below:

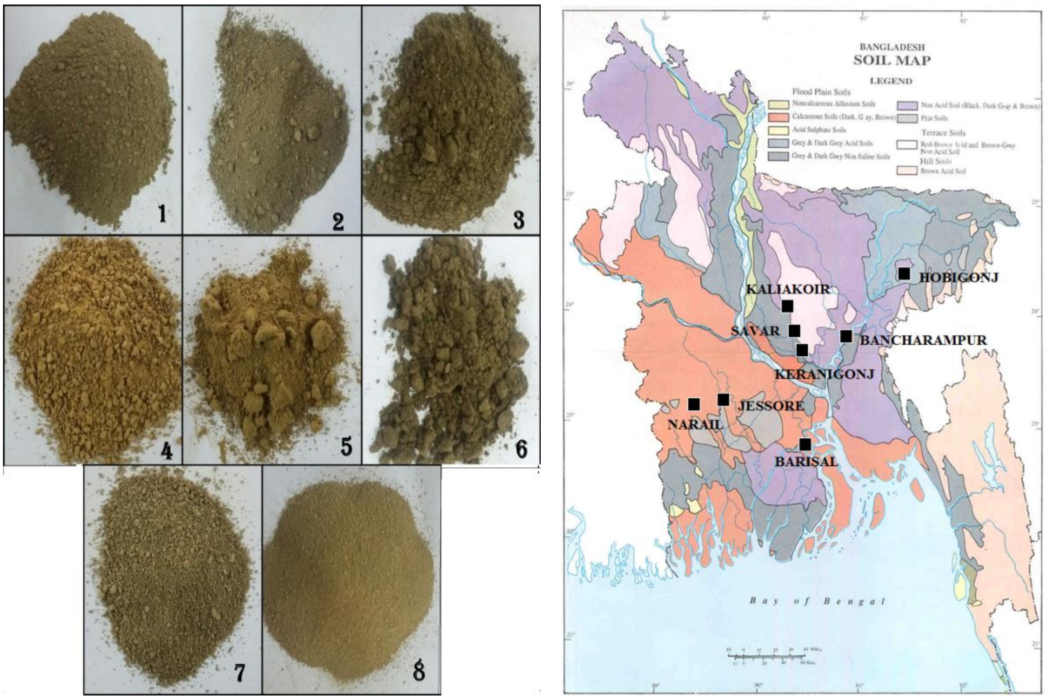
(1) To determine plastic limit using a modified load (240 gm where 30gm cone and 210gm load).
(2) Another was to determine a specific penetration value with the regular cone (80 gm) whose corresponding water content in the water content vs penetration graph can be stated as the 'Plastic Limit' of that soil.

METHODOLOGY

Soils from four different locations of Bangladesh have been collected and a series of laboratory investigations were performed in the Geotechnical Laboratory of BUET. Tests were done following the corresponding ASTM standards and BS standards.

Study Areas

To make the research more precise soils have been collected from eight different locations of Bangladesh. These locations are-
(1) Barisal, (2) Keranigonj, (3) Bancharampur, (4) Kaliakoir, (5)Hobigonj, (6)Narail, and (7)Jessore (8)Savar.



(a) (b)

Fig. 1:(a) Soil samples collected from: **(1)** Barisal, **(2)** Keraniganj, **(3)** Bancharampur, **(4)** Kaliakoir, **(5)** Hobigonj, **(6)** Narail, **(7)** Jessore, and **(8)** Savar and (b) locations shown on Bangladesh Soil Map

Hand Rolling Method

The plastic limit of each soil sample is determined using hand rolling method (ASTM Standard D 4318). About 30 gm of soil passing through 425 micron sieve is mixed with distilled water and left for a suitable maturing time. A ball is formed with about 5 gm of soil paste and rolled into a thread of 3 mm diameter on a glass plate with the fingers of one hand. This procedure of mixing and rolling is repeated till the soil starts crumbling at a diameter of 3 mm. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia. The water content of the crumbled portion of the thread is determined. The test is repeated at least thrice to get the average water content. This average water content is called the plastic limit.
Following are the two methods which are proposed here.

Proposed Method 1

This method is used to determine a specific penetration value corresponding to whose water content can be marked as the Plastic Limit of that soil.

Experimental Set-up

The experimental set-up is exactly similar to the liquid limit test. Neither the load nor the dimensions of the set up is changed.The British fall cone apparatus (BS 1377, British Standard Institution, 1990); manufactured by Wykeham Farrance, Inc; with a 30 degree cone and weighing 0.785 N (80 gm) was used during the experimental investigation. The fall cone apparatus includes a specimen cup of 55 mm in diameter and 40 mm in height. Fig. 2(a) shows the schematic diagram of the experimental set-up.

Procedure

At first, the soil samples were dried in air. If clods were there in soil sample, they were broken with the help of wooden mallet. It has to be ensured that the soils were fine enough. Then sieving was done. The soil samples were passed through #200 IS standard sieve. The plastic limit of each soil sample was known. Each soil should be mixed with water exactly by the percentage of the plastic limit. Distilled water was used to make the soil paste.

After making the soil sample it was taken in the mould and the mould is filled to the top. The top surface was made horizontal and taken care that too much hollow was not present in the sample. It should also be kept in mind that soil sample should not be pressed too much. Then it was placed below the cone and the load is dropped for 5 seconds. That penetration value was that noted. Fig. 2(b) shows the photograph of the experimental set-up.

Proposed Method 2

The load used for this test should be 240 gm. The load of the cone is 30 gm. So the modified load should be of 210 gm. Mild steel has been used to make the load. For inserting it in the cone penetrometer a small section has been cut off and the height is adjusted according to it.

The experimental set-up and procedure is exactly the same as the Liquid Limit Test using cone penetrometer apart from the load used for it. In Liquid Limit Test, 80 gm load is used for penetration where in Plastic Limit Test 240 gm load is used. It is based on the measurement of penetration into the soil of a standardized cone of specific mass. The plastic limit is defined as the water content of the soil which allows the cone to penetrate exactly 20 mm during that period of time. Because it is difficult to obtain a test with exactly 20 mm penetration, the procedure is performed multiple times with a range of water contents and the results are interpolated from water content vs penetration graph. Fig. 2(c) shows the custom made load and Fig. 2(d) shows the schematic diagram of the load.

(a) (b) (c) (d)

Fig. 2: (a) schematic diagram of the experimental set-up; (b) photograph of the test set-up; (c) The modified load and (d) schematic diagram of the load (dimensions in cm)

RESULTS AND DISCUSSIONS

After collecting the samples from different locations, various tests were performed on the soils such as: specific gravity test, sieve analysis, hydrometer analysis, liquid limit test etc. Grain size curves has been obtained by sieve analysis and hydrometer analysis. The curves for all soil samples are shown in Fig. 3 and the results are shown in tabular form in Table 1.

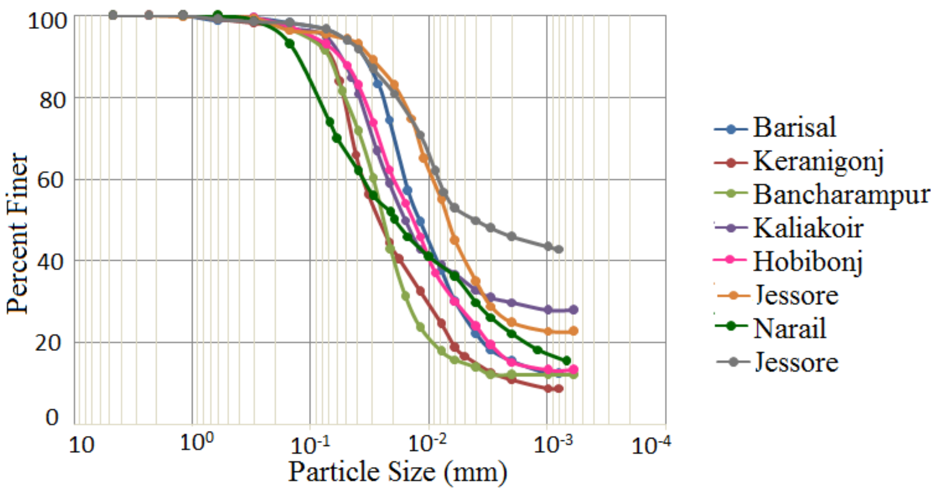


Fig. 3: Grain size curves of eight soil samples collected

Table 1: Grain size, soil classification and liquid limit

Name of the region	G_s	% Passing # 200 sieve	Sand (%)	Silt (%)	Clay (%)	Soil Classification (USCS)	Liquid Limit (%)
Barisal	2.64	96.8	3.2	71.0	15.5	CL	34
Keraniganj	2.72	85.6	1.0	88.0	11.0	CL+ML	23
Bancharampur	2.83	95.5	1.0	52.5	46.5	ML	37
Kaliakor	2.64	94.8	5.2	64.8	30.0	CL	36
Hobiganj	2.55	93.1	6.9	77.7	15.4	CL	31
Narail	2.71	88.9	1.2	66.8	32.0	ML	35
Jessore	2.57	95.5	3.9	71.1	25.0	CL	36
Savar	2.62	80	2.0	41.0	57.0	CL	49

Plastic Limit Obtained Using Modified Load and Penetration Values

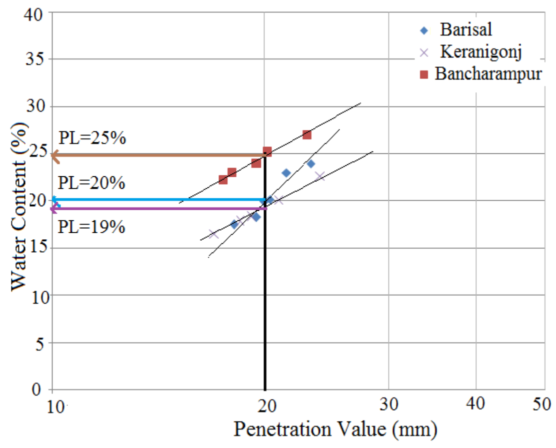
Method 1:

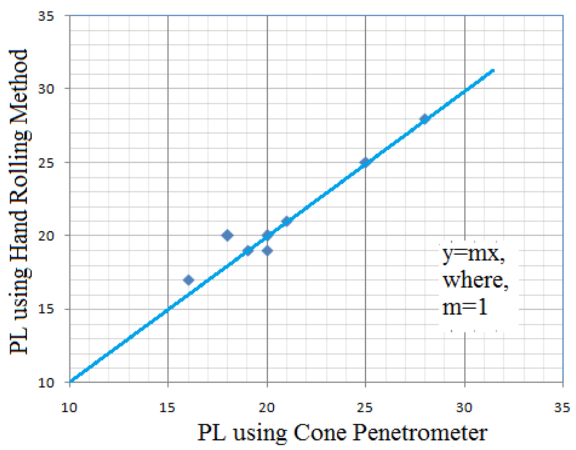
The predetermined plastic limit values using the Hand Rolling Method are used to determine the penetrations values. Table 2 depicts the penetration values obtained using cone penetrometer.

Method 2:

The water content corresponding to 20mm penetration is said to be the Plastic Limit of that soil. For three soils: Barisal, Keranigonj and Bancharampur, plastic limits are shown in the Fig. 4(a). Other values are obtained in the same process. Table 2 depicts are values of other five soil samples.

A correlation can be shown between the plastic limit values obtained in the hand rolling method and Cone Penetration's modified load method in Fig. 4(b). Since the values obtained in two methods are almost the same or near to each other, the plotted graph of plastic limit by hand rolling vs plastic limit by Cone Penetrometer is a straight line passing through the origin.





(a) (b)

Fig. 4: (a) Plastic Limit of Barisal, Keranigonj and Bancharampur soil with modified load and (b) correlation of plastic limits determined using Hand Rolling and Cone Penetrometer method

Table 2: Plastic limit of 8 soils using Hand Rolling and Cone Penetrometer (Method 1) and Penetration values found by Cone Pnetrometer (Method 2)

Name of the Locations	Method 1		Method 2	
	Values of Penetration (mm)	Average Penetration Value (mm)	PL (Hand Rolling Method) (%)	PL(Cone Penetrometer) (%)
Barisal	12.9	13 (approx.)	18	20
Keraniganj	13		19	19
Bancharampur	12.8		25	25
Kaliakor	12.3		16	17
Hobiganj	13.1		20	20
Narail	13.5		28	28
Jessore	11.6		21	21
Savar	Not obtained		20	19

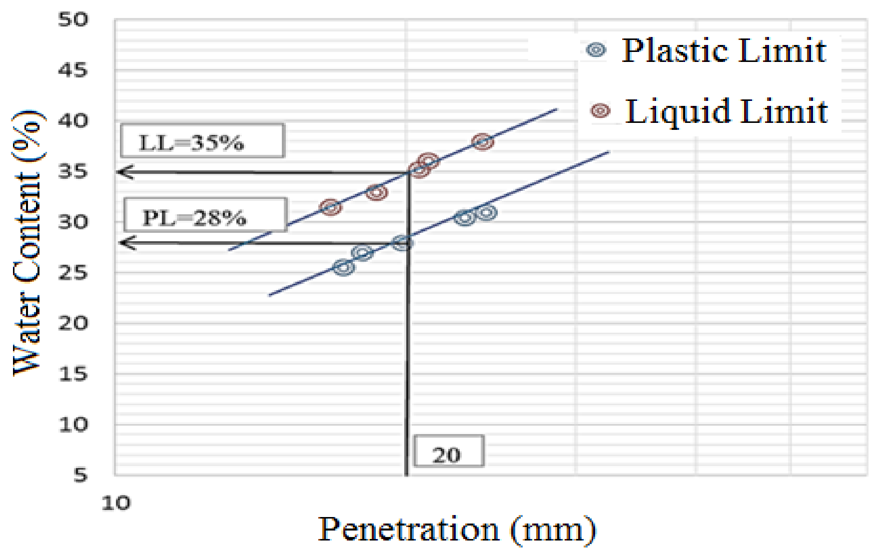


Fig. 5: Combined graph for liquid limit and plastic limit using Cone Penetrometer (for Narail soil)

Plastic Limit is determined by 240 gm load and Liquid Limit is determined using 80 gm load. If the water content vs penetration curves for both are plotted in the same graph, we can see that they are parallel to each other. The vertical difference between these two lines is the Plasticity index of this soil. Fig. 5 shows the liquid limit and plastic limit curves for the soil sample of Narail. The other soil samples also give similar graphs.

Following observations are made:

(1) The penetration values are nearly 13 mm for 6 soil samples-Barisal, Keranigonj, Bancharampur Kaliakoir, Narail and Hobigonj. It can be said that without modifying the load, the plastic limit values can be obtained from the penetration at nearly 13 mm.

(2) The study was done on eight soil samples. Among them six soil samples gave quite similar penetration values where in case of Savar soil sample we could not hydrate it with Plastic Limit to do the test. This is because there is a large difference between the liquid limit (49%) and plastic limit (20%) of this soil.

(3) This study was done only on eight soil samples. Use of more soil samples can give more precise and accurate results.

(4) Use of modified load for determining plastic limit is quite and accurate process and less operator dependent.

(5) When the plastic limit obtained by Hand Rolling method and Cone Penetrometer are plotted in plain graph, they give a straight line passing through the origin.

CONCLUSIONS

Findings of this research are summarized below:

(1) The specific penetration values obtained from different type of soils for determining plastic limit is nearly 13 mm. That means in the water content vs penetration graph from liquid limit test, water content corresponding to 13 mm penetration can be said the plastic limit.

(2) The penetration value of Jessore soil is quite far from the value 13mm.

(3) The penetration value of Savar soil sample could not be obtained because it did not make a proper soil paste to perform the test.

(4) The plastic limit values obtained by Cone Penetrometer by modifying the load were almost the same as the values obtained by Hand Rolling Method.

The accuracy of measuring plastic limit of fine grained soil has been debatable. The study proposed new techniques to determine the plastic limit which has rich significance on the soil characteristics. The results produced from these methods were compared to the results from standard method stated in (ASTM Standard D 4318) Hand Rolling method. It is apparent that method which is less operator dependent produce least variability and is expected to be more feasible means of measuring Plastic Limit of soil.

ACKNOWLEDGEMENTS

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CONSTRUCTION OF EARTHEN HOUSES USING CSEB: BANGLADESH PERSPECTIVE

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ABSTRACT
In a country like Bangladesh, endeavors to find a low cost, eco-friendly and sustainable building material that mitigates the problems caused by construction materials such as Fired Clay Bricks (FCB), Corrugated Iron Sheet (CI), concrete, wood, bamboo, etc is of paramount importance. This study aims to investigate the effect of different stabilizers on engineering properties of Compressed Stabilized Earth Blocks (CSEB). Blocks were prepared using local soil collected from Savar. In addition, compressed blocks containing three different stabilizers (lime, cement and jute-lime mixture) at various percentages were also prepared. After a 28 days curing period, it was observed that unstabilized and lime stabilized blocks had undergone significant cracking. It was found that blocks stabilized with a combination of jute fibre (0.3% w/w) and lime (5% w/w) can withstand large deformations. Although cement is effective in increasing the strength, it is not as effective in improving the ductility of the blocks. From the investigation, it was concluded that jute-lime mix stabilized CSEB can be used as a cost effective earthquake resistant building material.

Keywords: CSEB; earthen block; shrinkage; lime; jute fibre

INTRODUCTION
About one third of the world's populations live in some form of earthen construction (Houben and Guillaud, 1994). Raw earth was one of the first, oldest and most traditional building materials to be used by man and it has a heritage dating back over at least 10,000 years (Islam, 2010; Hossain, 2015). Sustainability of such earthen houses gained the attention of developed countries in the past 40 years (Islam et al., 2006; Islam, 2010). However, this interest is not growing in developing countries like Bangladesh. As a low income country, the common people of Bangladesh can only dream of building a decent shelter for themselves at a low cost. Therefore, a need arises to find a feasible building material which is not only locally available and economical, but is also a way towards sustainable development. As such, it could be hoped that if local people understands the feasibility and economy of earthen houses, its use would be maximized in the near future.
Earthen buildings have the benefit that they can be built from on-site materials rather than materials with high carbon footprints (Holliday et al., 2016). However, there are few undesirable properties such as loss of strength when saturated with water, erosion due to wind or driving rain and poor dimensional stability (Islam and Haque, 2009; Islam and Iwashita, 2010). Durability and strength are also major problems. Another severe problem of earthen building is its vulnerability to earthquake loading. Various researches have been carried out around the world to alleviate these problems. One such form of earthen building material is Compressed Earthen Block (CEB). When stabilizers such as cement, lime or jute are added in certain proportions to form CEB, then it is called Compressed Stabilized Earthen Block, CSEB (Mesbah et al., 2004; Marin et al., 2010; Ming, 2011). The numbers of factors influencing the properties of such blocks are many. Not only the stabilizers but also the clay content has significant effect on strength and erosion properties of CEB/CSEB (Walker, 2004). Due to affordability, local availability and ease of construction, earth blocks have huge potential as a low-cost building construction material in Bangladesh. This study focuses on improvement of earth block as a building construction material, determining the factors that drive environmental and economic impact and analyzing the tradeoff of properties germane to different earth blocks. The primary objective of this research is to determine the properties of lime stabilized clay and to obtain the

influence of stabilizers on strength properties of earthen block.

Advantages over other construction materials

Bangladesh needs an alternative building material to Fired Clay Bricks (FCB), Corrugated Iron Sheet (CI), concrete, wood, bamboo, etc. that is both cost effective and environment-friendly. CSEB can be a good alternative. Not only CSEBs are 25-30% cheaper than FCB, but also produce 70% less CO_2 emissions per m^2 wall than that of FCB. In addition, there is no deforestation for firewood, neither any top soil depletion. For the production of CSEB, mainly subsoil is needed and the abundant supply of riverbed sand in this region can also be used. Moreover, hollow-interlocking CSEBs allow for horizontal and vertical reinforcement for earthquake resistant construction. For constructing affordable, safe and eco-friendly housing, CSEB is a strong alternative in Bangladesh.

METHODOLOGY

Soil sample was collected from Christian Commission for Development in Bangladesh Human and Organizational Potential Enhancement Centre (CCDB HOPE CENTRE) which is located at Baroipara, Savar. All soil samples were collected from 15 cm below the ground surface. New soil samples underwent an initial visual inspection after being received in the laboratory. The amount of organic material in each of the sample along with the sample's in-situ moisture content has been determined. Tests pertaining to the engineering and index properties of soil have also been conducted following ASTM standards (ASTM, 2006). The soil used for CSEB production has been found to contain primarily silt and clay. Particle size distribution of the soils is presented in Fig. 1. The clay component provides the cohesion or binding forces necessary to hold the particles comprising the block together. Silt, sand and gravel particles contribute to the structural strength by combining to create a compact matrix with little void spaces. Lime was selected primarily for soil stabilization. Soil sample was first stabilized with different lime contents (3%, 6% and 9%) to observe the behavior of lime stabilized soil.

Soil mix was watered until it was plastic enough to mould. Water content less than optimum moisture content of the soil by weight has been used. The water and soil was thoroughly mixed. As the blocks may develop some cracks, to reduce the number of cracks and also to make the blocks more weatherproof, stabilizing materials have been added to the mix.

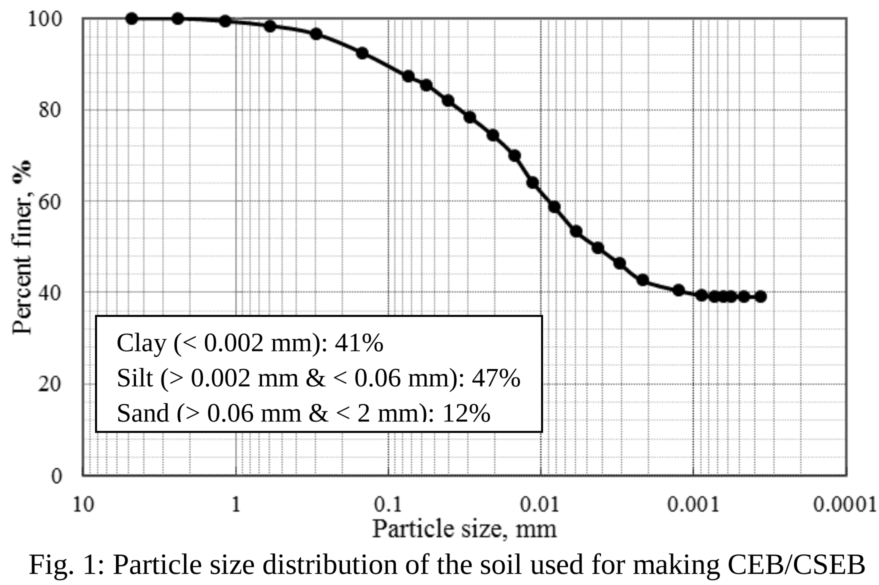


Fig. 1: Particle size distribution of the soil used for making CEB/CSEB

Here, soil samples were stabilized using either cement and sand, only lime, or a combination of lime and jute together. Another sample where no stabilizers were used was also prepared. Five blocks of each set were prepared using earthen block preparation machine (Press 3000 Multi-Mould Manual Press). Four groups of earth blocks were prepared for testing, unstabilized block (USB), blocks at field condition (HOPE Centre)/cement and sand stabilized CSEB (CSSB), lime stabilized CSEB (LSB) and lime-jute stabilized CSEB (LJSB). The combinations of various constituents of each type of block are given in Table 1. These blocks were cured for 28 days before testing for compressive strength. Blocks were cured by keeping them at room temperature (25°-30°C) and spraying water on them at two days interval. Fig. 2 shows the blocks after one day of production at the time of curing. After curing period, unconfined compressive strength (UCS) test was performed using Universal Testing Machine to study the strength and ultimate failure strain of different blocks. The strain rate used did not allow for the stress rate to surpass the rate given by the code.

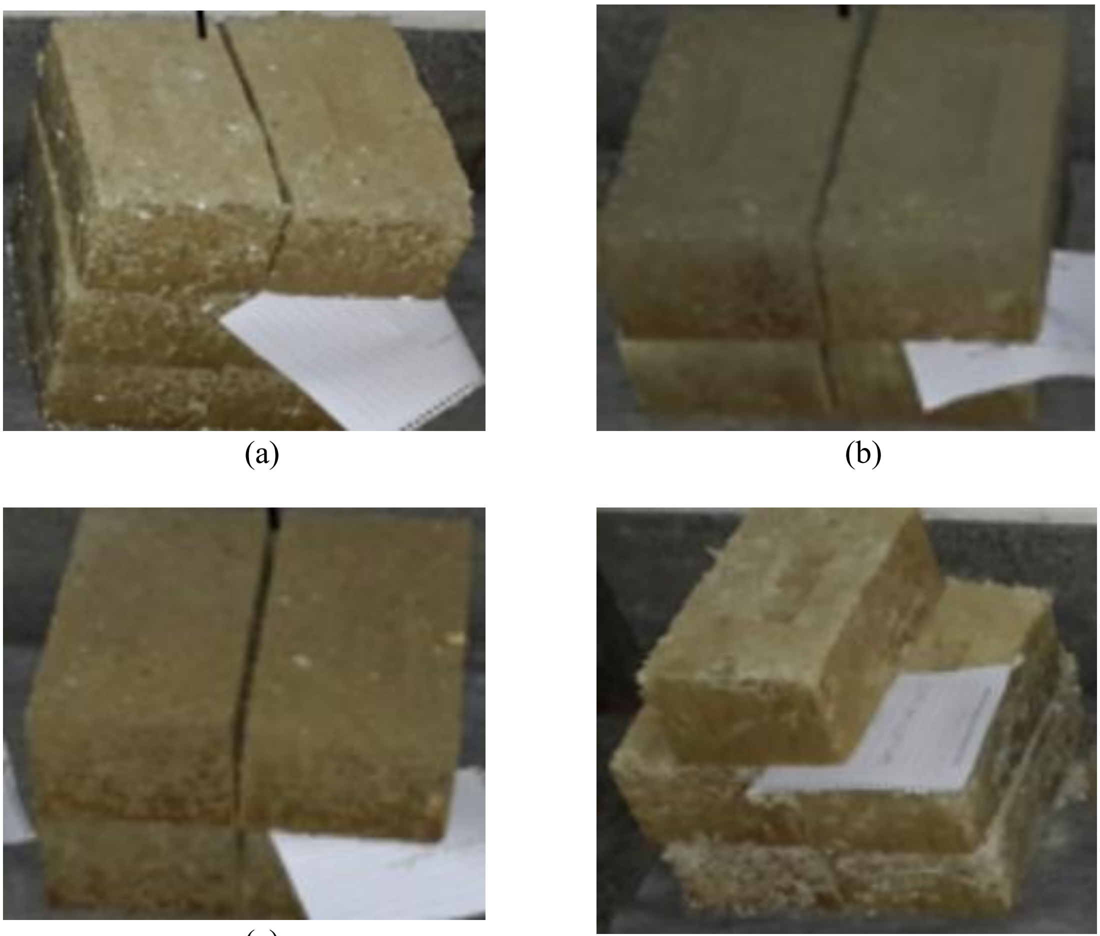
To calculate UCS, Equation 1 was used.

$$UCS = P/A \qquad (1)$$

Here, P = Force at failure (lbs)

A = Cross sectional area of top face of specimen (According to ASTM D143, ASTM, 2006)

After selecting the desired soil mix ratio, several trials were given by producing the blocks and testing them. If, at any point during the process, the proposed soil or soil mixture did not comply to the requirements, the soil sample was reselected and/or the sample modified.



(a) (b) (c) (d)

Figure 2: (a) Unstabilized CEB (USB); (b) lime stabilized CSEB (LSB); (c) cement and sand stabilized CSEB (CSSB) and (d) lime-jute mix stabilized CSEB (LJSB) samples during curing period.

Table 1 Combinations used for preparing earthen block

Group Name	Soil		Cement		Coarse Sand		Lime		Fiber (Jute)		Water		Number of block produced
	kg	%	kg	%	kg	%	kg	%	kg	%	Lit	%	
USB	16	91	-	-	-	-	-	-	-	-	1.5	9	5
CSSB	15	67	2	9	3	13	-	-	-	-	2.5	11	5
LSB	25	86	-	-	-	-	1.5	5	-	-	2.5	9	5
LJSB	20	82	-	-	-	-	1.2	5	0.07	0.3	3.0	12	5

RESULTS AND DISCUSSIONS

By performing soil identification tests, according to USCS the soil was found to be CL. After performing Atterberg limit tests of lime stabilized (3%, 6% and 9%) soil and unstabilized soil, a detailed comparison could be made based on their different properties as shown in Fig. 3.

In linear shrinkage tests, the soil behaved differently for different lime contents. For 3% of lime, the linear shrinkage result was approximately same as the unstabilized soil sample while for 6% and 9% lime contents, one and two cracks formed respectively along the lateral direction of the sample. From the above experiments and observations, it can be inferred that as the soil sample has relatively high amount of clay, it should be stabilized with lime (Ming, 2011; Ogundipe, 2013). Nevertheless, too much lime causes shrinkage crack. Henceforth, 5% lime was mixed with soil for preparing earth blocks. There is a likelihood of shrinkage crack if the soil is only stabilized with lime. Thus, some blocks were made using lime and jute with soil while others were stabilized with sand and cement. Also, some blocks were made using only soil to compare with stabilized bricks.

During the curing period, a lot of shrinkage cracks developed in the unstabilized and lime stabilized blocks and henceforth they were considered unsuitable for the construction of CSEB. The remaining two groups of specimens had undergone no cracks and seemed to have gained sufficient strength. Therefore, CSSB and LJSB blocks were tested using Universal Testing Machine. The average ultimate compressive strength and failure strain of each group of blocks are presented in Fig. 4. Typical condition of specimens before and after the test are depicted in Fig. 5.

It can be deduced from the test results that though the cement and sand stabilized blocks possess high compressive strength, they might not be suitable due to low failure strain and ductility and inadequate resistance against earthquake forces. Addition of small amount of lime and jute reinforcement with soil offers considerable bond strength with the development of no shrinkage crack at the time of curing. Jute and lime stabilized blocks maintained their integrity at the time of loading as they have

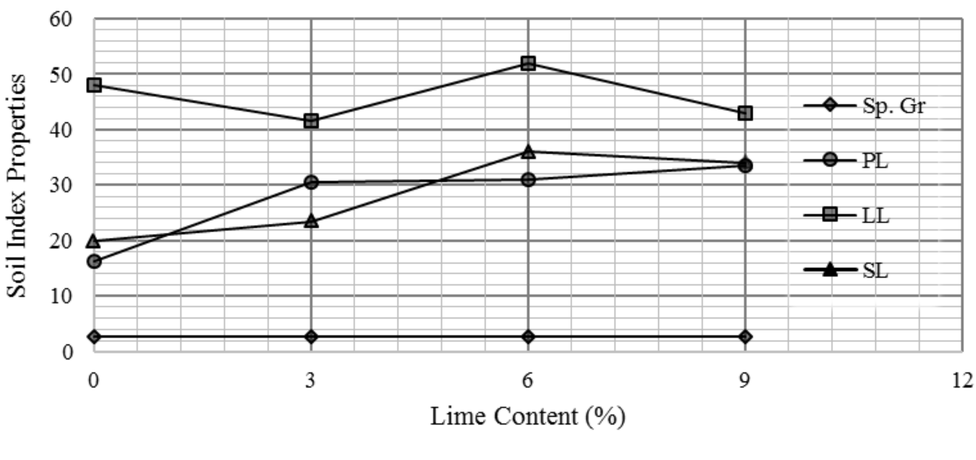
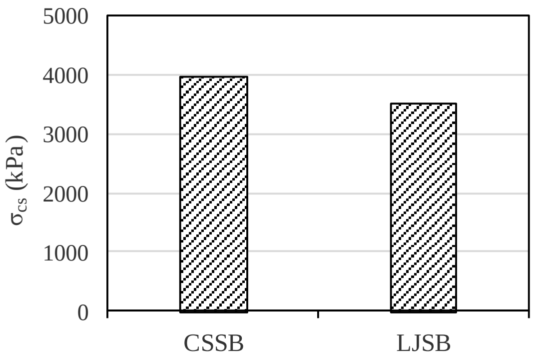
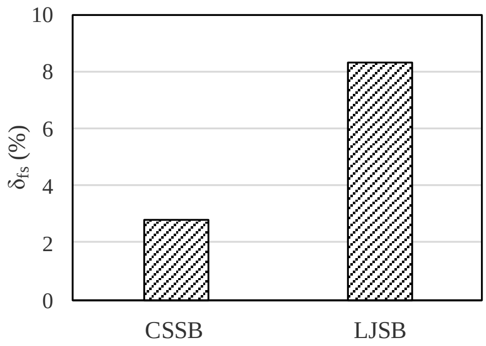


Fig. 3: Variation of index properties with lime content

high failure strength and considerably good ultimate strength. Also, jute and lime stabilized blocks have higher ductility and resistance against earthquake forces. If the jute fibres are mixed at a large scale (around 2%), higher ultimate compressive strength and failure strain can be achieved and the block properties will be enhanced significantly (Islam and Haque, 2009).



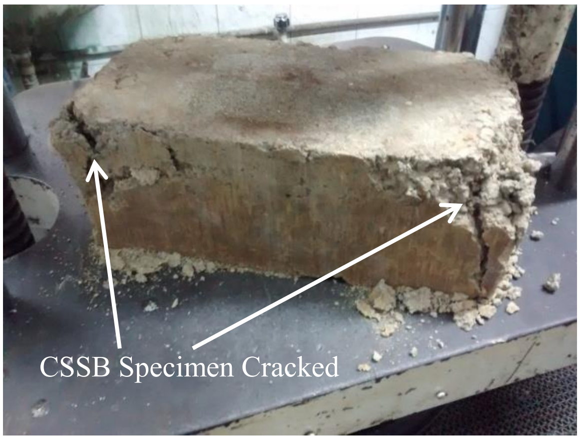
(a)



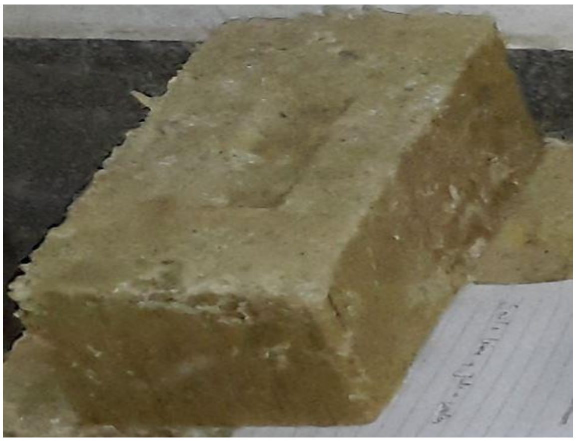
(b)

Figure 4. Comparison of (a) compressive strength and (b) failure strain of: cement and sand stabilized CSEB (CSSB) and lime-jute mix stabilized (LJSB).

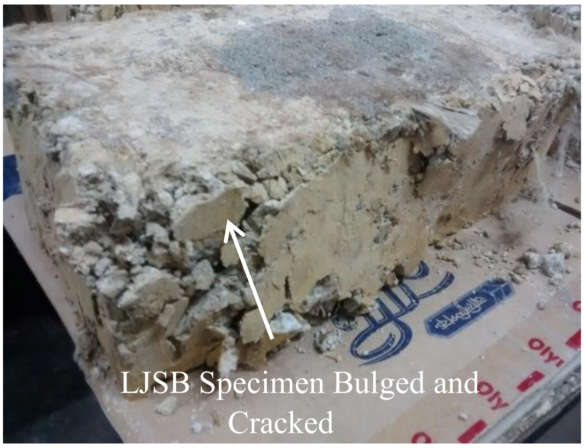
(a)



(b)



(c)



(d)

Figure 5. Photographs of: (a) cement and sand stabilized CSEB (CSSB) specimen before test; (b) cement and sand stabilized CSEB (CSSB) specimen after test; (c) lime-jute mix stabilized CSEB (LJSB) specimen before test; (d) lime-jute mix stabilized CSEB (LJSB) specimen after test.

CONCLUSIONS

Climate change impact on Bangladesh would only reinforce the existing problems that pose serious impediment to the economic development of the country. CSEB is a suitable alternative for constructing affordable, safe and eco-friendly buildings in Bangladesh and can be a positive shift towards a safe and pollution free environment to protect the world from climate change.

From the test results, it can be concluded that silty clay soil with high lime content exhibits high shrinkage crack. Moreover, CSEB blocks made using clay only without any type of stabilizer and blocks made using only lime as a stabilizer are not recommended for construction of any structures. All other clay blocks using sand-cement as stabilizers and using jute-lime as stabilizer may be used for construction works. Jute fiber with lime is found to be better with respect to shrinkage crack control and increased strength and reduced deformation properties of block. In addition, it increases bond strength, has higher ductility and can be considered as a low cost earthquake resistant material for earthen house construction. Due to limitation of scope, analysis of dynamic properties has not been carried out. Dynamic tests, for instance shake table tests of full scale model, are to be shown in authors' future publications.

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CLIMATE RESILIENT SLOPE PROTECTION FOR COASTAL REGIONS OF BANGLADESH USING BIO-ENGINEERINGTECHNIQUES

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ABSTRACT
One of the major maintenance challenges of rural roads, bridge approaches and minor embankments in Bangladesh is protection of respective slopes. Almost whole of the country remains inundated for 4 to 6 months of the year that loosen the earthen slopes resulting in erosion of the slopes. The traditional engineering solutions for this problem have been application of concrete blocks, palisade, sand bags, stone revetments, geo-textile, etc. that not only increases cost of construction and maintenance but is often found ineffective and unsustainable. Studies in many countries around the world has revealed that embankment stability can also be gainfully augmented by using bio-engineering techniques and can provide for long-term sustainable low-cost and maintenance free solution for slope protection. It has been widely recognized that plant root systems can improve the soil shear strength significantly. The technique envisages the use of appropriate vegetation with minimum artificial human intervention resulting in economic and ecological benefits. In this regard, a comprehensive field study on vegetative slope protection is being conducted in different parts of the coastal region under the 'Coastal Climate Resilient Infrastructure Project (CCRIP)'. Vetiver grass (*Vetiveria zizanioides,* locally known as *binna grass*) has been selected for its special attributes and easy availability throughout the country. Effectiveness of vetiver grass against rain-cut erosion and tidal wave in coastal zones of Bangladesh has been examined using model study and field trials. This paper presents three case studies of vetiver plantation in slope protection against rain-cut and wave-induced erosion. It has been found that the vetiver based bio-engineering technique can provide an effective and environment-friendly solution for embankment slope protection.

Keywords: Bio-engineering, embankment protection, vetiver, salinity, sustainability.

INTRODUCTION
Most of the traditional practices to protect embankments against erosion in Bangladesh are expensive and sometimes their performances are quite unsatisfactory with respect to their costs. Recently, around the world the bio-engineering techniques have widely gained popularity for embankment/slope protection (Hengchaovanich, 1998; Truong et al., 2002; Islam et al., 2014). These techniques are being increasingly favored to control soil erosion in general and for slope protection in particular. Vetiver grass (*Vetiveria zizanioides*, locally known as *binna grass*) has already been used in more than 100 countries around the world to curb soil erosion. Most developed and developing countries like Australia, Brazil, China, India, Kuwait, Malaysia, Spain, Thailand, USA and Zimbabwe use vetiver for erosion protection works (Islam et al., 2013a; Islam et al., 2013b; Suleiman et al., 2013; Islam, 2015; Parshi, 2016). However, efficacy of such systems has not been scientifically studied in Bangladesh perspective. Local Government Engineering Department (LGED) is one of the largest departments of Bangladesh Government and has constructed about 3,05,000 km of rural road. More than 50% of these roads are constructed on embankments. These embankments need to be protected from erosion and damage for overall structural stability and sustainability of roadway pavements. Henceforth, LGED seeks for cheaper, appropriate and effective protection measure for its vast road network infrastructures. To study the potential of bio-engineering as climate resilient slope protection measure in the coastal areas of Bangladesh, an action research project entitled "Investigation of

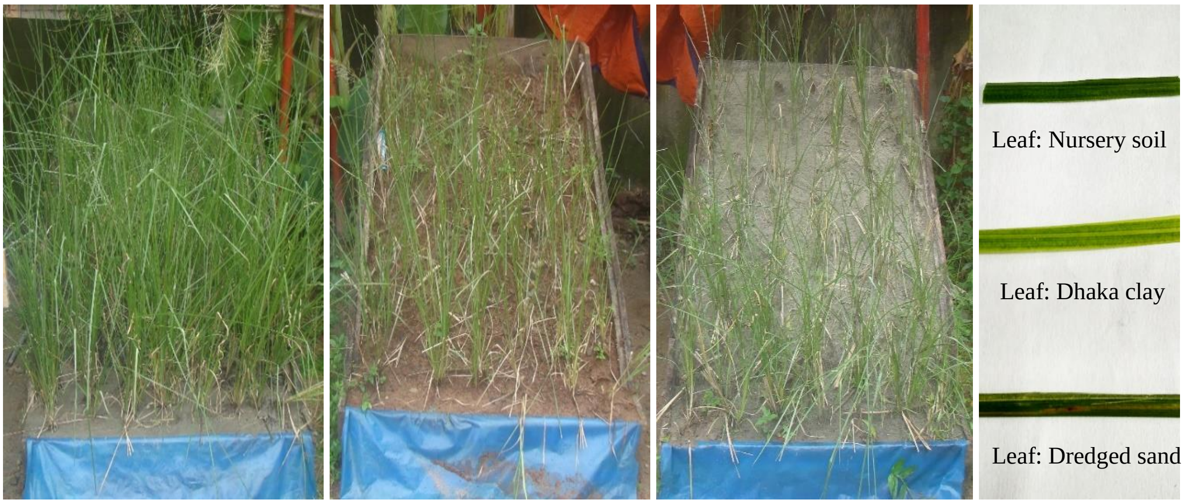
Climate Resilient Slope Protection of Embankments" was awarded to the Department of Civil Engineering, BUET by LGED in 2014. The project has started in July 2014 and the research activities of the project is planned to be carried out over a period of 36 months. The CCRIP intervention covers a total of 12 districts of the south-western coastal areas: Gopalgonj, Madaripur and Shariyatpur (Dhaka Division); Khulna, Bagerhat and Sathkhira (Khulna Division); Barisal, Patuakhali, Barguna, Jhalokathi, Bhola and Pirojpur (Barisal Division). Main objective of the study is to identify suitable plant species that would provide for a sustainable, environmentally safe and cost-effective anti-erosion cover for embankments/dykes/village and market mounds etc. to be adapted to local conditions of inundation and saline water, thereby making rural communication as well as community infrastructures more climate resilient and sustainable.

METHODOLOGY

Vetiver grass has been selected for its inherent special attributes and easy availability throughout the country. To determine the efficacy of vetiver grass in stabilizing road side slopes against rain-cut erosion and flood, at first the model (dimension: 180 cm ×100 cm × 130 cm containing different soils and with varying slopes) studies have been conducted at BUET premises. Vetiver grass was collected from a local natural source located in Pubail, Gazipur area. Then they were planted in cow dung mixed soil in poly-bags and nurtured for three months in an open place where these plants got enough sunlight and rainfall. Then healthy grown up plants were planted in grid, triangular and row patterns in the model slopes. Growth and performance of vetiver in slope protection are being monitored at 2-3 months' interval. At the beginning of dry season (winter) the shoots were trimmed and the plants were watered regularly. Figs. 1a-1c represent the condition of model slopes three months after plantation. Fig. 1d presents the close view of leaves of vetiver grown in different kinds of soils. For the model study, three types of soils namely: dredged sand, Dhaka clay and nursery soil were filled in six wooden slope models. The gradation curve of these soils has been presented in Fig. 3a. The general description of the models is presented in Table 1. After the model study, the field trials have been conducted in the selected coastal zones to study the performance of vetiver based protection measures under field conditions. Under CCRIP project so far field trial has been conducted in nine districts namely: Barisal, Khulna, Satkhira, Sariatpur, Patuakhali, Madaripur, Gopalganj, Pirojpur and Bagerhat. In this paper, findings of the field trials conducted at Barisal, Madaripur and Satkhira sites have been presented. Fig. 2a shows the districts under CCRIP project. All the selected roads were newly upgraded. Then the embankment slopes of these sites were prepared getting help from the members of labor contracting society (LCS). The selected vetiver tillers were planted at 30cm apart along the slope and perpendicular to the slope. The typical road section and plantation pattern are presented in Fig. 2b and

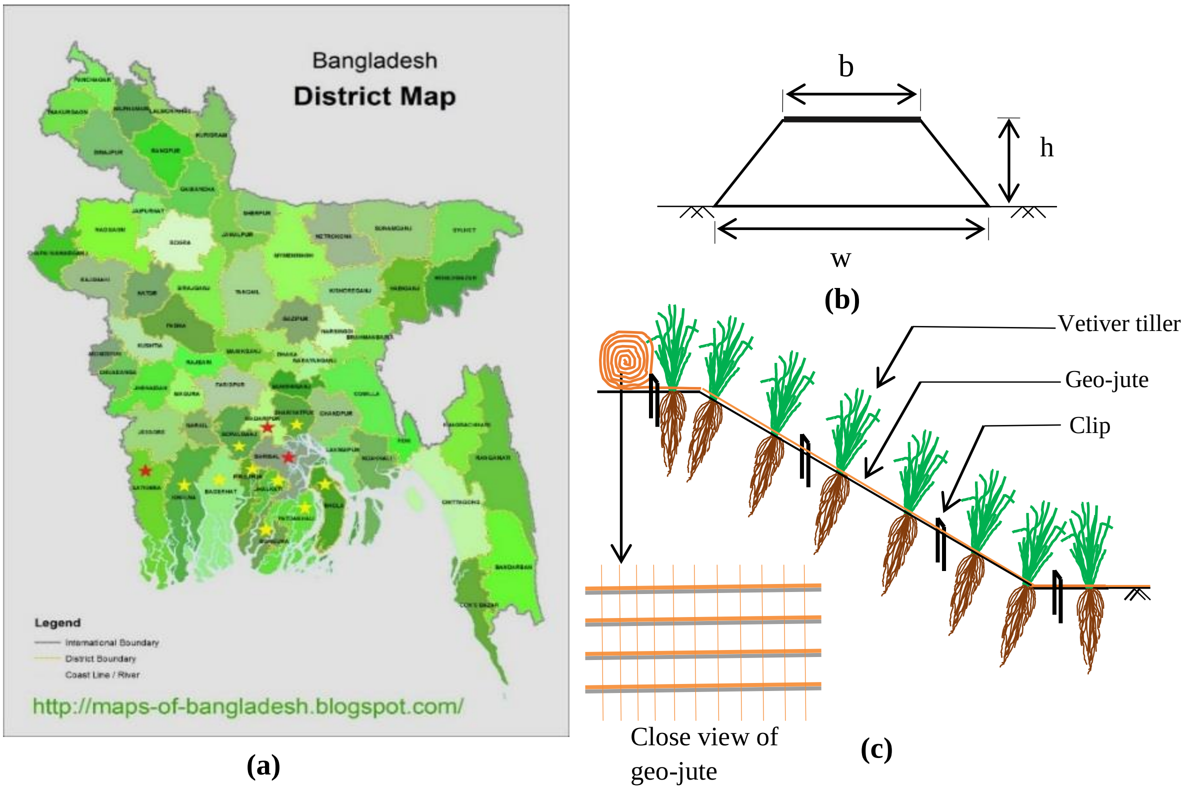
Table 1: Description of the models

Model name	Source	Type of soil	Plantation	
			Pattern	Spacing
M1	Dredged material	Silty sand	Grid	15 cm c/c
M2	Dhaka clay	Red silty clay	Grid	15 cm c/c
M3	Nursery	Sandy silt with organic content	Triangular	15 cm c/c
M4	Nursery	Sandy silt with organic content	Row	10 cm c/c
M5	Nursery	Sandy silt with organic content	Row	15 cm c/c
M6	Nursery	Sandy silt with organic content	Row	20 cm c/c
M7	Nursery	Sandy silt with organic content (Salinity: 10 to 12 ds/m)	Grid	25 cm c/c



(a) (b) (c) (d)

Fig. 1 Photographs showing the growth of vetiver in model slopes after three months of plantation: (a) nursery soil (leaf: dark green), (b) red clay (leaf: light green with multiple yellow strips), (c) dredged sand (leaf: dark green with single yellow strip in the middle) and (d) close view of leaves



(a) **(c)**

Fig. 2 (a) Study areas of CCRIP, (b) typical road section of LGED (top width: b, bottom width: w, height: h) (LGED, 2005) and (c) Schematic diagram showing vetiver plantation scheme

Fig. 2c, respectively. After completion of vetiver plantation, proper nursing and monitoring (that includes watering, measurement of root, shoot growth, erosion of the road embankment) was made at regular basis. The first trial was conducted at Barisal and then at Satkhira. These two trials were conducted in August, 2015. The trial in Madaripur was conducted in January, 2016. All of these roads are village road. A brief description about the selected field trial sites is stated in the Table 2. Fig. 4 states the condition of the field trial sites (including topography) at the time of plantation. Soil sample from each site was collected and index properties and grain size of the soil samples were determined. From the gradation curves shown in Fig. 3b, it is clear that, soil in these sites are basically clayey silt with 70-75% silt, 15-25% clay and 2-10% sand. The field trial sites were monitored closely and performance data were acquired on a regular basis.

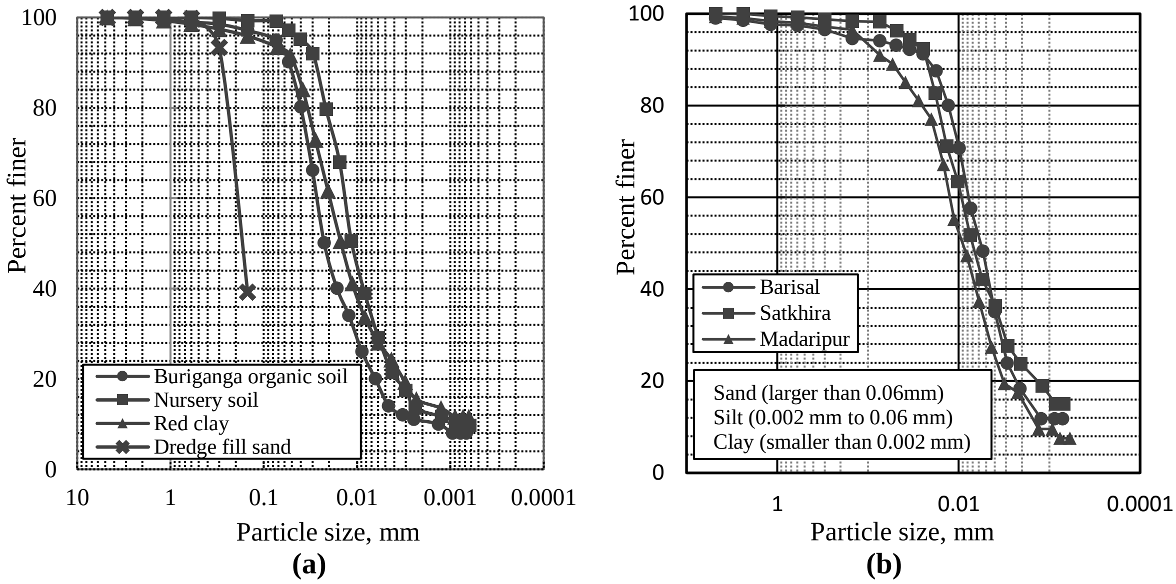


Fig. 3 Gradation curves for: (a) soils used for model and (b) soils collected from field trial sites

Table 2: Location, geometry of road embankment, traffic and climatic conditions of field trial sites

Site Name	District	GPS	Length of road (km)	b (m)	h (m)	w (m)	Traffic condition	Temp (°C)	Rainfall (mm)
Babuganj	Barisal	22.84° N, 90.29° E	2.3	4.4	2.4	11.4	Both motorized and non-motorized	25.9	2184
Rajoir	Madarip -ur	23.23° N, 90.07° E	5.0	4.0	2.43	11.2	Both motorized and non-motorized	33.8	1350
Asashuni	Satkhira	22.57° N, 89.19° E	2.13	5.5	2.13	11.2	Non-motorized	35.5	1710

(a) **(b)** **(c)**

Fig. 4 Photographs showing the site just after vetiver plantation: (a) Barisal (surrounded by wet land) (b) Madaripur (surrounded by green field with a small nearby canal) and (c) Satkhira (river *Morichop* flowing alongside creating wave action)

RESULTS AND DISCUSSIONS

With a view to investigating performance of the vetiver grass in protecting slope of road embankment, the field visits were made regularly in the intervention site areas. During filed visits as part of the plans monitoring process, the growth progress of both root and shoot were measured. Also it was keenly observed whether there was any damage to the road embankment slope occurred due to rain-cut or sliding. From the model studies, it has been found that vetiver growth is the best in nursery soil as compared to that of Dhaka clay and dredged sand (Fig. 1). Performance of vetiver grass against rain cut erosion of slopes has been found satisfactory in the model study. However, some part of the upper portion of the sand model eroded in the first few weeks of plantation. In these different types of soil, the color of the vetiver leaf has been found to be different (Fig. 1d). From Fig. 5, it was observed that after 24 weeks, roots grew up to 32 cm in Barisal, whereas in Madaripur the root grew up to 50 cm within the same span of time in spite of being planted in winter. This prodigious/accelerated growth in Madaripur was mainly due to proper nursing particularly during the dry season. In both Barisal and Madaripur, inflorescence has been observed which indicates maturity of vetiver grass. The growth of vetiver grass in Satkhira has been found to be poor, only a meager number of planted tillers did survive at this site with adverse climatic condition (Fig. 6c). The main reasons behind the poor growth in Satkhira intervention area have been identified as mainly due to the salinity as well as the improper time of plantation i.e., just after monsoon. In order to find a solution for growing vetiver in saline soil, vetiver was planted in a wooden slope that contained saline soil (sea salt mixed with nursery soil) with different alternative conditions. It was found that vetiver can be grown in saline soil adding organic fertilizer and ensuring proper watering. Moreover, it has revealed that the proper time of plantation would be well before the full monsoon period.

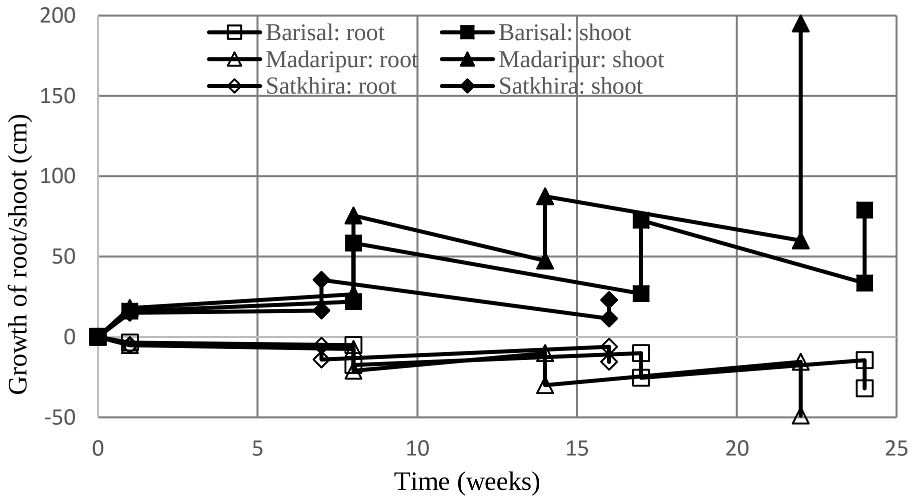


Fig. 5 Root and shoot growth of vetiver grass in the field trial sites

(a) **(b)** **(c)**

Fig. 6 Photographs showing: (a) Barisal site condition after 42 weeks, (b) Madaripur site condition after 20 weeks and (c) poor growth of vetiver in Satkhira site after 20 weeks

CONCLUSIONS

Bio-technology in contrast to current practices of embankment and slope protection around the world is more sustainable, practical and efficient. The data acquired from the field trials made in the intervention sites and model studies conducted at BUET premise, have revealed that the vetiver based slope protection measure is an effective method in protecting the slope of road embankment from rain-cut erosion. It is also found that to protect embankment slopes from erosion, these grasses need to be planted well before the monsoon period i.e. during May-June. Besides, proper nursing and monitoring need to be ensured until the plants grow quite well. It may take six weeks to eight weeks for vetiver to grow. It is also observed that to have better growth performance and survival rate, the vetiver grass should be planted at least six weeks before the monsoon starts. With this plantation arrangement, it is expected that the plants will get enough monsoon rainfall and grow very quickly and thereby would be ready for protecting slope from erosion more effectively. Even if the plantation is done in winter, proper nursing and watering can result in positive output. From the study, it is clear that vetiver plantation holds a good prospect for protecting slopes in different geographic setting with various soils and climatic conditions. Though, the growth rate of vetiver roots and shoots were found to be widely varied with soil conditions i.e., soil type, nutrient content, salinity and climatic conditions. It is recommended that if an embankment is constructed with sandy soil, the soil should be covered with geo-jute for ensuring the protection at the early stage. For saline zone, it is suggested that the plantation should be made on soil properly mixed up with cow-dung/fly ash and most importantly watering should be made regularly and vigorously for better growth and performance.

ACKNOWLEDGMENT

This research was supported by LGED and funded by IFAD. Sincere appreciation goes to LGED officials for their support and help in the field work. Authors would also like to show sincere gratitude to the Dr. M. Matiur Rahman, Ex-Director, Bangladesh National Herbarium for giving his valuable time and for sharing his wisdom with us in identifying proper plants. Author also thanks the Civil Engineering Department of BUET for facilitating the necessary support for this research.

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COST EFFECTIVE FOUNDATION ON PROBLEMATIC SOIL OF RECLAIMED AREAS IN DHAKA CITY

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ABSTRACT
Dhaka city is expanding rapidly on reclaimed land. Most of these sites are developed by filling marshy lowlands (1.5~13.5 m) using dredged materials from nearby river bed. Almost all fills are basically silty sand. The SPT N-value of the filling varies from 1~13. Current practice of foundation design for buildings in reclaimed areas is generally deep foundation (piles). Long piles are essentially vulnerable to lateral loads and construction of such long piles is difficult. Moreover, for five to six storied buildings pile foundation is uneconomical. This study has been conducted to propose an alternative and cost effective foundation system for building in reclaimed areas of Dhaka city. The study site was in Mirpur Defence Officers' Housing Scheme (DOHS), Mirpur, Dhaka. A shallow foundation with reinforced base soil was constructed at the selected site for full scale load test. Bearing capacity of the foundation soil was found to be increased by 15-20% due to base soil reinforcement. This test result eliminates the requirement of a deep foundation for a five to six storied residential building on such problematic soil. The finding of this study thus reveals a cost effective foundation alternative for medium rise structures in reclaimed areas of Dhaka city.

Keywords: Reclaimed area, geotextile, fill materials, cost effective foundation.

INTRODUCTION

Dhaka city is experiencing a rapid growth of urban population and it will continue to do so in future due to several unavoidable reasons. As a result, different new areas are being reclaimed near Dhaka city. In most cases, the practice for developing such reclaimed areas is to fill low lands (1.5-1.5m) by dredged soil collected from nearby riverbed (Islam and Nasrin, 2009; Ahmed, 2010 and Islam et al., 2013). Hydraulic filling procedure is the most widely used method among many filling procedures to reclaim such lands. Fig. 1 gives a pictorial view of reclamation procedure of the present day in Dhaka city. Details of land reclamation procedure are described in Islam et al. (2013).

Some studies have been carried out to evaluate the characteristics of dredged fill layer of the reclaimed sites (Ahamed 2005;). In most cases, the dredged material is silty sand with high fines content (Islam and Hossain, 2010). The presence of fines in hydraulic fill means greater compressibility and reduced permeability and hence it is subjected to long term consolidation. Soft organic clay layer beneath the filling layer may also cause excessive settlement problem to the structures lying on top of such soil with a shallow foundation (Islam et al., 2013).

Current practices of foundation design for buildings in such reclaimed areas are mainly construction of piles. Long piles are essentially exposed to lateral loads and negative skin friction (Islam and Nasrin, 2009). Negative skin friction produces a drag load that can be of substantial amount for long piles. Quality control of cast-in-situ long piles is also questionable. Moreover, for five to six storied building, pile foundation becomes uneconomical. Hence, the need for a cost effective foundation on such problematic soil has emerged as a potential subject to be addressed. The ultimate capacity of a reinforced shallow foundation on problematic soil is always estimated with empirical assumptions. Due to lack of analysis, the option of adopting reinforced shallow foundation on such soil has been neglected until very recently.

Fig. 1. Hydraulic filling procedure (a) dredged fill transported by barge (b) & (c) dredged fill transferred to the site in the form of slurry and (d) delivery of fill to the site.

CASE STUDY

In recent past use of geo-textile as reinforcement to improve the bearing capacity of foundation soil and settlement performance of shallow foundation has gained much attention in the geotechnical engineering field. However, this practice is yet to emerge as a popular solution. Geo-textile and other reinforcing materials at the base of a shallow foundation was used to construct a building structure at Khulna Medical College campus (Alamgir and Chowdhury, 2004). The performance of the adopted systems was found to be quite satisfactory. . Academic building of Khulna Medical College and its foundation system are shown in Fig. 2.

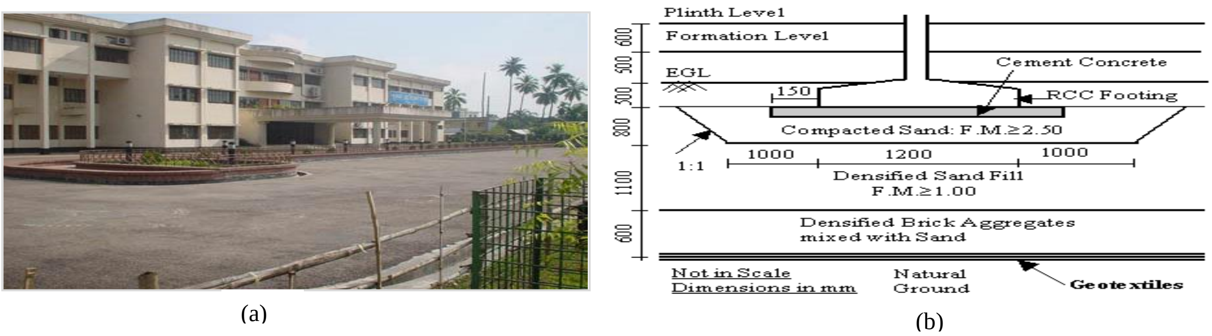


Fig. 2. (a) Academic building of Khulna Medical College and (b) foundation system (Alamgir and Chowdhury, 2004)

It is quite obvious that the dredged fill-soil of reclaimed areas demand special attention for designing foundation systems on it. There has to be a proper subsoil investigation, correct load-settlement pattern and proper numerical analysis for designing shallow foundation on these type of problematic soil. With this backdrop, the objective of this paper is to investigate the sub-soil characteristics of the selected reclaimed area and conduct full scale load tests on a reinforced shallow footing to investigate the feasibility of cost effective shallow foundation system on problematic soil of reclaimed areas in Dhaka city following numerical analysis.

METHODOLOGY

At first the sub-soil characteristics of selected reclaimed area (Mirpur DOHS) was determined for evaluating the bearing capacity of shallow foundation on it. Bearing capacity was estimated by analytical method. Then the bearing capacity of foundation soil and load-settlement behaviour of shallow foundation was evaluated by plate load test. A shallow foundation was constructed on reclaimed soil with base soil reinforcement by geo-textile for full scale load test. From the full scale load test, load-settlement behaviour of a shallow foundation was observed for 25 mm of settlement. From the load-settlement graph, the ultimate bearing capacity was determined for 25 mm

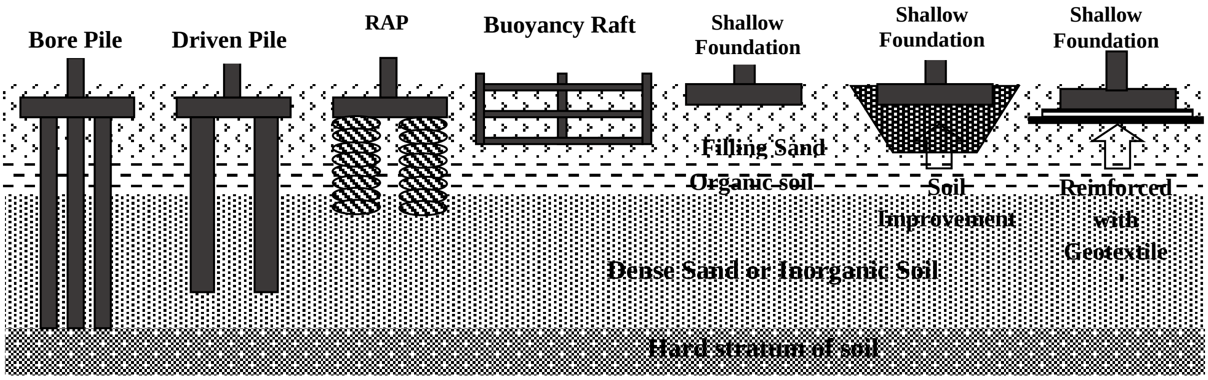


Fig. 3. Suitable foundation options for the study area (Fox and Cowell, 1998)

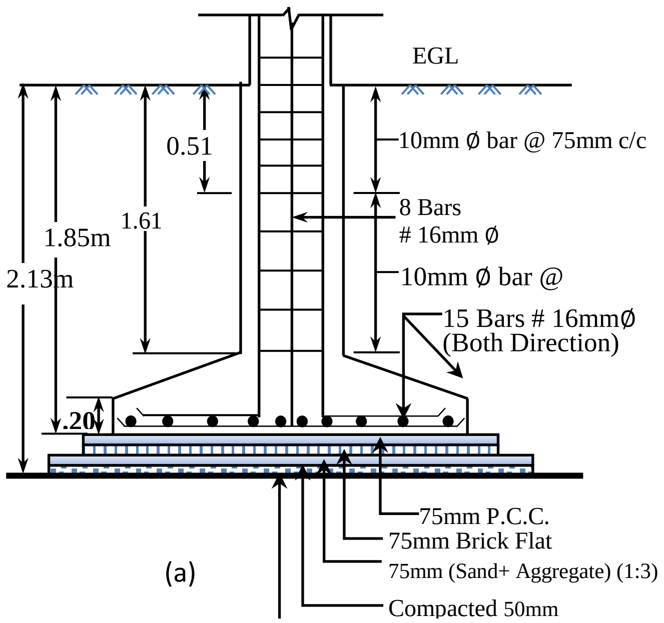
settlement. At the end, a comparative analysis of cost between reinforced shallow foundation and pile foundation was done to propose a cost effective shallow foundation on this type of problematic soil of Dhaka city.

FOUNDATION OPTIONS/ALTERNATIVES

At present, cast-in-situ pile foundations are generally used for building construction. Some other suitable foundation systems based on past studies (Fox and Cowell, 1998) that can be proposed for the study areas are shown in Fig. 3. These foundation systems can mitigate the problems that may occur to the structures in the reclaimed areas of Dhaka city. However, it is to be noted here that suitable foundation alternative is to be confirmed by field trials.

Shallow Foundation with Reinforced Footing

Traditional analysis usually discourages shallow foundation in reclaimed lands. However, bearing capacity calculation by appropriate soil characterization together with reinforced footing may lead us to adopt shallow foundation in many places of reclaimed land. Field load test verified by numerical analysis will clarify the effects of reinforcement used in the improved foundation system. The reinforcement option could be a geo-textile at the bottom over which there could be compacted sand and sand-aggregate layer. Details of the footing reinforcement is shown in Fig. 4.



(b)

Fig. 4: (a) Section view- sallow foundation with reinforcement and (b) photograph of constructed shallow foundation with reinforcement in field

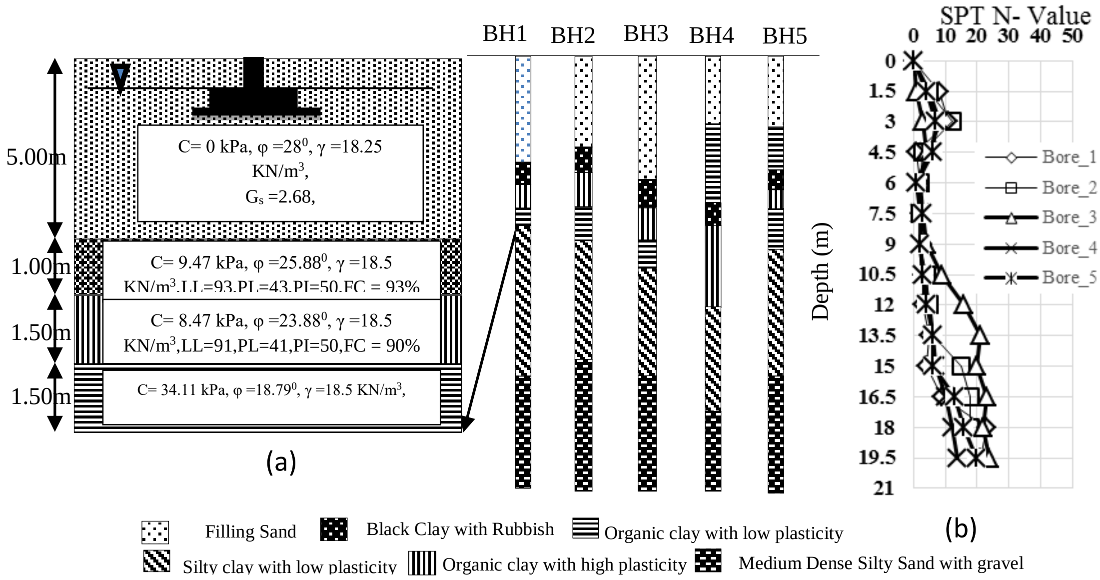


Fig. 5. (a) Sub-soil strata below footing and (b) Borelogs and SPT N-values of reclaimed

The study area is a typical reclaimed area of Dhaka city. It has a dredged fill layer of 3 to 6.5 m on top. Below the dredged fill soil there are black clay with rubbish, organic clay and silty clay. Beneath the clay layer from 16.5m below EGL, medium dense to dense silty sand layer exist having a higher SPT N-value. Details of sub-soil properties below footing and typical bore-logs of the study areas with field SPT N-value are presented in Fig. 5.

Sub-Soil Characteristics

In the reclaimed land generally it is found that the depth of filling sand layer varies from 4 to 6m from EGL. In most cases, the dredged material is silty sand with high fines content. The field SPT N-value of the filling layer varies from 1 to 11. It has been found that the value of specific gravity of the sand of the filling layer varies from 2.65 to 2.73. From the grain size analysis of dredged fill soil it is found that mean grain size (D_{50}) and fines content (F_c) of the sand of the filling layer vary from 0.12 to 0.15 mm and 17.4 to 30.7%, respectively. It is clear that the dredged fills are poorly graded sandy soil (SP). It has high void ratio and low density. The soil generally have low bearing capacity and is likely to experience local or punching shear failure (C_u<6, C_c<1). More details about the characteristics of organic soil are available in Islam and Nasrin (2009).

RESULTS AND DISCUSSIONS

Bearing Capacity Evaluation and Improvement

Various methods were adopted to evaluate the bearing capacity of foundation soil. Ultimate bearing capacity of foundation is shown in the graphical form in Fig. 6. At the very outset, empirical equation using SPT N-value was used to find out the bearing capacity of soil. The bearing capacity was found to vary between 304 kPa to 310 kPa. Based on various theoretical analyses, the bearing capacity was estimated to be within 257 to 350 kPa. Following plate load test, the bearing capacity of foundation soil was calculated to be 342 kPa. Foundation soil was also modelled with GEO5 software and estimated bearing capacity was found to be 267 kPa. Finally, a full scale load test was conducted on the shallow foundation constructed on reclaimed soil of Mirpur DOHS with the reinforced base layer. The reinforcement was done placing a geo-textile at the base of the footing. Full scale load test on the reinforced footing has raised ultimate bearing capacity of the foundation soil up to 400 kPa corresponding to a settlement of 25 mm. A comparitive analysis of load vs settlement curve of plate load test and full scale load test is shown in Fig. 7. At the end of the result analysis, it is found that the design bearing capacity of dredged fill soil is increased at least by 15%-30% due to foundation reinforcement.

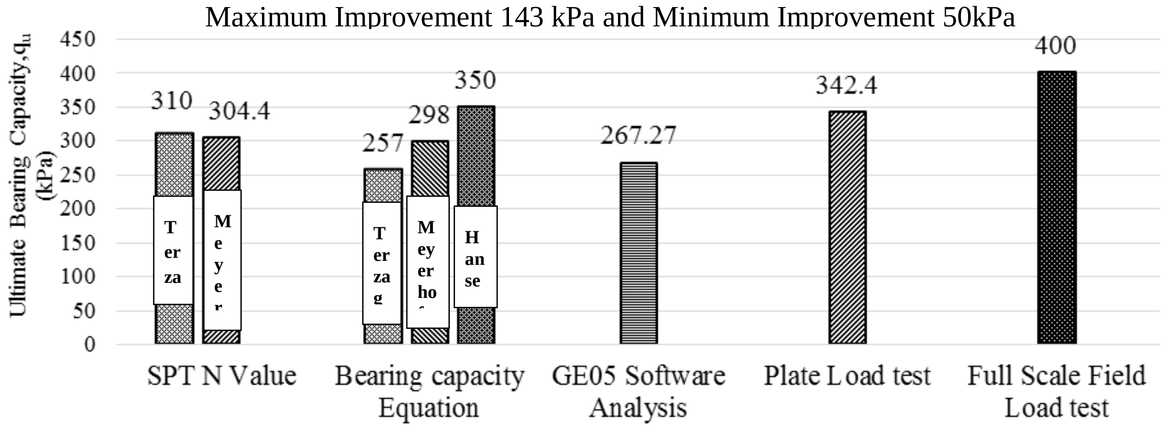


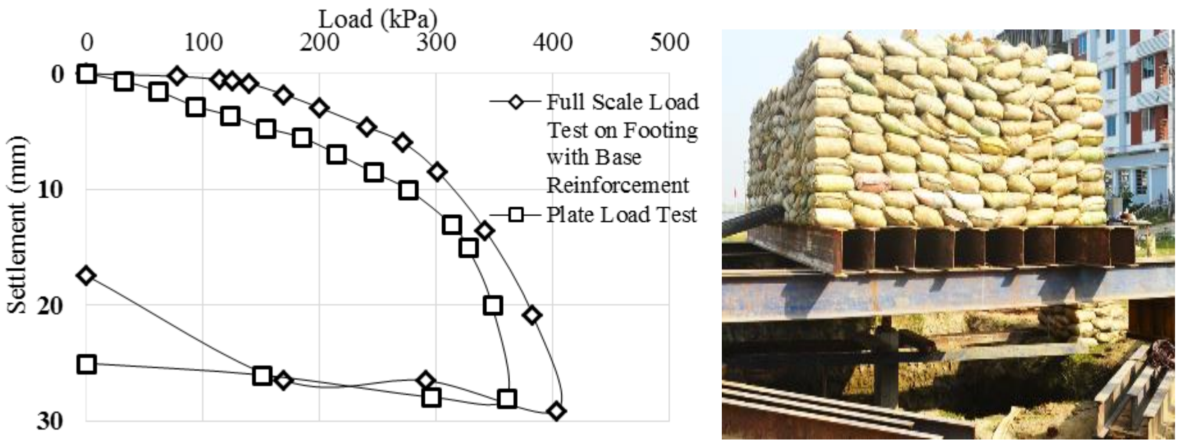
Fig. 6: Ultimate bearing capacity determined from different

Bearing Capacity and the Limit of Vertical Extension of Residential Building

ETABS software was used to analyse the maximum and minimum load that a G+6 storied building, having shallow foundation, exert on the foundation soil. The maximum and minimu base pressure was found to be 249.92 kPa and 90.98 kPa respectively. Average base pressure was noted to be 200 kPa. From the tests and analyses, it can be concluded that a medium rise building (up to six stories) can be constructed with reinforced shallow foundation on such reclaimed areas.

Settlement Analysis

Consolidation settlement of the shallow foundation on dredged fill soil was estimated considering design bearing capacity to be 300 kPa and due to the surcharge of the filling layer over the soft organic clay. It has been calculated that the total consolidation settlement of structure will be approximately 19mm. This total settlement is below the allowable limits and less likely to cause any structural damage.



(a) (b)

Fig. 7: (a) Load - settlement curve and (b) field load test arrangement

Cost Analysis

As per AASHTO design guideline; for 450kN load the dimensuions of pile for our site would be of 17m long having 0.508m dia (2X Piles together). Volume of concrete for two piles with pile cap is around 9.5m^3; whereas shallow foundation at 1.5m depth with base reinforcement requires around 1m^3 volume of concrete. In terms of cost, two piles with pile cap will cost around 140,000 BDT where reinforced shallow foundation will cost around 65,000 BDT. A comparative analysis of cost for pile foundation and shallow foundation is graphically presented in the Fig. 8.

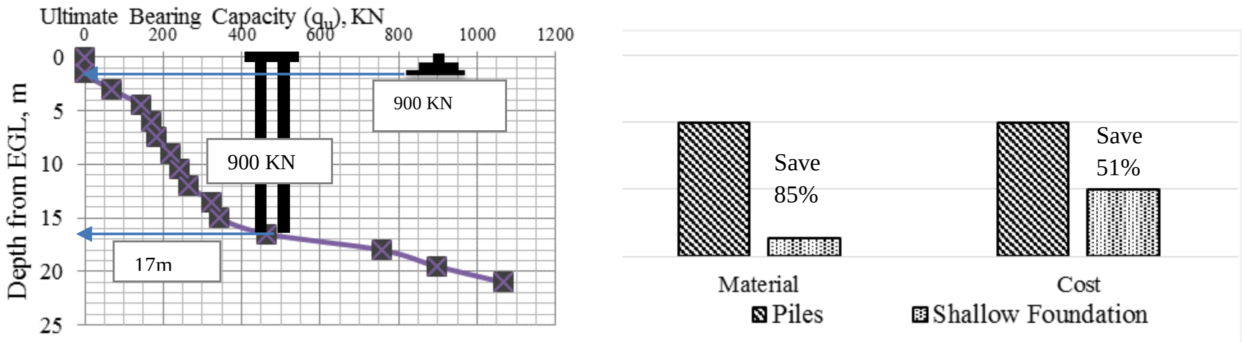


Fig. 8: Comparative analysis – shallow foundation vs. deep (pile) foundation
(a) Dimension (b) Material & Cost

CONCLUSIONS

Reclamation of Dhaka city without proper planning and design has made such lands problematic with respect to large settlement potential, negative skin friction and liquefaction. Current practice of foundation design for buildings in reclaimed areas is generally deep foundation (piles). Pile foundation may be used for high rise building to avoid liquefaction problem. RAPs and Buoyancy Raft foundation can also be used to avoid excessive settlement problem. However, special attention should be taken during foundation design in such soil conditions and field trials has to be done before implementation of such foundation system.

The proposed reinforced shallow foundation may be adopted for low rise buildings (especially up to G+6 stories) on reclaimed land where sub-soil characteristics are identical to that of studied site. Alternately this type of shallow foundation may also be used in conjunction with deep foundation specially for peripheral footing that experience comparatively low base reaction. From the field tests and analyses, it has also been revealed that the proposed shallow foundation is a cost effective foundation alternative for the reclaimed areas of Dhaka city.

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DETERMINATION OF OPTIMUM CEMENT CONTENT FOR STABILIZATION OF SOIL - A CASE STUDY

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ABSTRACT
This study investigates the effect of cement on the performance of soil, collected from Sothern University Bangladesh Campus, Mehedibag, Chittagong. The addition of cement was found to improve the engineering properties of available soil in stabilized forms specifically strength characteristic. Therefore, laboratory tests such as compaction, Atterberg limits, unconfined compressive strength tests for different percentages of cement content and original soil samples were performed. These test results show that the soil can be made lighter which leads to decrease in dry density and increase in moisture content due to the addition of cement with the soil. Besides that the unconfined compressive strength and shear strength of soil can be optimized with the addition of 3% cement content measuring by weight.

Keywords: Cement; unconfined compressive strength; Atterberg limits; index properties; strength properties

INTRODUCTION

Every civil engineering structure, whether it is a building, a bridge, or a dam, is founded on or below the surface of the earth. Foundations are required to transmit the load of the structure to the soil of sufficient strength. For small structure, shallow foundations are provided generally. If the soil of shallow depth is unstable then the soil is to be stabilized. Similarly in case of rigid and flexible pavement, often it is required to stabilize the sub base and base layer. Stabilization is the process of blending and mixing materials with a soil to improve the soil's strength and durability. The pavement soil qualities will be improved by thoroughly mixing and compacting with additives include portland cement, fly ash, bitumen, and combinations of any of the additives [3]. The properties of all the pavement layers are considered in the design of the flexible pavement system [5].The type of the additive and the amount required are dependent upon the soil classification and the degree of improvement desired [4]. Cement has been found to be effective in stabilizing a wide variety of soils, including granular materials, silts, and clays; by products such as slag and fly ash; and waste materials such as pulverized bituminous pavements and crushed concrete. Generally, the stabilization concept can be dated 5000 years ago. Treated earth roads were used in ancient Mesopotamia and Egypt, and that the Greek and Roman used soil-lime mixtures [1]. The first experiments on soil stabilization were achieved in the USA with sand/clay mixtures around 1906. In the 20th century, especially in the thirties, the soil stabilization relevant to road construction was applied in Europe [2].

Cement-modified soil is typically used to improve subgrade soils or to amend local aggregates for use as base in lieu of more costly transported aggregates. The improved engineering characteristics of materials which are treated with cement provide important benefits to Portland cement concrete (rigid) and asphalt (flexible) pavements.

The role of hydraulic cement such as portland or slag cement is to bind soil particles together, improve compaction, and decrease void spacing, improve the engineering properties of available soil such as, unconfined compressive strength, modulus of elasticity, compressibility, permeability, the drying rate, workability, swelling potential, frost susceptibility and sensitivity to changes in moisture content (Leonards, 1962; Woods, 1960; Robert et al., 1971). Cement can be used to stabilize any type of soil, without those having organic content greater than 2% or having pH lower than 5.3 (ACI 230.1R-90 1990).

The objectives of the study are given below:

- To classify the soil according to MIT.
- Correlate maximum dry density and optimum moisture content with various % of cement in soil.
- To show the effect of cement on unconfined compressive strength of soil and determine the optimum cement content.

Materials and Methods

Specification of soil

The collected soils were hard and it was pulverized manually by hammer. Then the soils were screened through the sieve of 4.75 mm aperture before preparing the specimens for testing. According to the MIT classification systems, the soil is classified as silty sand.

The particle size [redacted]

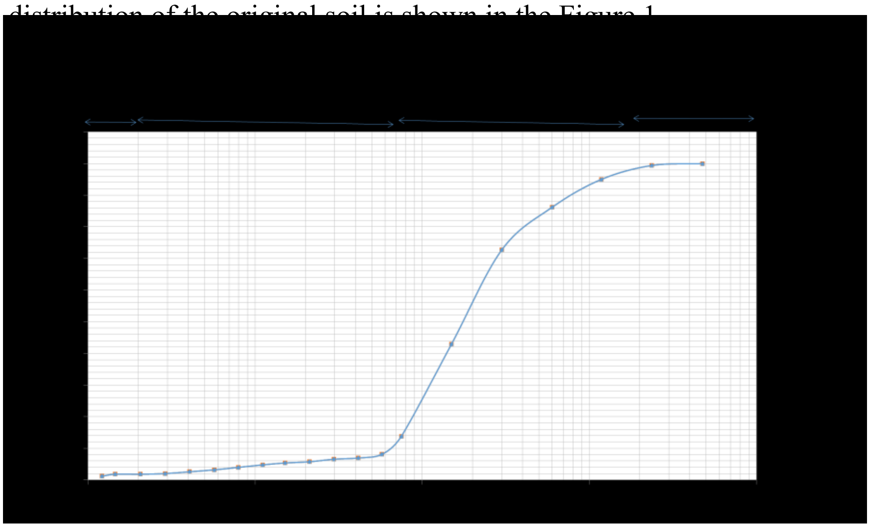


Fig. 1: Particle size distribution of original soil

Preparation of Testing Samples

The collected soils and cement contents were oven-dried at 105 ^{0}c overnight to remove moisture and repress microbial activity. Then the oven dried samples were mixed thoroughly by hand in a large tray in a dry state.

- The index properties of the collected soil samples are determined.
- Soil is classified according to MIT soil classification system.
- Laboratory tests standard proctor test, Unconfined compression test are done for only soil sample and soil mixed with 3%, 6%, 9%, 12% cement respectively.
- Optimum moisture content obtained from standard proctor test is used preparing the test specimen of unconfined compression test. The sample for unconfined compression test is cured for 1 day, 7days, 14 days and 28 days.
- Finally the unconfined compressive strength variation of soil for different curing period is analysed and an optimum value of cement content is determined.

RESULTS

From the grain size analysis, specific gravity test and Atterberg limit test, following properties of soil are found. Figure 2 shows that liquid limit of soil increases gradually with the increases in percentage of cement content. This improvement of liquid limit attributed that more water is required for the cement treated soil to make it fluid. This change of atterberg limit is due to the cation exchange reaction and flocculation–aggregation for presence of more amount of cement, which reduces plasticity index of soil.

Table 1: Index Properties of Soil

Properties	**Values**	Properties	Values
MIT classification	Silty sand	Coefficient of curvature (Cc)	0.873
Specific Gravity (G)	2.75	Liquid limit (wl)	16.6%
Effective size (D10)	.063 mm	Plastic limit (wp)	15.87%
Fineness modulus	1.0368	Plasticity index (Ip)	0.73%
Uniformity coefficient(Cu)	3.492		

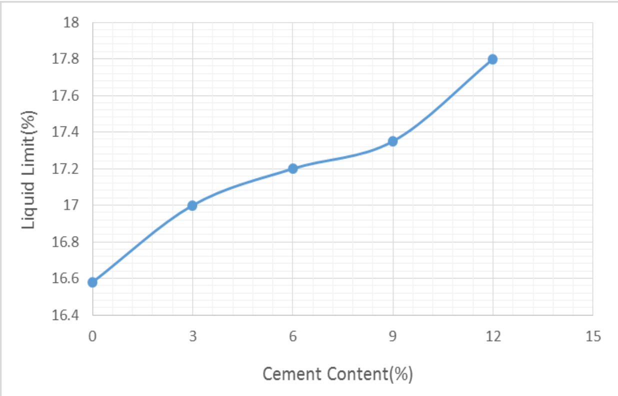


Fig 2: Variation of liquid limit with cement content

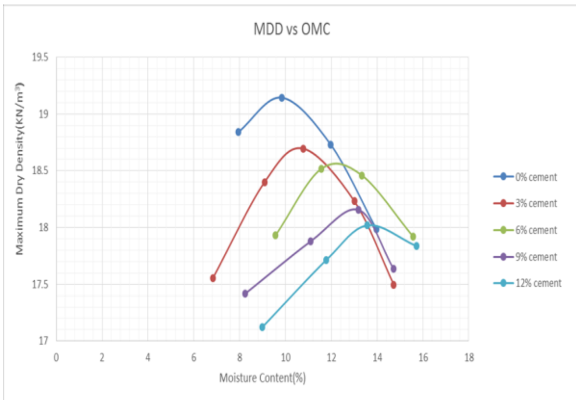
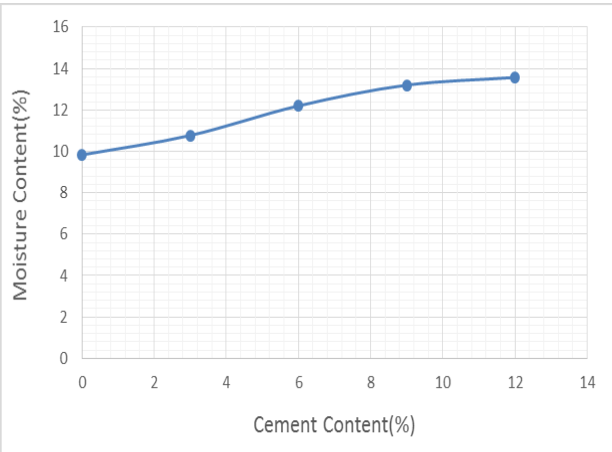


Fig 3: Correlation between Maximum Dry Density and Optimum Moisture Content with various % of cement in soil



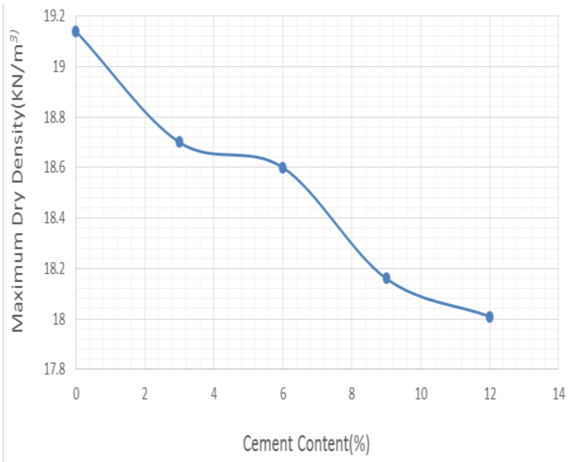


Fig 4: Variation of OMC with cement content

Fig 5: Variation of MDD with cement content

The variation of optimum moisture content and maximu
soil is shown in figure 3. This figure represents the maximum dry density of soil decreases gradually with an increase of cement content. It is the result of initial coating of soils by cement to form larger aggregate, which consequently occupy larger spaces.

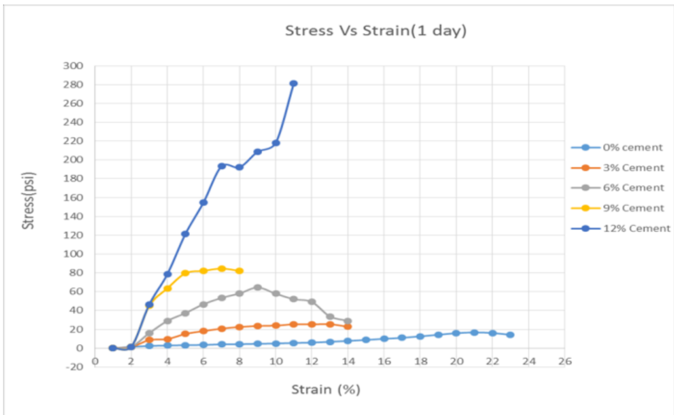
On the other hand, the optimum moisture content of soil increases with increase cement content, because cement is finer than the soil. The increase of water content was also attributed by the pozzalanic reaction of cement with the soil.

Strength Characteristics

Unconfined Compressive Strength (UCS)

The test result of unconfined compressive strength is shown in Figure 5. This figure illustrates the stress-strain behavior of original and cement treated soil under vertical load. Initially the stress is rapidly increases with the increase of strain but in case of soaking untreated soil sample, stress increases gradually with the increase of strain. After attaining the peak stress, it decreases with the increase of strain for all the combination of cement and soil. Approximately all the specimen shows shear failure after observing the failure plane of specimens.

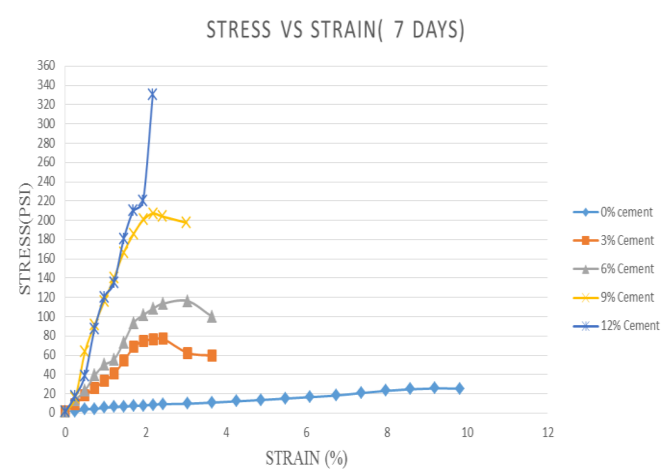
- Fig: 6(a) shows the variation of compressive strength of soil for different percentages of cement for curing period 1 day.



Cement content(%)	Unconfined Compressive strength(psi)	% increase in UCS
0	14.259	
3	22.749	59.54
6	29.093	104
9	70.054	391.3
12	281.139	1871.66

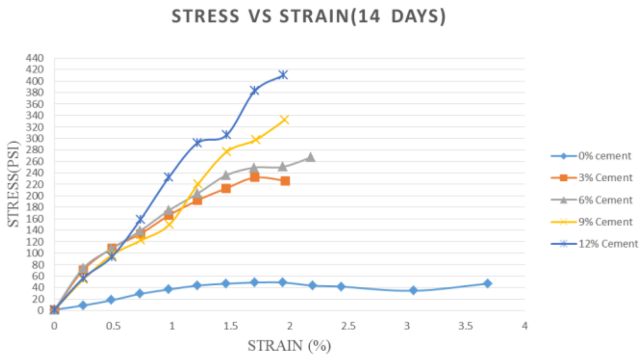
Fig: 6(a)

- Fig: 6(b) shows the variation of compressive strength of soil for different percentages of cement for curing period 7 days.



Cement content(%)	Unconfined Compressive strength(psi)	% increase in UCS
0	24.252	
3	59.595	145.73
6	100.159	68
9	197.515	97.20
12	330.598	67.37

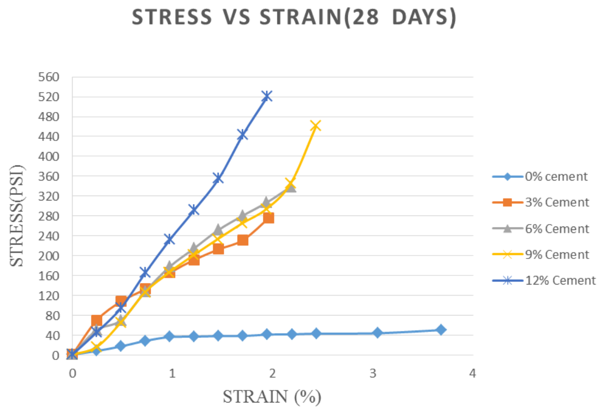
- Fig: 6(c) shows the variation of compressive strength of soil for different percentages of cement for curing period 14 days.



Cement content(%)	Unconfined Compressive strength(psi)	% increase in UCS
0	46.84	
3	226.14	382.79
6	267.24	18.17
9	331.90	24.19
12	410.31	23.62

Fig: 6(c)

- Fig: 6(d) shows the variation of compressive strength of soil for different percentages of cement for curing period 28 days



Cement content(%)	Unconfined Compressive strength(psi)	% increase in UCS
0	50.84	
3	276.14	443.15
6	339.09	22.79
9	460.9	35.92
12	520.31	12.88

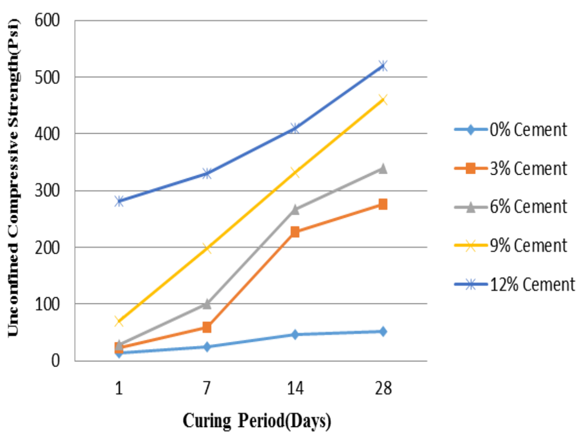


Fig 7: Variation of Unconfined Compressive Strength at different percentages of cement for different curing periods.

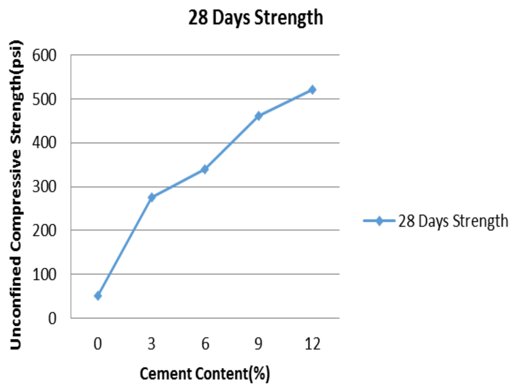


Fig 8: Unconfined Compressive Strength variation with cement content

From Fig: 7 it is observed that strength increases rapidly at 3% cement content where strength does not increase rapidly for other percentages of cement. Again fig:8 also shows the shows the same pattern.

CONCLUSIONS

A study has been conducted to investigate the fundamental properties such as consistency, compaction, compressive strength characteristics of untreated and cement treated soil. It can be concluded that there is an improvement of geotechnical properties of cement treated soil. The following conclusions, based on the test results in this study, are drawn.

- According to MIT classification system, this soil is silty sand.
- The maximum dry density of soil decreased with the addition of cement and value of optimum moisture content of cement treated soil increased because of the pozzalonic action of cement and soil, which needs more water.
- Comparing with the 28 days unconfined compressive strength, strength increases 5.43 times for using 3% cement, 6.67 times for 6% cement, 9.65 times for 9% cement and 10.4 times for 12% cement.
- From the economic consideration and the strength required in field, the optimum cement content is 3% as strength increases rapidly for adding 3% cement compared to other percentages of cement content.

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STRENGTH INCREMENT OF SAND COLUMN BY USING ADMIXTURES

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ABSTRACT
Fly ash is a coal combustion by-product. It is harmful for environment and can be classified as a hazardous waste. But it has cementious behaviour and may be useful for construction projects, engineering purposes. It is also helpful in solving the problems related to the geotechnical aspects under the process of ground improvement. By unconfined compressive strength test it is observed that the strength of a sand column increases when fly ash is used as one of the admixtures. The unconfined compressive strength test has been done using fly ash, cement and lime as admixtures for sand column considering different curing periods. With various proportions of admixtures the samples were prepared and the experiment was conducted. From the results obtained, it has been found that the percentage of fly ash increases the strength of sand column with respect to other admixtures.

Keywords: Fly ash; unconfined compressive strength; ground improvement

INTRODUCTION
Sustainable development is a vital concept to be considered in substitution of ground improvement methods in conventional construction practices. Inter-dependence of economy and environment factors are significantly important in regards to formulate policies and legislations in global aspect decisions, especially in terms of projects that impose massive impact on environment. It is imperative to study concurrent status of construction site prior to geotechnical design, especially in cases involving larger infrastructures. Most of construction practices use raw materials instead of recycling the waste materials. Waste production and increase of waste become worst as the wastes are harmful to the environment and human health.
Fly ash has been classified as hazardous Coal Combustion by-Product (CCP). This huge value of production is extremely dangerous if not dealt with right methods. As a result, many researchers have started to look into appropriate methods to be used and applied to the fly ash, reported that fly ash can be successfully used to improve the geotechnical parameters such as bearing capacity and shear strength. From research, it has been found that the use of fly ash can increase the potential for implementation as road sub-base for light to medium traffic. Through such research, it is stated that fly ash is a supplement that can be used to increase the strength, improvement of bearing capacity of soil and at the same time to solve the problems of settlement.

METHODOLOGY
Fly ash (FA), cement, lime and sand is used for the experimental investigations. FA is a solid waste from the combustion of coal with a high temperature (about 1000^0C) in coal power stations. For this study, the source of the FA has been taken from Barapukuria coal mine, Dinazpur, Bangladesh. Portland cement has been used based on the availability of this product. This type of cement has an ideal ratio of material properties needed for this study. Sand particles passing through 4.75 mm sieve were used to mixing with other materials. The optimum moisture content of sand is 14.7% with maximum dry density of 1.79 gm/ml. The classification of sand used is Well Graded Sand.

A mould of 4 inch diameter and 4.5 inch height is taken with collar attached to it. The sample prepared by mixing 20% water of its weight is placed in the mould in 3 layers and compacted by hammer with 25 blows at each layer. The collar is removed and the surface is smoothed by a sharp knife. The mould is placed on a sample ejector and the sample is ejected from it.

The sample is placed in the trimmer to make the size of the sample to the desired shape of 4 inch height and 2 inch diameter. The sample is now covered with plastic membrane and allowed for air curing. The curing is done for the samples maintaining the duration of 7, 14, 21, 28 days according to the contents of admixtures. After curing the sample is placed on the bottom plate of the compression machine and adjusts it with the upper plate for proper contact. Now both the dial gauge and proving ring gauge to zero is adjusted. Compression load is applied to cause an axial strain at rate of 0.5 to 2% per minute. The test is continued until the failure surface is clearly developed.

PROPORTION OF SAMPLE PREPARATION WITH 20% WATER MIXING

1. 40% fly ash, 5% cement, 5% lime and 50% sand mixture.
2. 40% fly ash, 7.5% cement, 7.5% lime and 45% sand mixture.
3. 40% fly ash, 10% cement and 50% sand mixture.
4. 40% fly ash, 15% cement and 45% sand mixture.
5. 50% fly ash, 5% cement, 5% lime and 40% sand mixture.
6. 50% fly ash, 7.5% cement, 7.5% lime and 35% sand mixture.
7. 50% fly ash, 10% cement and 40% sand mixture.
8. 50% fly ash, 15% cement and 35% sand mixture.

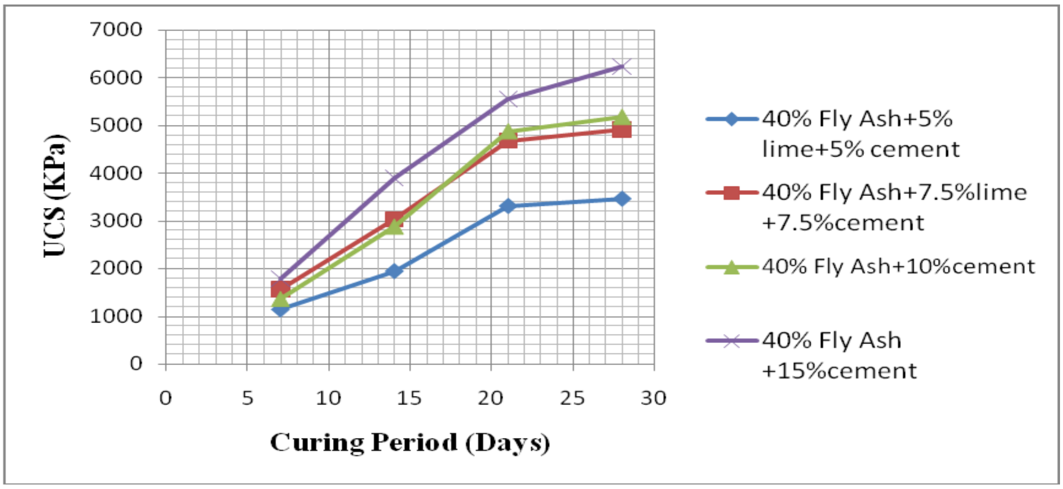

Fig-1: Unconfined Compressive_Strength vs. curing time curve for (5, 7.5, 0, 0) % lime and (5, 7.5, 10, 15) % cement with 40% fly ash

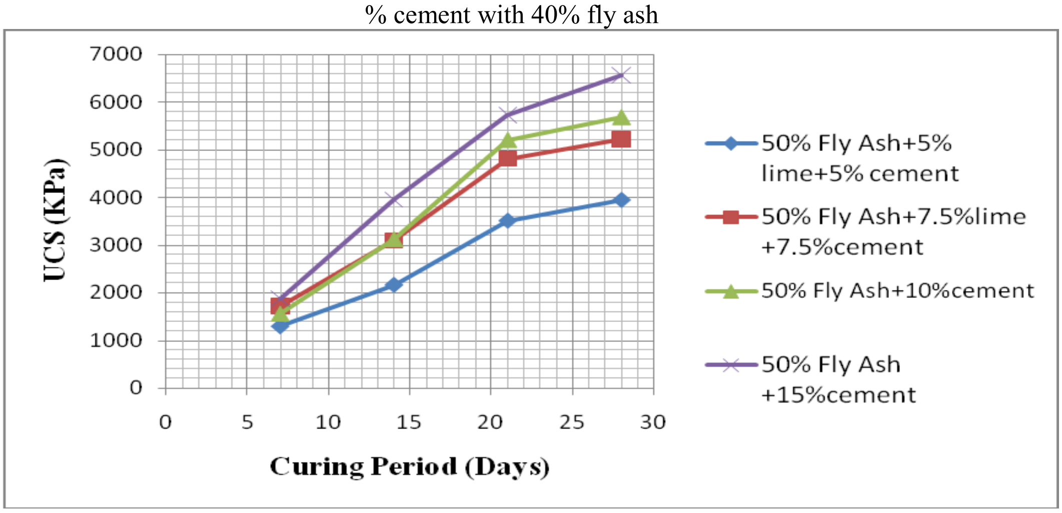

Fig-2: Unconfined Compressive Strength vs. curing time curve for (5, 7.5, 0, 0) % lime and (5, 7.5, 10, 15) % cement with 50% fly ash

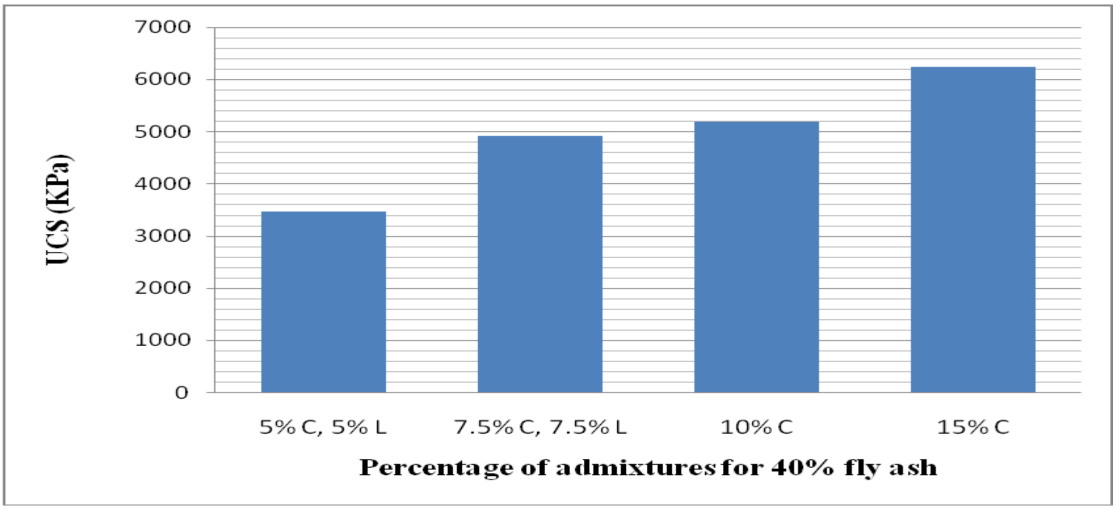

Fig-3: Percentage of admixtures used with 40% fly ash vs. unconfined compressive strength for 28 days

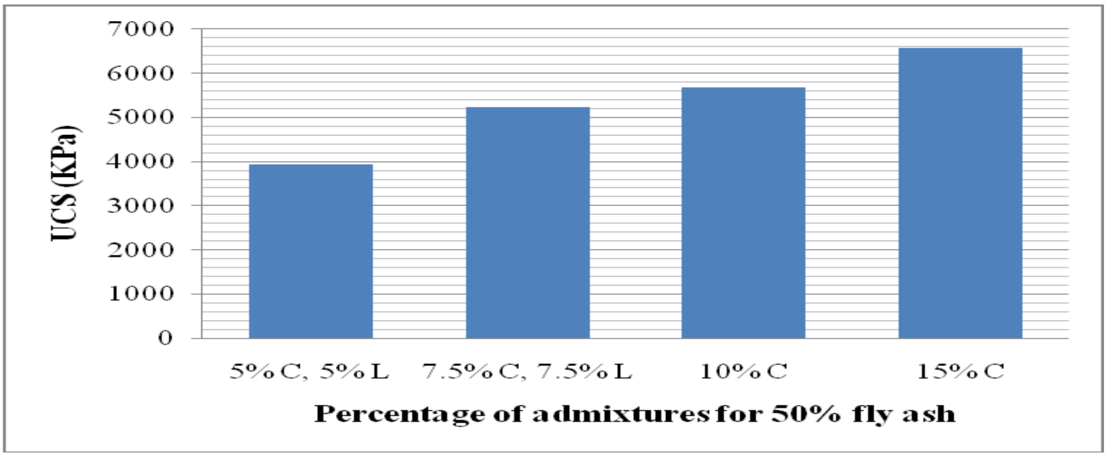

Fig-4: Percentage of admixtures used with 50% fly ash vs. unconfined compressive strength for 28 days.

Table-1: Comparison among unconfined compressive strength of sand column with different proportions of admixtures

Fly Ash (%)	Cement (%)	Sand (%)	Lime (%)	Unconfined Compressive strength (kPa) at different curing periods			
				7 days	14 days	21 days	28 days
40%	5	50	5	1148.00	1950.80	3316.36	3468.09
	7.5	45	7.5	1560.64	3034.58	4681.39	4920.03
	10	50	-	1365.60	2882.00	4877.00	5180.46
	15	45	-	1777.39	3901.60	5548.94	6242.56
50%	5	40	5	1300.53	2167.56	3511.44	3944.96
	7.5	35	7.5	1717.00	3099.61	4811.98	5223.81
	10	40	-	1560.60	3121.28	5202.14	5679.00
	15	35	-	1864.10	3953.63	5722.35	6567.70

RESULTS AND DISCUSSIONS

From the samples made for unconfined compressive strength test, it is observed that the strength gained in 7 days with different admixtures used do not differ much. In fact they are within the range of 1000 kPa to 2000 kPa. But strength increases much at 14 days curing time. The strength gained by 5% cement, 5% lime is less than the others. The strength for 10% cement alone and the combination of cement and lime (7.5%cement+ 7.5% lime) are very close to each other at 14 days curing. The strength gained by 15% cement sample is higher than the other proportions. At 21 days curing, the strength increases almost at the same rate as 14 days curing. From the graph it is seen that the slopes of the curves are almost same. From the Fig.1&2, it is seen that the rate of increment of strength from 21 days curing to 28 days curing is smaller for three proportions except for 15% cement.
Comparing the strength level of the samples of 40% and 50% fly ash admixtures, it is observed that the strength of the sample containing 50% fly ash is higher than that of 40% fly ash. This happens for every proportion of admixtures used for all curing periods (Table 1). For example, the strength of 15% cement sample with 50% fly ash is 4.95% higher than that of 40% fly ash at 28 days curing time (Fig.3 & 4). Hence it can be said that by increasing the amount of fly ash in the sand column the strength of it can be increased.

CONCLUSIONS

Sand column is a method to increase the strength of soft ground. By using fly ash as one of the materials in the sand column, it is useful not only in the geotechnical engineering, but also in the environmental issues. Fly ash and cement are the materials that have cementitous behaviour. This mixture process will continue under the chemical reactions of hydration and pozzolanic. It will harden and the void can be strengthened by unconfined compressive strength (Shakri S.et. al., 2013).
From the present study, it has been observed that by increasing the amount of the fly ash without changing the percentages of other admixtures, the strength of sand column is increased. As fly ash is a waste material and is harmful for environment, its proper use should be made. Use of fly ash in construction purposes increases the strength of the infrastructure and thus save the environment from pollution.

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STABILIZATION OF ORGANIC SOIL USING LIME ADDED SALT (NaCl)

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ABSTRACT
Lime stabilization is known to be an effective stabilization method for soil. However for organic soil it becomes less effective due to low increment in strength. Therefore salt is used to accelerate lime-organic soil reactions. Salt is introduced to remove the barrier in order to accelerate as well as help lime to increase the strength of organic soil. Hydrated lime & salt of Sodium (NaCl) is used here for the stabilization of organic soil. This study focused on the strength characteristic of the organic soil by using unconfined compression test. The soil samples have been collected from Koiya Bazar, Botiaghata, Khulna near Abul Kasem Degree College, Bangladesh. Various proportions of lime with addition of various proportions of sodium chloride (NaCl) are examined for organic soil stabilization. Results obtained are compared among the three different mixtures of organic soil, lime & salt. Hydrated lime is used in this research since it is not too exothermic and harmful to the skin compared to quicklime. On 36 remolded samples (38mm x 80mm),the percentage of lime used are 3%, 6% and 12% & variable concentration of sodium chloride (NaCl) as 2.5%, 5% and 10% at the curing period of 0,7,14, and 28 days have been used to observed the strength increment of organic soil. Here proportion of 10% NaCl & 3% lime shows best result among these three combinations.

Keywords: Lime stabilization; organic soil; sodium chloride (nacl); hydrated lime; unconfined compression test; strength increment

INTRODUCTION

Most of the problem encountered by geotechnical engineers at construction site is the properties of material are unable to reach the required specification. These problems normally face by soft soil such as organic clay which has low un-drained shear strength and low bearing capacity. This result influenced by some organic matter which consists of Humic acid more than 2%. Organic matter acts as 'masking' in which it coats the primary source of organic clay minerals (silica and alumina) causing the obstruction when lime is used as well as reducing the effectiveness of lime stabilization & also affects the pozzolanic reaction in stabilization process. Even though many research done proves that lime can be used as a method of ground improvement, but the significant increase in soil strength is still lower due to a reduction in compacted dry unit weight of clay soil (B. Dan Marks &T. Allen Halliburton, 1972).Which indicates that by using salts as an additive in lime, the strength of organic soils may be increased much better compared to the use of lime alone. The main objective of this research is to investigate the effectiveness of lime – salt mixture in stabilization of organic soil & to determine the percentage of strength increment in organic soil obtained from different proportions of lime (Hydrated lime) & salt (NaCl).

METHODOLOGY

To determine the physical & engineering properties of organic soil different laboratory tests have been performed like as Natural water content, Specific gravity, Particle size distribution, Atterberg limit, Standard proctor test, Consolidation test, Compaction Test etc. Different salt percentages (2.5%, 5%, and 10%) mix with different percentages of lime (3%, 6%, and 12%) and unconfined compression test was performed for these soil samples at the curing period 0, 7, 14, 28 days respectively.

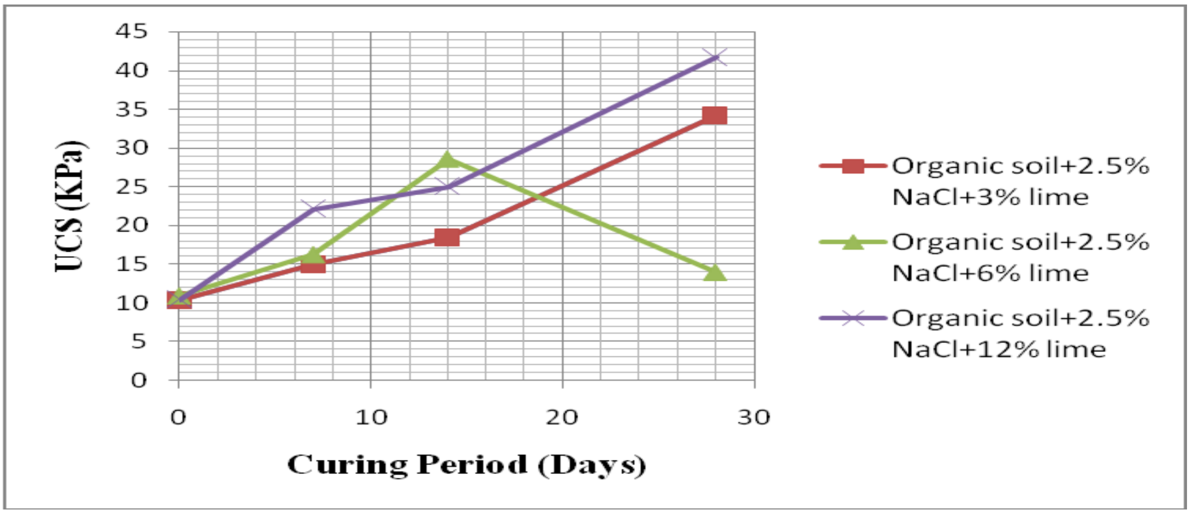

Fig. 1: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with 2.5% NaCl + (3, 6, 12%) lime

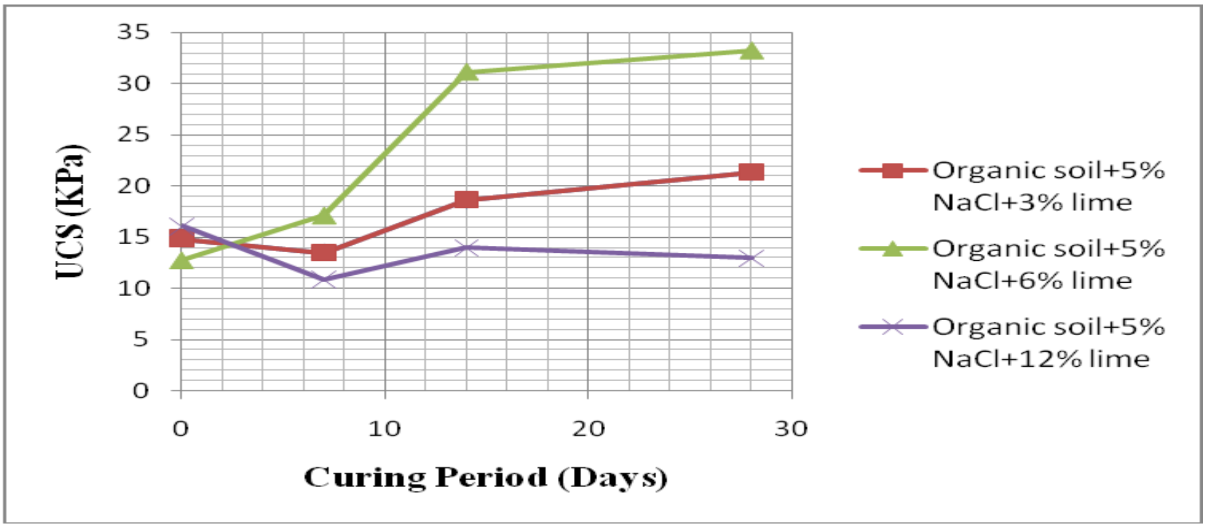

Fig. 2: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with 5% NaCl + (3, 6, 12%) lime

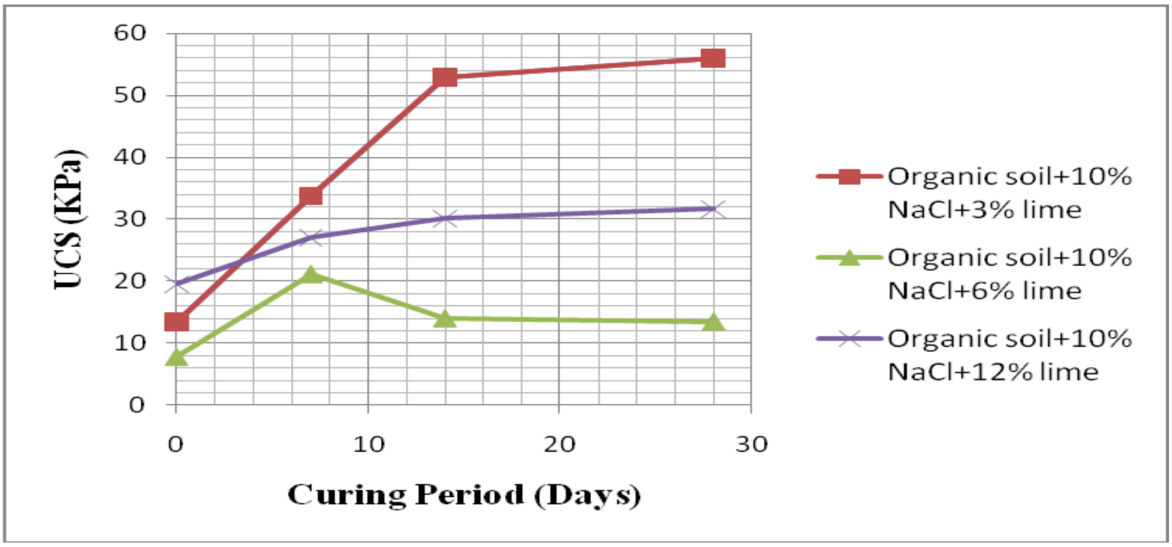

Fig. 3: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with 10% NaCl + (3, 6, 12%) lime

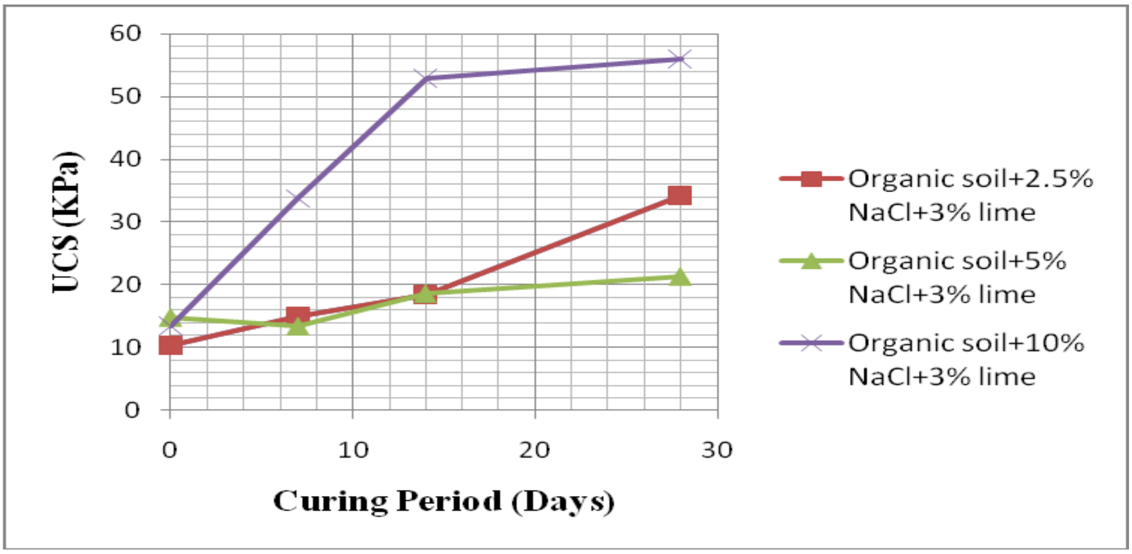

Fig. 4: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with (2.5, 5, 10%) NaCl + (3%) lime

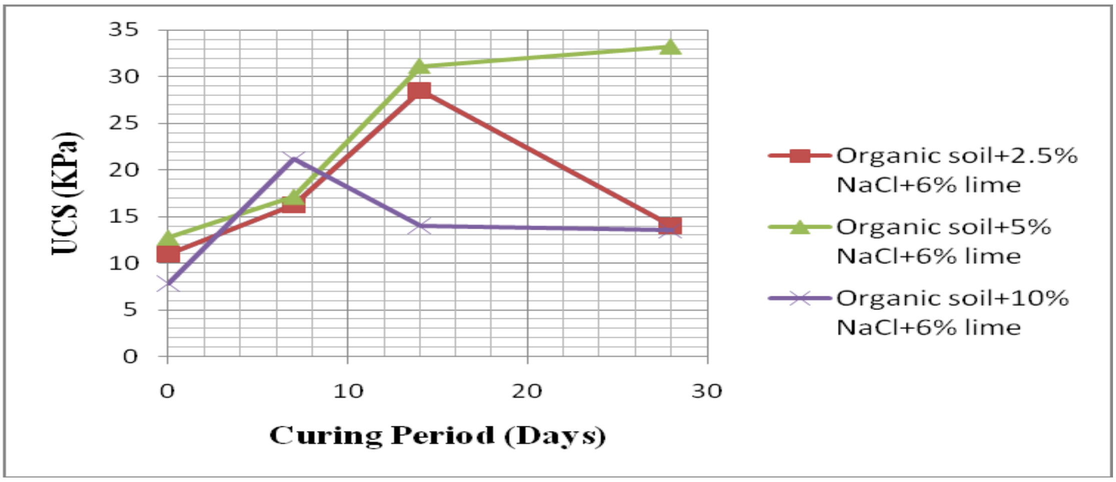

Fig. 5: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with (2.5, 5, 10%) NaCl + (6%) lime

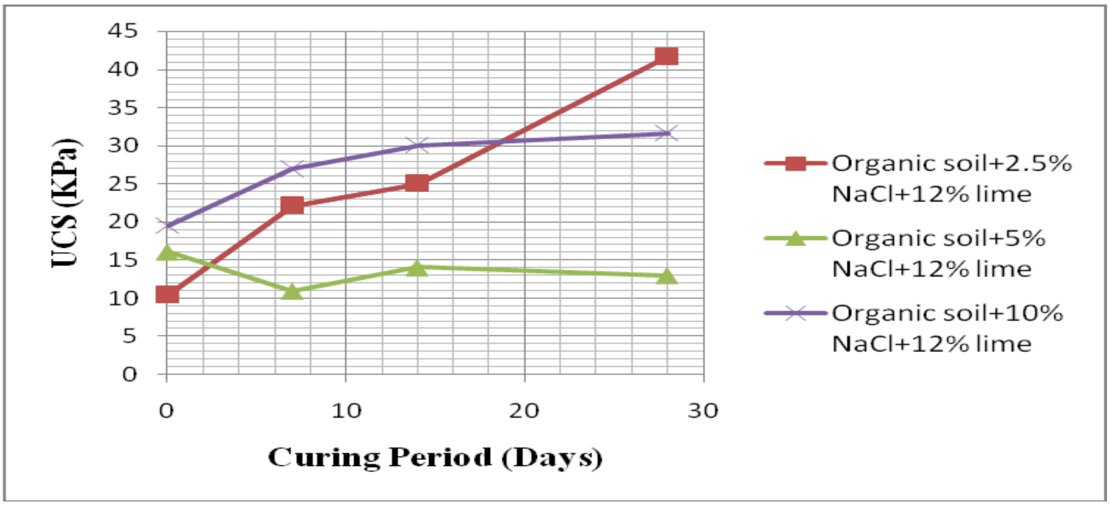

Fig. 6: Curing period vs. Unconfined Compressive Strength Curve for Organic Soil with (2.5, 5, 10%) NaCl + (12%) lime

Table 1: Summary Result of Unconfined Compressive Strength (UCS)

SAMPLE	UCS (psi) 0 day	UCS (psi) 7 days	UCS (psi) 14 days	UCS (psi) 28 days
Organic soil+2.5%NaCl+3%lime	10.38	15.05	18.47	34.26
Organic soil+2.5%NaCl+6%lime	10.97	16.27	28.55	14.02
Organic soil+2.5%NaCl+12%lime	10.44	22.16	25.06	41.75
Organic soil+5%NaCl+3%lime	14.89	13.49	18.65	21.37
Organic soil+5%NaCl+6%lime	12.77	17.13	31.14	33.22
Organic soil+5%NaCl+12%lime	16.1	10.89	14.02	12.98
Organic soil+10%NaCl+3%lime	13.35	33.74	52.94	56.05
Organic soil+10%NaCl+6%lime	7.79	21.16	14.02	13.49
Organic soil+10%NaCl+12%lime	19.51	26.99	30.1	31.66

RESULTS AND DISCUSSIONS

The present work studied the strength characteristics of organic soil using admixtures. The admixture used here lime & sodium chloride. The percentage of lime used are 3%,6% and 12% &variable concentration of sodium chloride (NaCl) as 2.5%,5% and 10% at the curing periods of 0,7,14 and 28 days have been used to observe the strength increment of organic soil. The soil has been collected from Koiya Bazar, Khulna from 10 feet depth. Its specific gravity & liquid limit are found to be 1.611 & 104% respectively. From the grain size distribution curve, the particle size of organic soil lies between (0.01- 0.4) mm.

For 2.5% NaCl concentration, initially the strength of soil is same for all the lime percentage used (Fig.1). Soil strength at 7 days is same for 3% & 6% lime but higher for 12% lime. At 14 days soil strength is found to be lowest for 3% lime & highest for 6% lime. At 28 days soil strength is higher for 12% lime & lower for 6% lime content. Here 2.5% NaCl & 12% lime combination shows higher values of strength among these three combinations considering curing period.

For 5% NaCl concentration, initially the strength of soil is same for all the lime percentage used (Fig.2). Soil strength at 7 days is found to be lowest for 12% lime but highest for 6% lime. At 14 days soil strength is lower for 12% lime & higher for 6% lime. At 28 days soil strength is higher for 6% lime & lower for 12% lime content. Here 5% NaCl & 6% lime combination shows higher values of strength among these three combinations considering curing period.

For 10% NaCl concentration, initially the strength of soil is higher for 12% than other percentages (Fig.3). Soil strength at 7 days is lower for 6% lime but higher for 3% lime. At 14 days soil strength is lower for 6% lime & higher for 3% lime. At 28 days soil strength is higher for 3% lime & lower for 6% lime content. Here 10% NaCl & 3% lime combination shows best result among these three combinations.

Among these combination when sodium chloride percentages are fixed then 10% NaCl & 3% lime combination shows higher values of soil strength. It is observed that with increase in percentage of NaCl causes reduce in percentage of lime to get higher strength.

For 3% lime concentration, initially the strength of soil is same for all the NaCl percentage used (Fig.4). Soil strength at 7 days is same for 2.5% &5% NaCl but higher for 10% NaCl. At 14 days soil strength is same for 2.5%&5% NaCl & higher for 10% NaCl. At 28 days soil strength is higher for 10% NaCl & lower for 5% NaCl. Here 3% lime & 10%NaCl combination shows higher values of strength among these three combinations considering curing period.

For 6% lime concentration, initially the strength of soil is higher for 5% NaCl than other two percentages (Fig.5). Soil strength at 7 days is same for 2.5% & 5% NaCl but higher for 10% NaCl. At 14 days soil strength is lower for 10% NaCl & higher for 5% NaCl. At 28 days soil strength is same for 10% & 2.5% NaCl & higher for 5% NaCl. Here 6% lime & 5% NaCl combination shows higher values of strength among these three combinations considering curing period.

For 12% lime concentration, initially the strength of soil is higher for 10% NaCl than other two percentages (Fig.6). Soil strength at 7 days is lower for 5% NaCl but higher for 10% NaCl. At 14 days soil strength is lower for 5% NaCl & higher for 10% NaCl. At 28 days soil strength is lower for 5% NaCl & higher for 2.5% NaCl. Here 12% lime & 2.5% NaCl combination shows higher values of strength considering curing period. Here also found that for higher percentage of lime reduces percentage of NaCl is required to get higher strength (Table 1).

CONCLUSIONS

Among these combinations when lime percentages are fixed then 10% NaCl & 3% lime combination shows higher values of soil strength. From the test result, it is observed that the strength is generally increases with curing period irrespective of the percentage of admixture used except for two cases (5% NaCl +12% lime +organic soil) & (10% NaCl +6% lime +organic soil).
The lowest strength is observed for 10 % NaCl+6% lime +soil & the highest value for 10%NaCl +12% lime at zero days of curing. For 7, 14, 28days curing , the corresponding strength shows minimum values for 5% NaCl +12% lime +soil which are 10.89 , 14.02 , 12.98 psi respectively. For the same curing period, 10% NaCl +3% lime soil shows the highest value of strength considering all the proportions. The increment of strength for 7, 14, 28 days are 209.83%, 277.61% &331.82% respectively. Higher percentage of NaCl with less percentage of lime mixed with organic soil gives higher strength than that of higher percentage of lime with less percentage of NaCl mixed with organic soil. Thus it can be concluded that NaCl may be used with lime to increase the strength of organic soil.

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A DETAILED ANALYSIS OF SLOPE STABILITY USING FINITE ELEMENT METHOD (FEM)

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ABSTRACT

Instability related issues in engineered as well as natural slopes are common challenges to both researchers and professionals. This paper mainly focuses on the analysis of some simple soil slopes using finite element method (FEM). This is not a very new concept, but not so practiced in the field of geotechnical engineering. The finite element method overcomes the assumptions in other analysis methods and gives more accurate result. In this paper, we use PLAXIS which is a finite element based software developed for the evaluation of the slope stability under various scenario. The stability of slope has been analysed for four types of soil which are not homogeneous and then evaluated the safety factors for varying slope heights. Then the same analysis has been done for the same slopes comprising homogeneous soil and compares the results. The surcharge load is another important factor for the slope stability. A stable slope can sustain certain amount of surcharge load until the safety factor reaches its minimum value. Higher the safety factors, higher the load carrying capacity of the slope.

Keywords: Slope stability; Finite element method; factor of safety; PLAXIS

INTRODUCTION

In construction areas, instability may results due to rainfall, increase in groundwater table and change in stress conditions. Similarly, natural slopes that have been stable for many years may suddenly fail due to changes in geometry, external forces and loss of shear strength (Abramson et al., 2002).The instability of a slope is an on going concern in most construction and infrastructure projects. The engineering solutions to slope instability problems require good understanding of analytical methods, investigative tools and stabilization measures (Abramson et al., 2002). According to (Nash, 1987), a quantitative assessment of the safety factor is important when decisions are made. Likewise, the primary aim of slope stability analysis is to contribute to the safe and economic design of excavation, embankments and earth dams (Chowdhury, 1978). To deal with these slope stability issues various approaches have been adopted and developed over the years. Finite element method has been increasingly used in slope stability analysis. The advantage of a finite element approach in the analysis of slope stability problems over traditional limit equilibrium methods is that no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions. The method can be applied with complex slope configurations and soil deposits in two or three dimensions to model virtually all types of mechanisms. The approaches now have been more of computational rather than the manual. There are a number of software packages that have been developed for geotechnical stability analysis which utilise the FEM. With the advancement in technology software packages utilising the FE methods have increased in popularity as they tend to possess a wider range of features (Hammouri et al., 2008).

Objectives of the study

The objectives of slope stability analyses are-

i) To determine the factor of safety for slopes of different heights.
ii) To assess the maximum surcharge load that can be carried by the stable slopes.
iii) To analyse the suitability of re-fill soil against the existing soils.

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PHI-C REDUCTION

Generally, there are two approaches to analyse slope stability using finite element method. One approach is to increase the gravity load and the second approach is to reduce the strength characteristics of the soil mass. The second approach is adopted in this study by using a powerful software finite element program called PLAXIS. In the *Phi-c reduction* approach the strength parameters $\tan\Phi$ and c of the soil are successively reduced until failure of the structure occurs. The strength of structural objects like plates and anchors is not influenced by *Phi-c reduction*. When using *Phi-c reduction* in combination with advanced soil models, these models will actually behave as a standard Mohr-Coulomb model. The Phi-c reduction approach resembles the methods of calculating safety factors as conventionally adopted in slip-circle analyses. The mathematical expression of this model, as well known, is given by the following formula:

$$\tau = \sigma_n \tan\Phi + C \tag{1}$$

where: τ = shear strength of soil material on a certain failure plane, σ_n = normal stress on the failure plane, Φ = angle of internal friction of soil material, and C = cohesion intercept of soil material.
The shear strength of the sliding surface is denoted by τ_f and expressed as follows:

$$\tau_f = \sigma_n \tan\Phi_f + C_f \tag{2}$$

where, C_f and Φ_f are the factored shear strength parameters and they can be given as follows:

$$C_f = C / SRF \tag{3}$$

$$\Phi_f = \Phi_f / SRF \tag{4}$$

For Mohr-Coulomb material model, six material properties are required. These properties are the friction angle φ, cohesion C, dilation angle ψ, Young's modulus E, Poisson's ratio ν and unit weight of soil γ. Young's modulus and Poisson's ratio have a profound influence on the computed deformations prior to slope failure, but they have little influence on the predicted factor of safety in slope stability analysis.

Dilation angle, ψ affects directly the volume change during soil yielding. If $\psi = \varphi$, the plasticity flow rule is known as "associated", and if $\psi \neq \varphi$, the plasticity flow rule is considered as "no associated". The change in the volume during the failure is not considered in this study and therefore the dilation angle is taken as 0. Therefore, only three parameters (friction angle, cohesion and unit weight of material) of the model material are considered in the modelling of slope failure.

GEOMETRIC MODEL OF SLOPE

Fig. 1 shows the generalized profile of slope used in the study. The height of the slope varies as 3m, 4m, 5m and 6m. The model slopes contain both heterogeneous and homogeneous soil. The soil properties were obtained from soil tests. Table 1 gives the soil parameters for the geometric models. To calculate the global safety factor for the slopes first the geometric profile of slope is incorporated in PLAXIS. The soil properties are also incorporated according to the soil parameters in Table 1.Then in the *initial conditions* option phreatic level is drawn. In this study, the pore water pressure is not accounted. Then the mesh was generated. In this study 15-node triangular elements were used. The powerful 15-node element provides an accurate calculation of stresses and failure loads. The two vertical boundaries are free to move, whereas the horizontal boundary is considered to be fixed. A mesh generated slope is shown in Fig. 2.

Table 1 Soil parameters used in this study

Soil type	Depth in meter	γ_{unsat} (kN/m^3)	Saturated unit weight,γ_{sat} (kN/m^3)	Friction angle,φ (Degree)	Poisson's ratio,ν	Elastic modulus,E (kN/m^2)	Cohesion, c (kN/m^2)
Soil 1	0.0 to 1.0	13.73	17.76	16.5	0.31	170	2.0
	1.0 to 2.0			15.1	0.33	340	2.0
	2.0 to 3.0			14.8	0.30	1360	0.9
	3.0 to 4.0			18.9	0.31	2210	2.0
	4.0 to 5.0			20.6	0.33	2550	2.0
	5.0 to 6.0			22.3	0.32	2720	2.0
	6.0 to 9.0			16.5	0.31	3640	1.9
	9.0 to 12.0			15.7	0.30	3750	1.0
Soil 2	0.0 to 1.0	13.54	17.95	13.5	0.30	340	1.9
	1.0 to 2.0			14.1	0.33	170	2.0
	2.0 to 3.0			15.8	0.33	680	2.0
	3.0 to 4.0			18.7	0.34	1530	1.9
	4.0 to 5.0			17.6	0.35	1870	0.9
	5.0 to 6.0			19.2	0.32	2210	2.0
	6.0 to 9.0			18.0	0.33	5123	2.0
	9.0 to 12.0			16.5	0.34	2730	1.0
Soil 3	0.0 to 1.0	12.75	17.23	16.7	0.33	170	1.0
	1.0 to 2.0			17.8	0.33	340	2.0
	2.0 to 3.0			21.7	0.34	340	1.9
	3.0 to 4.0			22.8	0.34	850	2.0
	4.0 to 5.0			18.9	0.31	1190	2.0
	5.0 to 6.0			21.9	0.32	3740	2.0
	6.0 to 9.0			22.7	0.31	6200	1.9
	9.0 to 12.0			23.5	0.34	2550	2.0
Soil 4	0.0 to 1.0	13.44	18.15	13.8	0.31	680	1.9
	1.0 to 2.0			14.8	0.31	680	2.0
	2.0 to 3.0			18.6	0.32	170	1.0
	3.0 to 4.0			20.3	0.33	1360	1.9
	4.0 to 5.0			16.6	0.34	2040	1.9
	5.0 to 6.0			15.7	0.32	3060	2.0
	6.0 to 9.0			22.6	0.33	4260	2.0
	9.0 to 12.0			24.6	0.30	3850	2.0
Re-fill	0.0 to 12.0	17.0	21.0	33	0.30	1.2E5	1.0

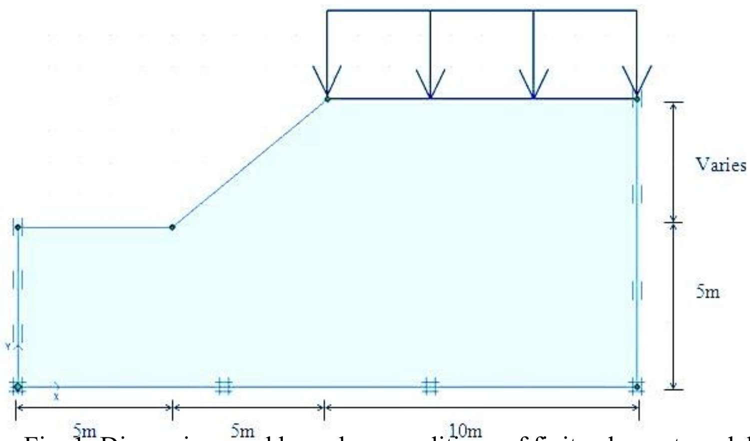

Fig. 1: Dimensions and boundary conditions of finite element model

Then the *calculation* window was opened to analysis. The *Phi-c reduction* calculation option is available in PLAXIS from the *calculation type* list box on the *General* tab sheet. If the *Phi-c reduction* option is selected the *Loading input* on the *parameters* tab sheet is automatically set to *Incremental multipliers.* A in the Parameters tab sheet the number of additional steps is automatically set to 100. Changing phenomena of stiffness matrix is followed after each iteration. The *Msf* value in the multiplier window is set to 0.1.

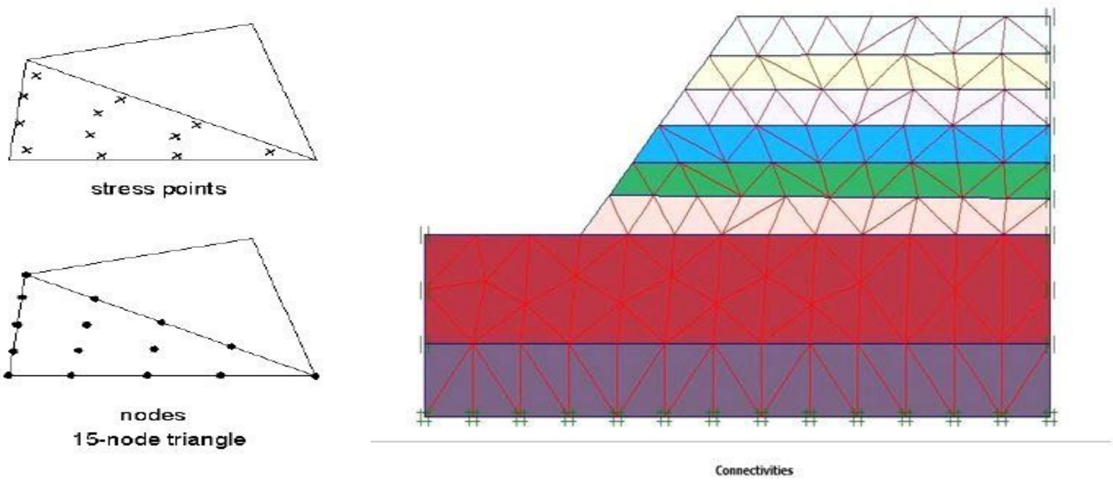

Fig. 2: 15-nodded triangular element and cross section of generated mesh

RESULTS AND DISCUSSIONS

Factor of safety

Fig. 3 shows the results of total displacement increments and factor of safety obtained from analysis for slope height of 4m consisting soil type 1 and re-fill soil.

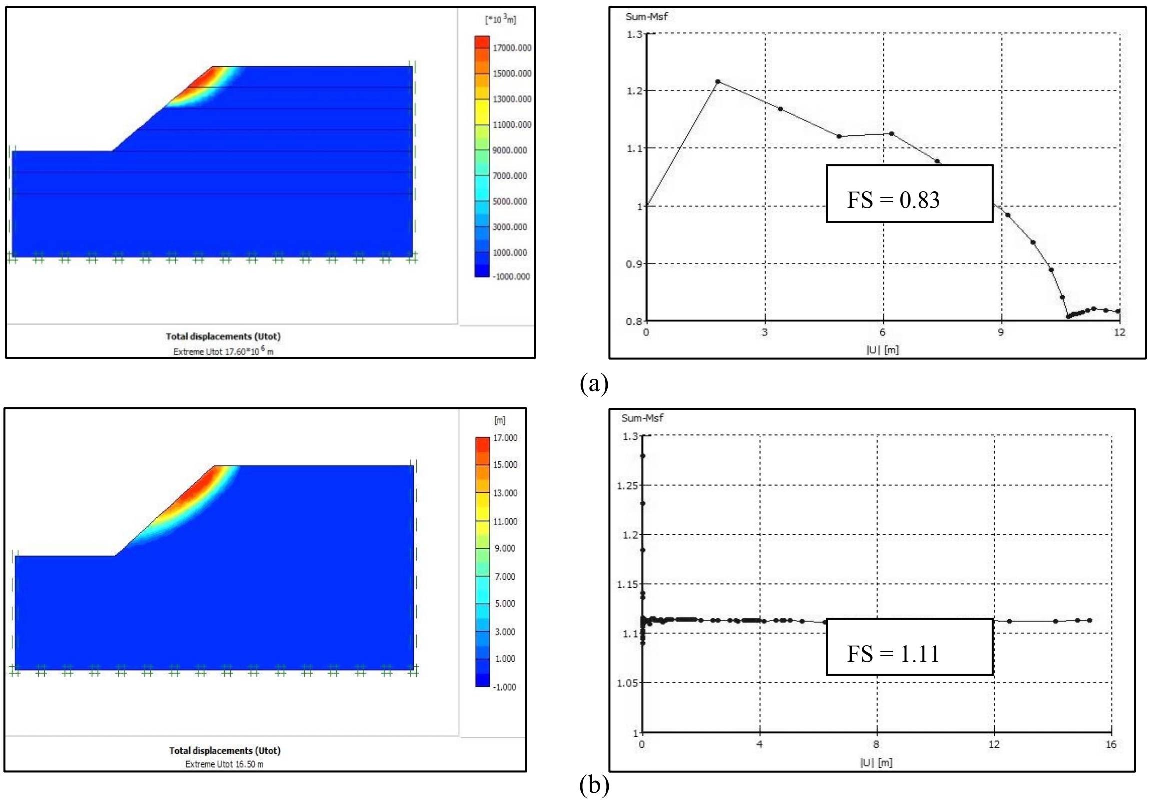

Fig. 3: Total displacement increments and factor of safety for (a) soil type 1 and (b) re-fill soil

Table 2 shows the factor of safety for slopes of different heights. It is obvious that, factor of safety gradually decrease as the slope height increases. From the table, we can depict that, only five times the value of factor of safety exceeds 1.0 which is the minimum value to call a particular slope stable.

Table 2: Factor of safety for varying heights of the slope

	3m	4m	5m	6m
Soil 1	0.95	0.83	0.72	0.64
Soil 2	1.33	0.98	0.80	0.62
Soil 3	1.41	1.08	0.89	0.76
Soil 4	0.98	0.83	0.73	0.63
Re-fill	1.50	1.11	0.90	0.76

Load carrying capacity

In this study the ultimate load carrying capacity of a stable slope is determined. The slope which has factor of safety greater than 1.0 can carry load. The uniform loads which can be carried by the stable slopes up to the verge of failure are tabulated in Table 3.

Table 3: Uniform load carrying capacity of the soil slopes

	Load carrying capacity (kN/m^2)			
Slope height	3m	4m	5m	6m
Soil 1	-----	-----	-----	-----
Soil 2	19.45	-----	-----	-----
Soil 3	14.00	8.00	-----	-----
Soil 4	-----	-----	-----	-----
Re-fill	46.20	22.00	-----	-----

CONCLUSIONS

In this study, detail analysis of slope stability is studied using FEM based software PLAXIS. The effect of slope height and the load carrying capacity is determined here. This evaluation is carried out for slopes comprising both existing in-situ soils and replaced re-fill soil. By comparing a number of data for both conditions some conclusions are drawn. As the height of the slopes increased the factor of safety decreased. In case of load carrying capacity, soil 1 cannot carry any load because the factor of safety is less than 1.0. Slope with height of 3m which is made of soil 2 can carry a surcharge of 19.45 kN/m^2 which is 28.02 % greater than that of the slope that made of soil 3 of same height. Re-fill soil has the most load carrying capacity. Thus, using PLAXIS, the stability and load carrying capacity of any proposed soil slope could be evaluated prior to the construction and thus safety is assured.

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A THEORETICAL FRAMEWORK FOR LANDSLIDE MONITORING AND EARLY WARNING GENERATION

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ABSTRACT
Landslides are a geomorphological process intricately linked to the landform, material, structural, hydrological, climatic and vegetative conditions within which they occur. Due to the increase in the number of population and industrialization, people need to live in landslide prone areas. There are many causes of landslides such as rainfall, rising pore water pressure or accelerating movement, earthquakes, volcanic activity, human work, flood, erosion, snowmelt and change in reservoir level etc. Due to the availability of powerful data acquisition devices, nowadays, we can easily capture huge volume of geometrical information. Proper analysis of such large volume of data can help us to take proper decisions and action plans. However, according to our knowledge, there is no work that performs the analysis of huge volume of such data under cloud computing framework. Considering this fact, in this paper, we provide a theoretical framework to analyze landslides related geometrical data with an aim of landslides monitoring and early warning generation. We hope that proper implementation of this framework will help us to obtain more accurate and precise information about landslides and the authority will be able to take more accurate and efficient action plans.

Keywords: Landslides; cloud computing; data transformation

INTRODUCTION

Nowadays we have many powerful data acquisition devices. As a result it is possible to capture large volume of geometrical information. Information obtained from landform, rainfall, rising pore water pressure or accelerating movement, earthquakes, volcanic activity, human work, flood, erosion, snowmelt and change in reservoir level etc. can be treated as geometrical information. Proper analysis of such large volume of data can help us to take proper decisions and action plans. However, according to our knowledge, there is no work that performs the analysis of huge volume of such data under cloud computing framework. Considering this fact, in this paper, we provide a theoretical framework to capture and analyze landslides related geometrical data for proper monitoring of landslides related issues and provide early warning so that the possibility of landslides can be minimized and proper actions can be taken accordingly.

Most of the tasks related to landslides monitoring so far are site-based and driven by development projects to test the stability of the sites for civil constructions. Conventionally this has been approached by stability analysis of the site, generally determined from the balance of shear stress and strength and expressed as a factor-of-safety. This type of assessment is not well-suited for the general people. Although, several initiatives [1, 2] have been taken for landslides detection and warning generation, they cannot detect and generate warning up to a satisfactory level. This is due to their limited capacity of acquiring and processing large volume of data. The availability of different sensor devices, cloud storage and big data analyzing tools creates an opportunity for us to gather, store and analyze large volume of heterogeneous data to identify important and useful patterns. This can help us to develop a system for accurate assessment about the possibility of landslides. The system generated results can then be evaluated and decisions can be made on whether the level of risk is intolerable, tolerable, or acceptable and actions can be taken accordingly.

Landslides generate a small but important component of the spectrum of hazard and increasing risk that faces mankind. If there were a choice, people would inhabit and rely for their wellbeing on the safe places of the earth – away from the threat of landslide. However, mankind has been placed progressively at the mercy of nature through population pressure, increasing demands for resources, urbanization and environmental change. It is the intersection of humanity with landslide activity that has recast a natural land-forming process into a potential hazard. Furthermore, economic globalization has enhanced reliance on communication and utility corridors. Fuel lines, water and sewage reticulation, telecommunication, energy, and transport corridors, collectively referred to as 'lifelines' in hazard studies, are highly vulnerable to landslide disruption.

Landslides present a threat to life and livelihood throughout the world, ranging from minor disruption to social and economic catastrophe. Spatial and temporal trends in the level of this threat have driven the current international and national concerns on the issue of hazard and risk reduction. However, these trends are difficult to determine accurately because of the variable quality and consistency of record keeping. These problems arise from a range of factors including: variability and improvements in observational techniques, changes in population density, the mix of different agencies involved and the variability of recording protocols, as well as heightened economic and social awareness. As well as economic loss, landslides have also caused numerous humanitarian disasters throughout history.

METHODOLOGY

Our framework for landslides monitoring and early warning generation comprises three main components. These are design, monitoring, and forecasting. The key tasks in the design phase of our landslide monitoring and early warning system include determining the needs and vulnerabilities of the population at risk, identifying any impediments to the population taking action if a warning is issued, and characterizing the geologic and meteorological setting and conditions that lead to landslide initiation. These conditions are referred to as the geo-indicators. Monitoring, which includes instrument installation and data communication and analysis, is a crucial activity. Forecasting represents the core element of the system as it includes the definition of thresholds, models and all the activities that lead to a warning. It is also the most problematic one, not just for the intrinsic difficulty of predicting natural events, but also for severe social and legal implications. An excessively high threshold value means that the lead time left for the emergency plans will be short and, in the worst case, that the event itself could be missed. Conversely, a threshold that is too conservative may lead to false alarms and to all the related problems.

In other words: acceptable risk criteria and tolerability of false alarms are two sides of the same coin; their definition helps to determine the possible range within which the value of the threshold can be set. In any case one has to keep in mind that the possibility of false alarms can be reduced, but cannot be completely nullified; therefore civil protection plans should encompass this chance as well. Figure 1 shows the three components of our proposed system.

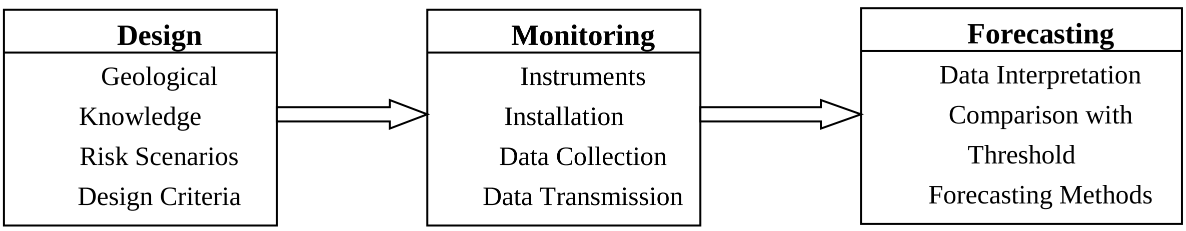

Fig. 1: Components of the proposed landslides monitoring and early warning generation system

Figure 2 shows the overall architecture of our system. In our architecture, we shall use special purpose sensors for collecting data. In the data collection phase, we need to consider the way of regional and site mapping, types of movements. In data collection phase, we also have to use displacement time series to define sliding mechanisms.

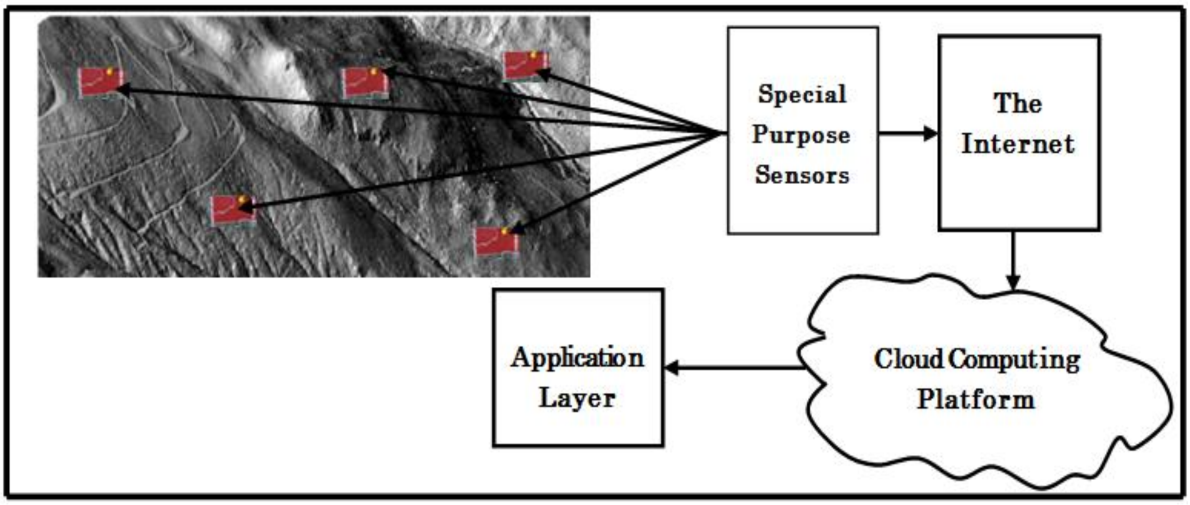

Fig. 2: Overall architecture of the system

The data collected via the sensors will be transmitted through the Internet to the cloud storage where data will be continuously analyzed under big data analysis framework to detect possibility of landslides. Finally, a platform independent application will be developed for the users so that they can obtain early warning about the landslides if they install the application.

EXPECTED RESULTS AND IMPACTS

By introducing a cloud-based framework for landslides monitoring and early warning generation, we can expect following results and impacts:

Results

- The system will be able to identify landslide distribution, process types and their state of activity for the test areas.
- It can be considered as an attempt to creation of local and regional landslide susceptibility, hazard, vulnerability, risk maps and management strategies.
- The system will be able to forecast future landslide risks in the test areas so that preventive measures can be taken in advance.

Impacts:

- The system will help in reducing the number of victims and losses caused by landslides.
- The system will provide a guideline for the authority to develop their action plan for reducing landslides related hazards.
- The system will help in selecting proper locations for the installation of gas, water and sewerage lines.

CONCLUSIONS

In this paper, we provide a theoretical framework for landslides monitoring and early warning generation. The implementation of this work will require collaboration between Civil Engineers and Computer Scientists. As interdisciplinary research is increasing nowadays around the globe such an initiative for landslides monitoring and early warning generation can help us to develop an efficient tool for serving the purpose and will help a lot to reduce the risks related to landslides.

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