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Effect of Super Water Absorbent Polymer and Watering Capacity on Growth of Tomato (*Lycopersicon esculentum* Mill)

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Abstract -*Supper Absorbent Polymers (SAPs) have been used as water retaining materials in agricultural fields. It can slowly release stored water and nutrients as required by the plants. The aim of the present study was to evaluate the effect of SAP named GAM-sorb, on growth of tomatoes. Two factor factorial experiments were carried out in Completely Randomized Design with three replicates. Water capacities in three levels; 75% of Filed Capacity (FC), 50% of FC and 25% of FC and SAPs in four levels, Low level (L) 2.625g, High level (H) 5.25g, Intermediate High level (M) 7.875g, Double High level (DH) 10.5g were applied as treatments. Control was carried out without SAP. The analysis data indicated that number of flowers/plant, Moisture Content of soil (MC), Relative Water Content (RWC) of plant leaves and % of dry weight/ fresh weight revealed significant different among different rates of SAPs. Plant height, yield and amount of chlorophyll present in plant leaves (SPAD values) were not significant. A significant difference observed in number of flowers/plant, plant height, yield, MC of soil, RWC of plant leaves (p<0.05) against three water rates. SPAD values, leaf area and % of dry weight/ fresh weight were not significant. Interaction effects were not significant for all parameters. Best SM retainging ability and growth parameters of tomato in sandy loam soil were prominent in combination of higest SAP (DH) and higest water rate (75% FC).*

***Key words**: field capacity, growth parameter, Super Absorbent Polymer*

1 INTRODUCTION

Supper Absorbent Polymers (SAPs) are compounds that absorb water and swell in to many times than their original size and weight. They are lightly cross linked networks of hydrophilic polymer chains. The network can swell in water and hold a large amount of water, while maintaining physical dimension structure (Buchholz and Graham 1997, Mahdavinia et al., 2004).

It was known that commercially used water-absorbent polymeric materials employed are partial neutralization products of cross linked polyacrylic acids, partial hydrolysis products of starch acrylonitrile copolymers and starch acrylic acid graft copolymers. The polymer consists of a set of polymeric chains that are parallel to each other regularly linked to each other by cross linking agents, thus forming a network. When water comes in to contact with one of these chains, it is drown in to the molecule by osmosis. Water rapidly migrates in to the interior of the polymer net work where it is stored. For these properties, they are widely utilized in various applications such as sanitary napkins, disposal dippers additives for drug delivery systems and soil in agriculture (Li et al. 2007). As the soil dried out, the polymer release up to 95% of the absorbed water to the soil.

Most authors agree that when super absorbent polymers are incorporated in the soil, the following can be observed; control of soil erosion and water runoff (Wallace and Wallace. 1990), increasing infiltration capacity (Zhang and Miller, 1996), increasing soil aggregate size (Wallace et al. 1986), reducing soil bulk density (Al-Harbi et al., 1999), increasing water retention (Johnson, 1984; Bres et al., 1993), improving the survival of seedlings subjected to drought (Huttermann et al., 1999), lengthening shelf-life of pot plants (Gehring et al. 1980), improving nutrient recovery from applied fertilizers (Smith et al. 1991, Bes et al., 1993), improving nutrient uptake by plants grown in poor soil, minimizing nutrient losses through leaching under highly leached conditions, (Mikkelsen, 1994) and reducing irrigation frequency (Taylor et al. 1986).

SAPs are mostly used in arid and semi arid regions of the World to overcome water scarcity problem. We introduce SAPs to urban agriculture, because urban agriculture contributes to minimize the problems of food security and food safety in the World. The present trend in Sri Lanka is more and more urban people to practice crop cultivation. When urban people focus on crop cultivation, they are confronted with severe water scarcity problems especially in Colombo area of Sri Lanka, mainly due to heavy evaporation rate during the day time. Further, highly polluted environment in urban city limits increases the temperature, hence increasing soil water evaporation. This will result in wilting of plant leaves during the daytime. Therefore, watering in the morning as well as in the evening is inevitable. However, frequent watering requires labour, time and money, in addition to water that may waste away during application. Apart from above, during the rainy seasons and when frequent watering is practiced, nutrient deficiency problems are prominent among the cultivated crops due to wash off of soil nutrients and fertilizers, followed by soil degradation. The present study focused to identify the effectual best SAP (GAM-sorb) rate and watering rate on growth of tomato in sandy soil under plant house.

2 LITRETURE REVIEW

2.1 Starch acrylic acid grafted super absorbent polymer

Starch acrylic acid graft SAPs produce according to irradiation of starch component with acrylic acid. The graft copolymerization of cassava starch with acrylic acid was investigated using a free radical initiator system (Fe^{2+})/$H_{(2)}O_{(2)}$ redox system) in water. A comprehensive understanding of the important variables and their interaction has been obtained by applying an experimental design method. In this approach, two ('high' and 'low') values of selected variables are considered. Important result parameters are add-on and the grafting efficiency. Out of eight reaction variables, it was found that only temperature, starch concentration and the starch to monomer ratio have a pronounced influence on these response parameters. Moderate reaction temperature (40 °C) and high starch concentration (10%) give relatively good results of add-on and grafting efficiency.

A low starch to monomer ratio favors add-on but decreases grafting efficiency. These findings can be used to optimize the production of cassava starch-acrylate copolymers and to gain insight in the process-product property interactions, for various applications.

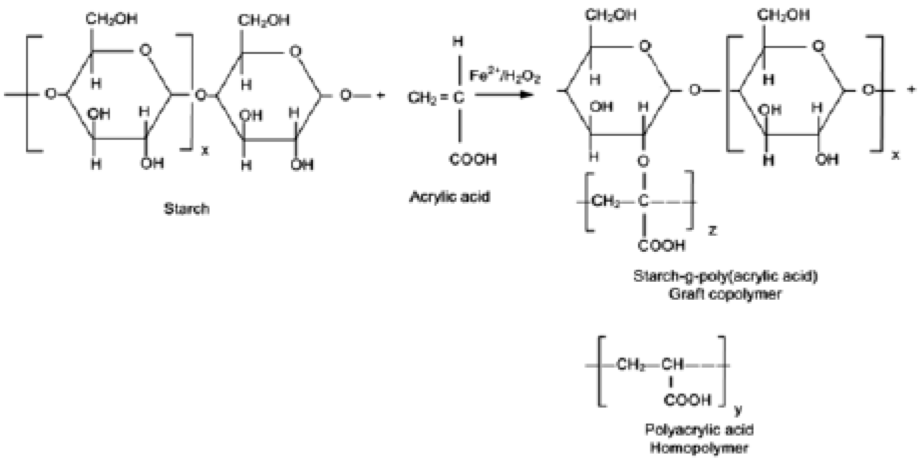


Fig. 1 The grafting reaction of acrylic acid with (cassava) starch

2.2 Swelling mechanisms in hydro gel copolymer

Grafting acrylic acid on to starch leads to the formation of polymer chain which are covered with negative charges (coo-) as shown in Figure 2 since negative charges will repel each other, the chain stretch out thereby providing spaces inside the polymer network which can absorb and retain a large volume of water or aqueous solution, e.g. body fluids or physiological salt.

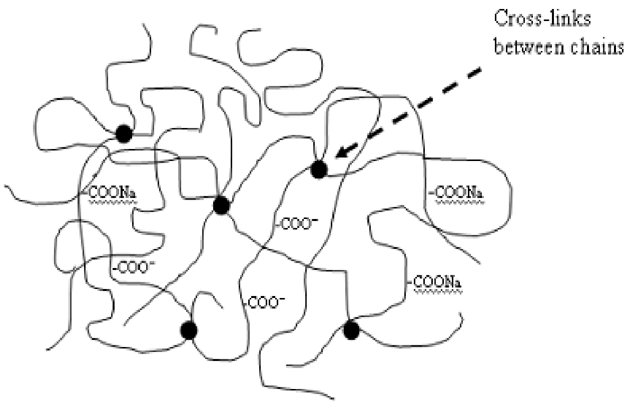


Fig. 2 Schematic of a cross-linked polymer network

Moreover, the hydroxyl groups from starch and the carboxyl groups from acrylic acid are hydrophilic and have a high affinity for water. Neutralization by adding sodium hydroxide replaces H^+ ions of carboxylic groups by Na^+. Upon contract with water these sodium ions are hydrated which reduce their attraction to the carboxylate ions. This allows the sodium ions to move more freely inside the network, which contributes to the osmotic driving force for the diffusion of the water in to the gel. Therefore, neutralization will increase the water absorption capacity.

2.3 Super absorbent polymer GAM-Sorb

Vinagamma affiliated to Vietnam Atomic Energy Commission has recently successfully manufactured water super-absorbent namely GAM-Sorbs which, when combined with organic or micro organic fertilizers, may raise plant productivity from 10-30% more than usual.

GAM-Sorb is made up from environmentally friendly and naturally born polymers, which may regenerate or degenerate in the soil. In order to make GAM-Sorb, manufacturers have to apply radiation techniques as physical agent to denature naturally born polymers (starch, for example), derivatives from cellulose, or poly-glutamic acid. The product may degenerate into humus, carbon dioxide, and water. GAM-Sorb, when combined with organic or micro organic fertilizers, may raise plant productivity from 10-30% more than usual, that is to say, without GAM-Sorb (http://www.vinagamma.com).

2.4 Effect of water deficit on plant growth of Fruit quality and yield

Several researchers have reported that frequency of irrigation and quality of nutrient solution provided to the plants affect yield and fruit quality (May and Gonzales, 1994; Peet and Willits, 1995; Singandhupe, Rao, Patil and Brahmananda, 2002).

Deficit irrigation could, however, cause substantial economic loss through decreased crop marketability. Shinohara et al. (1995) reported that water stress inhibits photosynthesis and translocation of photosynthesis to vegetative organs that may result in decreased plant growth and yield. Other investigations found translocation of photosynthate in to the fruit to be promoted by water stress with reduction in plant yield (Shinohara et al., 1995). In supporting the view to water stress, they confirmed that the water content of the fruit decreases and the concentration of fruit constituents increase due to concentration effects. In a study with irrigation, Yrisarry et al. (1993) concluded that over and under irrigation in processing tomato production could lead to low soluble solid contents with high crop yield or poor crop yield but high soluble solid contents, respectively. Yrisarry et al. (1993) mention that the failure of a crop to reach its water demand will result in reduced plant size and reduced total crop yield. Numbber of clusters/plant, number of fruit set/plant and yield was negatively affected by water deficit (Byari et al., 1999).

Fruit cracking is a problem that can lead to serious economic losses. According to Peet (1992) and Jones (1999), fruit cracking is caused by several factors, mainly associated with the water balance of the fruit. Recently, Dorais et al. (2001) indicated that fruit when cuticle elasticity and resistance are weak. Hence, fruit cracking has been reported to be the result of physical failure of the fruit skin and which is believed to result from the stresses acting on the skin (Milad and Shackel, 1992).

3 METHODOLOGY

3.1 Location and variety selection

The study was carried out in the plant house of the model farm that belongs to Department of Agricultural and Plantation Engineering, of the Open University of Sri Lanka.

3.2 Variety Selection

Tomato – "Bathiya" variety were selected as the experimental variety since it shows less vulnerability to diseases than other tomato varieties.

3.3 Preparing soil samples for experiments

Soil samples were taken from different locations of the Open University of Sri Lanka. Collected soil samples were mixed with compost and spread on the concrete floor and sealed with polythene for solarization.

3.3.1 Analyzing soil samples

Soil samples were analyzed in soil science laboratory in Makadura Research and Developing Centre. Soil pH was analyzed using pH meter. Soil Electrical Conductivity (EC) was determined using electrical conductivity meter, soil available forms of phosphorous and potassium concentrations were analyzed using flam spectrophotometer method and soil nitrogen concentration were analyzed using Kjeldhal method.

3.3.2 Finding the volume of water required to fulfill the Field Capacity (FC) Apparatus: Pressure plate

- 1 bar ceramic plate was dipped in a water bath and kept until it is saturated.
- Soil sample retaining ring was filled with soil and weighted (m_1).
- Soil sample was sprayed with water till saturate and it was placed on ceramic plate.
- Tip of the out flow tube was connected to the burette.
- 1/3 bar air pressure was applied and water was extracted to the burette.
- Extracted volume of water was measured (V).

Volume of water required for field capacity was calculated as follow,

Weight of required soil sample = m_2

Volume of water required for m_2 to fulfill the field capacity = $V \times m_2/m_1$

3.4 Experimental design

Tomato seeds were sown in nursery trays. Water was applied daily until two (02) weeks.

Black plastic pots were filled with 3.5 kg of soil. Two weeks later, young plants were transplanted to the pots. One plant consisted in each pot. Amount of GAMsorb to be applied were calculated according to the Hossein et al., 2010.

Low level of polymer	(L)	-	2.625 g per pot
High level of polymer	(H)	-	5.25 g per pot
Intermediate High level	(M)	-	7.875 g per pot
Double High level	(DH)	-	10.5 g per pot
Control	(C)	-	No GAMsorb

Four (04) rates of SAP and Three (03) rates of water capacity were applied as follow.

SAP rates	Water capacities
• L- Low level	● 25% Field capacity
• H- High level	● 50% Field capacity

- M-Intermediate High level
- DH- Double high level

- 75% Field capacity

Fifteen treatment (15) combinations were selected with above specifications,

T1- (Control: 25% FC)
T2- (Control: 50% FC)
T3- (Control: 75% FC)
T4- (low level: 25%FC)
T5- (low level: 50%FC)
T6- (low level: 75%FC)
T7- (High level: 25%FC)
T8- (High level: 25%FC)
T9- (High level: 25%FC)
T10- (Intermediate high level: 25%FC)
T11- (Intermediate high level: 50%FC)
T12- (Intermediate high level: 75%FC)
T13- (Double high level: 25%FC)
T14- (Double high level: 50%FC)
T15- (Double high level: 75%FC)

Two-factor factorial experiment was carried out Completely Randomized Design (CRD) with three (03) replicates. The pots were placed inside the plant house according to the CRD with the recommended spacing of tomato 45 × 45 cm. Additional ten (10) pots were maintained as guard rows. Water the plants daily to the field capacity until plants were established well for seven (07) days. After seven (07) days, seedlings were watered daily to the rates of 25% of Field Capacity (FC), 50% of FC and 75% of FC. Before the irrigation, the pots were weighted and confirmed the constant weight in all pots and appropriate amount of water was added to each treatment. When nutrient deficiency problems were prominent same volume of foliar fertilizer application was added to each plant. Pests were controlled only when the infestation was seen to be a threat to normal plant growth. Hand weeding was done when necessary.

Number of flowers per plant was recorded seven (07) days intervals, Weight of the harvested fruits per plant was recorded three (03) days intervals in grams (g), Leaf area was measured in randomly selected mature five (05) leaves per plant using leaf area index, Chlorophyll content in plant leaves (SPAD values) were measured in randomly selected fully expanded five (05) leaves per plant using the SPAD meter, Relative water content (RWC) was measured on flag leaves. Immediately after cutting the base of lamina, leaves were sealed in plastic bags and quickly transferred to the laboratory. Fresh weights (FW) were determined within 1h after excision. Turgid weights (TW) were obtained after soaking leaves with distilled water in test tubes for 16 to 18 hours at room temperature under low light condition. After soaking, leaves were carefully blotted dry with bloating paper to determine turgid weight. Dry weights (DW) were obtained after oven drying for 72h at 70^{0}C. The RWC was calculated according to Schonfeld et al. (1988) as RWC= [(FW-DW)/ (TW-DW)]. A fruits defect (cracks, blossom end rot) per plant was recorded Two (02) days intervals. Ratio of dry weight/fresh weight of each plant was measured in grams (g). Soil moisture content was measured three days after harvesting following by drying in an oven at 105^{0}C until it reached at a constant weight.

In addition, relative humidity and temperature was measured daily using wet and dry bulb thermometer inside the plant house in order to study the general climatic changes in the area. Statistical analysis was carried out through Minitab software version 14.

4 RESULTS

4.1 Soil and environment condition

Soil in the tested area was sandy loam and organic matter content, Electric Conductivity (EC) and pH value were 5%, 1.32 dS/m and 6.81 respectively. The average temperature and relative humidity of the research area was 34 ^{0}C and about 55%.

4.2 Finding the volume of water required to fulfill the field capacity

Table 1 Rates of water applied to find the field capacity

	1st soil sample	2nd soil sample	Average
m_1	26 g	24 g	
m_2	3500 g	3500 g	
V	3.25ml	2.3ml	
FCV*	437.5 ml	334.25 ml	386.00ml
FCV* 75%	330 ml	173.07ml	251.5 ml
FCV* 50%	220 ml	115.384 ml	167.5ml
FCV* 25%	131ml	69.230 ml	100 ml

* Felid Capacity Volume

4.3 Growth parameters

As per the results indicated in table 2, there was a significant different ($p<0.05$) between number of flowers per plant, moisture content of soil, Relative Water Content (RWC) of plant leaves and percentage ratio of dry weight/ fresh weight of harvested plant against different rates of SAP. However, plant height, yield and SPAD values were not significant. Further, there was a significant difference between Number of flowers per plant, plant height, yield, moisture content of soil, RWC of plant leaves ($p<0.05$) against three water rates. Since SPAD values, leaf area and percentage ratio of (dry weight/ fresh weight) harvested plant were not significant for three water rates. According to the results in table 3, number of flowers per plant, plant height, yield, moisture content of soil, RWC of plant leaves, SPAD values, leaf area, and percentage of dry weight/fresh weight in Double High level with SAP added were the highest when the lowest occurred at the control (No added SAP). Moreover, water rates were highest in all previous mentions parameters at water 75% FC compared to water 25% FC. Interaction effects were not significant for all above parameters. However, LS means of number of flowers per plant, plant height, yield, moisture content of soil, RWC of plant leaves, SPAD values, leaf area and percentage of dry weight/fresh weight were highest in treatment number 15 which were added SAP(DH): water 75% FC as shown in figures 3 to

10. At the same time lowest LS means of number of flowers per plant, plant height, yield, moisture content of soil and Relative Water Content (RWC) of plant leaves were observed in treatment number 1, control (not added SAP): water 25% FC. In addition, LS means of SPAD values, leaf area and percentage of dry weight/fresh weight have lowest at treatment number 3 which were added no SAP(C): water 75% FC. These results were evident in table 4 as well. Figure 11 (a to m) shows that the symptom of blossom end rod of some treatment separately. According to that figures, symptom of blossom end rot were not observed in treatment added in intermediate high level SAP (M): Water 75%FC and SAP (DH): water 75% FC. Predicted results of least square means were manually graded (a>ab>b>bc>c>cd>d) and grafted according to analysis data.

Table 2 Significant condition of measured parameters

Term	No of Flowers	Plant height (cm)	Yield (g)	Moisture content (%)	RWC	SPAD value	Leaf area (cm^2)	Dry weight/ Fresh Weight (%)
SAP rates	0.001	0.094	0.381	0.035	0.000	0.184	0.112	0.000
Water rates	0.000	0.000	0.000	0.002	0.001	0.910	0.299	0.152
Interaction	0.351	0.958	0.515	0.704	0.725	0.724	0.330	0.273

Table 3 Variation of least squares means for measured parameters

Term	No. of Flowers	Plant height (cm)	Yield (g)	Moisture content (%)	RWC	SPAD value	Leaf Area (cm^2)	Dry weight/ Fresh Weight (%)
SAP rates								
Control (C)	0.867	98.43	26.770	6.539	47.46	50.86	7.605	43.34
SAP (DH)	2.289	110.57	37.242	10.059	75.87	53.29	9.152	65.74
Water rates								
25%FC	0.778	77.33	7.265	6.002	50.45	51.99	7.947	51.13
75%FC	2.378	131.68	56.747	10.596	72.88	52.16	8.811	57.95

Table 4 Predictred results of least squares means for measured parameters

Treatment combinations	No. of Flowers	Plant height (cm)	Yield (g)	Moisture content (%)	RWC	SPAD value	Leaf Area (cm^2)	Dry Weight/ Fresh Weight (%)
T1 - C: water 25%FC	0 ^{d}	71.033^{d}	0^{d}	4.614^{d}	37.669^{d}	51.160^{c}	7.745^{c}	43.595^{c}
T2 - C: water 50%FC	0.867^{cd}	98.433^{bc}	26.77^{bc}	6.539^{cd}	47.46^{cd}	50.856^{cd}	7.605^{cd}	43.340^{cd}
T3 - C: water 75%FC	1.9^{b}	125.833^{b}	56.26^{b}	8.464^{d}	57.251^{c}	50.553^{d}	7.466^{d}	43.085^{d}
T4 - L: water 25%FC	0.306^{cd}	74.179^{cd}	2.272^{cd}	5.308^{cd}	44.060^{cd}	51.573^{bc}	7.846^{bc}	47.363^{bc}

Table 4 (Cont.)

Treatment combinations	No. of Flowers	Plant height (cm)	Yield (g)	Moisture content (%)	RWC	SPAD value	Leaf Area (cm2)	Dry Weight/ Fresh Weight (%)
T5 - L: water 50%FC	1.222cd	101.467bc	29.388bc	7.419bc	54.562cd	51.465bc	7.992bc	48.940bc
T6 - L: water 75%FC	2.139ab	128.754ab	56.504ab	9.530ab	65.064ab	51.358bc	8.138bc	50.517bc
T7 - H: water 25%FC	0.778cd	77.325cd	7.265cd	6.002cd	50.451cd	51.986bc	7.947bc	51.130bc
T8 - H: water 50%FC	1.578cd	104.5bc	32.006bc	8.299bc	61.664bc	52.075bc	8.379ab	57.949bc
T9 - H: water 75%FC	2.378ab	131.675ab	56.747ab	10.596ab	72.876ab	52.164bc	8.811ab	54.898bc
T10- M: water 25%FC	1.25cd	80.471cd	12.258cd	6.697cd	56.842cd	52.399bc	8.048bc	60.139ab
T11- M: water 50%FC	1.933ab	107.533bc	34.624bc	9.179ab	68.766ab	52.684bc	8.765ab	65.380ab
T12- M: water 75%FC	2.617ab	134.596ab	56.99ab	11.661ab	80.689ab	52.969ab	9.483ab	58.665b
T13- DH: water 25%FC	1.722c	83.617c	17.251c	7.391c	63.233b	52.811b	8.148b	65.739ab
T14 -DH: water 50%FC	2.289ab	110.567bc	37.242bc	10.059ab	75.867ab	53.293ab	9.152ab	72.812a
T15 -DH: water 75%FC	2.856a	137.517a	57.234a	12.727a	88.501a	53.775a	7.745c	43.595c

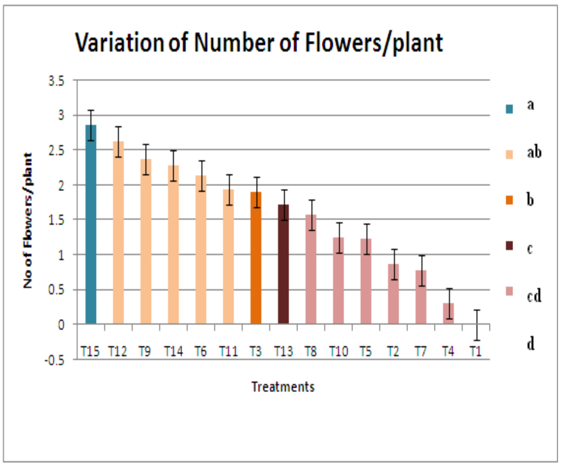


Fig. 3 Variation of LS means of treatment combinations for Number of flowers/plant

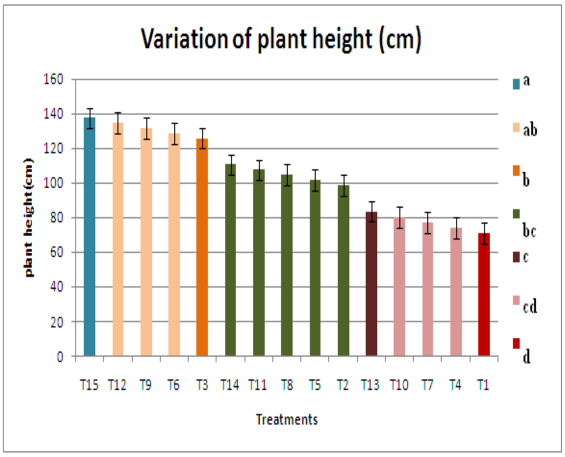


Fig. 4 Variation of LS means of treatment combinations for plant height

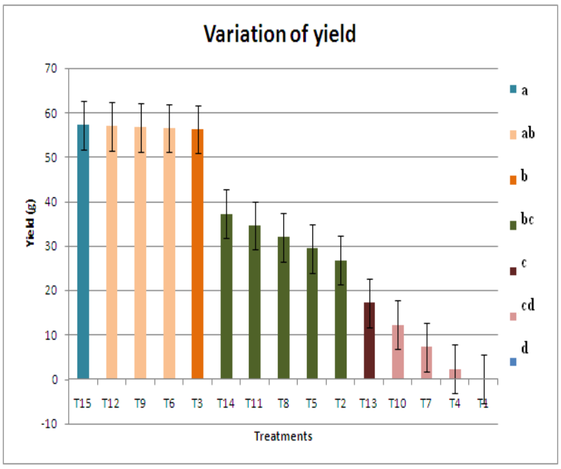


Fig. 5 Variation of LS means of treatment combinations for yield

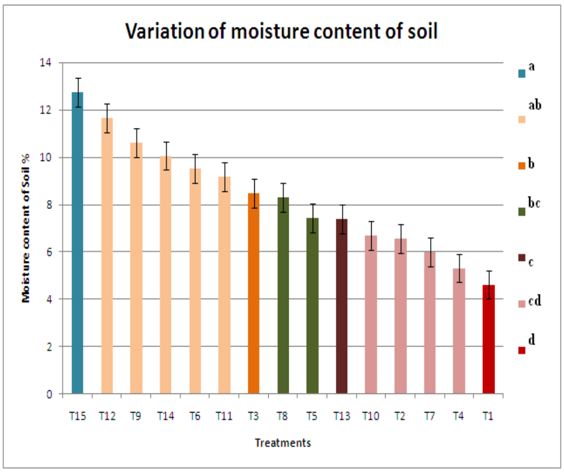


Fig. 6 Variation of LS means of treatment combinations for MC of soil

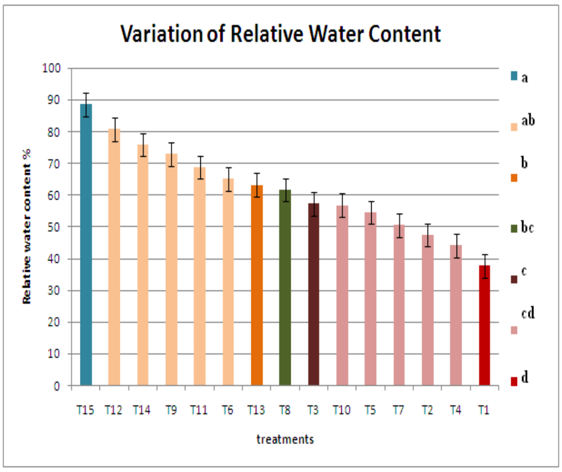


Fig. 7 Variation of LS means of treatment combinations for RWC

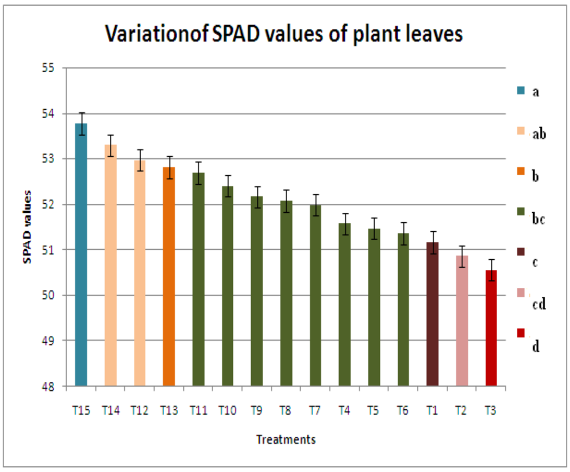


Fig. 8 Variation of LS means of treatment combinations for SPAD values

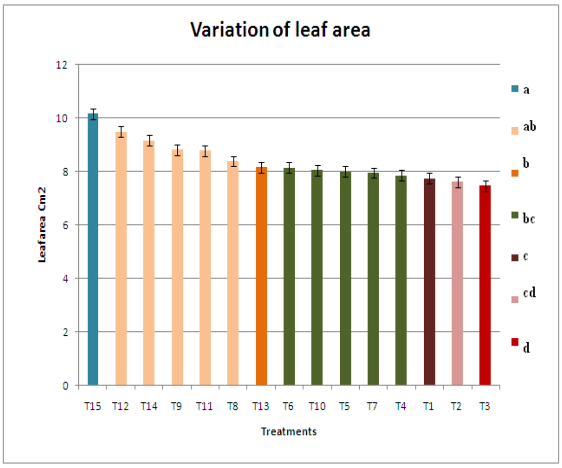


Fig. 9 Variation of LS means of treatment combinations for leaf area

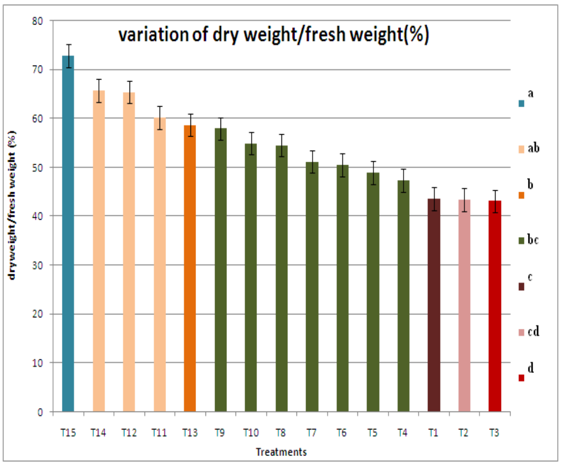
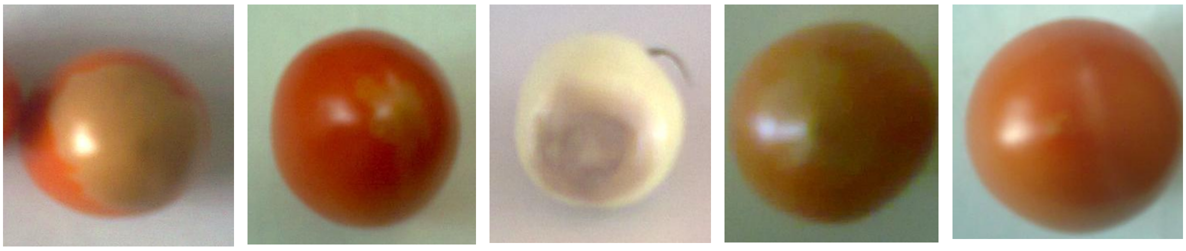
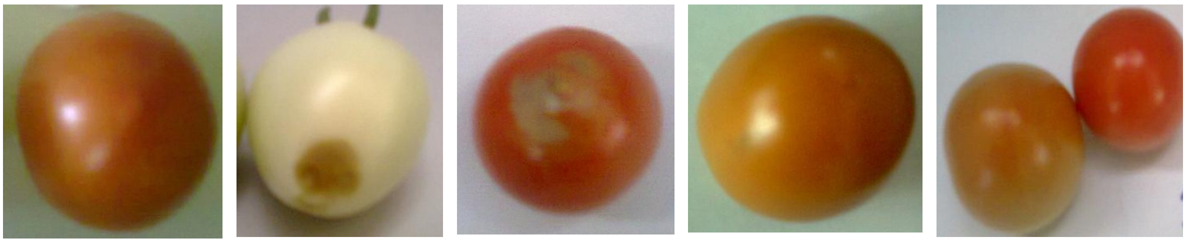


Fig. 10 Variation of LS means of treatment combinations for dry weight/fresh weight

(a) L25%FC (b) M25%FC (c) DH25%FC



(d)C50%FC (e) L50%FC (f) H50%FC (g) M50%FC (h) DH50%FC



(i)C75%FC (J) L75%Fc (k) H75%FC (l) M75%FC (m) DH75%FC

Fig. 11 (a - m) Variation of Blossom-end rot of harvested fruits

5 DISCUSSION

SAPs have been used in agricultural and horticultural fields Johnson (1984), Mikkelsen (1994), and Yazdani et al. (2007) due to their ability to retain water and nutrients for long time when incorporated into the soil. The stored water and nutrients are released slowly in required amounts to the plant that improves its growth under limited water supply condition Huttermann et al. (1999). Results of the present study indicated clearly a remarkable increase in soil moisture retaining ability and at the same time improving of some morphological parameters of tomato with limited soil-water supply.

According to the present study number of flowers per plant, plant height, leaf area, moisture content of soil, Relative Water Content (RWC) of plant leaves, chlorophyll present in plant leaves (SPAD values), yield and percentage ratio of dry weight/fresh weight has been increased in highest SAP and highest water added DH SAP: water 75% FC compared to the treatment which was not added with SAP and lowest water rate from FC (Control: water 25% FC). These results are in line with the Ghasemi et al. (2008) reported the best amount of super absorbent (0, 0.2, 0.4, 0.6, 0.8 and 1% of weight) and *Chrysanthemum morifolium* Ramat reaction to it, in different irrigation intervals (2, 3, 4 and 5 days). They announced that using these hydrophilic gels had positive and significant effect on number of flowers per plant, area of leaves, plant height, root/shoot proportion and coverage area in drought stress. The best performance in all indices was related to 0.8% treatment compared to control (without SAP) and also according to their study, water deficit reduces growth parameters. Further, Poormohammad Kiani et al. (2007) declared that RWC of well - watered plants in Recombinant Inbred Lines (RILs) of sunflower ranged from 80.4 to 91.7%, while in water-stressed plants, it ranged from 59.5 to 80.7%. Our results were concordant with those obtained by Jing et al. (2002) and Unyayar et al. (2004). Stomata are mainly sensitive to leaf water status, tending to close once with decreasing leaf water potential. In this situation, the rate of photosynthesis declined while the respiration rate increased. In addition Ahmadi et al. (2000) showed that the moderate water stress (15% of FC) has significantly reduced the content of chlorophyll from wheat leaf. Maslenkova et al. (1997) and Zhang et al. (2006) were also found that the retardation in the content of photosynthetic pigment, because of water stress was attributed to the ultra structural deformation of plastids, including the protein membranes forming thylakoids, which in turn cause untying of photo system 2, which captures photons, so it's efficiency declined, thus causing declines in electron transfer, ATP and NADPH production and eventually CO_2 fixation processes. That due to reduced SAPD values of plant leaves and yields of tomato. Blossom end rot is a physiological disorder that is caused by a lack of calcium uptake from the soil and transfer to the fruits during dry weather. The problem is costly to many tomato growers and disappointing to home gardeners. Although a sudden lack of water is the principal cause of blossom end rot, excessive soil moistures early in the season may smother the root hairs and cause blossom end rot to occur during sudden hot weather. (http://ohioline.osu.edu/hyg-fact/3000/3117.html) Our result indicated that the increasing SAPs rates reduced appearance of blossom end rot.

6 CONCLUSIONS

Best soil moisture retainging ability and growth parameters in sandy loam soil were prominent in combination of higest SAP (DH) rate and higest water rate (75% FC) added to the tomato plants. Other SAP rates as well as water rates added tomato plants grown in sandy loam soil showed lower gorwth performances and at the same time showed lower ability to retaining the soil moisture.

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An Investigation on actual Soil Skin Friction capacity of CIB Piles- Case study: Proposed Forty Two Storied Building Project, Colombo 03

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Abstract – *Cast in-situ Bored (CIB) piles in Sri Lanka are very often designed considering only the end bearing capacity, neglecting the soil and rock skin friction. This causes foundations to become very uneconomical. The study presented here was done as a case study, where the subsurface does not contain any compressible soil layers, which subsequently cause to buildup negative friction forces on piles during its consolidation process. Three different design methodologies were adopted and compared with field load test values to assess, which best simulate the realistic conditions. The Burland method, ICTAD method and O'Neil & Reese method were used to calculate the theoretical soil skin friction levels, whilst Williams and Pells method was used to calculate the skin friction in the rock socket. Both High Strain Dynamic Test (using Pile Dynamic Analyzer (PDA)) and Static Load Test (SLT) results were used to interpret the actual field skin friction values, keeping in mind about the relative merits and demerits of these techniques. It was revealed that the results obtained during field load tests are substantially higher when compared to the theoretical results obtained through all three methods. However, O'Neil & Reese method in combination with Williams and Pells method provides substantially higher values compared to other two, which are the most widely used methods in local pile design practice. Hence the most appropriate method of calculating soil skin friction is O'Neil & Reese method in local context compared to other two methods. The reason behind the large discrepancy between theoretical values and field load test values may be due to two reasons, viz., (a) soil parameters obtained from in-situ test results with the help of standard charts and tables do underestimate local subsurface conditions and (b) the methods used to calculate the rock socket friction highly underestimate the locally available high grade-high strength metamorphic bedrock conditions.*

Keywords: *Cast In-Situ Bored Piles, High Strain Dynamic Test, Skin Friction, Standard Penetration Test 'N' value, Static Load Test*

Nomenclature

CIB	Cast In situ Bored Piles
EB	End Bearing
SF	Skin Friction
PDA	Pile Dynamic Analyzer
SLT	Static Load Test

1 INTRODUCTION

Cast In-Situ Bored (CIB) Piles are widely used in Sri Lanka as foundations to support heavily loaded structure like high rise buildings, bridges, flyovers and towers. In most cases design engineers tend to follow the design parameters in the site investigation reports rather than going from the basics, mainly because of lack of access to latest engineering foundation design practices and construction methodologies and lack of confidence in the time effects of bentonite filter cake around the pile (Thilakasiri, 2006). In most cases it has been revealed that these design parameters underestimate the local subsurface conditions. Owing to above factors, CIB piles are very often designed considering only the end bearing (EB) capacity, neglecting the skin friction (SF) capacity.

When the compressive strength of the bedrock in the coastal zone, closer to Colombo is considered, the experimental compressive strength values are very much higher than those used in soil reports to estimate the end bearing capacity. For example, average UCS of moderately weathered rock core samples in CCR project is 45 MPa, OZO Colombo Hotel Project is 60 MPa and Shangri-la Hotel Project is 81 Mpa. In contrast, recommended end bearing capacity for moderately weathered bedrock is around 3-5 MPa as given by most of the soil investigations reports.

As the pile is loaded axially and forced to move downwards, the first type of resistance it has to undergo is the skin friction (Bowles, 1997 and Tomlinson et al, 2008). The skin friction is activated under very small displacement (Tomlinson et al, 2008) and its magnitude depends on the strength properties of the surrounding soil, method of installation of the pile, and the properties of the pile surface etc. (Poulos and Davis, 1980).In addition, the use of the bentonite slurry during the drilling process has a significant impact on the mobilized ultimate skin frictional resistance during loading as a filter cake formed on the drilled wall of the pile bore, which considerably reduces the mobilized skin frictional resistance (Tomlinson et al, 2008).

Skin friction of a pile includes two components, namely frictional resistance and the adhesive resistance. Skin friction is mobilized in both cohesive and cohesionless soils and can be calculated by various methods.

In order to estimate the skin friction, certain engineering parameters must be obtained, such as unit weight, shear strength and consolidation properties in addition to the lateral soil pressure coefficients. In most cases these parameters are obtained from correlations with in-situ test results (in most cases the SPT'N' value). However, when the bases of these correlations are investigated, it is found that they are developed in countries where more unfavourable geotechnical conditions exist compared to local context. Apart from few low land areas, the subsurface is composed of residual soils in most parts of Sri Lanka. These residual soils are of sub-rounded to angular grains of mostly quartzite in nature, which always generate much higher skin friction than the soils in the subsurface conditions where these correlations have developed.

Secondly the underestimation of local geotechnical conditions is created in the method of calculating the skin friction capacity. As mentioned above these methodologies and related equations (empirical equations related to in-situ test results) have developed for more poor subsurface conditions compared to local context. Therefore, authors believe that, it is worthwhile to look into the possibility of using much higher skin friction capacity levels than the levels currently being used, which will enable the use of reduced pile diameters and hence more economical design.

1.1 Objectives

This study was carried out to achieve the following objectives.

a. To investigate the generated skin friction distribution along the pile using different theoretical skin friction calculation methods.
b. To analyze the suitability of above theoretical concepts by comparing with the field load test results.
c. To identify the most appropriate method of calculating the skin friction distribution of a pile under local context.

2 METHODOLOGY

Following methodology was adopted in this study to achieve the above mentioned objectives.

- Determination of engineering parameters of different soil strata with the use of in-situ test data.
- Calculation of soil skin friction, rock socket skin friction and end bearing capacity for the closest pile to the particular borehole using three (03) different theoretical concepts.
- Determination of actual soil skin friction and end bearing capacity levels with the use of field load test results.
- Comparison of results under (1) and (2) and finding the most appropriate theoretical method of calculating soils skin friction for CIB piles for local context.

2.1 Determination of engineering parameters from borehole log data

Engineering parameters were determined from standard correlations between in-situ test results and particular engineering parameters or using standard tables and charts (Tomlinson and Boorman, 1995).

The subsurface under the study are composed of sands/silty sands and do not contain any compressible soils (Geotech, 2003). Hence, in-situ test results of Standard Penetration Test (SPT) '**N**' values (Geotech, 2003) were used to determine the engineering parameters of different soil strata. These SPT'**N**' values were initially corrected using Equation 01 and then these corrected 'N' values (Bowles, 1997) were used in determining engineering parameters.

$$N_{\text{corrected}} = N_{\text{field}}\, C_N \eta_1 \eta_2 \eta_3 \eta_4 \qquad (01)$$

Where;

C_N, η_1, η_2, η_3 and η_4 are the correction factors for overburden, hammer energy, rod length, sampler and the borehole diameter respectively.

2.2 Calculation of theoretical skin friction

Following three methods were employed in calculation of theoretical soil skin friction, where all three methods are valid only for cohesionless sandy soils.

1. Burland Method
2. ICTAD Method
3. O'Neill and Reese Method

The skin friction capacities in rock socket region were calculated using William and Pells method.

2.2.1 Burland Method

For coarse grained soils Burland, 1973, (Bowles, 1997)proposes that the ultimate shaft resistance (f_s) on bored piles in coarse grained soils at a point can be expressed in terms of effective stress as depicted in equation 02.

$$f_s = \beta\sigma_v^{'} \tan\delta \tag{02}$$

$$\beta = (1 - \sin\phi) \tag{03}$$

$$\delta = 0.75\phi \tag{04}$$

Where;

f_s- ultimate shaft unit side resistance (skin friction) at a point on the pile in kPa.

$\sigma_v^{'}$- effective vertical stress along the pile

ϕ- angle of internal friction of soil

β- shaft resistance factor for coarse grained soils.

According the Equation 02, the unit soil skin friction increases with depth. However this is only up to a certain depth called ***critical depth (z_c)*** and beyond which the imposed soil skin friction value will be constant. The critical depth was calculated using Equation 05 and chart proposed by Meyerhof, 1976 (Poulos and Davis, 1980), which provides z_c/d ratio for corresponding ($\phi^{"}$) and ('d'), the pile diameter.

$$\emptyset^{"} = \phi - 3 \quad \text{for CIB piles} \tag{05}$$

Here, ϕ and $\emptyset^{"}$ are expressed in degrees.

2.2.2 ICTAD Method

This is one of the simplest methods that can be used to evaluate skin friction of bored piles. In this method skin friction totally depends on the SPT 'N' values and hence the variation of skin friction along the pile shaft reflects the variation of SPT 'N' values. This is an extended version of Meyerhof, (1956, 1976) and Shioi and Fukui, (1982)(Bowles, 1997). The unit ultimate shaft resistance of bored piles was estimated using equations (06) and (07) (ICTAD, 1997).

$$f_s = 1.3 * N_{corr} \tag{06}$$

$$f_r = 2.0 * N_{corr} \text{ and } f_r < 200kPa \tag{07}$$

Where;

f_s and f_r- unit ultimate skin friction of soils and rock respectively in kPa

N_{corr} - corrected average SPT'**N**' value

2.2.3 O'Neill and Reese Method

O'Neill and Reese, 1999(Seavey and Ashford, 2004) is one of the methods that most commonly used in practice in most parts of the world. Here, the skin friction was estimated using equation (08).

$$f_s = \beta_i \sigma'_{vm} \quad (08)$$

(i) For $N_{corr} \geq 15$;
$$\beta_i = 1.5 - 0.245 * Z_i \quad (09)$$

(ii) For $N_{corr} < 15$;
$$\beta_i = \{(N_{corr}/15) * [1.5 - 0.245 * Z_i^{0.5}]\} \quad (10)$$

Where;

f_s - ultimate shaft unit side resistance (skin friction) in kPa

σ'_{vm} - effective vertical stress at the midpoint of the particular soil layer

β_i - dimensionless factor calculated from Equations (09) and (10) for sands

N_{corr} - corrected average SPT'**N**' value

Z_i - vertical distance from the ground surface to the middle of the soil layerin meters

2.2.4 Skin Friction in Rock

The skin friction capacity in rock socketed area (f_{rs}) was determined using the method proposed by William and Pells (1981) (Tomlinson and Woodward, 2008), using the relationship in Equation (11).

$$f_{rs} = \alpha \beta q_{uc} \quad (11)$$

Where;

q_{uc} - unconfined compressive strength of rock in socketed area in the rock.

α - rock socket reduction factor from the chart αVsq_{uc}

β - rock socket correction factor from the chart βVsj

j - mass factor Hobbs, 1975 from the chart fractures/m Vsj

These charts are given in Tomlinson and Woodward, 2008.

2.2.5 Calculation of end bearing resistance on Pile in Rock

Allowable end bearing capacity was obtained using chart for allowable bearing pressures for metamorphic rocks given in BS 8004:1986 clause 2.2.2.3.1 figure 1 (b) (BSI, 1998).

2.3 Field Testing of Piles

Field testing of piles is done with the use of basically two types of Pile load capacity testing in Sri Lanka, viz.(Thilakasiri, 2009).

1. High Strain Dynamic Test(HSDT) using Pile Driving Analyzer (PDA)
2. Static Load Test (SLT)

2.3.1 High Strain Dynamic Testing of Bored Piles (PDA)

Both the skin frictional and end bearing components of the developed resistance on a CIB could be estimated using the dynamic testing of piles using Pile Driving Analyzer (PDA). The PDA is both a field data acquisition unit as well as a computer unit for onsite data assessment. The CAPWAP computer software allows full and accurate analysis of the PDA field data. CAPWAP model is a match curve of computed pile top force to the measured pile top force time record. It is capable of providing total computed soil capacity, sum of skin friction and end bearing. Furthermore, it provides the skin friction force for the pile and its distribution along the pile shaft and pile toe bearing capacity can be obtained separately. It also computes Load Vs Settlement curve (Thilalkasiri, et.al. 2006).

In this study, the processed CAPWAP data for the particular pile tests were directly used. The data were constituted of skin friction distribution along the pile and the end bearing component separately (Geotech, 2006).

2.3.2 Static Load Test (SLT)

A constant axial load is applied on a pile for a predetermined time interval and the settlement is measured. This load is increased incrementally, generally up to 150% of the working load. The load shall be measured by a load-measuring device and by calibrated pressure gauges included in the hydraulic system (Thilalkasiri, et.al. 2006).

Thevariation of load-settlement with time was obtained from the particular pile test reports (Geotech, 2006) and then this data were processed to determine the skin friction and end bearing components of particular load tests. Van Weele, 1957 method (Bowles, 1997) was used in finding the skin friction and end bearing capacity levels of particular shafts. The applicability of this procedure has been tested for local context (Thilalkasiri, 2006) and has proven to be matching the results with CAPWAP results. The load-settlement curve interpretation is depicted in Fig.1.

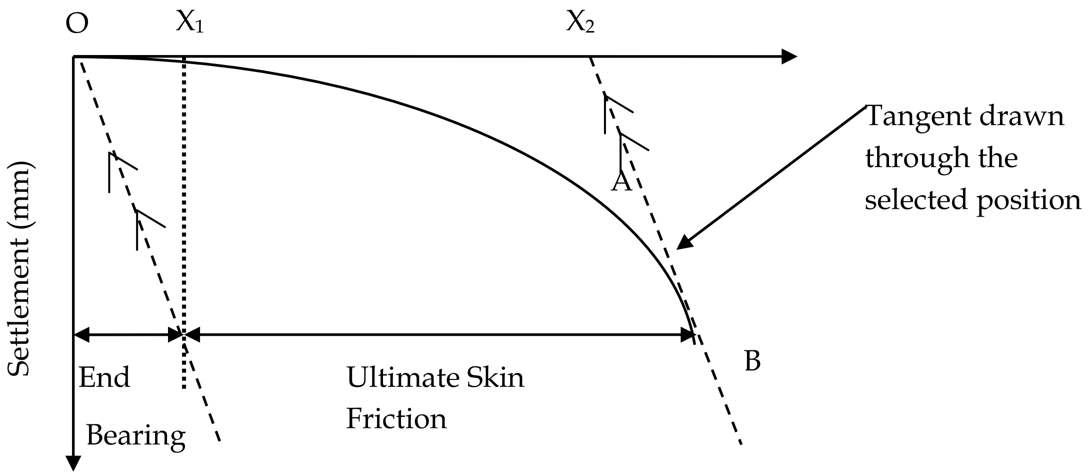

Fig. 1 Load vs. Settlement behavior proposed by Van Weele

The ultimate end bearing resistance is equal to OX_1.

Ultimate Skin Friction Capacity	=	Value of the tangent at B (OX_2)	-	Value of the parallel line through (0, 0) to the tangent at B (OX_1)

2.4 Data collection

Data for the analysis were collected from pile load test results of the particular project (Geotech, 2006). Details of Pile Numbers are presented in Table 1.

Table 1 PDA tested piles against total number of piles and respective pile diameters.

Pile Dia.(mm)	**1800**	**1500**	**1200**	**900**	**600**
Number of piles	90	55	25	7	5
Test piles	35	7	3	-	-

Altogether data of 15 boreholes were available for the particular site and there were 182 CIB piles of diameters ranging from 600mm to 1800mm (Table 1). Out of those 182 piles, 41 piles were tested using high strain dynamic tests (HSDT) using PDA (Pile dynamic Analyzer). Three (03) SLTs (Static Load Test) have been performed on three selected piles. The details of the tested piles in reference to borehole locations are given in Table 2, which were collected from soil investigation reports (Geotech, 2003)

Table 2 Details of load tested piles with respect to the nearest boreholes

BH-1	**BH-2**	**BH-3**	**BH-4**	**BH-5**	**BH-6**	**BH-A**	**BH-B**	**BH-G**	**BH-I**	**BH-J**
P012	P050	P101	P132	P040	P072	P015	P010	P131	P088	P042
P020	P049		P151	P042				P142	P093	P045
P014	P046	P114	P133	P061				P148	P087	P033
P003		P123	P149	P074					P78b	P032
P004		P126	P150						P094	
P087		P124							P78A	
P013		P130							P064	
P023		P121								

3 RESULTS AND ANALYSIS

3.1 Theoretical skin friction capacities

The theoretical ultimate skin friction on each pile was calculated using the methods mentioned under section 2.3, with the help of filed in-situ test values (SPT'**N**') mentioned under section 2.2. Later safe skin friction values, as shown in Table 3, were obtained by factoring the ultimate soils skin friction by a factor of safety of 3.0 and ultimate rock socket skin friction by a factor of safety of 2.5. Higher factor of safety in obtaining safe soil skin friction was used mainly because of uncertainties involve in the adjacent smear zone and the bentonite cake that forms around the pile borehole.

Table 3 Skin friction (SF) capacity levels acting on piles using theoretical methods

Pile	Reference Borehole	Depth of Pile (m)	Safe soil skin friction (kN)			Safe Rock socket friction (kN)
			Burland	ICTAD	O'Neil &Reese	
P020	BH-01	29.10	2250	2198	6508	791
P013	BH-01	28.35	1438	1319	4652	528
P014	BH-01	27.54	1377	1198	4354	528
P004	BH-01	30.00	2342	2347	7110	791
P023	BH-01	28.19	1520	1281	4672	528
P003	BH-01	29.70	2348	2387	7357	791
P012	BH-01	28.90	1875	1802	5900	659
P050	BH-02	28.19	2074	1834	6941	371
P049	BH-02	29.63	2485	2316	8453	371
P046	BH-02	31.05	2444	2565	8526	371
P124	BH-03	28.55	1943	2356	8069	661
P101	BH-03	28.80	3017	2615	10032	661
P121	BH-03	30.05	3275	2715	10317	661
P114	BH-03	26.20	2873	2263	10687	992
P140	BH-03	27.80	3007	2413	9928	827
P126	BH-03	27.85	3141	2564	10317	992
P109	BH-03	31.23	3543	2564	10388	661
P151	BH-04	27.90	1959	1681	5783	930
P130	BH-04	28.50	1952	1681	5975	1116
P132	BH-04	27.87	2342	2000	5673	1116
P133	BH-04	27.20	1875	1522	4728	1116
P150	BH-04	29.35	1963	1681	5975	930
P074	BH-05	30.15	1678	1636	5236	715
P040	BH-05	29.60	2308	2039	6779	858
P061	BH-05	30.83	2411	2209	6525	858

Table 3 (Cont.)

Pile	Reference Borehole	Depth of Pile (m)	Safe soil skin friction (kN)			Safe Rock socket friction (kN)
			Burland	ICTAD	O'Neil &Reese	
P015	BH-A	23.75	1532	1402	5482	848
P010	BH-B	29.40	2241	2515	6807	936
P165	BH-B	29.35	1620	1622	5265	520
P166	BH-B	31.45	1498	1442	5392	520
P131	BH-G	27.10	2242	1999	7837	936
P148	BH-G	27.50	2346	2109	7595	936
P088	BH-I	29.70	2525	2368	7860	834
P093	BH-I	27.30	2219	1969	7338	834
P078	BH-I	28.82	2421	2194	7343	834
P094	BH-I	30.20	2522	2368	7856	834
P064	BH-I	29.65	2624	2495	6808	834
P042	BH-J	29.98	2624	2084	6808	780
P032	BH-J	28.40	2168	1798	5227	780
P033	BH-J	26.60	2159	1472	5086	936
P045	BH-J	30.50	2440	1876	7413	936
P043	BH-J	30.70	2527	2021	5550	936

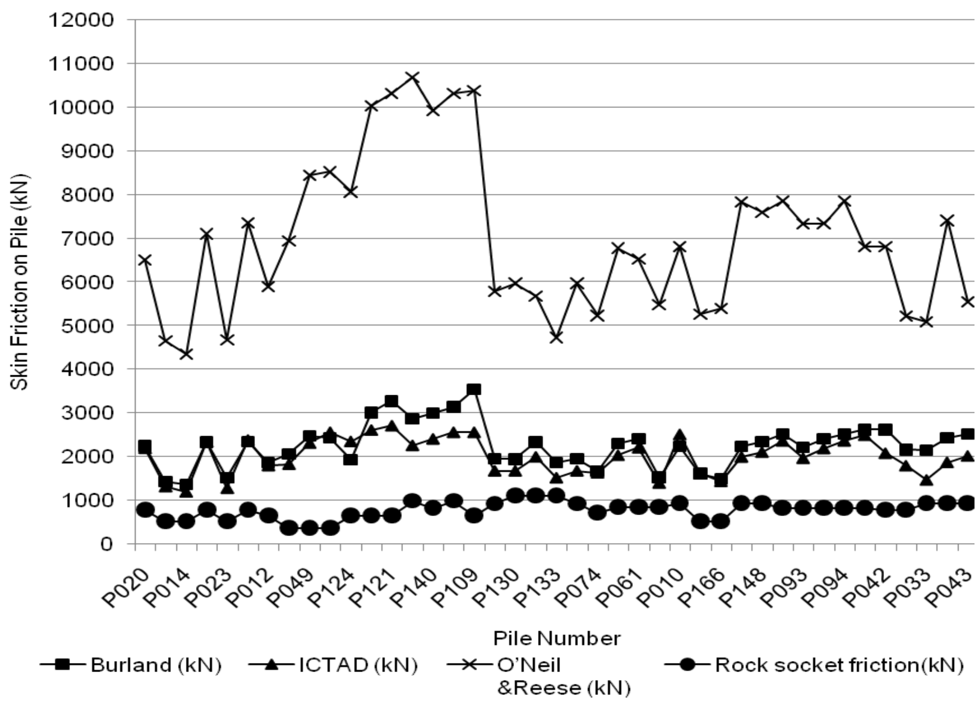

Fig. 2 Skin Friction on Piles vs. Pile Number

3.2 Allowable End bearing Resistance of Pile

The allowable end bearing capacities were calculated using the method mentioned under section 2.3.5 and given in Table 4. The * marked piles have been terminated after encountering much competent bedrock profiles (which depends on quality of cores recovered in pile boring process)compared to reference boreholes and below the termination depth of respective boreholes. Hence the UCS values and RQD values determined for upper layers during soil investigation stage had to be ignored in these cases.

Table 4 End bearing (EB) capacity levels acting on piles using theoretical methods

Pile	Reference Borehole	Depth of Pile (m)	Weathering grade	RQD %	UCS (MPa)	End bearing capacity (kPa)
P020*	BH-01	29.10	Fresh	52		12500
P013*	BH-01	28.35	Fresh	52		12500
P014*	BH-01	27.54	Fresh	52		12500
P004*	BH-01	30.00	Fresh	52		12500
P023*	BH-01	28.19	Fresh	52		12500
P003*	BH-01	29.70	Fresh	52		12500
P012*	BH-01	28.90	Fresh	52		12500
P050	BH-02	28.19	Fresh	33		5500
P049*	BH-02	29.63	Fresh	33		6000
P046*	BH-02	31.05	Fresh	33		6500
P124	BH-03	28.55	Fresh	28		5000
P101	BH-03	28.8	Fresh	28		5000
P121	BH-03	30.05	Fresh	28		5000
P114	BH-03	26.28	Fresh	Nil		275**
P140	BH-03	27.80	Fresh	10	7.13	330**
P126	BH-03	27.85	Fresh	10	7.13	330**
P109*	BH-03	31.23	Fresh	28		5500
P151*	BH-04	27.90	Fresh	80		10500
P130*	BH-04	28.50	Fresh	80		10500
P132*	BH-04	27.87	Fresh	80		10500
P133	BH-04	27.20	Fresh	80		10000
P150*	BH-04	29.35	Fresh	80		10500
P074	BH-05	30.15	Fresh	89		11500
P040	BH-05	29.60	Fresh	89		11500
P061*	BH-05	30.83	Fresh	89		12000
P015*	BH-A	23.75	Fresh	100		12500
P010*	BH-B	29.40	Fresh	93		12000

Table 4 (Cont.)

Pile	Reference Borehole	Depth of Pile (m)	Weathering grade	RQD %	UCS (MPa)	End bearing capacity (kPa)
P165*	BH-B	29.35	Fresh	93		12000
P166*	BH-B	31.45	Fresh	93		12000
P131*	BH-G	27.10	Fresh	91		11500
P148*	BH-G	27.50	Fresh	91		11500
P088*	BH-I	29.70	Fresh	70		10000
P093*	BH-I	27.30	Fresh	70		10000
P078*	BH-I	28.82	Fresh	70		10000
P094*	BH-I	30.20	Fresh	70		10000
P064*	BH-I	29.65	Fresh	70		10000
P042*	BH-J	29.98	Fresh	90		11500
P032*	BH-J	28.40	Fresh	90		11500
P033*	BH-J	26.60	Fresh	90		11500
P045*	BH-J	30.50	Fresh	90		11500
P043*	BH-J	30.70	Fresh	90		11500

Note *
The end bearing capacity of piles which have been terminated below the depth of reference boreholes was determined using standard tables (Tomlinson and Boorman, 1995), which gives general end bearing capacity levels for different RQD and degree of weathering levels.
Note **
In actual practice the Piles P114, P126 and P140 have been terminated much highly competent layer compared to the reference borehole level and the discrepancy in the borehole results may be due to variation of bedrock profile.

3.3 Practical (Field) pile capacities

3.3.1 High Strain Dynamic Testing (PDA) results

The Table 5 depicts the PDA data From CAPWAP analysis results of piles, providing the skin friction, end bearing and total capacity forces respectively.

Table 5 SF and EB capacity levels acting on piles given by Field PDA Test Results

Pile	Total skin friction (kN)	End bearing capacity (kN)	Total capacity (kN)
P020	8456	4483	12939
P013	5945	6602	12547
P014	9476	7112	16589
P004	15078	10281	25359
P023	6180	8211	14391

Table 5 (cont.)

Pile	Total skin friction (kN)	End bearing capacity (kN)	Total capacity (kN)
P003	18227	7299	25526
P012	13783	5641	19424
P050	19080	12125	31206
P049	5997	6871	12868
P046	14401	16589	30990
P124	23681	8731	32412
P101	17746	7819	25565
P121	19561	7534	27095
P114	17766	8260	26026
P140	9967	9163	19130
P126	21072	4199	25271
P109	16000	9339	25339
P151	14568	9251	23819
P130	13351	13606	26958
P132	23691	13165	36856
P133	15206	14529	29734
P150	13312	6573	19885
P074	6720	2727	9447
P040	13234	17383	30617
P061	18070	7161	25231
P015	11252	7495	18747
P010	18158	7269	25428
P165	5189	4189	9378
P166	5474	3571	9045
P131	20238	10899	31137
P148	20022	5935	25957
P088	18080	8780	26860
P093	16245	9006	25251
P078	13371	13705	27076
P094	14735	11223	25957
P064	18335	8348	26683
P042	15078	13508	28586
P032	8466	11811	20277
P033	17256	14558	31814
P045	11919	17658	29577
P043	15441	10556	25997

3.3.2 Static Load Test (SLT) Results

As the second field testing method, the field skin friction and end bearing capacity levels were estimated using the method described under 2.4.2. The values are presented in Table 6.

Table 6 SF and EB capacity levels acting on piles given by Field SLT Test Results

Pile	Total skin friction (kN)	End bearing capacity (kN)	Total capacity (kN)
P050	15696	9810	25506
P014	7995	4513	12508
P042	11772	6867	18639

3.4 Comparison of Theoretical Results with Field Test Results

The Table 7 summarizes the total skin friction levels (both soil as well as rock socket) acting on respective piles, estimated using theoretical methods against actual field observations.

Table 7 Comparison of theoretical SF capacity levels against field SF capacity levels

Pile	Total safe theoretical skin friction (kN)			Total Skin Friction from PDA (kN)	Total Skin Friction from SLT (kN)
	Burland	ICTAD	O'Neil & Reese		
P020	3041	2990	7300	8456	
P013	1966	1846	5179	5945	
P014	1904	1726	4881	9476	7995
P004	3134	3139	7901	15078	
P023	2047	1809	5199	6180	
P003	3139	3178	8148	18227	
P012	2534	2462	6559	13783	
P050	2445	2205	7312	19080	15696
P049	2856	2687	8824	5997	
P046	2815	2936	8897	14401	
P124	2604	3017	8730	23681	
P101	3678	3277	10693	17746	
P121	3936	3376	10978	19561	
P114	3865	3255	11679	17766	
P140	3834	3240	10754	9967	
P126	4133	3556	11309	21072	

Table 7 (Cont.)

Pile	Total safe theoretical skin friction (kN)			Total Skin Friction from PDA (kN)	Total Skin Friction from SLT (kN)
	Burland	ICTAD	O'Neil & Reese		
P109	4204	3225	11049	16000	
P151	2889	2611	6713	14568	
P130	3068	2797	7091	13351	
P132	3458	3116	6789	23691	
P133	2990	2638	5844	15206	
P150	2893	2611	6905	13312	
P074	2393	2351	5951	6720	
P040	3165	2897	7637	13234	
P061	3269	3067	7382	18070	
P015	2380	2250	6330	11252	
P010	3177	3451	7743	18158	
P165	2140	2142	5785	5189	
P166	2018	1962	5912	5474	
P131	3178	2935	8773	20238	
P148	3282	3045	8531	20022	
P088	3359	3202	8695	18080	
P093	3053	2803	8172	16245	
P078	3255	3029	8178	13371	
P094	3357	3202	8691	14735	
P064	3458	3329	7643	18335	
P042	3404	2864	7588	15078	7995
P032	2948	2578	6007	8466	
P033	3095	2408	6022	17256	
P045	3376	2812	8349	11919	
P043	3463	2957	6486	15441	

4 DISCUSSION

When comparing the results of theoretical concepts with field test results, the Burland method found to provide most conservative values compared to other two. The field PDA results are higher as much as 210% to about 910% of the skin friction values given by Burland method. Same trend was seen with SLT results and these values are 235% to 642% of Burland skin friction levels.

Even though ICTAD method shows a lesser conservativeness compared to Burland method, the values are unacceptably lower compared to field values. PDA and SLT values are of 223% to 865% and 280% to 712% respectively of the ICTAD method generated skin friction capacity levels.

O'Neil & Reese method provides the least conservative friction levels compared to other two. Apart from pile P049 (where field results are less than theoretical O'Neil &Reese values), the PDA results are of 90% to 350% and SLT results are of 105% to 214% of the O'Neil & Reese method generated skin friction values. Hence it is evident that O'Neil &Reese method provides the least conservative estimate compared to other two methods, with reasonable margin with ultimate skin friction. Skin friction values from this method are 185% to 335% of Burland and 220% to 360% of ICTAD methods. Comparatively lower values of Field test (PDA) results in piles P049, P140, P165 and P166 may be due to other associated quality factors during the casting of piles.

When considering the end bearing capacities, it is very difficult to compare field load test results with theoretical results, mainly due to two reasons.

1. The bedrock at the particular site is fractured and weathered to a considerable depth and thickness of this incompetent zone is highly variable within shorter distances. Hence the bedrock profile of the pile location may be completely different from that of the nearest reference boreholes. Therefore, it is unreasonable to compare the theoretical results with field test results.
2. In field tests, loading are carried out generally only up to 150% of the working load and behaviour of pile is only studied up to this limit only. The bearing component reflects only to this limit and to have an idea about the ultimate level of end bearing, it will be necessary to impose much higher percentage of the working load.

5 CONCLUSIONS AND RECOMMENDATIONS

Following conclusions and recommendations can be made based on the outcomes of this study.

1. When considering the skin friction distribution along the pile, even though theoretically Burland method initially considers the overburden effective stress, later it ignores this effect by the critical depth factor. Again, when estimating the shear strength parameters for the same method using correlations with in-situ test data, it underestimates local soil shear strength parameter levels.
2. In ICTAD method, relies only on SPT'**N**' values and does not consider the confinement effects of overburden effective stress as in Burland method. Even though field SPT'**N**' values reflect this overburden effect later these values are corrected for overburden effect during the determination of $N_{corrected}$. However, unlike in Burland method, a second type underestimation of local shear parameter conditions does not occur in this method and thus slightly better results are produced by this method.
3. As mentioned above the application of O'Neil &Reese method produces least conservative and most practical results compared to Burland and ICTAD methods. It may be mainly because it directly considers the depth from surface to particular layer

and thus the confinement effects of overburden effective stress levels.

4. The application of Burland and ICTAD methods will be useful after carrying out detailed studies on applicability of empirical relationships between in-situ test values and shear strength parameters for local conditions. Even the applicability of critical depth factor on locally available highly permeable coarse grained residual soils should be investigated.
5. Even though it has been proven that the O'Neil &Reese method best suits the local conditions, it should be emphasised that adoption of high quality construction techniques and monitoring is essential as it creates only marginal space for errors compared to other two methods.
6. To have a better understanding about merit and demerits of these methods, a detailed study with an instrumented pile testing program is essential. In addition, this study was conducted for a case where subsurface composed only of residual sands, without compressible clays. Therefore, it is recommended for future studies to investigate the applicability of O'Neil &Reese method for complex geotechnical conditions, where negative skin friction comes into the picture.

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Cost Effective Bus Route Information System

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Abstract – *At present, there is no Bus Route Information System operational in Sri Lanka. Even though this is fairly common in other countries, the initial high capital investment required has delayed the introduction of such a system locally. A cost effective solution is the only way to speed up such service in a developing country like Sri Lanka. This paper describes our prototype implementation of a cost effective real time Bus Route Information System for public transport system. This system provides related information such as route number, type of the bus service whether A/C, non A/C, semi luxury, emergency alerts and bus arriving time to the commuters during before and after transit period which helps passengers to take effective transit decisions. Passengers can access this bus information from on street displays at bus stops. Cost effectiveness comes from the semi-automated nature of the solution which requires a driver to be pro-active in providing status information. This contrasts with the high capital cost associated with the deployment of a fully automated GPS based solutions. Displays at the bus stops will be updated according to the received information from each and every driver interface devices.*

Keywords*: Bus Route Information System (BRIS), Driver Interface and Display Unit (DIDU), Bus Information Display (BID)*

1 INTRODUCTION

In Sri Lanka, there is no real time bus route information available to the commuters. Due to the lack of a proper bus monitoring system, bus drivers tend to take buses according to their own schedules. Even if there are predefined timetables driver may not adhere to due to reasons like traffic jams, excessive stoppage at bus stands to collect more passengers to meet the revenue target for the day etc. This causes increased waiting time and an uncertainty in bus arrival for the commuters.

By introducing a proper bus monitoring system, the government can offer a better and efficient bus service for the commuters. Proposed system has two main interfaces, namely, Driver Interface and Display Unit (DIDU) and Control Centre and Bus Information Display (BID). DIDU is used to transmit information via Short Message Service (SMS) to the control centre and BID unit which is fixed at the bus stops. These display panels receive information from DIDUs and panel flashes bus route number, expected time of next bus arrival, bus service type (AC / non AC / semi luxury etc.) and emergency alerts which are helpful to bus passengers to plan their journeys. Compared to existing Bus Route Information Systems (BRIS), proposed system uses a very low cost communication method which is suitable to our country. This system is operated using SMS technology and it is very easy to implement and expand the network. The rest of this paper is organized as follows: Section 2 is the literature survey related to this project. Section 3 gives a detailed description of the project with design details. Section 4 provides the implementation and testing results and finally the conclusion and future work.

2 LITERATURE SURVEY

Today many countries use BRIS to offer better service to passengers (Bangare et al., 2013; Ganesh et al., 2012; Nagaraj et al., 2011; Next Bus 2013). These systems make use of GPS technology and sophisticated software to track buses along their route and calculate their arrival time for specific stops. Unfortunately in Sri Lanka there is no such a system for passengers resulting in many difficulties due to uncertainties associated with the bus arrival times.

The passenger transportation service can be interrupted due to traffic variations and it is a daily problem faced by all transit providers. **NextBus** (Next Bus 2013) is an advanced vehicle tracking system which uses global positioning satellite information to predict when the next bus will arrive at any given bus stop, thereby eliminating waiting times and any need for schedules for all transit riders. NextBus was designed to keep customers on schedule even if their bus or train in Singapore. Without NextBus, the commuting experience of many passengers is disappointing because of the difficulties in predicting the arrival of the next bus. A GPS tracking unit can determine the precise location of any individual or vehicle carrying the GPS receiver. Data about location and other aspects can be stored in various forms, depending on the type of tracking unit. Tracking is accomplished through the use of Global Position System (GPS) receivers. Data communications provided through the use of Cellular Digital Packet Data (CDPD), a public data network. GPS provides the basic location (latitude and longitude) of the vehicle. CDPD transmits the location, vehicle ID, current route assignment and other data to the tracking system.

The core of the NextBus system is the predictive servers located at the NextBus Network Operations Center, or NOC. Hosted in secure facilities, these servers perform all of the calculations needed for predictions, serve up the data to riders and agencies, and communicate with the vehicles. Communication with the vehicles can be by a variety of methods from private radio to public data networks. The servers also provide the web pages seen by agencies and riders, as well as host the SQL database with all of the information needed to generate reports, maps, and predictions. This system is highly advanced and country like Sri Lanka cannot afford this kind of a system due to the high implementation cost.

3 SYSTEM DESCRIPTION

Proposed system consists of two interfaces, Driver Interface and Display Unit (DIDU), Control Centre and Bus Information Display (BID) (Refer Fig. 1). DIDU is used to transmit information via SMS to the control centre and BID unit fixed at the bus stops. These display panels consists of GSM modem to receive information. It flashes bus route number, expected time of next bus arrival, bus service type (AC / non AC / semi luxury etc.) and emergency alerts which are helpful for the bus passengers to plan their journeys.

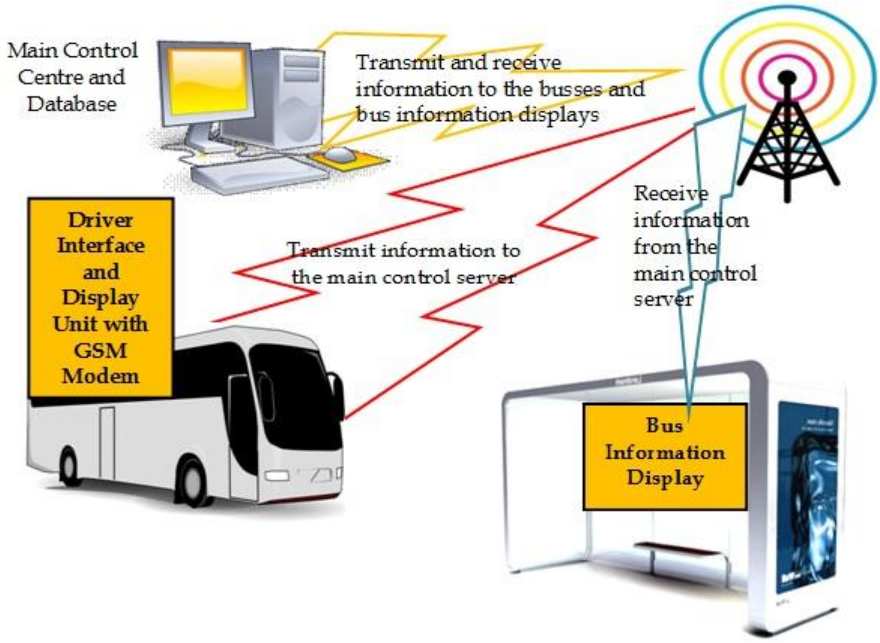


Fig. 1 Proposed system

3.1 Methodology

This BRIS, by design, facilitates staged implementation and can be easily extended to any number of bus stops. The prototype system is developed only for few bus stops. Data transfer between control centre and DIDU is done using a GSM modem. Data transfer between control center and BID at the bus stop is also done by the same technique. AT89S52 micro controller is used as the main control centre and database instead of a PC. It acts as a small PC which can be configurable to communicate with bus information display. Each display consists of AT89S52 micro controller to filter the received information.

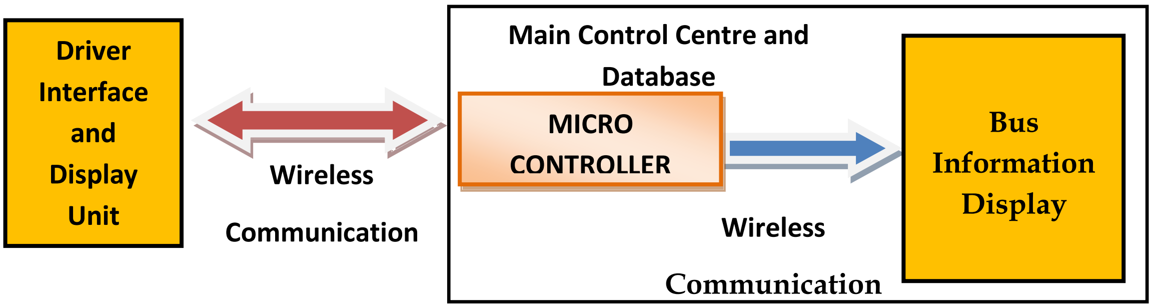


Fig. 2 Communication diagram of the proposed system

3.2 System Interfaces

This system is not based on the GPS and tracking of the vehicle is done by the DIDU. Initially some data is stored in two systems, DIDU and BID. Driver must drive the bus according to the information (Time table) in the driver interface and display unit. If the driver cannot reach the bus stop at the given time, he can alert the system by entering the expected new bus arrival time. Displays at the bus stops will update according to the received information from the DIDU from buses.

3.3 Driver Interface and Display Unit

3.3.1 Features and functions of the DIDU

a. Bus driver can alert the system whether bus is at the bus stop or not by pressing the Arrived / Delay button in the keypad (Fig. 4). This information helps to track the location of the vehicle and the driver needs to press this button each and every bus stop.

b. An emergency breakdown situation, bus driver can alert the system by pressing the breakdown button on the interface.

c. Bus driver can alert the system about the current bus stop number by pressing the 4x4 keypad.

d. LCD displays the next bus stop number and the time for the next bus stop. Bus driver must drive the bus according to this pre defined information.

e. If bus driver cannot reach the bus stop at given time, he can approximate the delay time and send delay time to the BID. If the driver cannot predict the delay time, he can update the display in the bus stop as bus is going to be delayed (In case of high traffic situation).

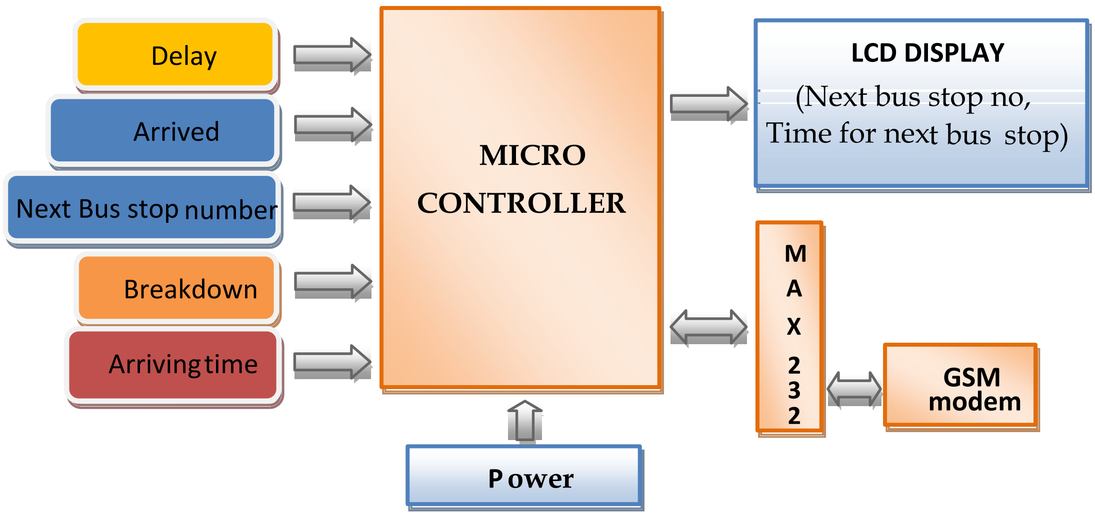


Fig. 3 DIDU block diagram

3.3.2 Instructions which needs to be followed by the driver when operating DIDU

Step 1: Switch on device

Step 2: Wait until “Enter Info:” appears

Step 3: Enter “0” in keypad; this will select the appropriate scheduled timetable which needs to be followed by the driver. Driver must drive according to the information on the LCD. Wait until the time appears for bus stand and drive the bus to the bus stand

Step 4: Wait until display appears "Enter Next Bus Stop Number:". Enter next bus stop number, driver needs to enter next bus stop number as 1,2,3,4…. sequence. If he avoids the sequence, system will not operate until he enters the correct next bus stop number.

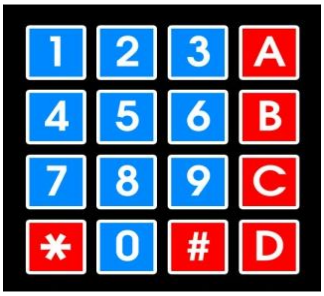
Step 5: Wait until display appears bus stop number and time for the next bus stop. For an example "BS 1 at 7.10pm" and "ACHIEVE Y/N?". If the driver can reach the next bus stop number at the scheduled time, driver needs to press the "YES (*)" button on keypad. Then information is sent via a SMS to the next bus stop. Driver needs to drive the bus according to the scheduled time. Sent information is always displayed on the display like "Info. Sent: Time", driver needs to drive the bus according to the information on the display. Second line of the display will appear as "ACHIVE Y/N?" again, which means driver can approximate the time again when he is unable to attend the bus stop at a given time. For that, driver needs to press the "NO (#)" button on the keypad and wait until "Approx Time:" appears. Enter approximate time and press "SEND INFO. (C)" button on the keypad and wait until "Info. Sent" appears. Driver can alert the system using above mentioned procedure at any time when he is unable to go to the bus stop at a scheduled time.

Step 6: When bus is arrived at the bus stop, driver must press the "Arrived" button (A) on the keypad to alert the system that the bus is at the bus stop. Display at the bus stop will update as bus has "Arrived". Wait until display appears "Enter Next Bus Stop Number:"

Note: Driver needs to enter the next bus stop number only when he is ready to go to next bus stop. Above steps repeated again and again (Step 4 and 5).

Step 7: Driver can alert the system by pressing the "BREAKDOWN (B)" button on the keypad when bus breakdown or meet with an accident. This information is transmitted to the next bus stop as "BREAKDOWN". This information will help passengers to take any other alternative decisions. If driver can fix the trouble, he can continue his journey again by updating the next bus stop number and approximated time.

Step 8: In high traffic situations, when driver cannot approximate time for the next bus stop, driver can alert the system by pressing the "ARRIVED / DELAY (A)" button on the keypad when display shows the "Approx Time:" where the bus is going to be delayed. Display at the bus stop will update as "DELAY" and this information will help passengers to take any other alternative transport method.



0-9 : Used to input next bus stop number and delay information
A : ARRIVED / DELAY
B : BREAKDOWN
C : SEND INFO.
D : " : "
***** : YES
: NO

Fig. 4 4x4 keypad arrangements

3.3.3 Flow chart – DIDU

The flow chart presents the operation of the DIDU. The semi-automated nature of the system is seen as there are several instances which the driver needs to interact with the system.

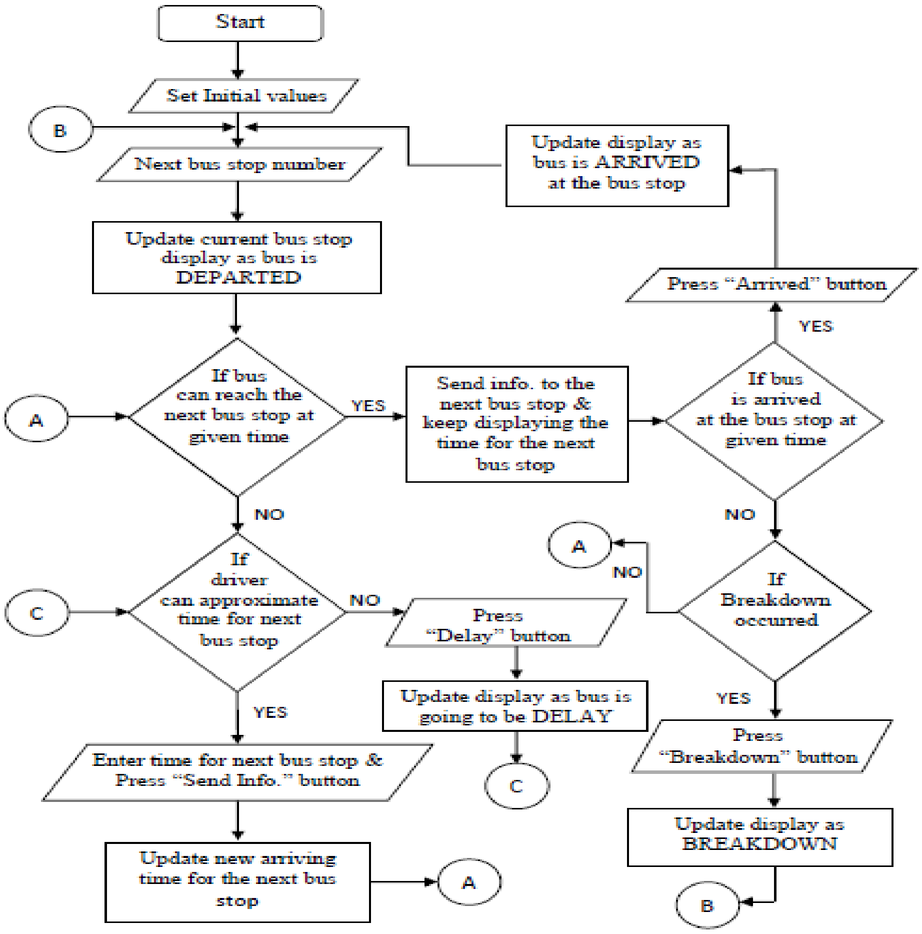


Fig. 5 Flow chart of DIDU

3.4 Bus Information Display Unit

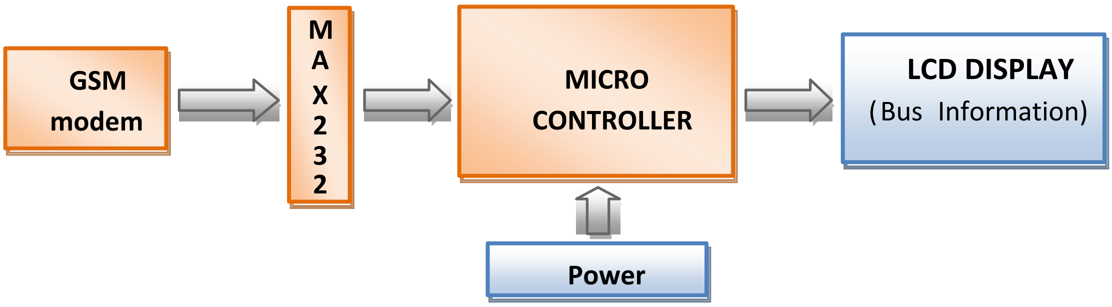


Fig. 6 Block diagram of BID unit

Microcontroller is programmed with some pre defined values and it displays bus arriving time. Display updates every time when it receives new information from the DIDU. If no information is received, display will update according to the initial set values. Every display circuit is unique; DIDU will identify each bus information display using the assigned number (GSM modem SIM card number). It is very easy to setup this display at the bus stop. It is needed to store assigned bus stop number to the DIDU database when scheduling a time table.

Example of a bus information display:

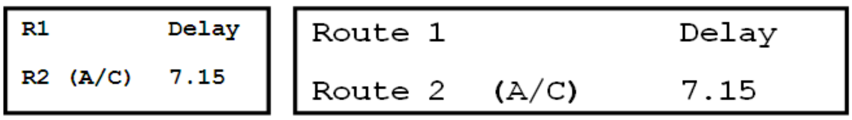


Fig. 7 Example of BID unit

3.4.1 Flow chart – BID Unit

The flow chart in Fig. 8 presents the operation of the BID unit.

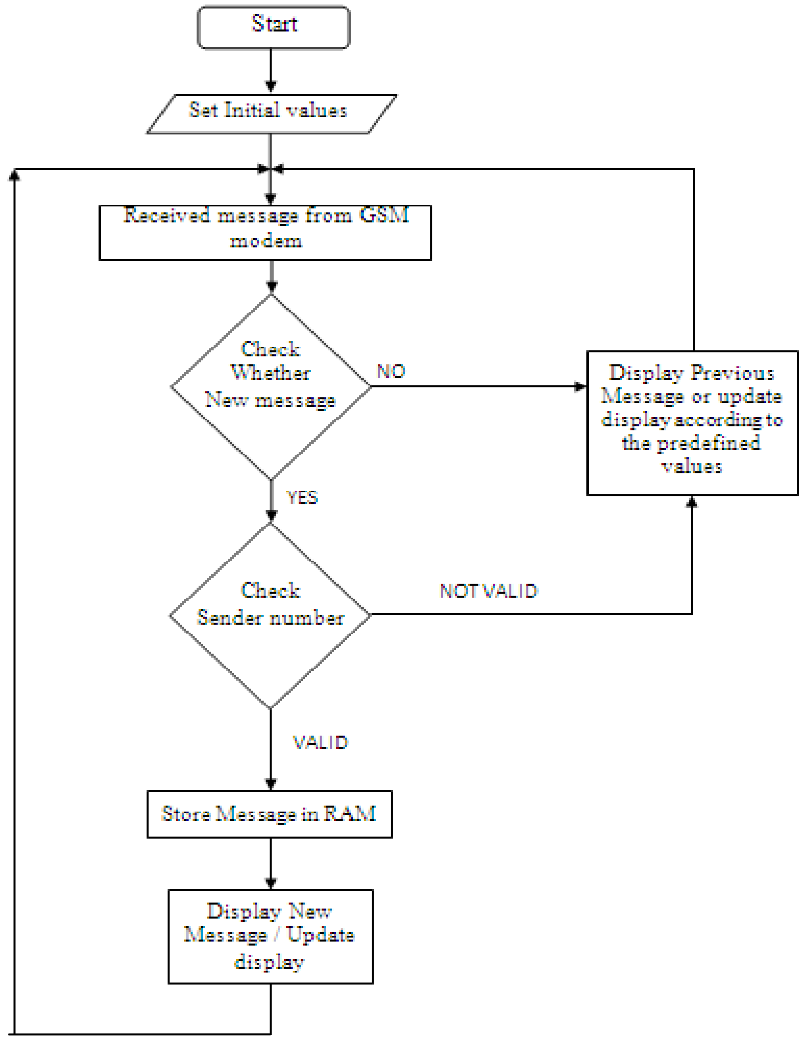


Fig. 8 Flow chart of BID

4 IMPLEMENTATION AND TESTING

This section presents the implementation and testing of the prototype BRIS.

4.1 Software used for implementation and testing

Keil µVision 3 Software (Keil µVision, 1997-2009) – C51 development tools, ISIS Simulation Software were used to develop the prototype BRIS. ISIS software provided the means for the design and simulation of the prototype system.

4.2 Implementation Setup (Fig. 9)

By design, BRIS implementation is in stages to facilitate easy expansion. The system can be extended to any number of bus stops. However, the prototype system was developed only for few bus stops and two routes (Route 1 and Route 2). One actual DIDU and one BID unit have been designed instead of designing many duplicated units. Isis Proteus Professional 7.2 software was used to implement virtual DIDU and virtual BID.

Each DIDU and BID has a unique number (SIM card number of the attached GSM modem) to identify each device in the network separately. For the implementation setup, bus stops are numbered at route 1 and route 2 from 0 to 6. Busses are travelling from bus stop number 0 to 6. Network will identify each device from a unique number, so that there should be a proper numbering plan as in Table 1. It can also reduce the 10 digit numbering plan to 4/5 digit numbering plan by attaching this system for a database which also helps passengers to remember bus stop number easily.

Table 1 Example of network numbering plan

(a) BID numbering plan

Bus stop number	**4 Digit number display at bus stop**	**BID GSM modem sim card number**
0	1000	07xx001000
1	1001	07xx001001
2	1002	07xx001002
3	1003	07xx001003
4	1004	07xx001004
5	1005	07xx001005
6	1006	07xx001006

(b) DIDU numbering plan

Bus Route No	**DIDU GSM modem sim card number**
Route **1** (Bus 1)	07xx100001
Route **2** (Bus 1)	07xx200001

(c) Numbering plan of Route 1 assuming it has more than one bus.

Bus Route No	**DIDU GSM modem sim card number**
Route **1** (Bus **1**)	07xx**1**0000**1**
Route **1** (Bus **2**)	07xx**1**0000**2**

Such numbering plan can be developed to this network by grouping numbers according to the type of the bus, bus routes and location of the bus stop. This will be helpful when expanding the network in future. DIDU and BID needs to be configured with bus arriving time schedules (timetables) and permission according to the above numbering plan.

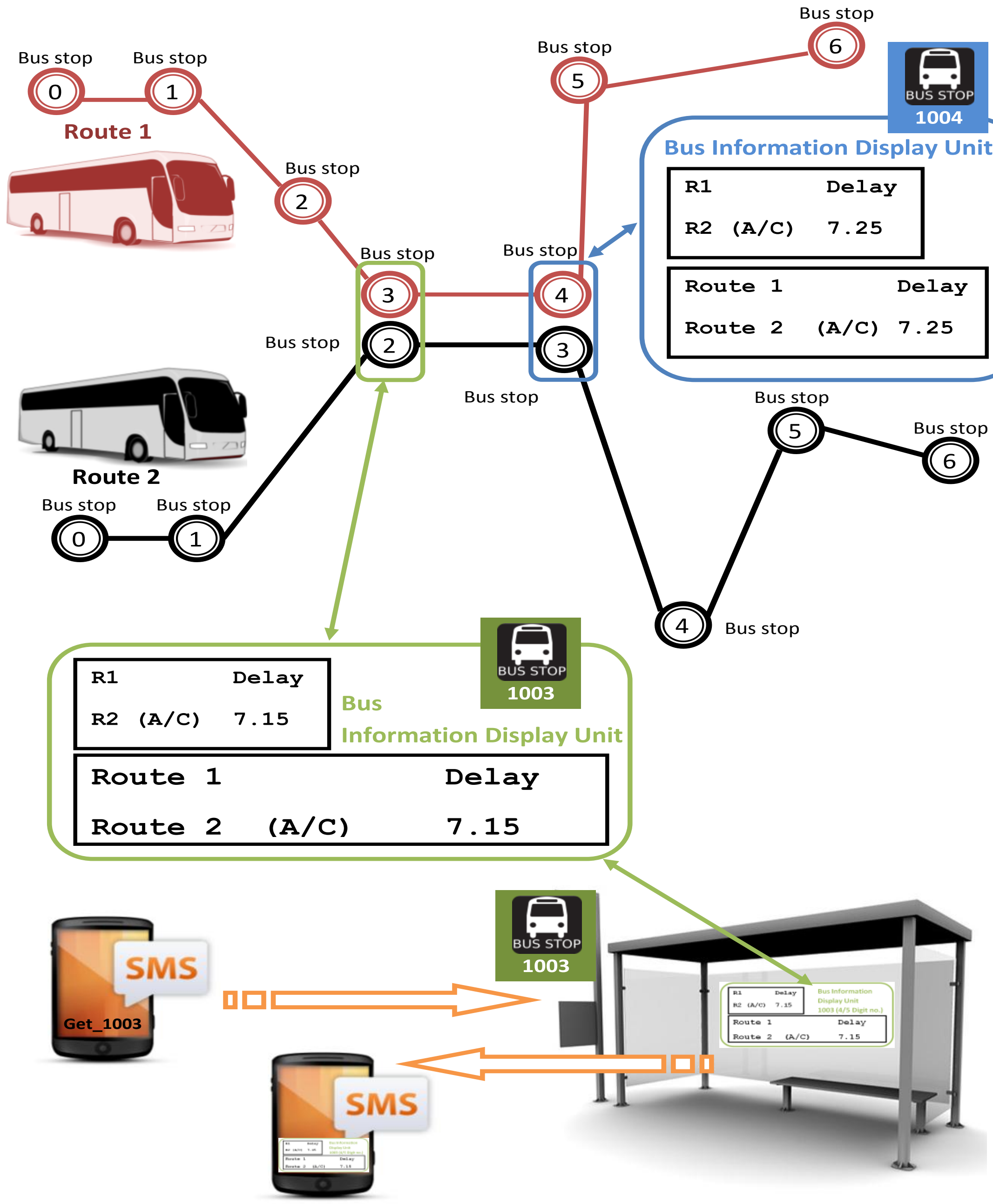
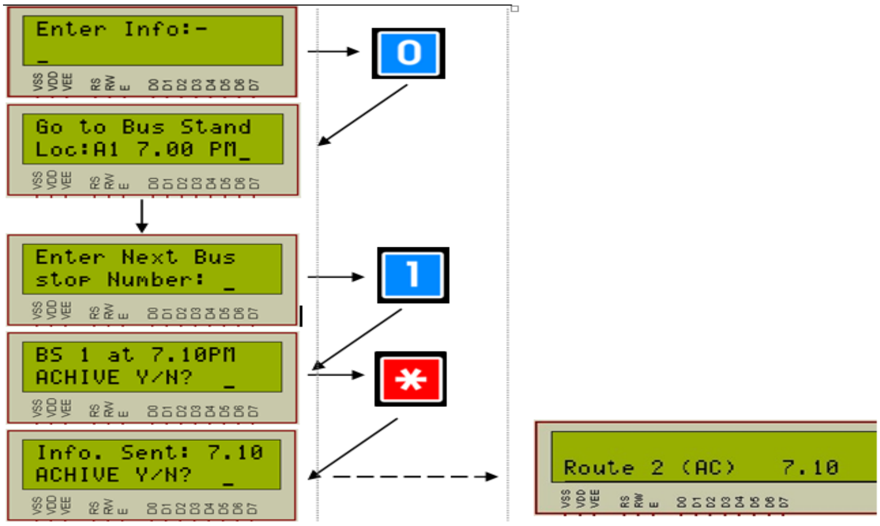


Fig. 9 Implementation setup

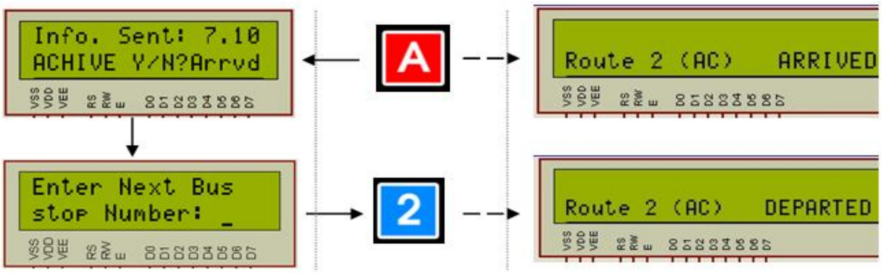
According to the Fig. 10, Route 1 and Route 2 busses go through bus stop 1003 and 1004. BID units display information as in Fig. 10. Passengers can get information which is at the bus stop by sending a SMS with the bus stop number (Ex. Get_1003) to the BID from their cellular phones. This network environment can be customized according to the preference of users.

4.3 System testing and results

Driver Interface & Display Unit		Bus Info. Display at bus stop
LCD status	Keypad entry	

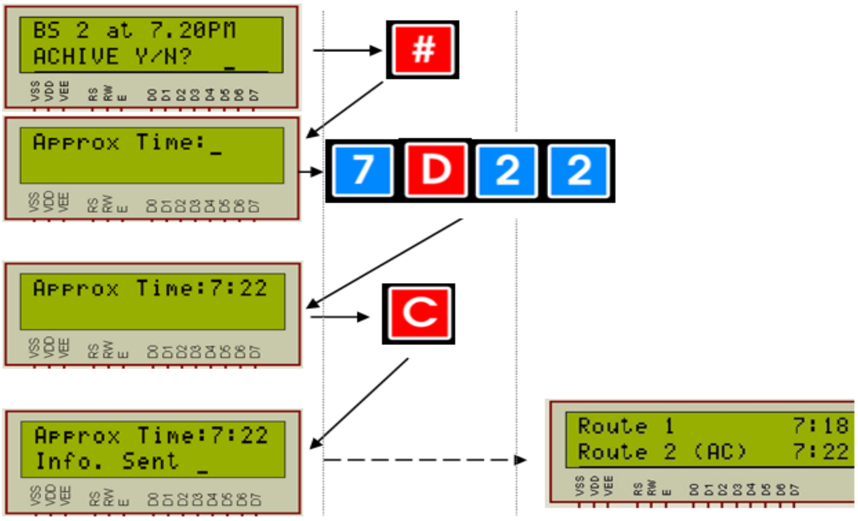


When bus arrives at the bus stop



Note: Enter next bus stop number only when bus is ready to go to next bus stop

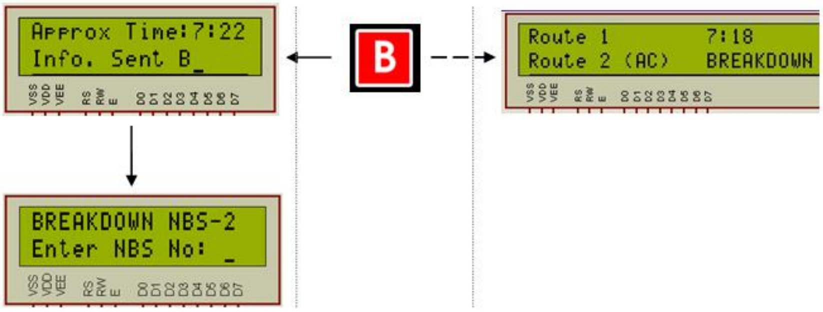
When driver is not able to reach the bus stop



Driver can approximate time and update display at the bus stop.

Driver Interface & Display Unit		Bus Info. Display at bus stop
LCD status	Keypad entry	

When breakdown occurs



Note: Driver can continue the journey after fixing the trouble by entering the NBS (Next bus stop) number

High traffic situation (when driver cannot approximate time)

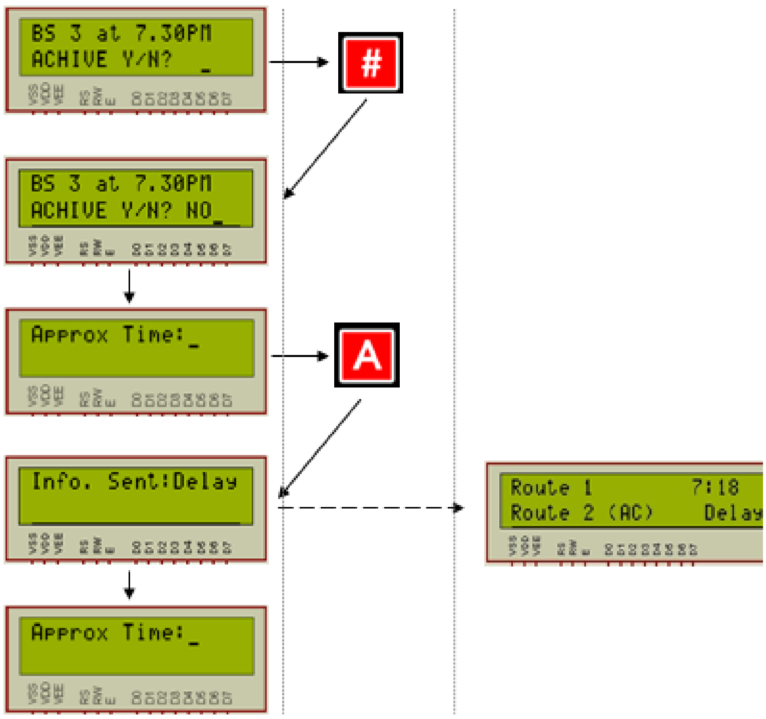


Fig. 10 DIDU and BID - virtual terminal information flow observations

5 CONCLUSIONS AND FUTURE WORK

This paper addresses an issue which touches the lives of many daily commuters who make use of the public transportation network, namely, non-availability of accurate arrival/departure information of buses and the resulting accumulated economic loss due to unnecessary wasting of time on the road. A Bus Route Information System with on-street displays increases the perceived reliability of the transport system because of the dynamic updating of arrival/departure information and the indication of any delay. The direct management benefit of this system is that it can be used to identify bottlenecks in the transportation network. The information gained from the system can be used in combination with passenger data to carry out better planning of the public transport network according to the needs of its users.

Compared to existing NextBus information system, this system is operated using very low cost SMS technology and it is very easy to implement and expand the network. One of the main advantages of this system is that establishing a new network structure is not needed for communication between the devices used in this system. This system can use the existing network structure which is used for SMS technology and it is very cost effective than establishing a new GPS network or RF structure. Proposed system is also capable of informing any breakdown or delay information to the people who are waiting for the busses at the bus stop. This information helps to take alternative decisions to the passengers in case of any emergency. This system can be linked to a bus priority system, which can speed up journey times, resulting in increased efficiency and lower costs. Bus information displays at bus stops can also be configurable to display advertisements and notices. Government or any organization can use this method to take an extra income from this system.

With the accuracy of the system increases the number of complaints is likely to decrease. The stress experienced by passengers due to uncertainty of bus arrival times will get reduced as they can access the information about the arrival times of the bus in advance. Since drivers know that they are being tracked they are less inclined to start their services early/late or deviate from the route.

In future, this system can be combined with a database which helps passengers to get information which is at the bus stop displays via SMS or web browsing by using their own cellular phones. This will help passengers to get information without going to the bus stop. By interfacing a voice recognition system to the DIDU it can reduce the risk factor of entering the information while driver is driving the bus. This system can be further improved to display the information of number of available empty seats in the bus. This information will help passengers to take alternative decisions and alternative transport method in case of high crowded situations.

This bus information system can also be applicable to other transport systems like railway transportation system, etc. Transport authorities can offer better and efficient transport services to the passengers by combining all these types together. Importance of this kind of information system is very essential to our country as the existing public transportation system can be greatly improved and benefit of using this system will be credited to both passengers and the government.

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Prospects of Using Geosynthetic Materials for Disaster Mitigation – A Case Study

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Abstract – *Natural disasters result in death, economic and environmental damages, and severe impediments to social development. They affect the economy immediately and directly, as well as having a long-term impact on the social life of the community. Some of the major natural disasters that have made severe damages with respect to loss of lives and property include floods, windstorms, earthquakes, and droughts. In the recent years geosynthetics have been successfully used, especially in the developed countries, for the mitigation of natural disasters such as floods, landslides, rock-falls, debris flows and avalanches. This paper discusses the possibility of using geosynthetics in mitigating natural disasters such as floods and earthquakes from a South Asian scenario with Pakistan as the case study. The work involves reviewing the nature and frequency of these natural disasters; identifying the areas where geosynthetics can play a vital role and juxtaposing the existing literature on geosynthetics for disaster mitigation with the risk situation. Conclusions are drawn emphasizing the seminal nature of the work and outlining the need for more focused studies on the use of geosynthetics for disaster mitigation in the South Asian context.*

Keywords: *Geosynthetics, Geotextiles, Natural disaster, Technical textiles, Disaster mitigation*

1 INTRODUCTION

Natural disasters lead to losses of human lives and property and adversely affect the national exchequer. Natural disasters are often frightening and difficult to understand, because humans have no control over when and where they would happen. What can be controlled is how prepared people are as communities and governments to deal with the dangers that natural disasters bring.

According to the International Disaster Database (EM-DAT), in the year 2012 alone 9330 people lost their lives due to natural disasters and some 106 million people were affected by disasters (Ferris, et al., 2013). Based on the statistics from the International Strategy for Disaster Reduction (ISDR), an organization headed by the United Nations Secretary for Humanitarian Affairs, between the years 1970 to 2005, the most common ten natural disasters are flooding (30.7%), windstorm (26.8%), epidemic (11.2%), earthquake (8.9%), drought (7.8%) landslide (5.1%), extreme temperature (3.5%), wild fire (3.4%), volcanoes (3.4%) and insect infection (1.0%) (Wayne, n.d.)

According to the statistics provided by the US Geological Survey between 2000 and 2012, around the world 813,856 have died due to earthquakes. According to a new study that has been published, it is predicted that 3.5 million people will have died in catastrophic earthquakes between 2001 and 2100 (Oskin, 2013). In a study published in 2009, scientists calculated that an earthquake with a million fatalities could be expected once a century if the world's population reaches 10 billion, as the United Nations predicts will happen in 2083 (Oskin, 2013). Four catastrophic quakes (those that kill 50,000 or more people) have already hit since 2001.

In many countries immediately after a natural disaster the authorities activate a Centralized Disaster Management System, where much of the work done is focused on disaster recovery and response and creation of awareness to deal with the disasters. Even though in some of the countries, certain amount of work is focused on reducing vulnerability through sustainable construction methods and materials, in only very rare occasions use of geosynthetics have been considered for disaster-proof infrastructure designs.

This paper intends to fill this void and introduce geosynthetics as a key material for mitigating natural disasters, with Pakistan as the case study. Since the subject is of a wide scope owing to the different types of disasters, the variety of damages caused by each and the multifarious functions of geosynthetics, the paper follows a short listing methodology to arrive at areas where geosynthetics can help reduce vulnerability.

The methodology of review is detailed below:

- Natural disasters are beyond human control and cannot be fully eliminated; the phenomenon can only be mitigated by manipulating a particular component of a natural disaster. This component is identified and highlighted for further exploration.
- Natural disasters inflicting the world are of various kinds. Only those that are highly frequented and have caused considerable damage lie within the scope of this paper.
- Frequently occurring natural disasters cause damage by destroying and disrupting a number of infrastructures and systems on which a country operates. Only those infrastructures that are essential and can be fortified with geosynthetics form the scope of this paper.
- Some explanation and details are provided of how the shortlisted infrastructures could be fortified with geosynthetics and hence reduce their susceptibility to natural disasters.

2 INTRODUCTION TO NATURAL DISASTERS

United Nations defines disaster as a natural or manmade event that disrupts a society to an extent that it cannot reestablish or resume its functions with the existing resources (Department of Humanitarian Affairs, 1992).

Not every natural occurrence such as a flood or an earthquake can be termed as a disaster. For an event to be a “disaster” it has to incur “losses” be it of financial, human or environmental nature. The following equation illustrates this (Cyr, 2005).

Hazard + Vulnerability = Disaster

Hazard is an event that is *potentially* harmful to life, property or ecology but may not necessarily cause such losses. (Khan & Khan, 2008). These may be classified under three main heads:

1- Hydrological hazard: Floods, limnic eruptions, tsunamis
2- Meteorological hazards: Billiards, cyclones, droughts, heat waves, hailstorms, tornadoes
3- Geological hazards: Volcanic eruptions, earthquakes, avalanches

Vulnerability of a group of people refers to its extent of preparedness or otherwise against hazards (Guzman, 2003).

There are several factors that results in a populations' increased level of vulnerability. These include poor construction practices, inadequate disaster warning and monitoring systems, lack of awareness, proximity to hazardous terrains etc. (World Bank, 2009). In other words, a disaster occurs only when a natural hazard meets vulnerability; and since it is beyond the control of the human beings to effectively limit a geological, meteorological or hydrological activity (hazard), what humans can do is only mitigate disasters by limiting the vulnerability.

The role of geosynthetics in limiting vulnerability to failures in soil structures is well established. Here, the primary function performed by the geosynthetic is reinforcement. As a reinforcement agent, geosynthetics work in conjunction with a soil mass to improve its tensile strength and elasticity properties as compared to the corresponding properties of an unreinforced mass (Bathurst, n.d.). With reference to disaster mitigation, geosynthetics can be used in the construction of sustainable structures, thus reducing the vulnerability towards natural disasters. This is graphically illustrated in Fig. 1.

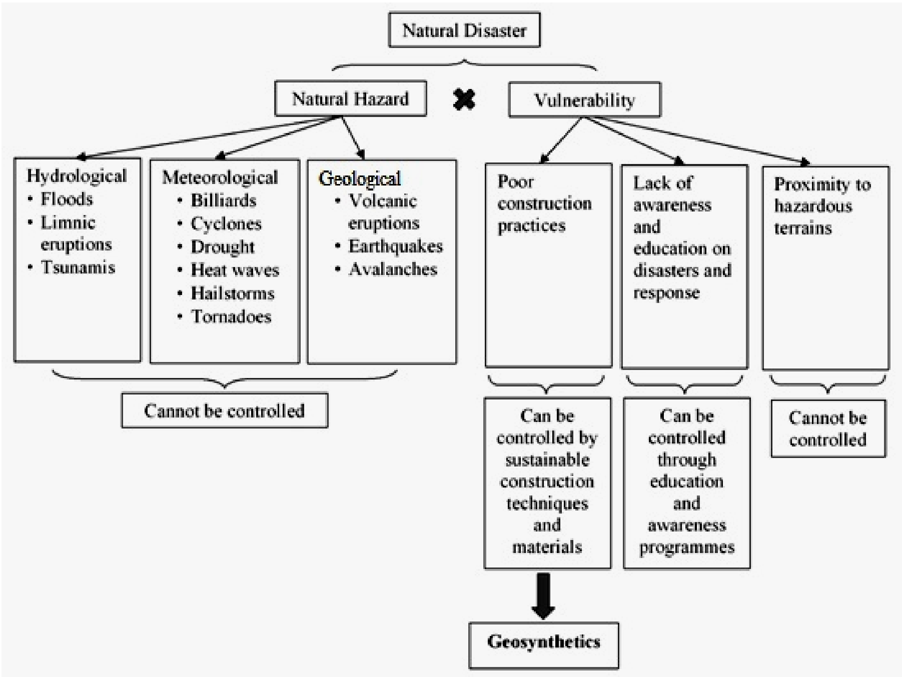

Fig. 1 Role of geosynthetics in disaster mitigation

3 THE PAKISTAN SCENARIO

Pakistan has an area of about 796,096 Sq. Km. Its topography consists of coastal beaches, sandy deserts, plateaus, plains, mountains, and snow covered peaks. The population is around 170 million of which 34% reside in urban areas and the rest in the rural areas. Like many other regions of the world, the areas comprising Pakistan are not free from the risk of natural disasters. It is highly susceptible to floods and, due to its closeness to the tectonic plates of Indo-Australia and Eurasia, is also at great risk of earthquakes (World Bank, 2009).

Table 1 shows the frequency of the top five frequented natural disasters that have inflicted Pakistan from the year 1954 - 2004.

Table 1 Frequency of natural disasters in Pakistan (%)

Natural Disaster	Frequency (%)
Floods	33
Earthquakes	18
Cyclones	16
Extreme Temperature	12
Landslides	10

Source: World Bank (World Bank, 2009)

Though Table 1 clearly points out floods and earthquakes as the top two frequently occurring disasters in Pakistan, the statistics do not include two of the worst catastrophes of recent years; the earthquake of 2005 (GLIDE number: EQ-2005-000174-PAK) and the floods of 2010 (GLIDE Number: FL-2010-000141-PAK).

3.1 Earthquake of 2005 (EQ-2005-000174-PAK)

The earthquake that happened on 8th October 2005 in Northern region of Pakistan measured 7.6 on Richter scale and covered a total area of some 30,000 square kilometers. The number of people killed due to this earthquake was 73,330 (Idris, 2007). Two aspects of material and environmental damage that took place during this earthquake, namely destruction of buildings and failure of lands, are discussed below as a background to the later discussion of the use of geosynthetics as a disaster mitigation material.

It is estimated that 780,000 buildings were destroyed or damaged beyond repair due to the earthquake, majority of which were of unreinforced masonry (URM) (Earthquake Engineering Research Institute, 2006).

The earthquake also set off numerous landslides. Owen et al. (Owen, et al., 2008) have classified the failures resulting in these landslides as being largely located in six geomorphic–geologic–anthropogenic settings, one of which is associated with road construction. In fact, landslides largely occurred on the roads that were built along the slopes exceeding 50 degrees (Owen, et al., 2008). This also supports the earlier findings that road construction on slopes triggers landslides (Barnard, et al., 2001) (Keefer, 1984).

3.2 Floods of 2010 (FL-2010-000141-PAK)

The monsoon of 2010 brought unprecedentedly heavy rainfall in Pakistan. The rains caused unprecedented floods in Khyber Pakhtunkhwa province, followed by Punjab and Sindh provinces. The damages and losses due to the rains resulted in 1985 deaths with further 1.7 million houses being damaged and 20 million people getting affected (Pakistan Disaster Management Authority, n.d.).

The damages were even more exacerbated due to major failures in river embankments, prominently along the course of the main Indus river. These failures of embankments occurred in the left marginal bund of Taunsa barrage, Rangpur canal, Muzaffargarh canal, Jampur flood bund in Punjab, Tori flood bund, Ghouspur bund, Beghari Sindh feeder bund, old Ghora Ghat bund, Haibat loop bund, MNV drain, Khirther canal, Moolchand Shahbundar bund and Manchar lake in Guddu – Kotri downstream reach (Sindh) of Indus river (Pakistan Disaster Management Authority, n.d.).

Due to these breaches and failures nearby towns and villages were inundated killing people, destroying crops and causing enormous infrastructure damage. Such failures of embankments are unfortunately not new to Pakistan. For example, during the floods of 2005 (GLIDE Number: FL-2005-000158-PAK) a total of five dams gave way with the biggest of these being the 35 metre Shadikor dam (McCully, 2005), which failed and drowned the nearby areas (United Nations Institute for Training and Research, 2005). This dam was built in the year 2003 with a cost of 45 million Pakistan Rupees., but it failed because of overtopping, which is a condition where the water is so abundant that it overflows from the dam and then causes it to collapse.

4 GEOSYNTHETICS IN SUSTAINABLE CONSTRUCTION AND DISASTER MITIGATION

Geosynthetics hold immense potential for sustainable construction and disaster-proof infrastructure. Their ability to bond with earthen / soil structures and yet retain flexibility make them a viable choice for reinforcement purposes.

Furthermore, they are inert and environmental friendly. They are also inexpensive as compared to other reinforcing materials such as steel. This section deals with the possibility of deploying geosynthetics for preventing some of the damages / failures caused by floods and earthquakes. Table 2 gives a shortlist of some of the damages caused by earthquakes and floods.

Table 2 Damages caused by earthquakes and floods

Disaster	Damage
Earthquakes	Collapse of masonry structures and mud houses Seismic-induced landslides
Floods	Failure of levees and dams Overtopping of dams

4.1 Collapse of masonry structures

Masonry constructions are primarily designed to support vertical loads such as their own self-weight and that of their contents. They are weak against complex seismic forces. Seismic retrofitting refers to the concept of modifying the existing masonry structures to enable them to withstand seismic loads (Nanda, et al., 2011). Two ways in which geosynthetics can be employed for retrofitting are *isolation of the base* and *textile reinforcement of the walls*.

4.1.1 Foundation isolation

In conventional designs the buildings are fixed to the ground. The seismic forces due to ground shaking are transferred to the superstructure. The seismic forces then induce lateral forces in the building and cause a part of it to shear off. The probability of this failure mechanism can be greatly reduced by introducing 'foundation isolators' at the plinth level to separate the superstructure from the ground. A foundation isolator works by shifting the natural period of the building away from that of the earthquake and providing additional damping to absorb the energy (Yegian, et al., 1999).

Several studies (Nanda, et al., 2011) (Yegian, et al., 1999) (Yegian & Kadakal, 2004) (Kevazanjian, et al., 1991) (Xiao, et al., 2004) (Yegian & Catan, 2004) have demonstrated the effectiveness of geotextile and geosynthetics as base isolators for seismic hazard mitigation. Though the existence of other base isolators are all too well known, they pose problems that can be avoided with the use of geosynthetics. For example, steel sheets are expensive and lead to construction complications; graphite, grease, screened sand, dry and weight sand cannot be used for a long term as graphite can be affected by chemical, grease can be contaminated by debris, dirt etc., and sand gets crushed after the shock which will increase the frictional characteristics (Nanda, et al., 2011). There are different ways by which the geotextiles and geosynthetics have been applied as base isolators in building constructions. In one of the studies (Thurston, 2007), the proposed base isolation system consisted of 40mm thick sheet of polystyrene followed by two slip layers of Ultra High Molecular Weight Polyethylene (UHMWPE) and Typar™ geotextile. As there could be a slurry seep through the geotextile, another layer of polyethylene sheet is placed on top of the geotextile. On top of this polyethylene sheet the concrete is casted in-situ and clearances are provided around the slab perimeter to prevent soil passive pressure from resisting slab movement.

4.1.2 Textile reinforcement of the walls

Poly-functional Technical Textiles (against natural hazards) (Polytect, 2008) is an EU funded research project under 6th Framework Programme (Yegian, et al., 1999). The aim of the project was to produce multifunctional sensor-embedded textiles that would provide both reinforcing strengths and monitoring capabilities for masonry structures and geotechnical applications.

One such research product is *Seismic Wall Paper*. It consists of a textile interface between outer mortar / cement / plaster layer and the inner URM (stone or brick) (Zangani, 2010).

There are several advantages of using sensor embedded textiles for reinforcing masonry structures. These include high strength to weight ratio, ease of handling and speed of installation. At the same time, monitoring the structure during its lifetime (strain, cracks, temperature, etc.) is very important, in order to predict possible anomalous situations, such as diffused cracking caused by additional unexpected loads or seismic events, soil subsiding, etc. In this way, the real-time monitoring of the structure allows repairing in the early stage, avoiding possible successive retrofitting and reducing maintenance costs (Coricciati, et al., 2010).

4.2 Collapse of mud houses

While foundation isolation and textile reinforcement of walls could make masonry structures resistant to damage by earthquakes, the fact remains that a large majority of housing sector in Pakistan is composed of *katcha* house rather than the *pucca* house. *Katcha* house is made of dry stone walls, and mud walls. Earth, which is compacted dry or mud plaster is used in floor and roof (UN Habitat, 2010). The resistance of mud house against earthquake is very low. Over 200 mud houses collapsed in Dalbandin, Baluchistan in the earthquake of January 2011 (Kassim, et al., 2011). A similar incident had taken place three years earlier when in 2008 an earthquake of magnitude 6.4 in Baluchistan destroyed more than 15,000 mud-walled and timber homes because of their low resistance to earthquakes (Karwal, 2008) (Maqsood & Schwarz, 2010).

An easy way to increase the compressive strength and hence the earthquake resistance of the mud houses is to reinforce the mud with fibres. The flexibility of the reinforced structures is also greater than the unreinforced ones (Akinmusuru & Adebayo, 1981) (Binici, et al., 2005). The fibres that can be used are easily available fibres such as straw, coir, jute, wheat, maize and bamboo.

Natural fibres cause an overall reduction in weight and density of the material. Also, as the composite mass dries, the stresses are distributed across the entire mass of the material reducing chances of breakage (Becker, 2010).

4.3 Seismic-induced landslides in areas of road construction

Road construction is done by cutting large areas of earth/soil from the upside of the slope, and then using the cut mass as a fill on the downside.

Owen at al (Owen, et al., 2008) have stated that in 2005 Kashmir earthquake, landslides occurred in almost every section of the road that was built on slopes with an angle greater than fifty degrees. Furthermore, the fill sides of many of the roads failed because the soil mass was not integrated enough during the construction.

The failures of road embankment occur due to non provision of slope strengthening works. One of the techniques suggested for slope stabilization by Bukhari et al (Bukhari, et al., 2006) is the provision of geosynthetic liner technology. Geogrids are spread out horizontally on the fills and then hooked to the walls by means of pins.

In cases where landslides are induced by excessive rains or inadequate drainage, and in areas where soils are susceptible to spontaneous liquefaction or quick clay, the non-woven fabric helps mitigate landslide by not only generating the tension forces to enhance the overall stability of slopes, but also preventing the development of pore water trapped in clayey soils by providing horizontal drainage through the fabric (Choobbasti, et al., 2009).

4.4 Failure of levees and dikes

Levees fail because of poor design, substandard construction, poor maintenance, or the reduction of their channel capacity because of sedimentation of the riverbed. Levees also fail because their "design flood" is exceeded. Any flood larger than the design flood will most likely overwhelm the levee (McCully, 2007). However, this is not the case with geosynthetic-reinforced levees. They are known to hold their ground during severe overtopping and storm conditions resisting scour and erosion. The procedure is simple and involves placing geotextile tubes horizontally, end-to-end and then pumping them with sand slurry. The tubes are then covered with sand forming the core of artificial dune structure. Such levees have proven their worth in the past; the most notable example of their integrity is from the Katrina storm that inundated the town of New Orleans in 2005.

It was revealed in the post-Katrina analysis that the geosynthetic reinforced levees (St. Charles and Jefferson) remained intact in the face of severe storm, while all other levees were breached (Dendurent & Woodward, 2009) (Dendurent & Woodward, 2009).

4.5 Overtopping of Dams

Studies on the reasons of overtopping of dams show that the failure occurs due to such factors as the area on which the dam is constructed, the type and capacity of dam, properties of construction material etc. Interestingly, all these failures start in the downstream side of the dam and then proceed to affect the entire dam body. It is therefore obvious that to arrest dam failure at an early stage, protection of its downstream face is very important (Cazzuffi, 2000).

Geosynthetics are widely used to prevent erosion caused by over topping of the dam. They are a good reinforcement option since they are inexpensive and are easy to install as compared to concrete slabs. They are also durable, flexible and provide good aesthetic appeal.

Geotextiles and geosynthetics have been used for reinforcement of downstream face for protection from overtopping in the earthen dam, Lake-in-the-Sky, in United States (turf biomat), Maraval dam in France (PET Woven geotextile) and in Bass Lake dam, Trout Lake dam and Price Lake dam in USA again.

Based on the discussions, it can be deduced that geosynthetics can be successfully used in Pakistan and other countries with similar economic and social backgrounds, for various disaster mitigation applications. Figure 2 summarizes these areas through a flow chart diagram.

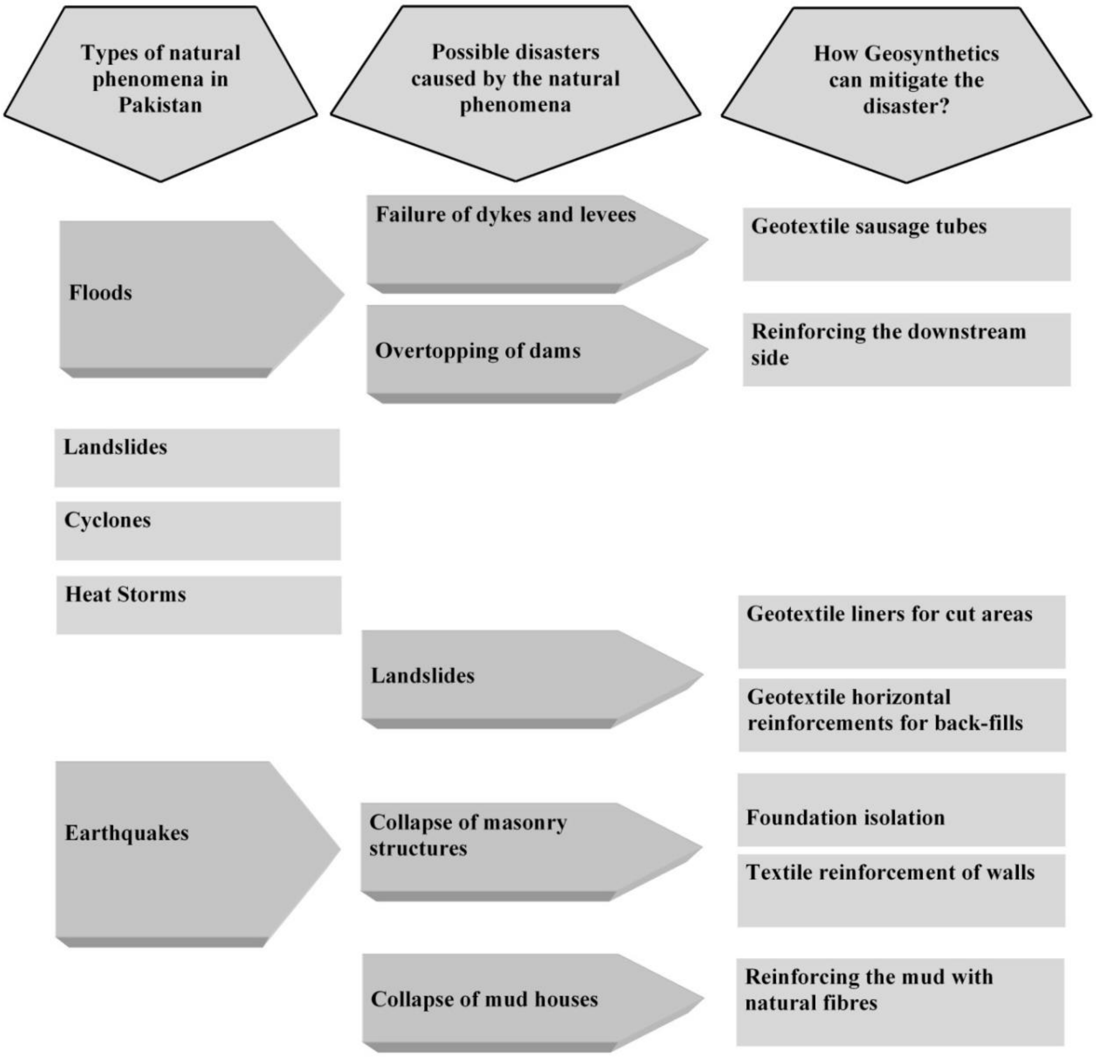

Fig. 2 Role of geosynthetics in disaster mitigation in Pakistan

5 CONCLUSIONS

Unfortunately many of the countries in the world, including those in the South Asian region are prone to natural hazards of various kinds including earthquake and floods. However the positive aspect is the fact that a natural disaster, as mentioned earlier, is a product of natural hazard and vulnerability. Therefore, it is apparent that if measures are taken to decrease the vulnerability by seeking and employing sustainable and affordable building construction and technologies for disaster-resistant infrastructure and design the disaster resulting out of the hazards can be greatly reduced. Geosynthetics have long been known as durable and easy to use reinforcement material. Their importance, in the wake of increasing susceptibility of the region to natural disasters, is more than ever before.

This article summarizes some of the areas in which geosynthetics and geotextiles could be used to mitigate the damage caused by natural disasters. Further focused studies need to be done in each of the specific areas discussed to accrue benefits by geosynthetics for a disaster-resistant South Asia.

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