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Use of Centrifuge-Based Physical Modelling for Understanding the Performance of Geostructures

B.V.S Viswanadham

Professor, Department of Civil Engineering, IIT Bombay, Powai, Mumbai-400076, India
E-mail: viswam@civil.iitb.ac.in

Abstract. Centrifuge-based physical modelling is widely adopted for understanding the performance of geostructures, like levees subjected to flooding and drawdown and geogrid reinforced soil walls subjected to seepage. In this paper, an attempt has been made to bring-out the advantage of centrifuge-based physical modelling to understand (i) the performance of levees subjected flooding using a custom designed and developed in-flight flood simulator at 30 gravities with and without chimney drain and (ii) the performance of geogrid reinforced soil walls with and without chimney drain subjected to seepage at 40 gravities. In both the cases, silty sand was used model soil and fine sand was used in chimney drain. All centrifuge model tests were performed using the 4.5 m radius large-beam centrifuge facility available at IIT Bombay. Models were instrumented with Linearly Variable Differential Transformers (LVDTs) for measuring surface settlements and Pore Pressure Transducers (PPTs) to measure raise in pore water pressure within the soil at the onset of flooding for levees and at the onset of seepage for geogrid reinforced soil walls. Additionally, digital image analyses of photographs of front elevation of levee models and geogrid reinforced soil wall models was carried-out to obtain face movements, movements of markers embedded within the levee, markers stuck to geogrid layers of reinforced soil walls at the onset of flooding and seepage. The developed inflight flood simulator was found to capable of generating the flood rate ranging from 2.2 m/day – 7 m/day. Further, results of centrifuge model tests conducted on levees without any chimney drain was noticed undergo catastrophic failure within 4.25 days of flooding induced seepage, whereas a levee with chimney drain was found to sustain flooding induced seepage of 37.5 days. Geogrid reinforced soil wall constructed with silty sand as a structural as well as backfill, without any drainage system experienced catastrophic failure. Contrary to this, geogrid reinforced soil walls with chimney drain as an external drainage system helped in averting catastrophic failure. However, probability of piping failure near the toe region of the wall can not be ruled-out. Further, the use of geocomposite layers as an internal drainage system within the reinforced zone also explored and placement of geocomposite layers at one-third portion of height from bottom was found to be effective.

Keywords: Physical models; Centrifuge model tests; Levee; Flooding; Seepage; Geogrid reinforced soil walls; Digital image analysis.

1 Introduction

The performance of geostructures (levees, embankment dams, geogrid reinforced soil walls, tunnels, retaining structures, etc.) subjected to self-weight loading and forces due to flooding, seepage, rainfall, external loading, earthquakes and waves is very much required to understand their behavior before and at failure. Two typical geo-structures, namely (i) levees subjected to flooding and (ii) geogrid reinforced soil walls subjected to seepage especially constructed with low-permeable fills is focused. The problem aggravates if the backfill material of geogrid reinforced soil walls s is not adequately selected. Koerner and Koerner (2011) [1] reported that 68% of the 82 cases of reinforced soil-wall failures in their database were due to improper drainage control during rainwater infiltration. The negative pore-water pressure in unsaturated soil is highly influenced by the external climatic conditions and flux boundary changes involving infiltration, evaporation and transpiration. As per the findings of Fredlund and Rahardjo (1993) [2], the negative pore-water pressure contributes in imparting additional shear strength to unsaturated soils. As water infiltrates into the slope, positive pore water pressure in the reinforced soil wall section increases, causing the additional shear strength due to matric suction to decrease and eventually disappear, thereby making the reinforced soil walls constructed with low permeable backfills vulnerable to failure. The rise and depletion of water level in the upstream side of levee/embankment dam structures are inevitable. The rise in water level can occur due to man reasons such as seasonal rains, flash flood, initial impounding of reservoirs, etc. Similarly, the drawdown can take place because of sudden loss of water on the upstream side of a levee/embankment dam. These failures may pose multiple adverse impacts, like disruption to services, infrastructure, economical loss etc.

Figure 1 shows typical cross-section of levee section with Chimney drain subjected to flood level of hf. As a drainage layer, it can be conventional sand layer of thickness ranging from 600 - 800 mm, chimney drain with batter of δ_v°.

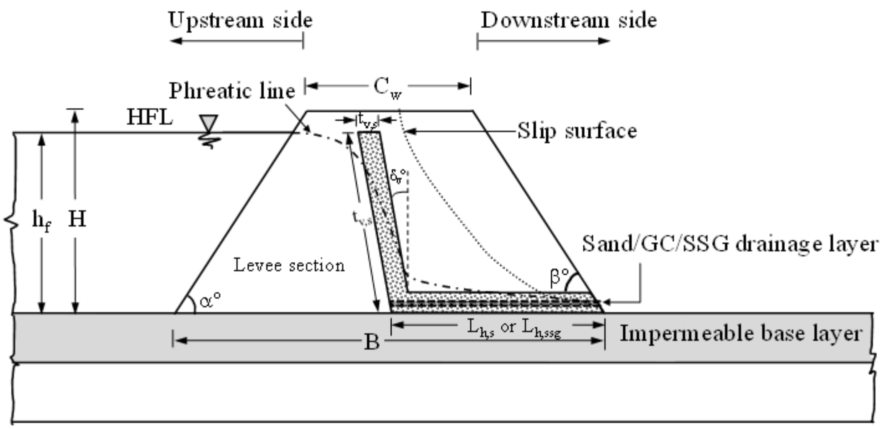


Fig. 1. Schematic cross-section of levee section with chimney drain subjected to flooding

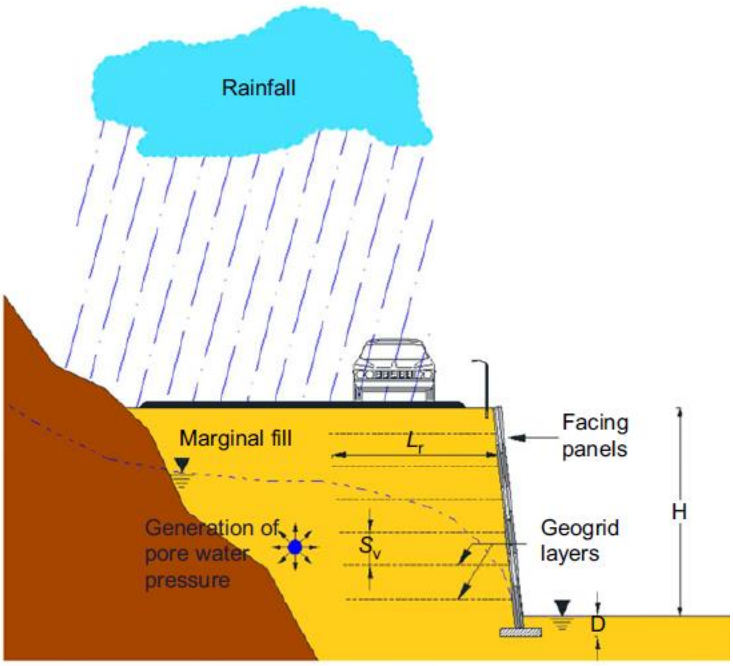


Fig. 2. Schematic cross-section of geogrid reinforced soil wall with low permeable backfill subjected to seepage

A geogrid reinforced soil wall consists of three main components: backfill soil, reinforcement layers and facing (Fig. 2). Full-scale modelling of geostructures subjected to rainfall/seepage/flooding are warranted in this regard for a comprehensive understanding of the actual physical phenomena involved in the process. However, full-scale physical model tests are expensive, time-consuming and difficult to replicate in case of a complex natural hazard or climatic events like rainfall, flooding, seepage, etc. At the same time, small-scale physical modelling cannot predict the behaviour of prototype structures accurately owing to dissimilarities in stress levels and strains existing between the laboratory *1g* model and corresponding prototype. In such situations, geotechnical centrifuge modelling can be used as an effective tool for investigating the response of geostructures: i) Model levees subjected to flooding and ii) Model geogrid reinforced soil walls constructed with low permeable backfill subjected to seepage. In centrifuge-based physical modelling technique, 1/N times scaled down models were subjected to a centrifugal acceleration of magnitude of N times greater (Ng) than that off earth’s gravitational acceleration (1g).

The present paper focuses on investigating: (i) the performance of levee subjected to flooding at 30 gravities using inflight flood simulator with and without chimney drain and (ii) the performance of geogrid reinforced soil walls constructed with low permeable backfill subjected to seepage and also explored enhancing the behavior using geocomposites and chimney drain. For both the cases considered, a blend of commercially available kaolin with the locally available sand to formulate silty sand with 20% fines. Especially for geogrid reinforced soil walls, the soil type was selected as model soil in view of the acute scarcity of good quality permeable granular material in recent times, which leads to the eventual use of low-permeability soils or marginal soils available locally at the site, as per the findings of Yoo et al. (2004) [3 and Pathak and Alfaro (2010) [4]. The results are interpreted in terms of crest settlements, face movement, deformed wall profiles and phreatic surfaces developed during seepage based on instrumentation data and digital image analysis.

2 Modelling Considerations

The basic principle of centrifuge-based physical modelling for geotechnical purposes is based upon the requirement of achieving identical stress fields in the model and prototype. This is achieved by developing appropriate scaling laws relating the model behaviour to corresponding prototype, as outlined in Schofield (1980) [5] and Taylor (1995) [6]. The model soil used during slope preparation is assumed to have identical mass density as that of the prototype. However, unlike 1*g* laboratory model, the centrifuge model is subjected to high gravitational field (*N*g) by rotating a vertical axis in a horizontal plane at a desired angular velocity denoted by ω. Thus, the centrifuge model is subjected to an inertial acceleration field of *N* times the earth's gravity, where *N* indicates the gravity level (or scale factor). The scale factor (model : prototype) for linear dimensions of slope is thus 1:*N*. Since the model is a linear scale representation of the prototype, slope/wall displacements will also have a scale factor of 1:*N*, and in accordance, strains are scaled by a factor of 1:1. Therefore, stress-strain variation of the soil mobilized in a centrifuge model will be identical to that of prototype.

2.1 Modelling of flooding and drawdown

The performance of levees subjected to flooding and drawdown is essentially governed by the stress conditions and pore water pressures present within the levee/embankment dam section. And the application of centrifuge modeling technique is pertinent for investigating their actual behaviour, where seepage boundary conditions significantly influence pore water pressures within the levee. Additionally, climatic conditions such as flooding and drawdown can be replicated and reproduced efficiently using centrifuge modeling technique.

The relevant scaling factors involved in modelling of flooding and drawdown in a geotechnical centrifuge is presented in Table 1.

2.2 Modelling of geogrid reinforced soil walls

Considering the components of the geogrid reinforced soil wall, the parameters related to the wall geometry, the backfill soil, the reinforcement and facing materials should be scaled down using scaling relations. Moreover the parameters related to the seepage phenomena should be taken into account properly. These relations have been discussed in detail by Viswanadham and König (2004) [7] and Raisinghani and Viswanadham (2011) [8]. Table 2 summarizes all related scaling factors in this study. Modeling considerations of geogrids and geocomposites are discussed elsewhere.

Table 1. Summary of scale factors for centrifuge modelling of levees/embankmant dams

Parameters	Units	Prototype	Model
Soil parameters			
Unit weight of soil (γ)	kN/m^3	1	N
Stress (σ)	kN/m^2	1	1
Cohesion (c)	kN/m^2	1	1
Angle of internal friction (ϕ)	Degree (°)	1	1
Levee parameters			
Levee height (H)	m	1	1/N
U/s and D/s slope of Levee (α and β)	Degree (°)	1	1
Length of chimney drainage layer (L_h *and* L_v)	m	1	1/N
Thickness of chimney drainage layer (t_{cd})	m	1	1/N
Thickness of chimney drainage layer (δ)	Degree (°)	1	1
Seepage parameters			
Pore water pressure (u)	kPa	1	1
Seepage time after commencement of flooding or drawdown(t_{sf} or t_{sd})	sec	1	1/N^2
Coefficient of permeability (k)	m/sec	1	N
Discharge through soil (Q)	m^3/sec	1	1/N
Flooding parameters			
Flood level (h_f)	m	1	1/N
Flood rate (r_f)	m/day	1	N
Drawdown parameters			
Drawdown level (h_d)	m	1	1/N
Drawdown ratio (R_d)	-[a]	1	1
Drawdown rate (r_d)	m/day	1	N

N = gravity level or scale factor; -[a] Dimensionless parameter; U/s: Upstream; D/s: Downstream

Table 2. Scaling laws adopted for centrifuge modelling of reinforced soil walls with permeable reinforcement layers

Parameters	Unit	Prototype	[a]Model
Geosynthetic parameters			
Tensile load, (T)	kN/m	1	1/N
Secant stiffness (J)	kN/m	1	1/N
Strain (ε)	%	1	1
Percentage open area (f)	%	1	1
Soil-geogrid interface friction angle (ϕ_{sg})	°	1	1
Transmissivity (θ)	m^2/s	1	1
In-plane coefficient of permeability (k_h)	m/s	1	N
Reinforced wall facing parameters			
Length (L_f)	m	1	1/N
Flexural rigidity/m ($E_f t_f^3$)	kN-m	1	$1/N^3$
Reinforced wall geometry parameters			
Height (H)	m	1	1/N
Inclination (β)	°	1	1
Length of reinforcement (L_r)	m	1	1/N
Vertical spacing of reinforcement (S_v)	m	1	1/N

[a]N = scale factor or gravity level.

3 Selection of Model Soil

In view of the significance of backfill material in controlling the performance of reinforced soil structures, special emphasis has been given to studies performed in low-permeable soils, and the detrimental effect of water on the performance of these structures. The Ministry of Road Transport and Highways (MORTH), Government of India specifies up to 10% fines, whereas The Federal Highway Administration (FHWA) allows up to 35% fines (passing 0.075 mm) in the reinforced soil zone. Accordingly, the model soil used in slope preparation was formulated in the laboratory to arrive at 20% fines by blending locally available fine sand and commercially available kaolin. The Goa sand is classified as SP according to USCS, whereas, the kaolin sample is classified as CL as per USCS, and consists predominantly of silt and clay in the ratio of 47% and 53% respectively. The blended silty sand is SM as per USCS, and consist of sand and kaolin in the ratio of 4:1 by dry weight with a saturated permeability (k_{sat}) of 1.54 x 10^{-6} m/s. The model soil is thus representative of the properties of locally available marginal soils which contain fines in excess of 15% and exhibit a saturated coefficient of permeability of less than 1x10^{-6} m/sec, prescribed respectively by Christopher and Stuglis (2005) [9] and Holtz and Kovacs (1981) [10].

4 Model Preparation, Test Program and Results

All centrifuge model tests were conducted using the 4.5 m radius beam centrifuge facility of 2500 g-kN capacity available at IIT Bombay, INDIA. Detailed specification of the centrifuge equipment is available in Chandrasekaran (2001) [11].

4.1 Centrifuge model tests on levee sections subjected to flooding using In-Flight Flood and Drawdown Simulator (IFDS)

A custom designed and developed IFDS works on the simple mechanism, in which the flood is generated by using the pumping mechanism and drawdown is simulated by draining solenoid valves and utilizing the flow of water under gravity. The set-up was calibrated and used in a 4.5 m radius large beam centrifuge facility available at IIT Bombay for simulating desired flood rate, flood duration, drawdown rate and drawdown ratio on the upstream side. The developed IFDS set up is capable of generating the flood rate ranging between 2.2 m/day and 7 m/day and sustaining the high flood level for maximum duration of 37.5 days. Similarly, it can also simulate the drawdown having drawdown rates between 2 m/day and 4.4 m/day and achieve the drawdown ratio of 0 to 1. Here, drawdown ratio can be defined as ratio of drop in water level in upstream side to the height of levee from the top surface of the base layer.

Figure 3 demonstrates the schematic diagram of model test package used in this study. The front elevations of models were captured using a digital camera and further analysed using digital image analysis to obtain the profile of levee models during various stages of centrifuge tests. The variation in pore water pressure within the levee was recorded with the help of pore pressure transducers (PPTs), whereas the surface settlements were measured using linearly variable differential transformers (LVDTs). The phreatic surfaces were obtained from recorded PPTs data during the tests. Figure 4 shows instrumented levee model mounted along with Inflight Flood Simulator (IFDS-2) on the swinging basket.

After mounting the whole model test package on the centrifuge basket, it was rotated at 80 revolutions per minute (rpm) to induce acceleration field replicating the 30 gravity level. After reaching at 30g, waiting period of 5 minutes in model dimensions was considered. After that, the flood was initiated by supplying the required input voltage (v_{in}) of 20 V to the pump with the help of control unit placed in the control room.

Figure 5 depicts the front elevation of both the levee models at the penultimate stage during centrifuge test at 30 gravity level. As can be noted from Figs. 5a, due to the increase in flood level on the upstream side, phreatic surface approached to the downstream slope, which resulted in the softening of toe region and subsequent catastrophic failure of downstream slope. However, in Model S-6, the phreatic surfaces could not approach to the downstream slope even after the seepage duration of 37.5 days, which indicates the efficacy of the chimney drain layer in the dissipation of the pore water pressure within the levee section(Fig. 5b).

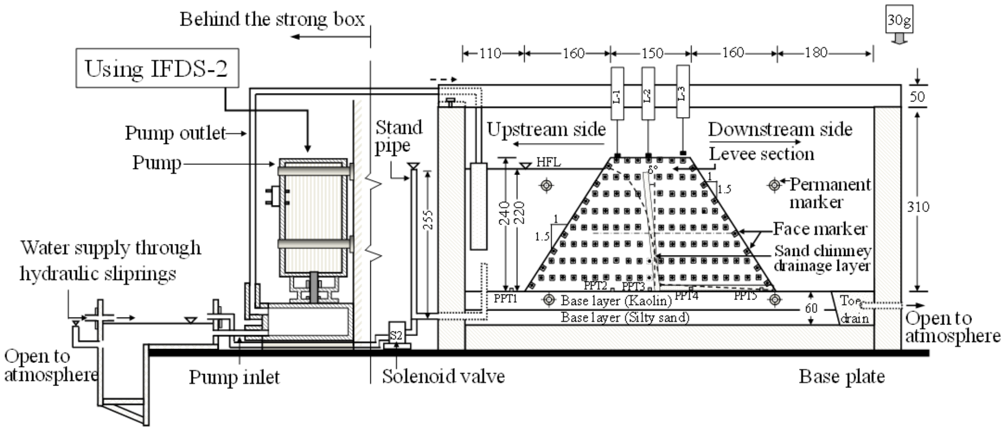


Fig. 3. Details of model test package [Model: S-6] (All dimensions are in mm)

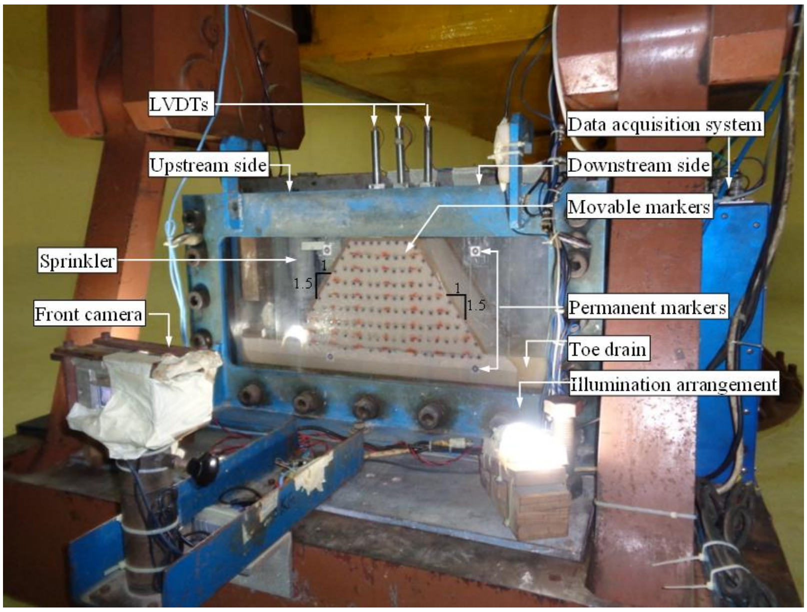
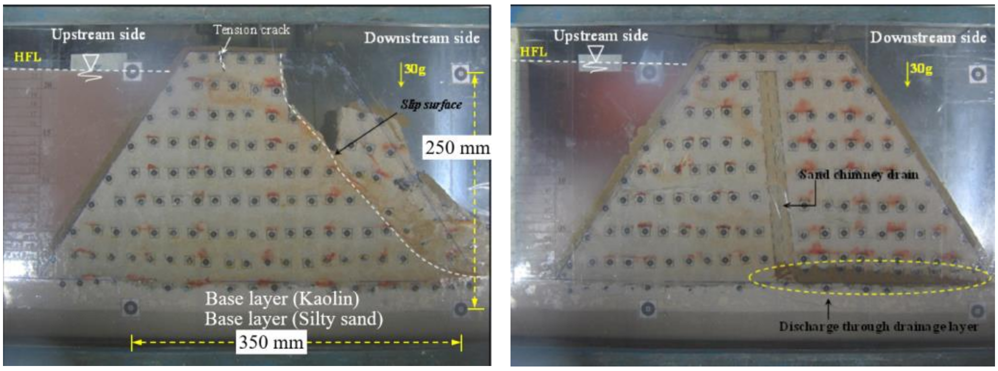


Fig. 4. Perspective view of the instrumented levee model [Model: S-1]



a) Levee model: S-1 b) Levee model: S-6

Fig. 5. Status of levee models at penultimate stage during centrifuge tests

The levee without any drainage layer underwent catastrophic failure, experiencing maximum crest deformation of 1.31 m (measured with the help of LVDT placed at the crest, as shown in Fig. 4), after seepage time of 4.25 days in prototype dimensions, whereas the levee with sand chimeny drain could sustain the stability even after the seepage of 37.5 days. As can be noted from Fig. 5b, levee model with chimney drain was found to be intact even after subjecting a sustained flooding of 60 minutes (in model dimensions). Measured crest deformation ($S_{c,max}$) during penultimate stages of model S-6 is 0f 0.008 m. In model S-6, chimney drain is modelled with fine sand having 20 mm thickness in model dimensions and this corresponds to 600 mm thick in the field. The seepage and deformation behaviour of the levee was investigated by measuring the pore water pressure within the levee section and crest settlements with seepage time.

Figure 6 depicts the variations of normalised pore water pressure at the toe ($u_{toe}/\gamma H$) and the factor of safety (FS) against slope failure with the seepage time (t_{sf}) for both the models. The pore water pressure at the toe of the embankment (i.e. towards downstream side) was measured with the help of pore-water pressure transducer for levee model with and without chimney drain. The factor of safety at the onset of flood and subsequent seepage is obtained by carrying out ϕ-c reduction based stability analysis with the help of finite element based software Plaxis-2D (Plaxis, 2012) [12].

As can be noted from the Fig. 6, in the case of model S-1, the FS decreased up to 0.92 at the onset of flooding and subsequent seepage. In comparison, a higher value of a minimum factor of safety of 1.10 was obtained in the case of S-6. It can be attributed to the efficacy of sand chimney drainage layer to dissipate the pore water pressure. With this the efficacy of developed IFDS for initiating flooding to a levee section with and without chimney drain could be demonstrated adequately. It implies that the setup can also be used to study the seepage and deformation behaviour of geotechnical structures such as earthen dams, canal, dikes and tailings dams involving large deformation at the onset of flooding or drawdown. For enabling drawdown to happen, with the help of piping connected to solenoid valves towards upstream side were used to drain water.

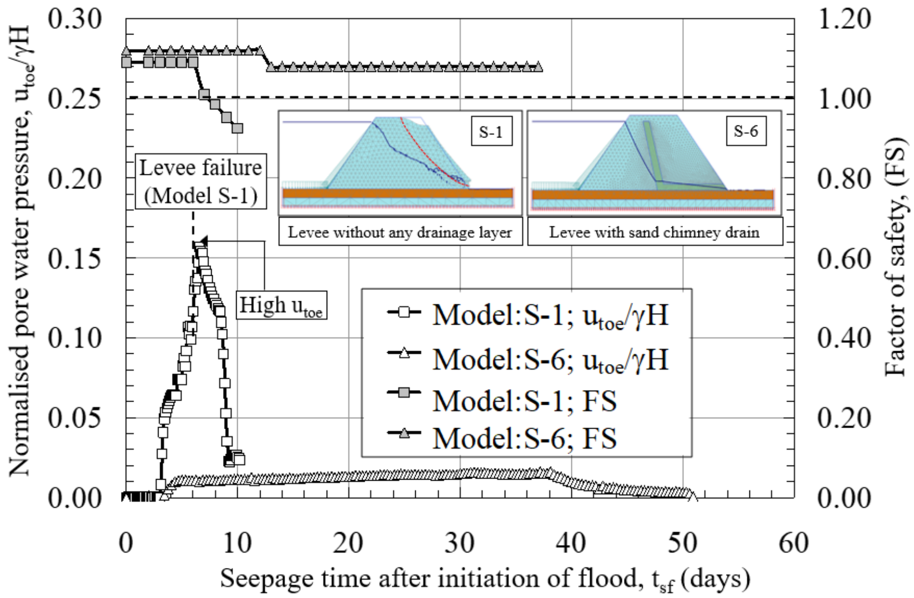


Fig. 6. Variation in $u_{toe}/\gamma H$ and FS with seepage time during flooding

4.2 Centrifuge model tests on geosynthetic reinforced soil walls subjected to seepage

A strong model container with internal dimensions of 760 mm (length) × 200 mm (width)× 410 mm (height) was used in this study. This box has a transparent Perspex front wall with 100 mm thickness for capturing front elevation of the model continually during the flight by using an on-board digital camera. The other three walls of container are made of mild steel plates with 100 mm thickness. A rectangular grid of four permanent markers (350 mm in horizontal and 200 mm in vertical) were placed inside area of Perspex glass. These permanent markers were used as reference points in digital image analysis to find the displacement field of the wall models with the help of discrete markers stuck to geosynthetic layers. A special procedure was adopted to reduce the friction of front and rear walls in order to meet plane strain condition. Due to this, a very thin cover of flexible polyethylene sheet was provided on the inner side of front and rear walls lubricated using white petroleum grease. Figure 7 shows perspective view of geogrid reinforced soil wall model mounted on the swinging basket.

A special seepage setup was used in this study to induce rising ground water condition in the wall models. As shown in Fig. 7, this setup has three main parts including: seepage tank, water reservoir and solenoid valve. Seepage tank is placed at the left hand side of the model and one of its walls in contact with soil is provided with perforated holes. To prevent clogging of the perforated wall by soil particles a thin layer of nonwoven geotextile was placed. The solenoid valve allows the flow of water from water reservoir placed at the top of the model container into the seepage tank and then into the soil medium. This has facilitated triggering of seepage during centrifuge test at 40 gravities.

Three Linear Variable Differential Transformers (LVDTs) were used at the top surface of the wall models to get surface settlement profile. One LVDT (L1) was placed at the wall crest, while other two LVDTs were put at a distance of 95 mm (L2) and 200 mm (L3) from the crest. Four miniature Pore water Pressure Transducers (PPTs) were used to depict the flow of water inside the reinforced soil wall. One PPT (PPT1) was put inside the seepage tank while other three PPTs were placed within the backfill soil along the base layer at a distance of 20 mm (PPT2), 125 mm (PPT3) and 250 mm (PPT4) from the seepage tank. The data from PPT3 (placed at the middle portion of the wall) and PPT4 (placed near the toe of the wall) were used to compare the results of centrifuge model tests.

In all centrifuge models the height of reinforced soil wall (H) was 250 mm (10 m in prototype dimensions). A base layer of 50 mm thickness was provided for all models and therefore the total height of models was equal to 300 mm. After placing the base layer, the geogrid reinforced soil wall was constructed in seven layers with equal thickness of 40 mm in all layers except in layers 1 and 7 with 30 mm thickness. The same soil was used for both reinforced soil portion and base layer at its optimum moisture content and maximum dry unit weight (standard Proctor compaction test). After compaction of each soil layer six geogrid layers with equal length of 200 mm (0.8H) were placed. Then thin transparent plastic markers with L-shape were glued properly along the reinforcement layers in equal distance of 20 mm. White grease was applied on one side of the markers to facilitate its movement in contact with Perspex wall of the container. By tracing the location of these markers' displacement field and therefore strain field of reinforced portion can be obtained. Moreover, red food dye was placed at some points on top of each soil layer to depict the flow of water through the wall models during the seepage.

By switching on the solenoid valve after elapsing 5-10 minutes at 40g the water was allowed to flow into the seepage tank and thereby into the soil medium. As described before, the facing panels were connected together in the form of Mortise and Tenon joint. Therefore, the narrow gap between panels was enough to dissipate cumulated pore water within the backfill and in geocomposite layers and it was not necessary to provide additional drainage at the facing.

In this paper the results of four centrifuge tests were reported and discussed. These four models were aimed to study the effect of geocomposite layers and chimney sand drain on the performance of geosynthetic reinforced soil walls with marginal backfills. The wall height (H = 250 mm), facing inclination ($\beta = 84^{\circ}$ with horizontal), reinforcement length ($L_r/H = 0.8$) and spacing ($S_v/H = 0.160$) was kept constant in all models. The facing and marginal soil type was also the same in all models. Model M1 was reinforced with six layers of geogrid (GG). In model M2 bottom two layers were of geocomposite (GC) and the remaining four layers were of geogrid (GG). In model M3 bottom four layers were of geocomposite (GC) and the remaining two layers were of geogrid (GG). In model M4, same as model M1, six layers of geogrid (GG) were used and a chimney sand drain was also provided. The thickness of chimney sand drain was 15 mm (600 mm in prototype dimensions) and grade II standard Goa sand was used to construct it. This sand has particle size ranging 0.4 - 1 mm and coefficient of permeability of 1.54×10^{-4} m/s (constant head test on sand at 85% relative density).

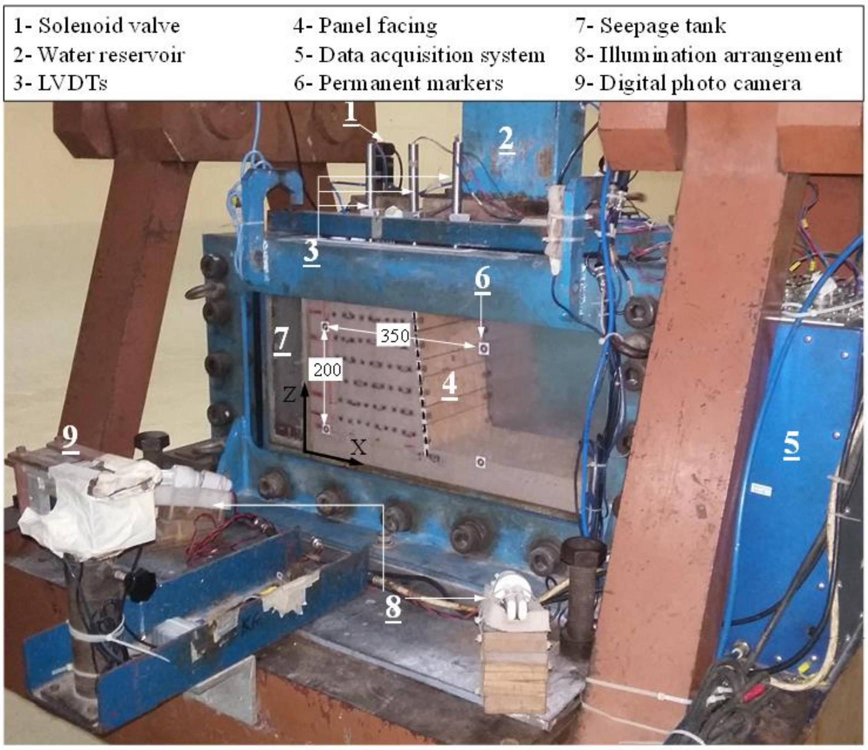


Fig. 7. Perspective view of reinforced wall placed on swinging basket (All dimensions are in mm)

Centrifuge model tests on geogrid reinforced soil walls were monitored in terms of pore water pressures, surface settlements, wall face movement, displacement field and strain along the reinforcement layers. PPTs and LVDTs were used to measure pore water pressures and surface settlements respectively. Digital Image Analysis (DIA) technique was also performed to obtain displacement field and strain along the reinforcement layers using discrete markers glued on to reinforcement layers and facing panels. From here onwards, the dimensions are given in prototype scale.

It is remarkable that in models M2, M3 and M4 after attaining steady state seepage conditions within 2 or 3 days of seepage, a relatively uniform variation of surface settlement was observed. Figures 8(a-d) illustrate the measured variations of top surface settlement with horizontal distance from the crest of the wall at various times after inducing seepage. In comparison, model M4 (with chimney sand drain) has the least deformation and model M3 (with 4 GC layers) has the less deformation than model M2 (with 2 GC layers).

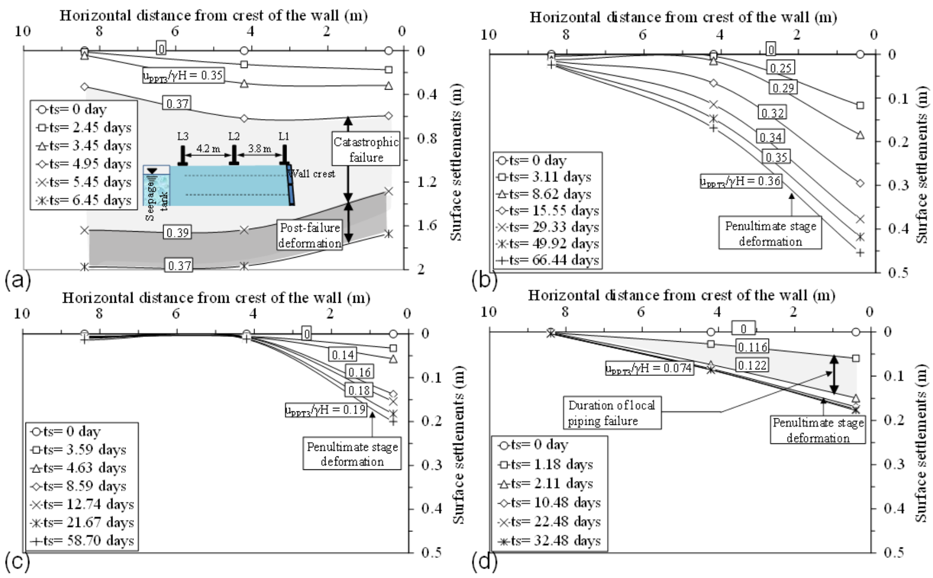


Fig. 8. Measured variation of surface settlement with horizontal distance from crest of the wall: (a) Model M1; (b) Model M2; (c) Model M3; (d) Model M4

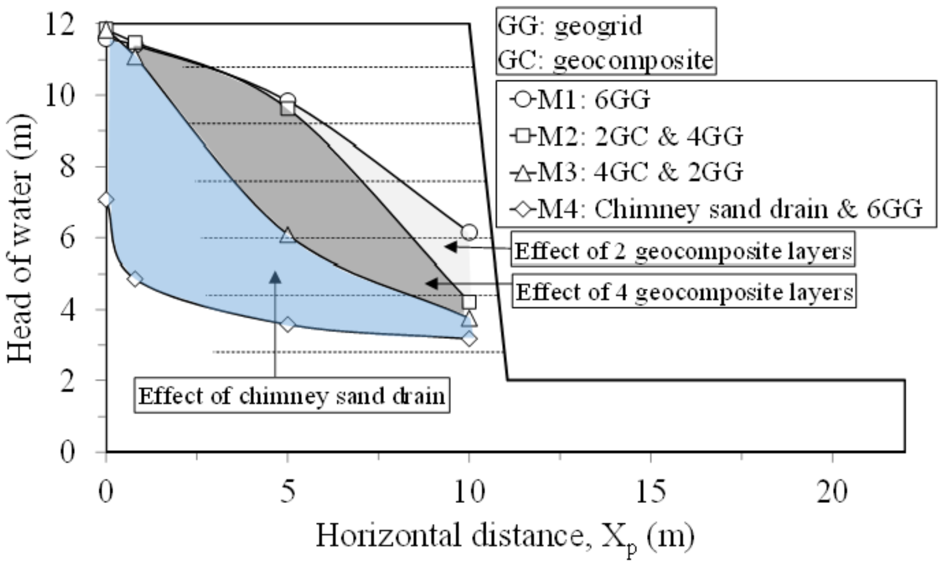


Fig. 9. Development of phreatic surface within geosynthetic reinforced soil walls at penultimate stage with and without internal or external drainage system

Interestingly the better comparison can be found in Figure 9 with phreatic surfaces obtained from PPT readings within the wall models during the penultimate stage of the centrifuge tests. In Figure 9, horizontal dashed lines indicate the location of reinforcement layers in the model. At first glance, the higher location of phreatic surface in model M1 could be noticed. The greatest depletion of phreatic surface is registered for model M4 with chimney sand drain application. This is attributed to the high drainage capability of chimney sand drain system. The second highest reduction of pore water pressure was achieved with model M3 with 4 GC layers application. Model M3 with 2 GC layers only affected the reduction of pore water pressure at the toe of the wall.

Figures 10(a-d) indicate the displacement fields of model geosynthetic reinforced soil walls M1, M2, M3 and M4 from the moment when seepage induced to the penultimate stage of the test in prototype dimensions.

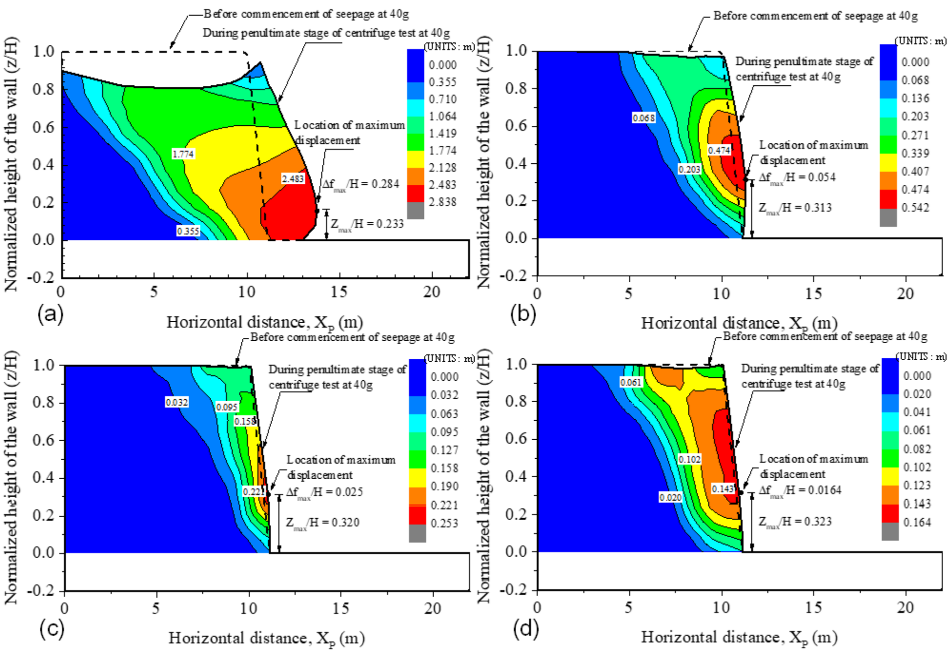


Fig. 10. Displacement fields obtained by digital image analysis: (a) model M1; (b) model M2; (c) model M3; (d) model M4

Figure 10(a) represents the large surface settlement and face movement occurred in model M1 as a result of catastrophic failure. An overview of failure surface is obvious in Figure 10(a). In comparison, surface settlements and face movements were observed to be controlled in models M2 (with 2GC layers), M3 (with 4 GC layers) and M4 (with chimney sand drain) as a result of drainage system application. The value of maximum facing displacement during penultimate stage of the test was normalized with respect to the height of the wall ($\Delta f_{max}/H$) and presented in Figure 10. The value of $\Delta f_{max}/H$ in model M1 at the onset of failure was equal to 0.284. The value of $\Delta f_{max}/H$ in models M2, M3 and M4 was equal to 0.054, 0.025 and 0.016 respectively.

The height of point of maximum facing displacement from the base layer of the wall was also marked in Figures 10(a-d) with respect to the height of the wall (Z_{max}/H). The value of Z_{max}/H was equal to 0.284 in model M1. Interestingly this value was near 0.320 in models M2, M3 and M4. Therefore, it can be inferred that the face movements are prone to be maximum at the bottom one-third of the wall height irrespective to the different drainage systems applied in wall models. However, it seems that the effect of type of facing in face deformation should be investigated further. Centrifuge model tests confirmed the effectiveness of providing geocomposite layers as internal drainage system in mitigating the problems of marginal backfills. Moreover, the tests indicated the application of chimney sand drain as external drainage system can be also a solution for this purpose. However, the probability of local piping failure occurrence near the toe region of the wall should be considered in design process.

5 Conclusions

This paper aims to present the use of centrifuge-based physical modelling of geostructures, like levees and geogrid reinforced soil walls. Based on analyses and interpretation of centrifuge model tests on levee sections with and without chimney drain subjected to flooding and geogrid reinforced soil walls subjected to seepage, the following conclusions can be drawn:

a) Performance of Levees subjected to flooding

1. The IFDS set up can readily simulate the flood rate from 2.2 m/day to 7 m/day in 30 gravities at the input voltage supply (v_{in}) from 16 V to 22 V to the pump.

2. Finally, the efficacy of the IFDS set up to commence the flood and sustain high flood level for the long durations was demonstrated by conducting centrifuge tests on levees with and without chimney drain subjected to flooding. The levee without any drainage layer experienced catastrophic failure after seepage time of 4.25 days in prototype whereas levee with sand chimney drain could sustain the stability even after the seepage of 37.5 days.

b) Performance of geogrid reinforced soil walls with low permeable backfill

1. Centrifuge model tests confirmed the effectiveness of providing geocomposite layers as internal drainage system in mitigating the problems of marginal backfills. Moreover the tests indicated the application of chimney sand drain as external drainage system can be also a solution for this purpose. However the probability of local piping failure occurrence near the toe region of the wall should be considered in design process.
2. Geogrid reinforced soil wall without any drainage system experienced catastrophic failure as a result of pore water pressure development within the reinforced zone during 5 days after seepage onset. The wall experienced the

maximum crest settlement of 1.69 m (S_{c-max}/H = 0.169) and maximum facing displacement of 2.84 m (Δf_{max}/H = 0.284) at the onset of failure.

3. This study clearly indicates the effectiveness of increasing the number of geocomposite layers result in enhancing the performance of the wall. The values of S_{c-max}/H and Δf_{max}/H were 0.045 and 0.054 respectively for wall with two geocomposite layers. The wall with four geocomposite layers indicated the values of 0.021 and 0.025 for S_{c-max}/H and Δf_{max}/H respectively.
4. Geogrid reinforced soil wall with four geocomposite layers (or up to half height from bottom) is found to perform superior than wall with chimney sand drain. The wall with chimney sand drain experienced piping failure. However in the wall with 4 geocomposite layers occurrence of piping was not registered. This is attributed to the distribution of dissipation of pore water pressure uniformly over half height of wall rather than concentrated at bottom close to the foundation pad level.

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