The impact of soil stratigraphy on liquefaction-induced settlements of Flood Embankments

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ABSTRACT: This research examines the effects of soil stratigraphy on the seismic performance of flood embankments, focusing on earthquake-induced settlements and the development of cracks. The study involves three dynamic centrifuge tests, each with a flood embankment resting on stratigraphically different soil foundation layers. The first centrifuge model, acting as a benchmark, had a uniform liquefiable sand layer as the foundation soil. In the second test, the liquefiable layer was overlain by a thin clay crust, while in the third test, it was overlain by a thick clay layer. The embankment was constructed using naturally occurring silt, mixed to the optimal moisture content and compacted in three layers within a wooden mould. The embankment was then frozen to ensure structural integrity before being inverted and positioned atop the centrifuge model foundation. The foundation consisted of Hostun Sand (HN31), while the clay layers were prepared using pre-consolidated Speswhite Kaolin Clay. The centrifuge models were subjected to consecutive dynamic excitations of input motions of increasing intensity. Two high-speed cameras recorded the movement of soil particles during shaking and measured the embankment's displacement. The impact of soil stratigraphy on liquefaction-induced settlements and development of cracks in the body of the embankment dams is examined here for a selected input motion, and an analysis of settlements and crack development in the embankments is constructed. The findings indicate that the presence of clay layers significantly reduces liquefaction-induced settlements and excessive deformation of the embankment body.

1 INRODUCTION

Flood embankments are the primary structural defence against floods, shaping interactions between human settlements and natural water systems. Despite their importance, they remain highly vulnerable to seismic events. Failure of these structures can trigger catastrophic flooding, as highlighted by Kishore and Madabhushi (2025). Their greatest seismic vulnerability arises from soil liquefaction, where saturated loose sands lose shear strength under dynamic loading due to excess pore water pressure. This strength loss leads to permanent ground settlement, causing liquefaction-induced freeboard reduction or cracking within the embankment body, both of which can result in uncontrolled seepage.

Historical earthquakes—such as the 1960 Chilean, 1971 San Fernando, 1995 Kobe, and 2011 Tohoku events—have repeatedly demonstrated the widespread consequences of liquefaction in deep, loose deposits, even under moderate-to-long shaking durations (Duke 1960; Seed et al. 1971; Oka et al. 2011; Yamaguchi 2011). Such failures emphasised the need for improved analytical and experimental methods to predict deformation and assess structural resilience.

Liquefaction research accelerated after the 1964 Niigata and Alaska earthquakes, with early work focusing on criteria for triggering. The widely used Simplified Procedure by Seed and Idriss (1971) relates seismic demand (CSR) to soil resistance (CRR) using in-situ tests. Later contributions by Ishihara (1985; 1993) linked deposit characteristics

with surface damage potential, advancing understanding of liquefaction-induced deformation.
For flood embankments, settlement is largely governed by soil–structure interaction. The embankment's self-weight imposes large static shear stresses on the softened foundation during shaking, producing shear-induced settlements far exceeding volumetric reconsolidation. This mechanism, highlighted in centrifuge modelling by Adalier and Sharp (2004), often results in substantial freeboard loss—a key determinant of post-earthquake serviceability and public safety.

The role of a crust layer is also critical. Studies by Towhata et al. (1992) and Yasuda et al. (1994) showed that silt or clay layers overlying liquefiable soils can restrict vertical deformation during shaking but may cause delayed settlement as pore pressures dissipate. These delayed effects influence assessments of cracking, overtopping risk, and overall serviceability. Centrifuge modelling is particularly effective at capturing such time-dependent responses.

This study examines how clay layers of varying thickness above a liquefiable foundation influence the seismic performance of embankments. It investigates the dynamic response of flood embankment founded on (a) loose sand, (b) loose sand with a thin clay layer, and (c) loose sand with a thick clay layer, focusing on the stratigraphic control of liquefaction-induced settlement.

2 CENTRIFUGE MODEL AND EXPERIMENTAL SETUP

Centrifuge modelling is a powerful and unique technique to study geotechnical engineering problems under controlled conditions. This research is based on physical centrifuge modelling, which involves experiments conducted on scaled models of the flood embankments resting on liquefiable sand foundations. Three dynamic centrifuge tests were conducted in the Turner Beam centrifuge facility at a 60-g level. This centrifuge facility at the Schofield Centre in Cambridge University has a rotational radius of 4.125 m. Further details about the centrifuge and its operation are presented by Schofield (1980) and recently by Madabhushi (2014). Table 1 presents the centrifuge scaling laws presented by Schofield (1980).

The models of stratigraphically different soil foundations were prepared in a rigid model container to be tested at a 60-g level. Details about the preparation of the model are in the next section. Figure 1 shows the schematic cross-section of the three centrifuge tests, illustrating the locations of the embankments and the instrumentation used in the tests. The three models featured three different foundations: (a) sand foundation, (b) thin clay layer on top of the sand foundation, and (c) thick clay layer on top of the sand foundation.

Table 1. Centrifuge scaling laws, Schofield (1980)

Property	Scaling Factor	Unit
Length	1/N	m
Stress	1	N/m^2
Strain	1	-
Gravity	N	m/s^2
Time (consolidation)	$1/N^2$	S
Time (dynamic)	1/N	s

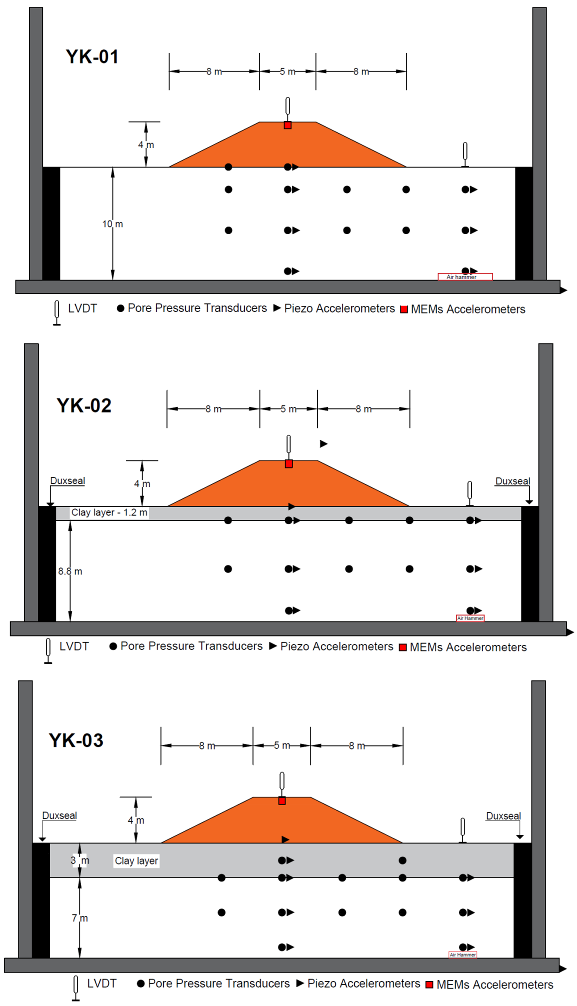

Figure 1. Cross-sectional views of the three dynamic centrifuge tests, a) YK-01, b) YK-02, and c) YK-03

2.1 Flood embankment specifications

For the three dynamic tests on the flood embankments, a wooden mould was designed to make the embankments upside down by consolidating the silty sand with the standard proctor hammer. The soil was first mixed with the optimum moisture content, which is 13.28% in this case, and then compacted in three layers in the mould. The mould was then kept in the freezer to freeze the embankment. Once frozen, the embankment was taken out, and control points were glued to one side of the embankment for PIV (particle image velocimetry) analysis, then carefully inverted and positioned atop the prepared soil foundation within the model container. This process facilitated accurate placement and, more importantly, ensured structural integrity for subsequent testing. The centrifuge tests were started after allowing time for the frozen embankment to defrost, as it could simulate a real-world structure.

Figure 2. Model flood embankment prepared in the wooden mould

2.2 Foundation and test preparation

For these dynamic tests, the foundations were made of Hostun sand HN3; the details are given in Mitrani (2006) and Haigh et al. (2012). Some properties of the sand are provided in Table 2. The saturated sand model foundations were prepared by air pluviation using a 3D automatic sand pourer present at the Schofield Centre, University of Cambridge. The clay layers were prepared from the pre-consolidated Speswhite Kaolin Clay; the detailed characteristics of the clay can be found in Chan (2020) and Guadio et al. (2022). Some properties of the clay material are provided in Table 3.

All three tests were conducted in an aluminium window container which has a rigid Perspex observing window glass panel in the front. For Particle Image Velocimetry (PIV) analysis, this type of model container with an observation window was used; it has been cited in previous research, such as Fern et al. (2015) and Kobayashi and Murakami (2019), to capture the dynamic behaviour of the embankment with high-speed imaging.

A servo-hydraulic shaker, shown in Figure 3, was designed by Madabhushi et al. (2012) to simulate dynamic loading. The shaker can apply historical recorded earthquakes as well as the sinusoidal model earthquakes to the centrifuge models. Table 4 provides the details of each input motion and the earthquake sequences for all three tests. The sand foundation relative densities are identical in these tests, which helps in better understanding the effects of dynamic loading on the flood embankments. The sand foundation models were saturated using an automated saturation system called CAM-SAT available at the Schofield Centre, University of Cambridge, further explained by Stringer and Madabhushi (2009).

Table 2. Properties of Hostun Sand HN31

Parameter	Value
e_{min}	0.555[a]
e_{max}	1.01[a]
G_s	2.65[a]
K (m/s)	1×10^{-3}[b]
D_{10}	0.209
D_{50}	0.335
φ (degrees)	33[a]

[a]Data from Mitrani (2006). [b] Data from Haigh et al. (2012)

Table 3. Properties of Kaolin Clay (Chan 2020)

Parameters	Value
PI (%)	41.8
ωL (%)	75
Gs	2.61
Κ	0.07
Λ	0.19

3 LIQUEFACTION-INDUCED SETTLEMENTS

Figure 4 presents the embankment settlement of all three tests during the first shake. The recorded settlements, shown in Figure 4, provide compelling evidence of a significant loss of soil stiffness and subsequent settlement of embankment consistent with liquefaction. The first graph shows the permanent and irreversible loss of flood embankment free board, whereas the second graph shows the heave in the ground near the embankment, which is also permanent. This heave occurs due to the undrained behaviour of foundation soil and occurs in response to the subsequent settlement. The comparison of the three tests shows that the soil stratigraphy plays a significant role in the liquefaction-induced settlements. In the first test with no clay layer, the embankment had settled around

0.29 m, in the second test with a thin clay layer, the embankment had settled around 0.25 m and in the last test with a thick clay layer, the embankment had only settled around 0.21 m. Figure 5 shows the rapid pore pressure buildup during the shaking, a sign of liquefaction in the foundation soil. The upper graph shows the excess pore pressure buildup in the deep ground below the embankment, and the graph in the middle shows the same for the free field.

The combination of co-seismic, large, and largely permanent settlements in the embankment structure and the residual volumetric change following the dynamic event confirms that the excessive pore pressure generation led to a profound reduction in effective stress, allowing the large-scale and rapid plastic deformation and reconsolidation of the soil mass.

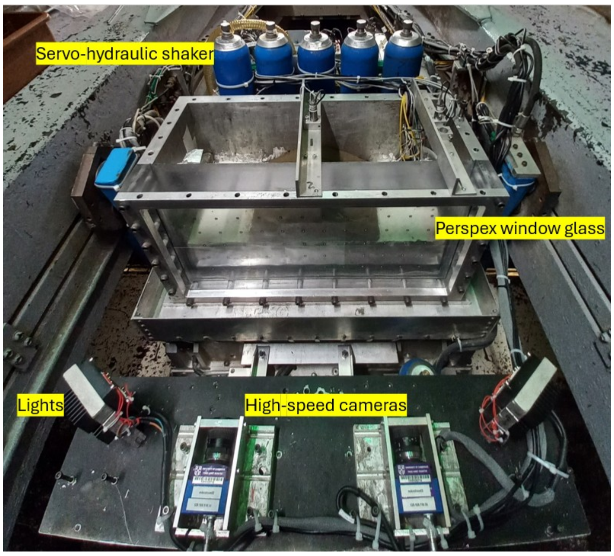

Figure 3. Flood embankment model resting on the servo-hydraulic shaker

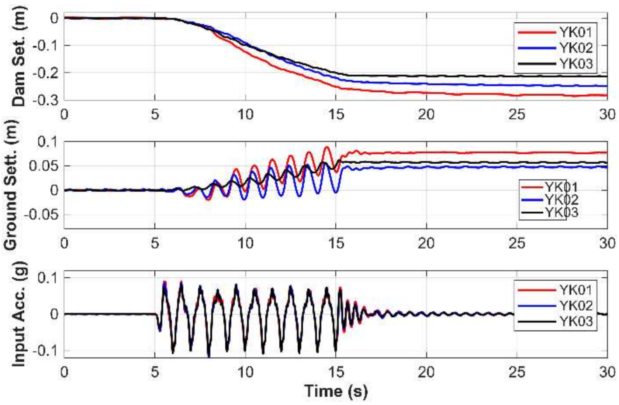

Figure 4 Liquefaction-induced settlements

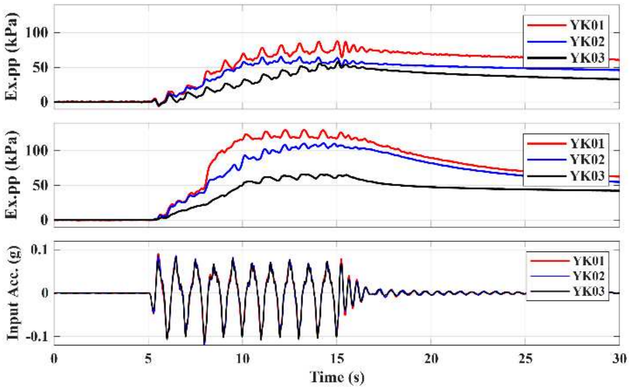

Figure 5 Excess pore pressure build up in all three tests

4 CREST ACCELERATION

Figure 6 compares the crest accelerations of the flood embankment from the three tests. The base accelerations in all three tests are uniformly applied to the model; however, the crest accelerations generated during the earthquakes show critical differences in the response of the structure. In the first test, when the embankment is resting on only the sand foundation. The embankment crest underwent the highest peak amplitude in the initial cycles of approximately 0.18g, which shows the extreme inertial force generation due to direct energy transfer through the foundation to the embankment body. In the following cycles, the crest accelerations decrease with the onset of excess pore pressure generation. The amplitude is less when the embankments are resting on clay layers atop soil layers. The presence of a clay layer seems to smoothen these crest accelerations over the time histories.

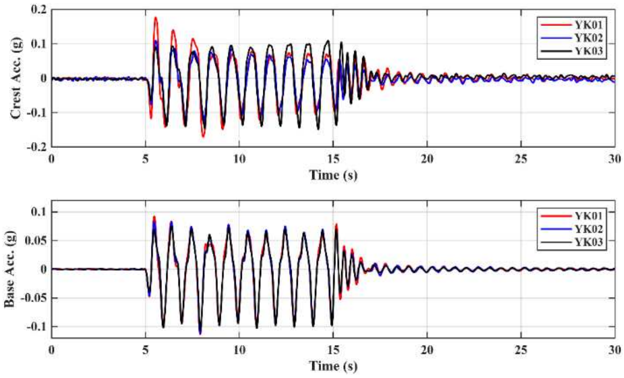

Figure 6 Embankment crest accelerations

5 CRACK DEVELOPMENT

Figure 7 compares the cracks developed in the model embankments during consecutive dynamic events. The figure shows that as the soil stratigraphy changes or with the presence of a non-liquefiable foundation layer, the chances of occurrence of cracks in the embankment structure decrease. Figure 7(a) shows the plan view of the first test with no clay layer; the cracks are wider, and there are cracks in each part of the structure. Figure 7(b) shows improvement in terms of structural integrity but still has developed wide cracks in the both sides of the embankment, whereas, Figure 7(c) shows the embankment resting on a thick clay layer and shows the resilience of the embankment towards the shaking and it can be seen that there are no wider cracks in the embankment body, there are still small cracks on the both toe sides of the embankment.

(a) (b) (c)

Figure 7 Plan view of the flood embankments after going through the consecutive shakings, showing the cracks development (a)YK01, (b)YK02, and (c)YK03

6 CONCLUSION

The centrifuge test results reveal that the soil stratigraphy plays a significant role in the dynamic response of the embankments. The presence of a non-liquefiable crust enhances the structural stability. The results also show that liquefaction-induced settlements decrease significantly when non-liquefiable layers are present in the foundation. The trend continues in the successive shakes as well. Moreover, the presence of a crust layer of sufficient thickness also reduces the development of cracks in the body of the embankments, which is also observed in the earthquakes that followed the first earthquake. It is not only the presence of the non-liquefiable layer but also its thickness that plays a very important role in the structural stability of the embankment.

ACKNOWLEDGMENT

The authors are grateful for the technical support extended by Mr John Chandler, Mr Kristian Pether, Mr Mark Smith, Mr Chris McGinnie, and Mr David Layfield at the Schofield Centre.

Table 4 Configurations of model centrifuge tests

Test ID	**Earthquake**					**Em-bank-ment height (m)**	**Foundation depth (m)**			
	ID	**Type**	**Fre-quency (Hz)**	**Cy-cles**	**PGA (g)**		**Hostun Sand**	**Kaolin Clay**	**RD, Sand, %**	**Clay su, (kPa)**
YK01	EQ1	Sinusoidal	1	10	0.11	3.96	9.96	N/A	48.5%	N/A
	EQ2	Sinusoidal	1	10	0.13					
	EQ3	Sinusoidal	1	10	0.24					
	EQ4	Imperial Valley	-	-	0.07					
	EQ5	Kobe	-	-	0.27					
YK02	EQ1	Sinusoidal	1	10	0.12	3.96	8.76	1.2	50.5%	14
	EQ2	Sinusoidal	1	10	0.15					
	EQ3	Sinusoidal	1	10	0.23					
	EQ4	Imperial Valley	-	-	0.07					
	EQ5	Kobe	-	-	0.29					
YK03	EQ1	Sinusoidal	1	10	0.11	3.96	6.96	3	49%	29
	EQ2	Sinusoidal	1	10	0.15					
	EQ3	Sinusoidal	1	10	0.23					
	EQ4	Imperial Valley	-	-	0.08					
	EQ5	Kobe	-	-	0.30					

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