

**Reinforcement approach based on arch
action in liquefaction-susceptible
embankments studied by dynamic
centrifugal model tests**

A SIGA

Abstract

The embankment structure is widely used in civil engineering, playing critical roles in transportation and water control. However, under seismic loading, these essential earthworks are often susceptible to deformation and failure, including a novel failure mode known as the arch collapse mechanism. In this mechanism, soil arching occurs within the embankment due to basal settlement. If saturation occurs, dynamic loading can cause the collapse of the soil arch, leading to the upper part caving into the liquefied zone and damaging the embankment. Recognizing that embankment damage is primarily caused by the collapse of the soil arch due to liquefaction has prompted a crucial exploration of strategies to minimize such damage.

A series of preliminary studies were conducted to modify the experimental procedure and measurement instruments to reproduce the arch collapse failure mode in centrifuge tests. Subsequently, centrifuge experiments were carried out under various conditions to investigate the influence of liquefaction and soil arch collapse on embankment damage modes. The results indicate that soil arching in embankments with basal settlement can mitigate large deformations caused by dynamic loads, as long as the arch action endures, regardless of whether internal liquefaction occurs or not. Embankment damage occurred in cases lacking a soil arch or where the arch collapsed under dynamic load. In summary, reinforcing the soil arch can effectively improve the deformation resistance of embankments. However, due to the complex stress conditions within the embankment during dynamic loading, the reliability of liquefaction evaluation methods has been discussed. Using the results of triaxial tests, an algorithm for data analysis was developed, and the energy-based method (EBM) was used to determine the liquefaction-triggering energy to predict liquefaction occurrence in centrifuge tests.

This dissertation aims to contribute valuable knowledge to the field by offering insights into soil arch interactions, liquefaction dynamics, and the influence of soil arch reinforcement on embankment behavior under seismic loading.

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Chapter 1. Introduction

1.1. Background

Japan, nestled in the Pacific Ring of Fire, is known for its frequent seismic activity and vulnerability to earthquakes. The unique geological setting of the region, coupled with the tectonic plate boundaries, makes it susceptible to intense seismic events. These earthquakes have had a profound impact on the stability of critical infrastructure, including road embankments, dams, and dikes. In geotechnical engineering, embankments are vital structures that ensure the stability and support of buildings and other constructions on the earth's surface. However, these embankments are vulnerable to numerous challenges, such as earthquakes and the settlement of the soil beneath them.

Most recently, after the 2011 off the Pacific Coast of Tohoku Earthquake, the main patterns of the damaged river embankments were reviewed based on the in-situ research (Oka et al., 2012a). The primary factors contributing to the observed damage were identified, notably including the liquefaction of the foundation ground, liquefaction of water-saturated soil within the embankments above ground level, and the consequential overflow and erosion induced by the accompanying tsunami (Oka et al., 2012b). While the conventional understanding attributes river embankment damage primarily to the liquefaction of sandy foundation ground, a noteworthy revelation emerged from the study—numerous embankments constructed on clayey ground also suffered significant damage due to liquefaction.

Fig. 1.1 illustrates typical soil profile patterns observed at the foundation of damaged levees resulting from the 2011 off the Pacific coast of the Tohoku earthquake. Categorically, type A embankments were built on a sand layer with the water table positioned below ground level. In contrast, types B to D embankments were situated on clayey soil, with the water table located within the embankment, signifying the presence of a saturated zone.

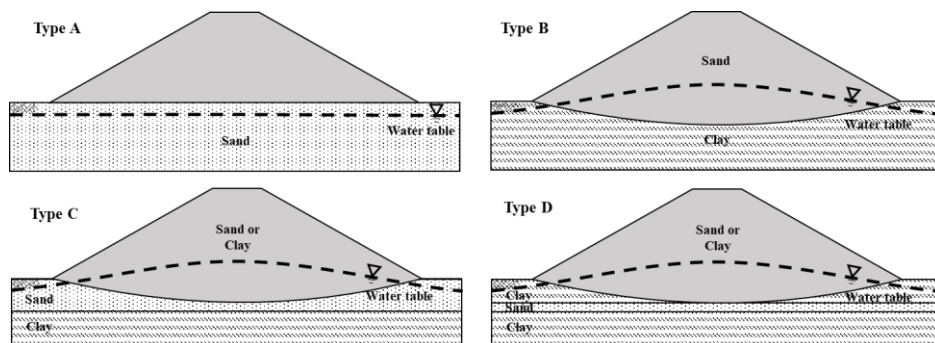
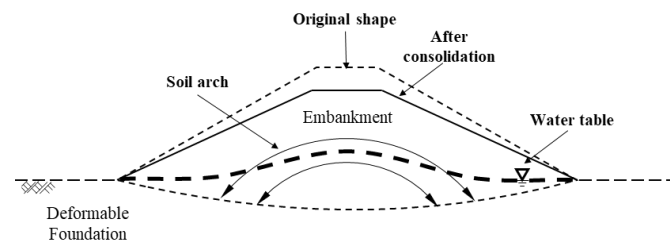


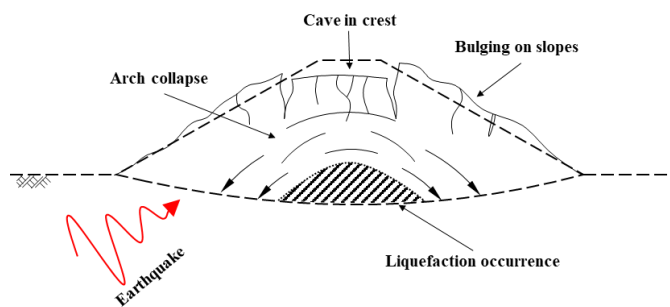
Fig. 1.1. Patterns of soil profiles(Oka et al., 2012a)

Based on the observed embankment pattern, a novel failure mode known as the arch collapse mechanism has been proposed, as illustrated in Fig. 1.2(Pipatpongsa, 2019). This theoretical framework identifies two key mechanisms leading to soil arch action: (1) Basal Settlement: The soil arch action initiates promptly after the commencement of centrifugal loading, primarily triggered by basal settlement. (2) Pore Fluid Supplying: The occurrence of soil arch is associated with centrifugal acceleration reaching 50G, driven by pore fluid supplying. The manifestation of soil-arch action results in a reduction of confining pressure, as depicted in Fig. 1.2 (a). This phenomenon amplifies the potential for liquefaction triggering. Under dynamic loading conditions, liquefaction transpires in the saturated zone beneath the soil arch. Consequently, the shear resistance in this zone approaches zero, rendering the upper part of the embankment highly susceptible to dynamic loadings. Concurrently, liquefaction-induced deterioration of confining pressure on the soil arch, which plays a pivotal role in the persistence of the arch(Michalowski & Park, 2005).

As liquefaction progresses, the collapse of the soil arch transpires, leading to the upper part caving into the liquefied zone and causing damage to the embankment. Observable effects include cave-ins on the crest and bulging on the slopes, as depicted in Fig. 1.2 (b). This comprehensive understanding of the arch collapse mechanism provides valuable insights into the vulnerability of embankments constructed on soft ground subjected to seismic loading.



(a) Arch action under the effect of the basal subsidence



(b) Arch collapse mechanism due to the seismic loading

Fig. 1.2. Schematic diagram of arch collapse mechanism(Pipatpongsa, 2019)

The realization that embankment damage is primarily caused by the collapse of the soil arch resulting from liquefaction has led to a crucial exploration of strategies to minimize such damage. The focus has shifted towards reinforcing the soil arch itself, aiming to increase confining pressure on the arch action and prevent its collapse during dynamic loading. This approach, reminiscent of the principles behind the New Austrian Tunneling Method (NATM) employed in tunnel construction, introduces a promising avenue for enhancing stability in embankments.

The New Austrian Tunneling Method (NATM) stands out as a widely embraced and cost-effective alternative to traditional shield techniques in the construction of tunnels, particularly in cohesive soil. One of its key components, rock bolts, plays a pivotal role in transferring loading or forces from unstable sections to a confined interior that is significantly stronger. Drawing inspiration from the success of NATM in tunnel construction, a parallel strategy is proposed for embankments facing basal settlement.

In Fix Principal Axis (FPA) hypothesis(Wittmer et al., 1997; Wittmer et al., 1996), the principal stress axes in a material element maintain a constant orientation fixed. By this hypothesis, in trapezoid embankment, the direction of major principal stress Ψ can be determined using Mohr's circle, as showed in Fig. 1.3.

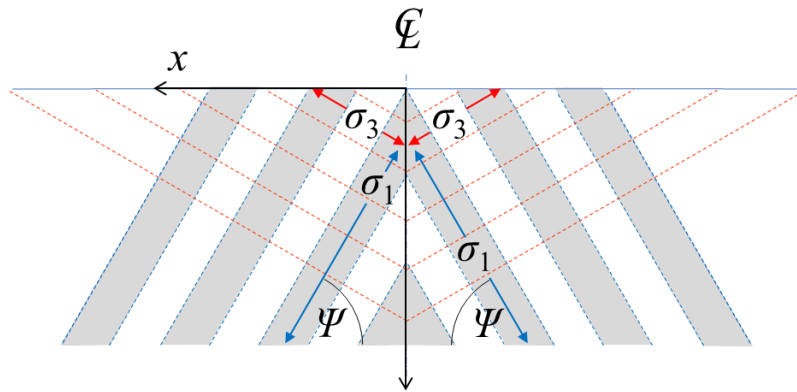


Fig. 1.3. Direction of principal stress in trapezoid embankment (Miyata et al., 2015)

The embankment reinforcement process involves careful consideration of soil-structure interactions, loading conditions, and the specific characteristics of the embankment in question. The collapse of the soil arch and the subsequent redistribution of earth pressure pose significant threats to the embankment's stability. To mitigate this risk, soil nails can be inserted into the embankment along the σ_3 direction as showed in Fig. 1.3, the soil arch collapse can be prevented and minimizes damage to the embankment. The placement of soil nails must be meticulously designed to optimize

their effectiveness in reinforcing the soil arch and preventing collapse during liquefaction-induced pressure variations.

1.2. Literature review

1.2.1. Embankment damages induced by earthquakes

The embankment structure, such as road embankments or dams, is a basic geotechnical structure in civil engineering, plays critical roles in transportation or water control. However, these important earthworks often susceptible to various threats, such as seismic damage. Under seismic loading, deformation and failure often occurred. In this occasion, the pattern of damage was influenced by several factors like: the properties of the earthquake wave, for example the frequency, the amplitude and duration; the properties of the embankment like the dimension or reinforcement condition; the materials of embankment or sublayers.

Japan is a country vulnerable to earthquakes, and have experiences many earthquake damages. The modern science and technology began to be introduced into Japan in the second half of the 19th Century(Towhata, 2008), and many important researches were conducted in the first half of 20th century, especially after the 1923 Kanto earthquake caused a great damage in Tokyo and Yokohama area. Then, in the second half of 20th century, people's lifestyle and types of infrastructures became drastically different from previous ones. In 1964, a great earthquake has occurred in Niigata, and after this earthquake, the liquefaction phenomenon has been started to be recognized as one of the earthquake-induced disasters. After the Niigata earthquake, the damage condition was reviewed based on the in-situ research, which pointed out that two kinds of damage to earth structures was found, the first one was the damage due to vibration of structure themselves, and the second one was the damage due to liquefaction of the sandy ground, and much damage of earth structure and foundation was resulted by the liquefaction of saturated loose sand. Also, the damage pattern of embankment was shown (Kawakami & Asada, 1966; Yamada, 1966). After this earthquake, the researchers started to investigate the mechanism of liquefaction, the evaluation of liquefaction potential and liquefaction mitigation based on case history studies, field investigations, and laboratory soil tests as well as shaking model tests (Koizumi, 1966; Yoshimi, 1967).

Then, after experienced such heavily damage caused by Niigata earthquake, a number of researchers have started to investigate the evaluation of liquefaction potential. A well-known method to evaluate liquefaction potential by compare the dynamic shear strength and dynamic shear stress was

first proposed Seed and Idriss (Iwasaki, 1986), and a series of research, experiment and in-situ investigation were conducted then (Iwasaki, 1978; Seed, 1979; Seed & Idriss, 1967, 1971).

The damage of embankment under seismic loading have been reported by many researchers. After Nihonkai-Chubu earthquake, the liquefaction-induced lateral displacement e in Noshiro City was reported (Hamada et al., 1986), and this damage have destroyed important pipeline for urban gas supply. It could be challenged to improve the soil and prevent liquefaction due to the financial problem, so a new technology should be found that allow liquefaction but avoid the fatal damage to the business and local communities. After 1993 Kushiro-Oki earthquake and 1995 Kobe earthquakes, it was reported that due to the remarkable length of existing dikes on potentially liquefiable subsoil, complete prevention of liquefaction is financially impossible. However, the function of embankment will not be influenced even if the height of embankment reduced, so measures that allow the liquefaction occurrence but can prevent the large deformation of embankment came into reality (Towhata, 2008).

In 1993, the Kushiro-Oki Earthquake struck the eastern part of Hokkaido, resulting in significant damage, including dike failure, revetment failure, and sluice damage. Notably, a novel failure mode was identified, wherein the earthquake-induced motion led to the development of cracks at top, and liquefaction of the saturated zone at the embankment's base (Kaneko et al., 1995; Sasaki, 1994). This unique failure mechanism arises from water being trapped within the embankment, rising above the ground surface, and the embankment settling into its base and below the water table before an earthquake occurs. The increase in water levels within the embankment can be due to rainwater seeping in, while subsidence is mainly seen in embankments built on compressible or weak soils, such as loose sand or soft peat foundations. Understanding the mechanisms of this specific failure mode is crucial for enhancing the resilience and seismic stability of embankment structures founded on diverse soil conditions (Sasaki, 1994; Shahbodagh et al., 2020).

During the 2011 Off the Pacific Coast of Tohoku Earthquake, the most severe damage was documented in specific sections exhibiting the following characteristics: (1) Continuous Liquefiable Layers in the Foundation: Instances where the foundation featured uninterrupted liquefiable layers experienced significant damage. (2) Compressible Foundation Beneath the Embankment: Sections with a compressible foundation situated below the embankment witnessed substantial impact. The basal settlement phenomenon occurred, leading to the lower part of the embankment settling into the ground below the water table. This subsidence caused the embankment to become saturated during the earthquake, intensifying the extent of the damage. (Harder et al., 2011; Sasaki et al., 2012). The failure mode and embankment patterns have been completely reviewed based on in-situ research (Oka et al.,

2012a). In sum, the failure mode of the embankment damage induced by earthquake can be schematized into 6 modes as showed in Fig. 1.4 (Sasaki et al., 2012; Shahbodagh et al., 2020).

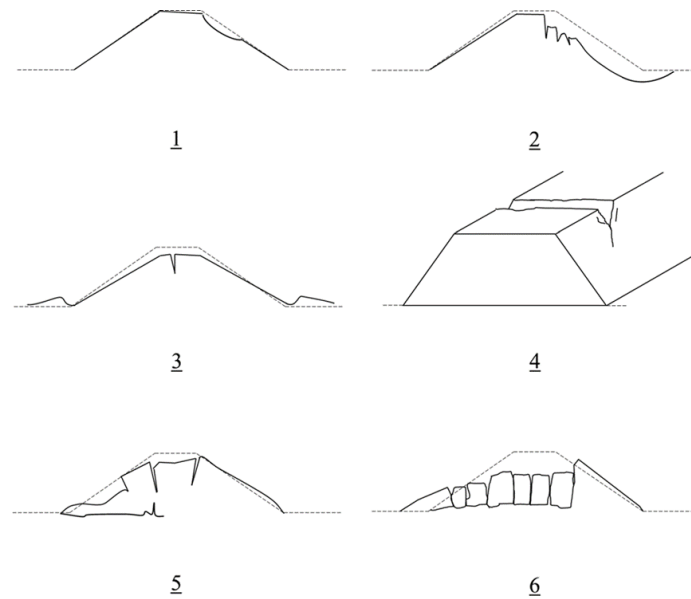


Fig. 1.4. Failure modes of river embankment (Sasaki et al., 2012; Shahbodagh et al., 2020)

As depicted in Fig. 1.4, Pattern 1 and Pattern 2 demonstrate shallow and deep circular sliding, traditional failure mechanisms frequently addressed in geotechnical engineering. Pattern 3 describes the settling of an embankment along with the upward movement of its toes, commonly seen in embankments on stiff or non-compressible foundations. Pattern 4 depicts transverse cracks and fissures along the embankment's length, caused by asynchronous lateral excitations. Pattern 5 occurs when the soil foundation weakens due to liquefaction, resulting in significant settlements of the embankment crest and shoulders. The lateral spreading of the embankment toe creates tension zones within the body, leading to longitudinal cracking. While liquefaction is a primary factor, highly compressible and soft foundations can produce similar effects even without liquefaction. Pattern 6 represents one of the most severe forms of embankment damage induced by earthquake-triggered liquefaction. The lateral spreading of the toe and the movement of the embankment slopes, driven by the destabilized liquefied or overly settled soft foundation, form fissured blocks within the embankment with considerable settlements.

Except the mentioned failure modes, a novel failure mode known as the arch collapse mechanism has been proposed, as illustrated in Fig. 1.2 (Pipatpongsa, 2019). In this mechanism, liquefaction occurrence will induce the confining pressure on the soil arch, the confining pressure is an important

property to for the arch stability (Michalowski & Park, 2005). So, the arch collapse will occur due to liquefaction, leading to the upper part caving into the liquefied zone and causing damage to the embankment. Observable effects include cave-ins on the crest and bulging on the slopes. This research provides an insight into the embankment's failure mode due to dynamic load.

1.2.2. Dynamic experiment of embankment

The extensive damages to embankments caused by earthquakes have spurred numerous researchers to delve into the failure mechanisms and behaviors of embankments under dynamic loading. A large number of physic experiments were conducted on this purpose. Notably, shaking table tests have played a crucial role, particularly in exploring embankment responses to soil ground liquefaction. One noteworthy set of experiments was conducted by Iwasaki in 1986, providing valuable insights into the effects of soil liquefaction on the behavior and stability of both pile foundations and soil embankments (Iwasaki, 1986). Koga et al. carried out six reduced-scale embankment shaking table model tests, revealing that, under shaking conditions, the ground beneath the embankment did not liquefy, in contrast to the easily liquefied free ground (Koga & Matsuo, 1990). Further shaking table tests were performed by Park et al. to examine the behavior of an embankment situated on liquefiable sand ground (Park et al., 2000). These tests reported significant deformations in the embankment, including crest settlement and lateral displacement.

Nonetheless, data derived from model tests in the laboratory may lead to misconceptions. This is because the self-weight of soil in a small model test, subjected to the acceleration of normal gravity, results in significantly lower effective stresses at various points in the model compared to real-world conditions. Therefore, in addition to conventional 1G shaking table tests, centrifuge modeling stands out as a superior alternative for studying embankment failure induced by liquefaction. The performance of such embankments has been a subject of investigation, leading to notable research outcomes. Adalier explored the dynamic response of embankments on loose saturated sand layers (Adalier et al., 1998). In a series of centrifuge tests, the behavior of embankments on saturated medium-dense sand, overlain by a low-permeability silt deposit, was examined, shedding light on the liquefaction process and mechanisms involved (Zeghal et al., 1999). Moreover, dynamic centrifuge tests were performed to evaluate the seismic performance of embankments, both with and without countermeasures, with the goal of developing standard methods for assessing crest settlement. (Okamura & Matsuo, 2002). To probe into the seismic stability and potential flow liquefaction of loose fill slopes, dynamic centrifuge tests were carried out in uni-axial and bi-axial directions on saturated

model embankments composed of loose, completely decomposed granite (Ng et al., 2004). A series of centrifuge model tests was conducted by Adalier in 2004, the behavior of embankment and saturated sandy soil foundation under dynamic load was investigated (Adalier & Sharp, 2004).

In 2008, embankment model tests were conducted to explore various conditions, including pile reinforcement and changes in height (Yun-Min et al., 2008). Okamura (2011; 2013) conducted a significant series of centrifuge tests using two distinct models: embankments resting on a rigid foundation soil layer and embankments situated on a soft clay deposit. He pointed out that while well-compacted embankments on non-liquefiable foundation soils with level surfaces are usually considered stable even under strong earthquake ground motions, certain factors, such as an increased thickness of the saturated zone within the embankment and decreased stresses in the embankment soil due to subsidence, may lead to potential instability. Higo et al. (2015) conducted a series of dynamic centrifuge model tests with varying water contents to explore how water content influences the deformation and failure behaviors of unsaturated embankments. More recently, Tiznado have conducted a series of centrifuge test to investigate the effect of Dense granular columns (DGCs) to mitigate the liquefaction (Tiznado et al., 2020). The deformation, liquefaction pore pressure ratio and the settlement of embankment on soil ground were investigated by centrifuge tests (Pramaditya & Fathani, 2020). Adapa (2021) conducted research on the behavior of embankments under different upstream conditions and densities. The study revealed that upstream conditions are critical in determining the embankment's failure mechanism, especially in low-density scenarios. Additionally, it was found that the distribution of water content within the embankment significantly affects its response to shaking, more so than the water level height on the upstream side.

PIV technique was used to obtain the strain distributions inside the embankments, the stress-strain relationship of embankment material can be used to evaluate the level of embankment damage (Izawa et al., 2022). The failure modes of embankment on liquefiable ground with columns was investigated by centrifuge test, the research also investigated the difference of columns with and without the steel reinforcement cores (Pourakbar et al., 2022). Centrifuge experiments and numerical simulations were combined to study the behavior of embankments constructed on liquefiable sand foundations. The research discussed excess pore pressure, acceleration response, settlement, and the deformation pattern of the embankment model. The numerical simulation showed good agreement with the centrifuge test results. As depicted in Fig. 1.5, the embankment was built on a liquefiable base (Saade et al., 2023).

interaction between a buried cylindrical tube and the surrounding soil, the pressure redistribution and arching were discussed (Luscher & Høeg, 1964). Handy have developed the mathematical theory of arch development, the develop stages of soil aching was investigated (Handy, 1985). In 1997, centrifuge tests were used to conduct the trapdoor experiment to investigate the soil arching (Adachi, 1997). Furthermore, static centrifuge experiments were conducted on active trapdoor models (Dewoolkar et al., 2007). The results show that the behavior of soil arch in a high gravity field was different with that in 1G condition.

In geotechnical engineering, the soil arching usually occurred in embankments reinforced by pile and geosynthetics. A number of experiments of embankment have been conducted in different conditions. For instance, embankment model tests were conducted to explore various conditions, including pile reinforcement and changes in height (Yun-Min et al., 2008). Han et al. (2015) investigated the properties of soil arching under dynamic loads by conducting numerical studies using the Finite Element Method (FEM) and model tests. Patel et al. (2021) conducted time-history analyses on basal geogrid-reinforced, pile-supported embankments, varying the embankment height and the tensile modulus of the geogrid.

In fact, except the piled embankment, the studies of soil arching induced by settlement began many decades ago. The experiments and have been conducted by Trollope to investigate the soil arching induced by the base deflection (Trollope, 1956; Trollope & Burman, 1980). The study of Lee investigated stress distribution in a granular embankment, concluding that linear elastic theory is suitable for rigid bases, arching theory is not applicable to rigid bases, and vertical settlement alone is insufficient for a fully plastic state without horizontal displacements (Lee & Herington, 1971).

In 2010, a closed-form solution for three stress components was determined (Pipatpongsa, 2010). The typical geometries of sand heap of constant unit weight γ lying at the angle of repose ϕ with the horizontal are illustrated in Fig. 1.6. This closed-form stress solution provides a method to obtain the soil-arching effect of embankment. A series of embankment model tests was conducted in 1G condition, the influence of embankment height was investigated by compare the result of test and solution as showed in Fig. 1.7 (Miyata et al., 2014).

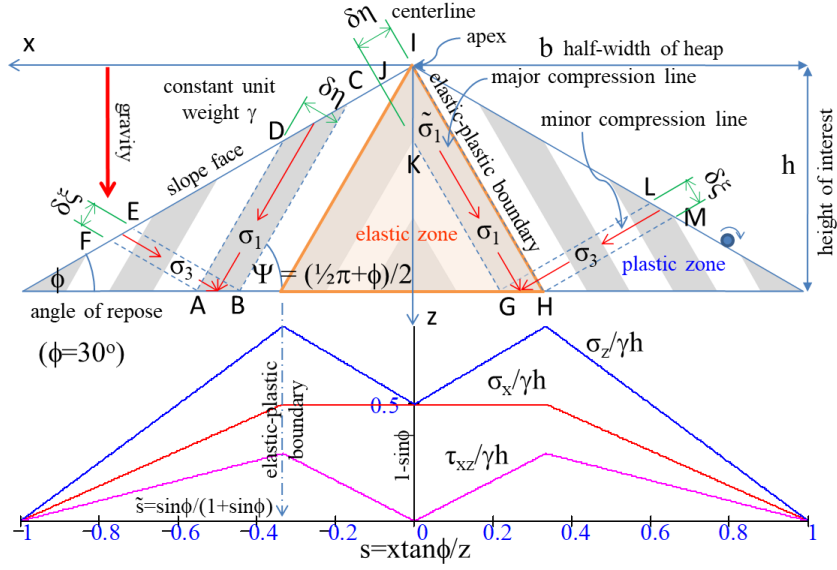


Fig. 1.6. Analyzing the stress distribution across the relative width of a symmetrical planar sand heap, formed at its angle of repose, involves scaling the stress from the apex to the height of interest (Pipatpongsa, 2010)

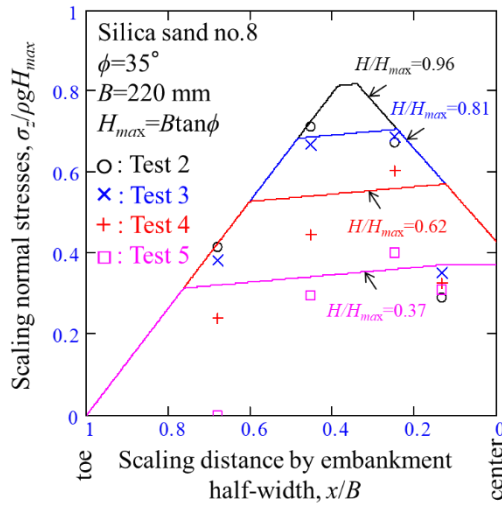


Fig. 1.7. Stress condition of embankment in different height (Miyata et al., 2015)

Additionally, Pipatpongsa et al. (2015) investigated the active and passive limits in sand heaps using the Newton method. They validated their findings through physical models of an inclined embankment, showing reasonable consistency between the active and passive limits and measurements within a specific range of deformation. Sand heaps were studied under incipient active failure conditions, exploring stress distribution at the angle of repose; through experiments with sinusoidal seismic loading, it was observed that the theoretical and experimental vertical pressure profiles gradually aligned after the state of incipient failure (Wanjala et al., 2020). The numerical stress

solutions of sand heaps under both active and passive conditions were solved by Nguyen (Nguyen & Tran, 2021).

1.2.4. Soil nail reinforcement

Piles play a crucial role in civil engineering, such as enhancing stability in embankments. Various methods have been employed for landslide stabilization, with investigations into pile design considering factors like interval, length, and diameter (Ito et al., 1981). Studies extended the design method to multiple rows (Ito et al., 1982). Chen explored the theory of vertical piles under lateral soil movements, leading to further developments based on various analysis methods (Chen & Poulos, 1997). For instance, the three-dimensional elasto-plastic shear strength reduction finite element method predicted slope stabilization (Cai & Ugai, 2000), and limit equilibrium analysis proposed a design method to prevent slope failure (Yamagami et al., 2000). Additionally, researchers utilized limit analysis to examine the stability of pile-reinforced slopes (Ausilio et al., 2001), contributing to the well-developed theory of reinforcement behavior analysis.

Experiments on piled slopes were also conducted to investigate slope stability behaviors. Extensive 1G model tests were performed to study strip-footing behavior, demonstrating a significant increase in the bearing capacity of the footing due to the presence of piles (El Sawwaf, 2005). Experimental research investigated the effectiveness of small-diameter, structural pile elements in stabilizing slopes through full-scale lateral load tests. These tests validated an analytical method and introduced a novel approach to assess the stability of pile-reinforced slopes, addressing both strength and service limit states (Thompson & White, 2006). Full-scale experiments using controlled blasting were conducted in the Port of Tokachi on Hokkaido Island to assess the behavior of piled slope (Ashford et al., 2006). The dynamic centrifuge model tests, considering factors like nail length, spacing, and slope inclination, revealed that the use of nails effectively prevented progressive failure during an earthquake, altered the acceleration response, and decreased slope deformation within a nail-influence zone, highlighting the significance of nail length and spacing for enhanced slope stability (Wang et al., 2010). Dynamic centrifuge tests demonstrated that stabilizing piles enhance slope stability during earthquakes, reducing deformation and exhibiting a triangular bending moment. Factors such as pile spacing, placement, slope gradient, and earthquake input influence the response of pile-reinforced slopes (Wang & Zhang, 2014). Rawat and Gupta (2016) studied the response of unreinforced and soil-nailed slopes to gradually increasing surcharge loads, both experimentally and numerically using finite element analysis. The study highlighted variations in maximum load-bearing capacity and crest settlement among different

slope configurations. Centrifuge experiments were conducted by Fang (2019) to investigate the behavior of undercut slopes reinforced by soil nails. The prediction technique for undercut slope failure was proposed using geotechnical centrifuge modeling, recording slope surface movements with a high-speed camera and analyzing them with image processing software (Pipatpongsa et al., 2021).

The NATM method, originating in Austria in the 1960s (Rabcewicz, 1964), entails initial support through shotcrete and reinforcement such as fiber or welded-wire fabric, steel arches, and occasionally ground reinforcement. The NATM has gained popularity in Japan for shallow tunnels and challenging ground conditions. The behavior of rock bolts was found through the experiments of two lowering panels including theoretical consideration (Yamamoto & Ohno, 1985). The high-friction anchored bolts were used to address bonding issues by pull-out test results, 3D finite difference method, and field application examples (Okabe et al., 2014). Comparative bench tests were carried out by Pytlik (2019) on steel arch supports, with and without reinforcement by self-drilling bolts and rock bolts. The study showed that employing mixed support systems substantially improves stability.

While the New Austrian Tunneling Method (NATM) may not have direct applicability to embankments, the utilization of rock bolts in tunnel construction offers inspiration for reinforcing arches in embankments. Drawing parallels, if soil nails employed in embankments can emulate the supportive function of rock bolts, there is potential to enhance the stability of soil arches, thereby mitigating the risk of embankment collapse.

Soil nailing is a widely utilized technique for reinforcing and stabilizing slopes, excavations, and embankments. Originally developed in the 1960s in Europe, it has since become a standard practice in geotechnical engineering for enhancing the stability of soil structures (Bonazzi & Colombet, 1984). The fundamental principle of soil nailing involves inserting slender reinforcing elements, typically steel bars or rods, into the ground to form a composite mass with increased shear strength and stability. These nails are usually installed at a slight inclination and are often combined with shotcrete or other surface protection layers to control erosion and improve the overall strength of the embankment or slope.

In engineering practice, soil nailing has proven effective in a wide range of applications, such as stabilizing highway and railway cut slopes, supporting deep excavations in urban settings, and reinforcing retaining walls and bridge abutments. It is particularly well-suited for stabilizing existing soil structures with minimal disturbance to the surrounding environment. One of the key advantages of soil nailing is its adaptability to different site conditions, making it a flexible solution for complex geotechnical challenges. For example, in urban areas where space is limited and traditional retaining

structures may be impractical, soil nailing can be implemented quickly and efficiently with minimal excavation.

Additionally, soil nails can be used to stabilize landslide-prone slopes or embankments that experience gradual movement. In these cases, the soil nail system increases the internal friction of the soil and provides long-term stability without the need for massive earthworks. The method's ability to be retrofitted into existing structures further enhances its versatility, allowing engineers to upgrade older infrastructure without significant reconstruction efforts (Byrne et al., 1996).

Since the 1980s, the use of soil nail reinforcement has expanded across Europe and North America (Bo et al., 2015). Its benefits include improved cost efficiency, reduced environmental impact, faster installation, and lower material consumption compared to traditional methods such as soldier piles or cast-in-place walls. Due to its relatively low footprint and faster installation process, soil nailing is also often preferred in environmentally sensitive areas and locations where construction downtime needs to be minimized (Dey, 2015).

In this study, soil nails have been used to reinforce embankments subjected to basal settlement and dynamic loading. By incorporating soil nails, the stability of the soil mass is significantly enhanced, mitigating the collapse of soil arches during dynamic events such as liquefaction. The nails increase the confining pressure along the potential failure planes, thereby improving the embankment's resistance to deformation and collapse. The practical application of soil nailing in this context demonstrates its efficacy in addressing the challenges of embankment reinforcement under complex loading conditions, including dynamic forces and settlement-induced stresses, which are common in geotechnical engineering projects.

1.2.5. Energy-based method in liquefaction evaluation

Liquefaction of saturated sand beneath the soil arch during the earthquakes will cause the arch collapse in embankment. The liquefaction phenomenon may occur due to the density decrease caused by basal settlement, and saturation. To evaluate the occurrence of liquefaction, some methods was used to determine the liquefaction potential of a soil deposit. In 1971, Seed and Idriss have developed stress-based method (SBM) of liquefaction evaluation. This simplified procedure provides a way to evaluate the liquefaction potential using the shear stress and the number of cycles. This method was wildly used in many engineering practices. However, the cyclic resistance ratio (CRR) is defined for a harmonic motion of a given amplitude and a number of cycles to represent liquefaction resistance

under a given design earthquake. Hence, the liquefaction evaluation in an irregular earthquake should be discussed.

Except from the stress-based method, the dynamic response stress-strain behavior of soil layers has been recognized to have a correlation with the soil liquefaction. A nonlinear relationship was described by Towhata and Ishihara (1985). as showed in Fig. 1.8. The soil was characterized as viscoelastic, indicating that when subjected to cyclic loading, the resulting strain shows a phase shift relative to the applied stress. As shown in Fig. 1.8 , this phase shift δ results in a closed-loop strain-stress relationship. The enclosed elliptical loop, represented by the area ΔW , denotes the energy dissipated during a single loading cycle (Kokusho, 2017). Conversely, the maximum elastic energy stored during cyclic loading occurs at the peak stress or strain, corresponding to the areas of triangles OAB or OA'B'.

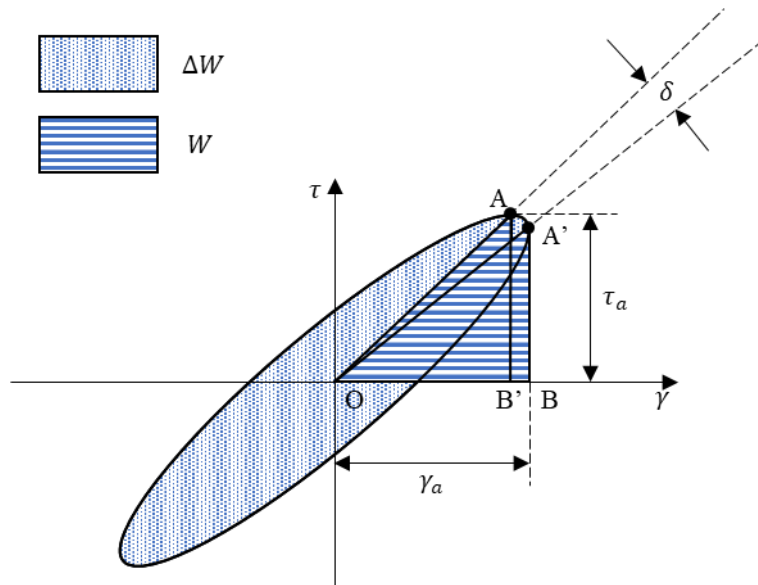


Fig. 1.8. General stress-strain relationship of viscoelastic material

The dissipated energy during cyclic loading has been identified as having a correlation with liquefaction, regardless of the amplitude, period, and irregularity of earthquake waves. The Energy-Based Method (EBM) is grounded on the principle that these soils dissipate energy into the ground during deformation under dynamic loads. Therefore, an energy-based method (EBM) could potentially be more straightforwardly applied compared to the Stress-Based Method (SBM), by comparing dissipated energy with the required energy without needing to consider differences in earthquake motions. Based on the research of Nemat-Nasser and Shokooh (1979), the EBM have been firstly

proposed by Davis and Berrill (1982). Davis and Berrill assumed a linear relationship between the pore pressure build up and the dissipated energy. Simcock et al. (1983) have conducted a series of cyclic triaxial tests and have found the relationship between pore water pressure, then, Davis and Berrill modified this relation and provides an energy model as:

$$\frac{\Delta u}{\bar{\sigma}_c} = a \left[\frac{\delta W}{\bar{\sigma}_c} \right]^b \quad (1.1)$$

Where $\bar{\sigma}_c$ is the effective confining pressure, δW is the dissipated energy per unit volume.

Then, a series investigation was conducted and the results have showed the correlation between dissipated energy and excess pore pressure independent of shear stress history (Figueroa & Dahisaria, 1991; Figueroa et al., 1994; Law et al., 1990; Liang, 1995). In situ liquefaction potential evaluation was conducted by Kazama et al. (1999) and compared with the laboratory test result. Here, a series of research was conducted to investigate the correlation between the dissipated energy and pore water pressure during dynamic process. However, the earthquake demand energy was not directly calculated, the application of EBM in engineering was limited, so from 2013, a new type of EBM was developed by Kokusho (2013), a comprehensive study was conducted, combining laboratory test results on energy dissipation and in situ seismic energy evaluation methods. This study aimed to propose a simplified Energy-Based Method (EBM) and compare its results with a conventional Stress-Based Method (SBM) using typical soil models. In 2018, Kokusho and Kaneko have confirmed the uniqueness of dissipated energy not only in harmonic motion but also in irregular earthquake. Using the EBM, a correlation between dissipated energy and CRR was investigated by a series of research, which was described as a quadratic relationship showed in Eq. (1.2) and (1.3) (Kokusho & Kaneko, 2018; Kokusho & Mimori, 2015; Kokusho, 2017, 2021; Kokusho & Tanimoto, 2021).

$$\sum \frac{\Delta W}{\sigma'_c} = 3.5 \cdot (CRR_{20} - 0.1)^2 + 0.008 \quad (1.2)$$

$$\sum \frac{\Delta W}{\sigma'_c} = 2.7 \cdot (CRR_{15} - 0.1)^2 + 0.008 \quad (1.3)$$

The number of cycles for the initial liquefaction based on energy method were agreed well with those based on stress method through results obtained from a series of undrained cyclic triaxial tests (Pipatpongsa, 2022), as showed in Fig. 1.9.

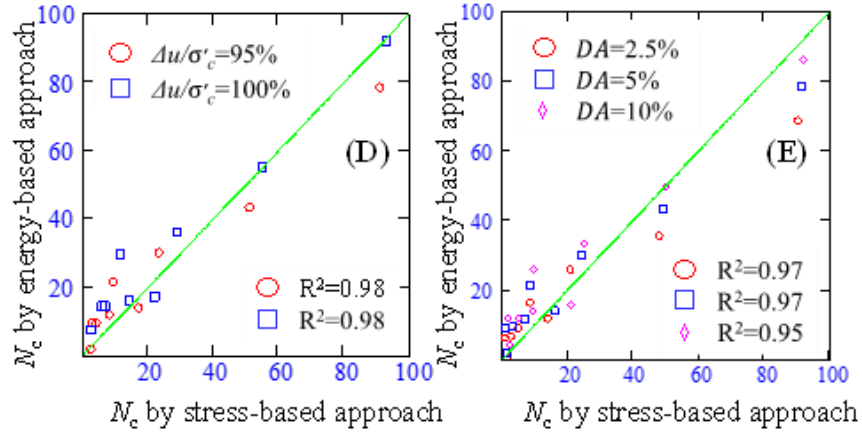


Fig. 1.9. Number of cycles of initial liquefaction (Pipatpongsa, 2022)

1.3. Scopes and organization

The literature review introduced the failure patterns of embankments, the soil-arching action, and the use of centrifuge experiments on embankment models. However, in the previous study, the research about arch collapse mechanism was insufficient. Specifically, there has been a lack of experimental investigation into the arch action induced by basal settlement in well-compacted trapezoidal embankments. Additionally, experiments with a saturated zone inside embankments have been rare, and the occurrence of liquefaction within these zones has not been thoroughly studied. Furthermore, the numerical stress solutions for trapezoidal embankments have not been systematically explored. The interaction between liquefaction and soil-arching, and their combined impact on the stability of embankments, remains insufficiently investigated.

Also, this research underscores a critical finding: the most substantial damage occurred due to the liquefaction of the water-saturated region confined by soft clay layers characterized by lower permeability levels. The elevated water table within embankments, surpassing ground surface levels, is a result of various factors, including rainfall and capillary action. This dissertation uniquely focuses on embankments constructed on impermeable layers with saturated zones, recognizing the specific challenges and vulnerabilities inherent in such geological configurations.

This dissertation addresses these significant gaps by advancing our understanding of liquefaction, soil-arching, and the arch collapse mechanism. In the context of embankments, which are prone to damage induced by liquefaction, the strategic placement of soil nails towards the bottom center emerges as a potential solution. Soil nails, hold the capability to fortify the soil arch by amplifying

confining pressure. This proactive reinforcement aims to avert the collapse of the arch triggered by a decrease in confining pressure during liquefaction-induced loading.

The research involved conducting centrifugal shaking table model tests and dynamic triaxial tests, developing closed-form stress solutions using the fixed principal axes method, and utilizing energy-based analysis. Based on the reviews, following objectives should be studied to delve further into the mechanics of soil arching effects, the stability of the arch itself, and the interplay between soil arching and liquefaction:

- (1) To study static and dynamic load transmissions in embankments with a saturated zone subjected to basal settlement using geotechnical centrifuge model tests.**
- (2) To explore the interaction between soil arching and liquefaction, and their combined impact on embankment failure modes and the onset of collapse mechanisms.**
- (3) To propose an innovative concept for reinforcement to inhibit the damage in embankments prone to internal liquefaction and seismic instability.**
- (4) To investigate the application of energy-based method (EBM) in evaluating liquefaction susceptibility of embankments under various conditions.**

In order to achieve these objectives, a series of studies was conducted, as shown in Fig. 1.10, outlines of **Chapter 2** to **Chapter 8** are summarized below.

Chapter 2: This chapter delves into the fundamental principles of centrifuge model testing and the basic properties of materials. The basic test conditions are introduced, laying the groundwork for the subsequent experimental chapters.

Chapter 3: This chapter details a preliminary study involving a series of centrifuge experiments designed to trigger soil arching effects and liquefaction, and to reproduce arch collapse failure modes in laboratory tests. It also examines the dynamic response of embankments at different depths and introduces the methodology for data analysis.

Chapter 4: This chapter focuses on the improvement of the experimental procedure and the assembly of a two-dimensional earth pressure sensor using a 3D printer. The reliability of its measurement for the two-dimensional state of stress is evaluated through experiments under centrifuge acceleration, contributing to the accuracy of data collection and interpretation.

Chapter 5: Conducting a series of centrifuge experiments under varying embankment heights, this chapter explores the soil arching effect and its formation development under different height conditions.

Chapter 6: This chapter introduces the use of soil nail reinforcement in embankment model construction. A series of centrifuge experiments is conducted under various reinforcement methods to

investigate arching effects, liquefaction resistance, and deformation behavior. Through dynamic centrifuge tests, this study aims to provide comprehensive insights into the behavior of shear-reinforced embankments under various conditions.

Chapter 7: The algorithm for data analysis developed in this chapter focuses on energy analysis and its use in future research. The results of triaxial tests were used to obtain energy-based method parameters and predict the occurrence of internal liquefaction of an embankment model in centrifuge tests using the energy method.

Chapter 8: This chapter provides the conclusion of this dissertation and outlines future work.

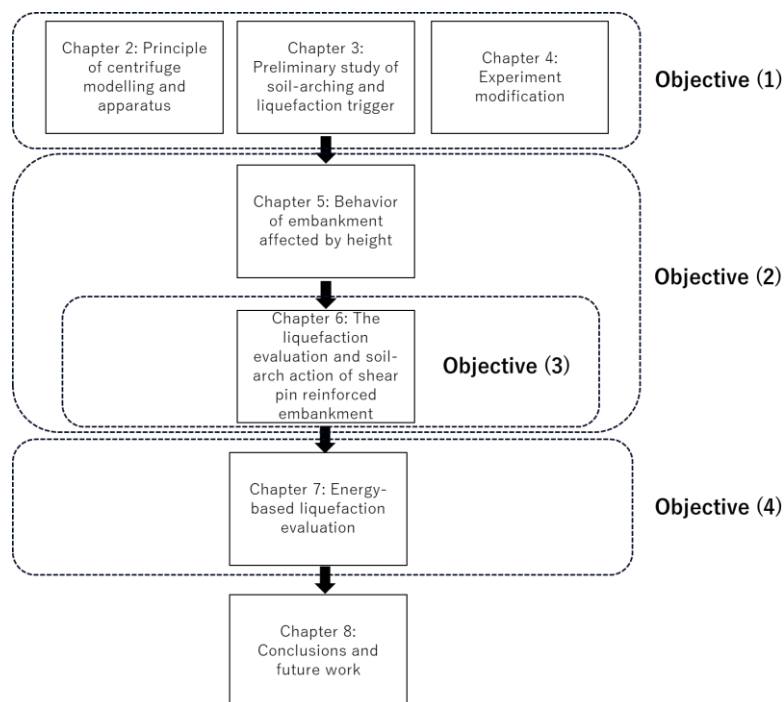


Fig. 1.10. Structure of the thesis and corresponding objectives

Chapter 2. Principle of centrifuge modelling and apparatus

2.1. Introduction

Due to the limitation of size, boundary condition or material properties, conducting the in-situ geotechnical tests can be challenged. Instead, the laboratory tests of soil structures provide an effective way to investigate a wide range of problems in geotechnical engineering. Nevertheless, data from model tests in the laboratory can be deceptive due to the reduced effective stresses caused by soil self-weight in a small-scale test under normal gravity acceleration, which differs significantly from the conditions encountered in the field. Hence, the centrifuge tests, as model test, were used to simulate the soil condition in full-scale condition more precisely.

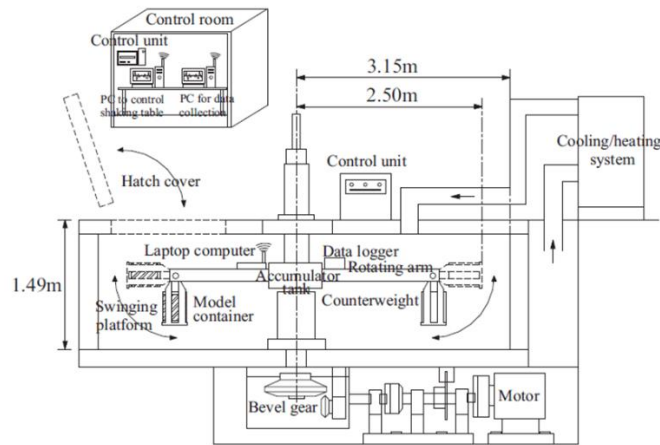
The earliest concept of using a centrifuge to simulate increased self-weight in small-scale models was pioneered by Phillips in Paris in 1896 (Craig, 2018). However, he only suggested but did not conduct a real centrifuge test. Subsequently, in the 1930s, significant early advancements in geotechnical modeling using centrifuges took place in the USSR, following independent proposals made by Davidenkov and Pokrovskii. In the 1960's, the first geotechnical centrifuge was built in Japan by Mikasa (Kimura, 1998). Today, centrifuge testing has been successfully applied in various fields, including retaining walls, embankments, slopes, and more. The mechanical behavior of a prototype soil mass under the Earth's gravity, g , can be replicated in a small-scale model at a scale of $1/N$. This model needs to be subjected to an inertial acceleration field N times that of Earth's gravity to achieve an acceleration field similar to that experienced by the prototype. Ovesen (1979) proposed a set of scaling factors derived from dimensional analysis. The typical scaling relationships used in centrifuge modeling are summarized in Table 2.1.

Table 2.1. Scaling factors in centrifuge test

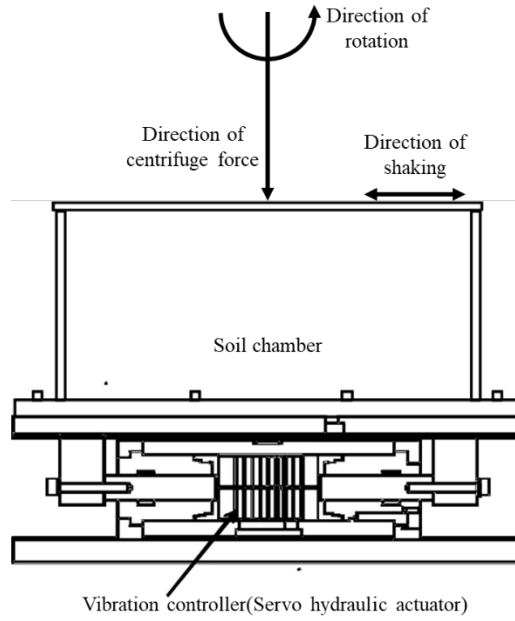
Parameter	Scaling factor (Prototype/Model)
Length	N
Mass	N^3
Force	N^2
Frequency	$1/N$
Stress	1
Strain	1
Pore water pressure	1
Displacement	N
Acceleration	$1/N$
Time(consolidation)	N^2
Time(shaking)	N
Hydraulic conductivity	$1/N$

2.2. Centrifuge apparatus

In this thesis, the geotechnical centrifuge machine located at Disaster Prevention Research Institute (DPRI), Kyoto University was used to investigate the behavior of embankment under dynamic loading. The schematic diagram and the specifications of centrifuge machine was showed in Fig. 2.1(a) and Table 2.2 (Higo et al., 2015).



(a) Schematic diagram of centrifuge of centrifuge machine



(b) Schematic diagram of centrifuge of shaking table

Fig. 2.1. Schematic diagram of centrifuge apparatus(a) Centrifuge machine; (b) Shaking table (Higo et al., 2015)

Table 2.2. Specification of centrifugal machine

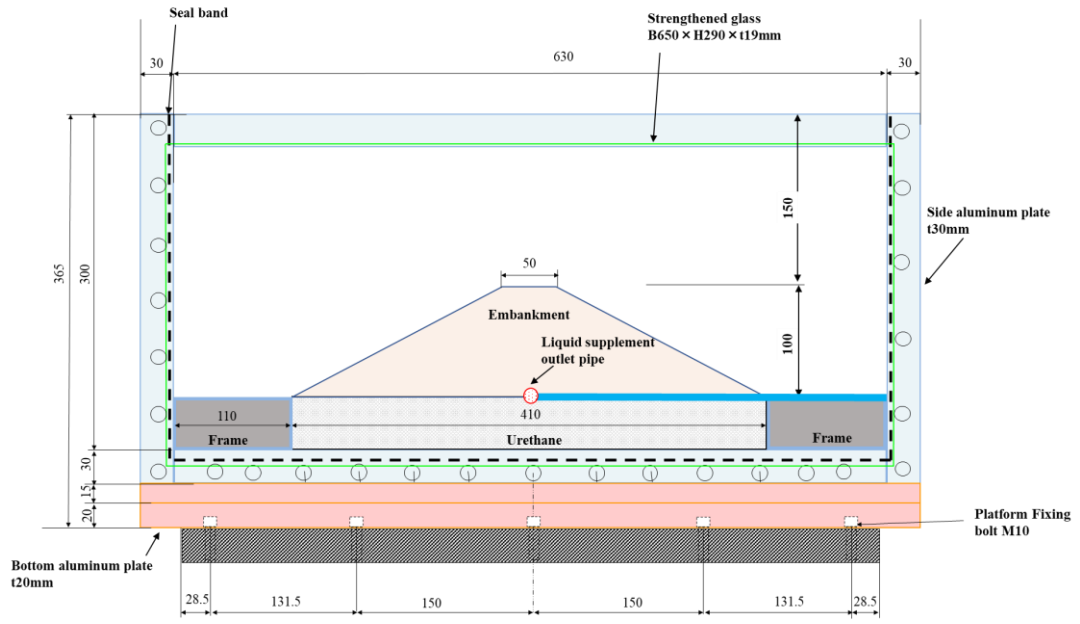
Effective rotation radius	2.50(m)
Effective space for model installation(L×W×H)	800×320×800(mm)
Allowable weight of model	120(kg)
Test capacity	24(g/t)
Maximum centrifugal acceleration	200(G)

The dynamic load was applied by the shaking table, the schematic diagram and the specifications of centrifuge machine was showed in Fig. 2.1(b) and Table 2.3(Higo et al., 2015). In this study, the amplitude of input wave was controlled by displacement, the input acceleration was detected by the accelerometer attached on the shaking table directly.

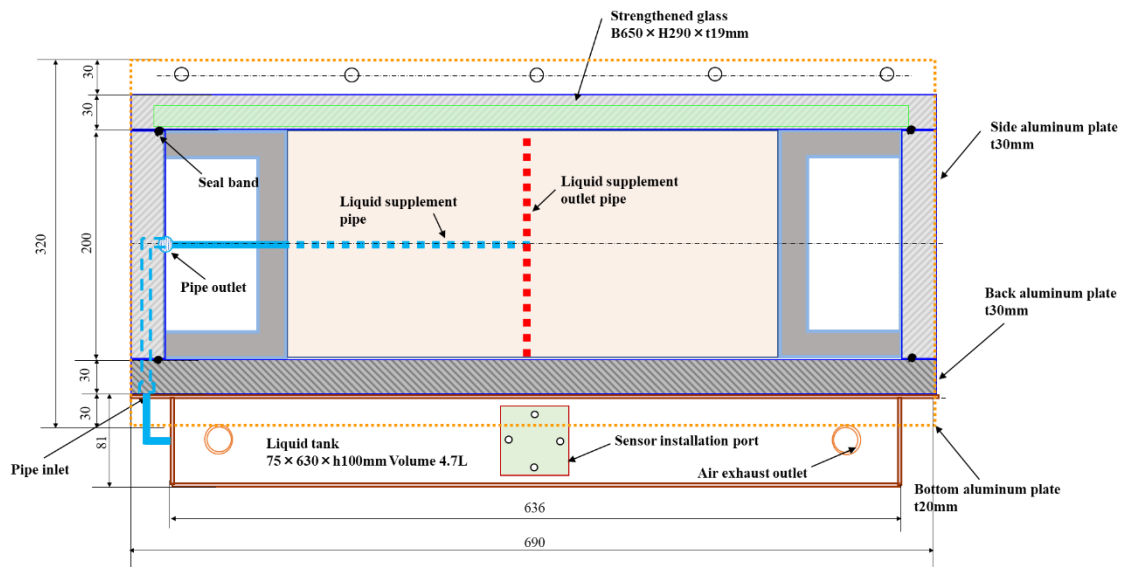
Table 2.3. Specification of shaking table

Vibration control method	Servo oil pressure control
Allowable centrifugal acceleration	50(G)
Displacement	± 5 (mm)
Maximum frequency	200 (Hz)
Input of waveform	Sine wave/Arbitrary wave
Allowable weight of model	100 (kg)

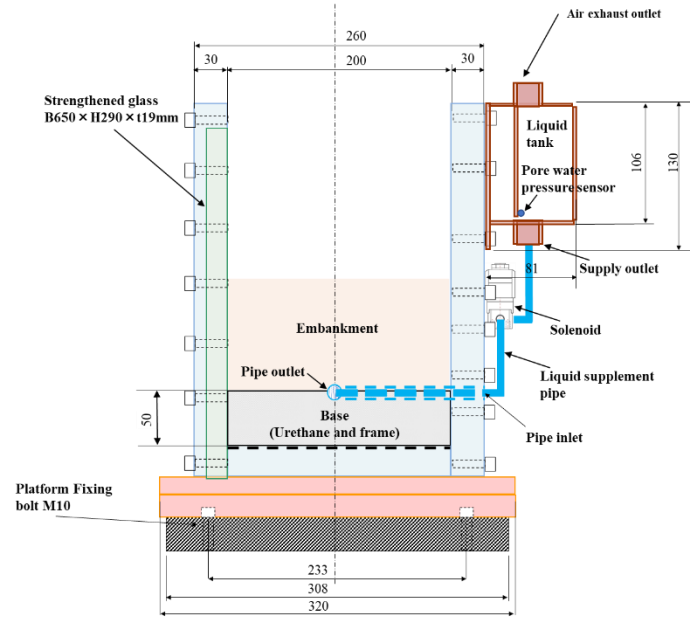
In this experiment, an aluminum tank with internal dimensions measuring 63cm $\times$ 20cm $\times$ 30cm (width $\times$ depth $\times$ height) was used. The tank features a glass side, enabling the observation of the vertical cross-section of the model. On the back of the tank, an additional liquid tank with dimensions of 63cm $\times$ 8cm $\times$ 10cm (width $\times$ depth $\times$ height) was attached, serving as the reservoir for the fluid supplied during the experiment. Fluid is delivered to the embankment through a tube connected from the liquid tank to the interior of the embankment. Fluid supply is regulated based on the dynamic water gradient, determined by the difference in height between the tank water surface and the supply position. The height difference between the tank bottom and the embankment bottom is 20 cm in model scale. During the experiment, the liquid-supplying was controlled by the solenoid, the solenoids can be controlled by the PC in control room, once the solenoids opened, the liquid will flow into the embankment through the water pipe. The schematic diagram of soil chamber was showed in Fig. 2.2. The picture of front and back view of soil chamber was showed in Fig. 2.3.



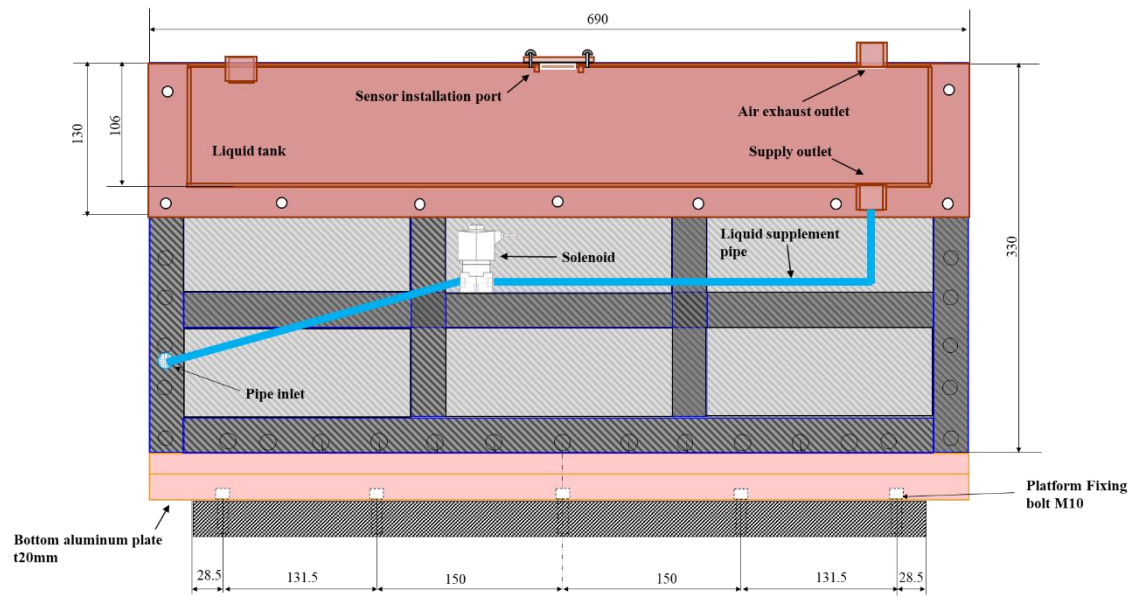
(a) Front view



(b) Top view

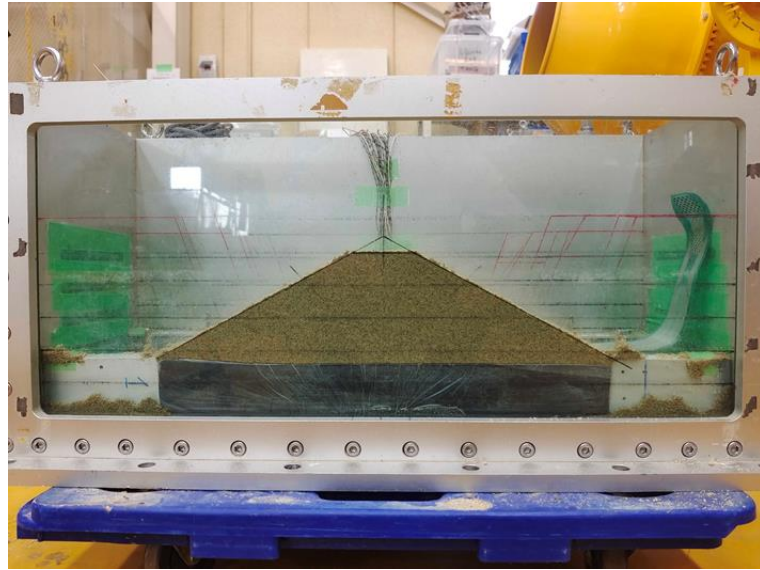


(c) Right view

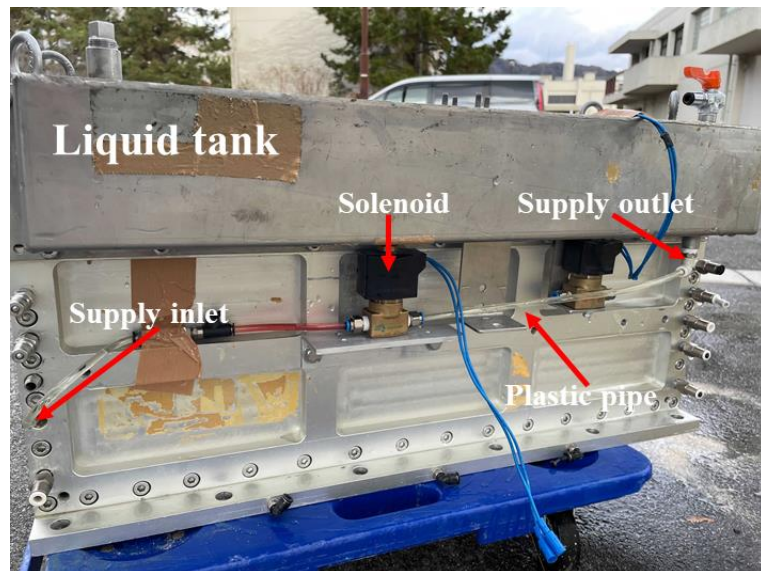


(d) Back view

Fig. 2.2. Schematic figure of soil chamber(a) Front view; (b) Top view; (c) Right view; (b) Back view



(a) Front view of soil chamber



(b) Back view of soil chamber

Fig. 2.3. Appearance of soil chamber(a) Front view; (b) Back view

2.3. Materials

The soil used for constructing the embankments was sieved Hiroshima sand sourced from Kure City in Hiroshima Prefecture, provided by Yamamoto Kenzai Co., Ltd. The raw sand was collected from a mountainous area, resulting in a high gravel content and low uniformity. To improve the distribution curve and ensure it stayed within the very easily liquefiable range of grain-size distribution

(Iwasaki, 1986) , the sand was sieved to achieve a maximum particle diameter of 1mm, as shown in Fig. 2.4.

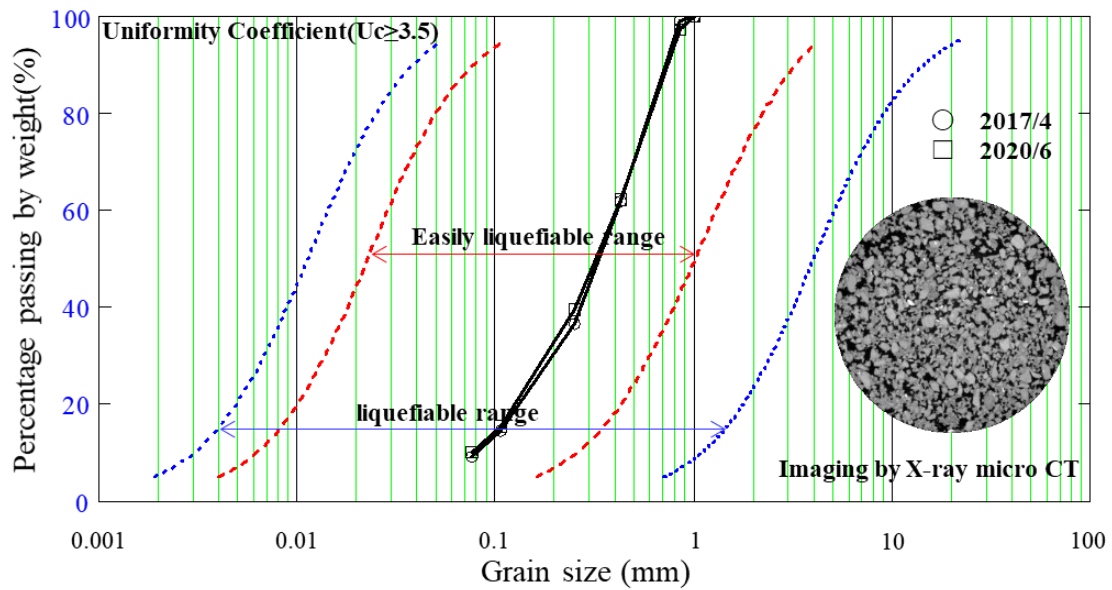


Fig. 2.4. The grain size distribution of the sieved Hiroshima sand

Compaction test was conducted to investigate the compaction curve of sieved Hiroshima sand, the results are presented in Fig. 2.5. In this thesis, the compaction degree in all centrifuge cases were 90%, and the water content varies from 4% to 14%, the compaction curve and the water content-dry density points of centrifuge cases were marked in figure too. The condition of sand is defined as humid with water content of 4% and 6%, as damp in 8%, 10%,12%, and as moist in 14%. The soft base was constructed by Urethane. The properties of Sieved Hiroshima sand and Urethane was showed in Table 2.4.

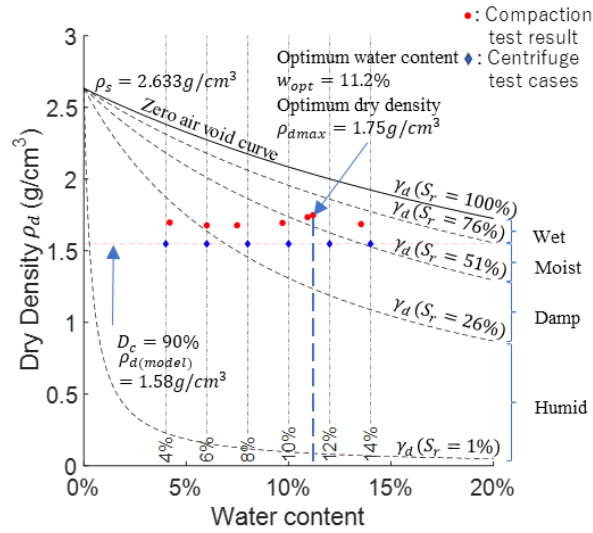


Fig. 2.5. Compaction curve of Hiroshima sand

To adjust the permeability coefficient of Hiroshima sand under 50G centrifugal acceleration, Metolose (Shin-Etsu Chemical Co., Ltd.) was used as a replacement for water during the centrifuge test. The kinematic viscosity was adjusted to 50 cSt, which is 50 times that of water.

Table 2.4. Physical properties of materials

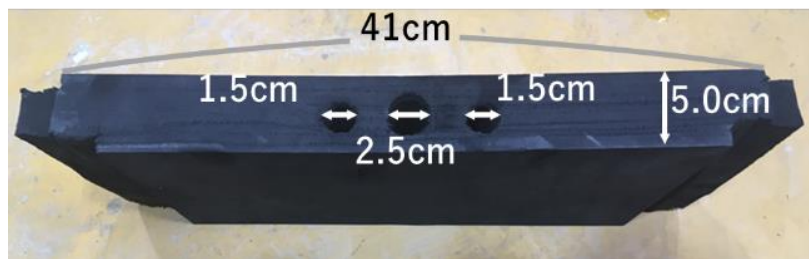
Hiroshima sand	
Soil particle density ρ_s	2.633 g/cm ³
D_{10}	0.08 mm
D_{30}	0.20 mm
D_{50}	0.34 mm
D_{60}	0.40 mm
Fine fraction content FC	9.2%
Maximum void ratio e_{max}	1.003
Minimum void ratio e_{min}	0.441
Dry maximum density ρ_{dmax}	1.75g/cm ³
Optimum water content w_{opt}	11.2%
Degree of compaction D_c	90%
Coefficient of permeability k	$2.4 \times 10^{-5} m/s$
Urethane	
Young's modulus	126~129kPa
Poisson's ratio	0.282±0.044
Tensile strength	0.39MPa
Elongation at break	150%
Density	0.18±0.03g/cm ³
25% compression load	29.4~58.8kPa

2.4. Model preparation

The preparation of embankment model in all cases have followed a same method. As the first step, the urethane material which has been mentioned earlier was crafted into dimensions of $50\text{cm} \times 20\text{cm} \times 5\text{cm}$ (width $\times$ depth $\times$ height), with indentations created at both the top and bottom ends, as showed in Fig. 2.6(a). In the lower indentation, steel weights ($5\text{cm} \times 20\text{cm} \times 1\text{cm}$, width $\times$ depth $\times$ height) were securely fastened using wire, to avoid the material floating during the process of fluid supplying. Two kinds of Urethane base were used in this thesis, the solid base and drilled base. In some cases, several holes were drilled at the center of the Urethane base to induce a larger basal settlement, as depicted in Fig. 2.6(b). The entire base was then covered by the vinyl, to simulate an impermeable flexible ground. Since the length of Urethane base was shorter than the length of soil chamber, two frames were affixed on both sides to stabilize the Urethane base under dynamic load. The frames and urethane base were then placed in the bottom of embankment. The schematic diagram of embankment experiment was presented in Fig. 2.8.



(a) Urethane base



(b) Drilled urethane base

Fig. 2.6. Urethane base (a) Solid; (b) Drilled

Then, the plastic pipe connected with the liquid tank (as showed in Fig. 2.3(b)) was introduced into the soil chamber, the inside diameter of the pipe was 3mm, and the ending side of the pipe was connected with the plastic nozzle. The nozzle was made by plastic stiff tube, which many tiny holes with diameter of 0.3 mm was drilled, to lest the liquid go pass through. The nozzle was placed on the center line of the Urethane base top, as shown in Fig. 2.7.

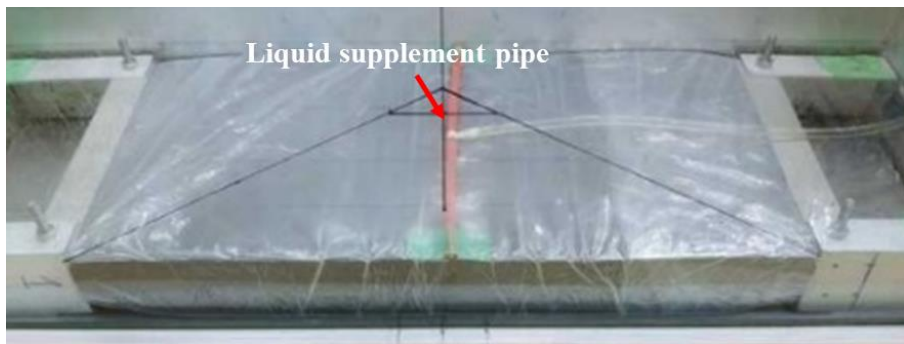


Fig. 2.7. Location of liquid supplement pipe

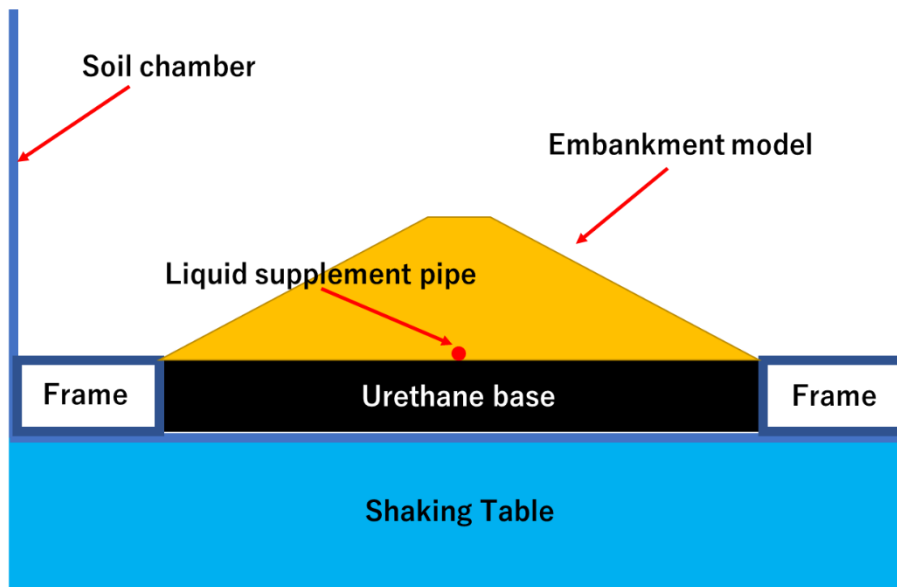


Fig. 2.8. Schematic diagram of model preparation

The embankment model was constructed following the standardized procedure outlined below:

(1). **Adjusting Water Content:** The water content of the Hiroshima sand was adjusted using water as the pore fluid. Metolose was avoided during this step to prevent its high viscosity from influencing compaction.

(2). **Stacking Wooden Plates:** Wooden plates, with thickness ranging from 11.5 to 17.5 mm, were stacked to provide temporary boundaries for each layer during compaction.

(3). **Compacting the Sand:** The sand was compacted within the boundaries defined by the wooden plates to maintain a constant 90% degree of compaction. Sensors were installed inside the model during this process to monitor its behavior. The embankment model after compaction is shown in Fig. 2.8.

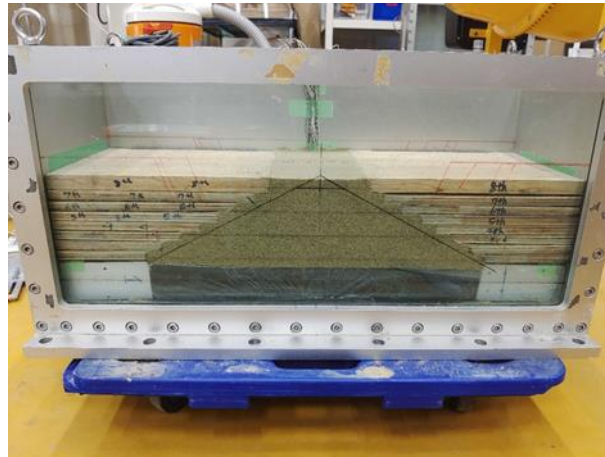


Fig. 2.9. The model after compaction

(4). **Layer-by-Layer Construction:** Steps 1 to 3 were repeated layer by layer until the embankment model reached the target height. After compaction, the wooden molds were meticulously removed, and the embankment model was trimmed to achieve the desired shape. Fig. 2.10 shows the model after removing the wooden plates and after trimming.

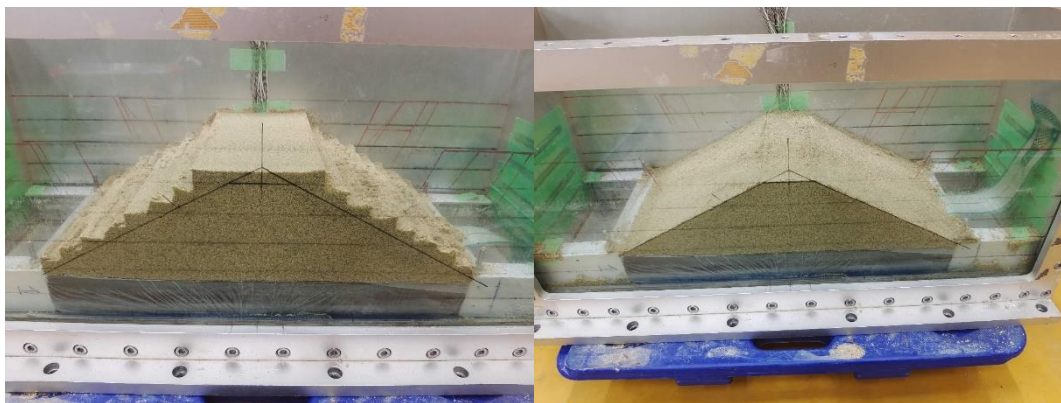


Fig. 2.10. The model before and after trimming

The test conditions vary in some case, the embankment height may be different and the soil nail may be inserted into embankment as reinforcement. But the base setting compaction were same in each case. More details will be introduced in future chapters.

2.5. Test conditions and experiment program

After the model preparation, the soil camber was putted and fixed with the shaking table, and the centrifuge tests were conducted using following steps:

(1) The centrifugal acceleration was applied into the model until 50G, and hold for a while until the recording data become stable. The data was collected by the sensors, and the sampling frequency was set to 1Hz before the dynamic loading.

(2) Using the remote PC in control room to switch the solenoid on, let the liquid flow into embankment model. Precisely control the flow rate was challenged, the amount of liquid was controlled by the decreasing of liquid pressure in liquid tank.

(3) After the saturation step, the data recording was turned off, then the sampling frequency was adjusted into 10 kHz. The recording was turned on again, and the dynamic loading was applied.

It should be noticed that in some cases, the white-noise will be introduced after the centrifugal loading step. And the amplitude and type of input wave varies with cases, which will be introduced in each chapter.

Based on the above-mentioned procedures, there are 17 cases conducted with various test conditions, such as base features, water content, embankment height and reinforcement condition. The model condition was listed in Table 2.5

Table 2.5. Model conditions

Test No.	Test ID	Base feature	Height	Water content	Reinforcement
1	SB-1-4-N	Solid	100%	4%	None
2	SB-1-6-N	Solid	100%	6%	None
3	SB-1-8-N	Solid	100%	8%	None
4	SB-1-10-N	Solid	100%	10%	None
5	SB-1-12-N	Solid	100%	12%	None
6	SB-1-14-N	Solid	100%	14%	None
7	DB-1-8-N-1	Drilled	100%	8%	None
8	DB-1-8-N-2	Drilled	100%	8%	None
9	DB-0.75-8-N	Drilled	75%	8%	None
10	DB-0.5-8-N	Drilled	50%	8%	None
11	DB-1-8-R6-1	Drilled	100%	8%	6 nails
12	DB-1-8-R6-2	Drilled	100%	8%	6 nails
13	DB-1-8-R6-3	Drilled	100%	8%	6 nails
14	DB-1-8-R4-1	Drilled	100%	8%	4 nails
15	DB-1-8-R4-2	Drilled	100%	8%	4 nails
16	DB-1-8-R6-4	Drilled	100%	8%	6 nails
17	DB-1-8-R4-3	Drilled	100%	8%	4 nails

The test identity was abbreviated as follows: DB-1-8-N-2 is identified as **D**rilled Base; Relative height **100%**; Construction water content **8%**; **N**o reinforcement; Test Number **1**.

Chapter 3. Preliminary study of soil-arching and liquefaction trigger

3.1. Introduction

The dynamic behavior of the embankment centrifuge will affect by many factors like embankment height or compaction degree. Many researchers have tried to study the dynamic behavior of embankments by centrifuge tests(Enomoto & Sasaki, 2015; Higo et al., 2015; Okamura et al., 2013). In addition, seismic behavior affected by construction water content and those mechanisms have not been systematically studied.

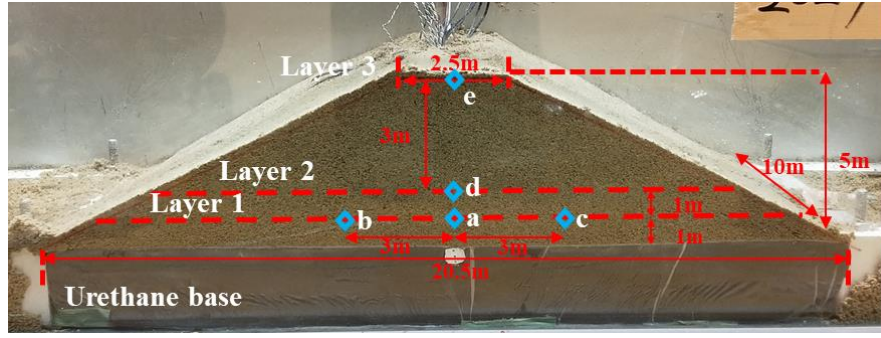
The shear stress condition is the basis for evaluating the deformation and strength of the embankment. The dynamic shear stress is the important basis for evaluating the behavior of embankment in a centrifuge test. Many researchers have studied the shear stress and shear strain in the embankment centrifuge test (Kazama et al., 1996; Koga & Matsuo, 1990).

The preliminary study was conducted in this chapter to reproduce the soil arching effect under basal settlement, and trigger the liquefaction by dynamic load. In this study, dynamic centrifugal model tests of embankments under different water contents with a fluid supply system are conducted to verify the dynamic behavior of the embankment in different water content. Embankment models are constructed in the same compaction degree with a water content from 4% to 14%. The Metolose will supply into the embankment model after the centrifugal acceleration reaches 50G, and the dynamic wave has been applied to the model. Through the shear stress, deformation, water level, and shear strain of embankment model provided by the earth pressure sensor, pore water pressure sensor, accelerometer and image analysis, the behavior of embankment in different water content was studied.

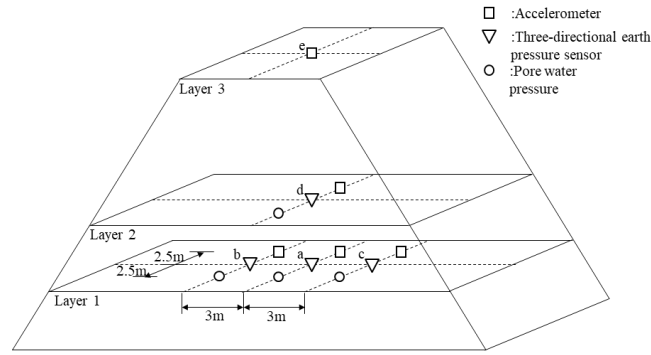
3.2. Centrifuge tests conditions

3.2.1. Model preparation and test conditions

The setup of the model and the positions of measuring instruments are depicted in Fig. 3.1(a) in prototype scale. The embankment model, which had varying water content, rested on an impermeable Urethane base, which has been mentioned earlier. We followed the specific standardized method to prepare the model, the water content of Hiroshima sand was adjusted in each case (4%, 6%, 8%, 10%, 12%, 14%), and the model height was 10cm (5m in prototype under 50G). The test condition of all cases was showed in Table 3.1.



(a) Embankment model



(b) Measuring instruments

Fig. 3.1. Centrifuge model at prototype scale (a) Shape and sensor location of embankment model; (b) Installation of measuring instruments

Table 3.1. Physical properties of materials

Case No.	Test ID	Water content
1	SB-1-4-N	4%
2	SB-1-6-N	6%
3	SB-1-8-N	8%
4	SB-1-10-N	10%
5	SB-1-12-N	12%
6	SB-1-14-N	14%

The types and locations of the sensors are also depicted in Fig. 3.1(b). In total, there were 5 accelerometers (ACC), 4 pore pressure transducers (PP), and 4 three-directional earth pressure gauges (TEP) installed to measure the physical behavior of the embankment during dynamic processes, the details of the three-directional earth pressure gauges will be provided in a subsequent section. For example, Fig. 3.2 illustrates the positions of measuring instruments in layer 1. The experimental data were analyzed under the assumption of plane strain conditions to ensure longitudinal consistency.

Moreover, one accelerometer was directly installed on the soil chamber to detect the input dynamic load, and one pore pressure transducer was placed inside the water tank to monitor water levels, which will be described later. Additionally, two laser displacement sensors were positioned at the top of the soil chamber to track settlements and horizontal displacements of the embankment crest.

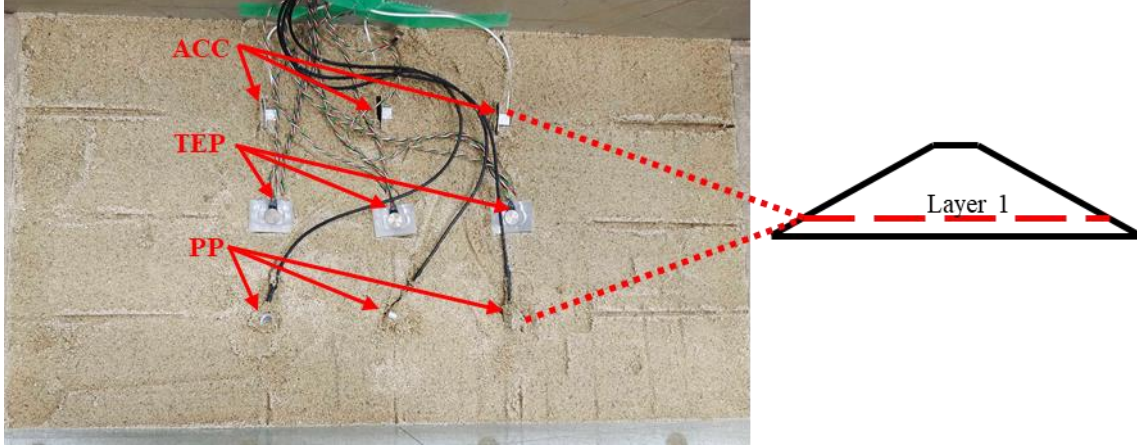
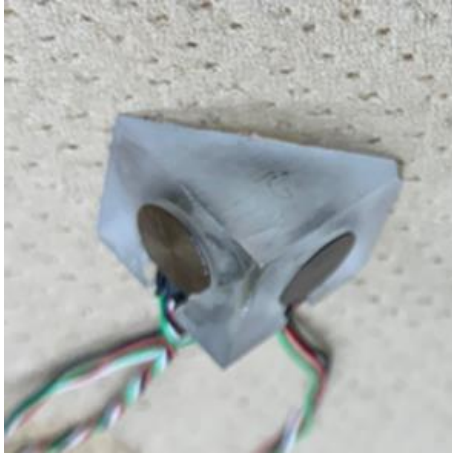


Fig. 3.2. Installation of measuring instruments

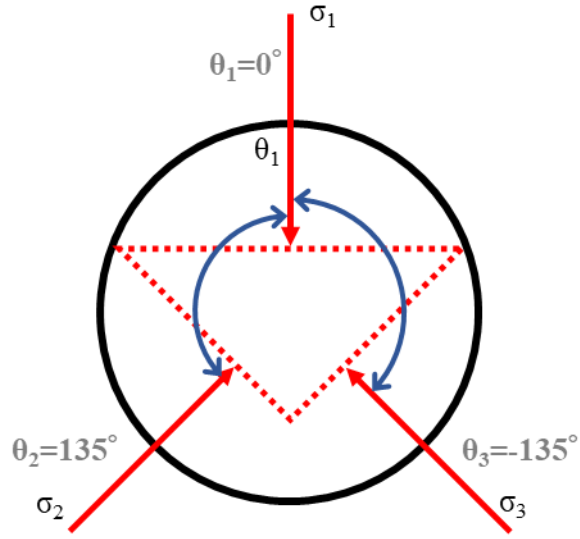
In this research, a test device assembled as three-directional earth pressure sensor was used, and the shear stress, vertical earth pressure, and horizontal earth pressure were calculated by decomposing the measured values. The device was made by processing a high-strength acrylic cube into a triangular prism shape, and three earth pressure sensors were installed on three faces of it. The two-dimensional stress tensor formulation for stress decomposition is as follows. A photograph of the acrylic triangular prism and the components of σ_1 , σ_2 , and σ_3 are shown in Fig. 3.3.

$$M = \frac{1}{2} \begin{bmatrix} 1 + \cos(2\theta_1) & 1 - \cos(2\theta_1) & 2 \sin(2\theta_1) \\ 1 + \cos(2\theta_2) & 1 - \cos(2\theta_2) & 2 \sin(2\theta_2) \\ 1 + \cos(2\theta_3) & 1 - \cos(2\theta_3) & 2 \sin(2\theta_3) \end{bmatrix} \quad (3.1)$$

$$\begin{Bmatrix} \sigma_y \\ \sigma_x \\ \tau_{xy} \end{Bmatrix} = M^{-1} \cdot \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{Bmatrix} \quad (3.2)$$



(a) Acrylic triangular prism



(b) Stress component

Fig. 3.3. Three-directional earth pressure sensor (a) Acrylic triangular prism; (b) Stress component

3.2.2. Experimental program

The dynamic centrifuge test is comprised of three main steps: centrifugal loading step, liquid supplement step and dynamic loading step. During the centrifugal loading step, the centrifugal acceleration gradually increased from 1G to 50G. Then, the white noise will be introduced first to alleviate the local soil arch caused by compaction and sensor installation. Following this, a controlled volume of Metolose-based liquid was supplied. This liquid was stored in a water tank attached on the back side of the soil chamber, and the liquid flow was controlled by the solenoids, once the solenoids opened, the liquid will flow into the model through the water pipe above the urethane base. Precise control of liquid volume was achieved by monitoring the pore pressure transducer within the water tank, with water level measurements derived from water pressure readings. The fluid was supplied until the pore water pressure gauge in the tank decreased around 5 kPa.

After the liquid supplement, the model was then kept at 50G for a while until the water level reached a stationary state. Then, five ramped sine waves with varying amplitudes were independently applied. The ramped sine wave is based on a 1Hz sine wave. Across a total of 30 cycles, the amplitudes for the first 5 cycles progressively increase from 0 to their maximum, while in the final 5 cycles, they gradually decrease from the maximum back to 0. The single amplitude in prototype scale for each input wave was 0.40 m/s², 0.70 m/s², 1.10 m/s², 1.85 m/s², and 3.35 m/s², as measured by the

accelerometer placed at bottom of the soil chamber. Fig. 3.4 showed the input acceleration of white noise, and an example of dynamic load.

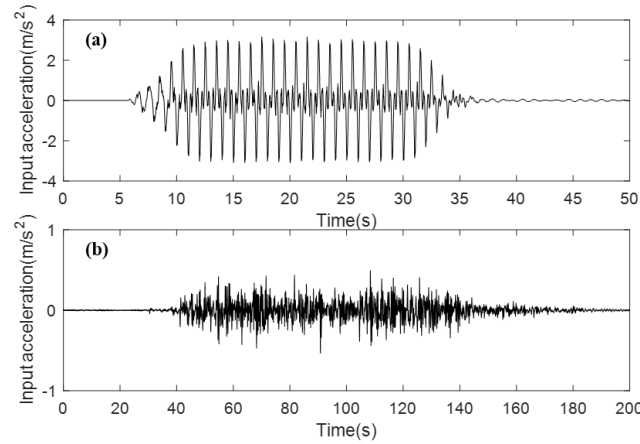


Fig. 3.4. Input acceleration time histories (a) Ramped sine wave; (b) White noise

3.3. Accelerometer-based calculation

3.3.1. Analysis methodology of stress and strain

While the three-directional earth pressure sensors mentioned earlier could provide shear stress data, their accuracy was significantly affected by embankment settlement resulting from the soft base deformation and the imposition of dynamic loads. Any shifts in sensor orientation or position compromised their precision. The laser displacement sensor, although capable of measuring displacement at the embankment's top, couldn't directly capture internal embankment displacement. Consequently, in this study, acceleration was employed to calculate shear stress and shear strain within the embankment.

In accordance with the research of Koga and Matsuo (Koga & Matsuo, 1990), the horizontal displacement of each accelerometer (d_n) can be calculated by double integrating acceleration (a_n) using Eq.(3.3). However, integrated displacement often tends to accumulate unexpected drift due to data recording errors. To address this, a high-pass filter and frequency integration were employed during calculation, the results before and after filter was showed in Fig. 3.5.

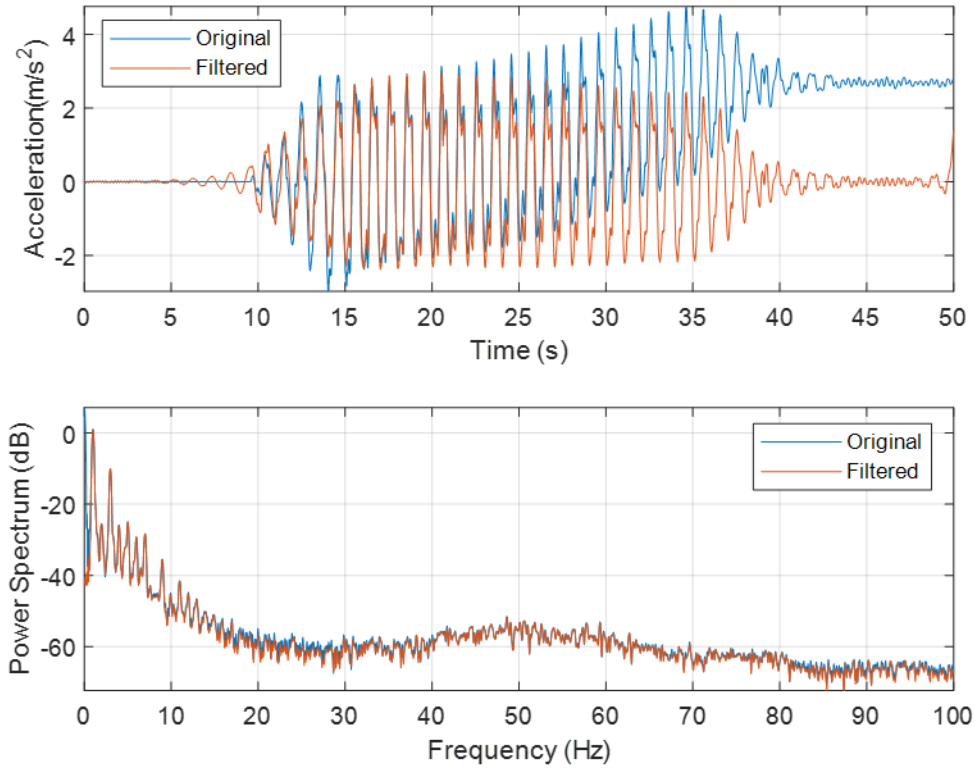


Fig. 3.5. Acceleration after a high-pass filter

$$\iint (a_n(t) dt) dt = d_n \quad (3.3)$$

Dynamic shear strain γ was determined by the differential displacement between two adjacent accelerometers, as illustrated in Fig. 3.6, using Eq.(3.4):

$$\gamma = \frac{\Delta d}{\Delta h} \quad (3.4)$$

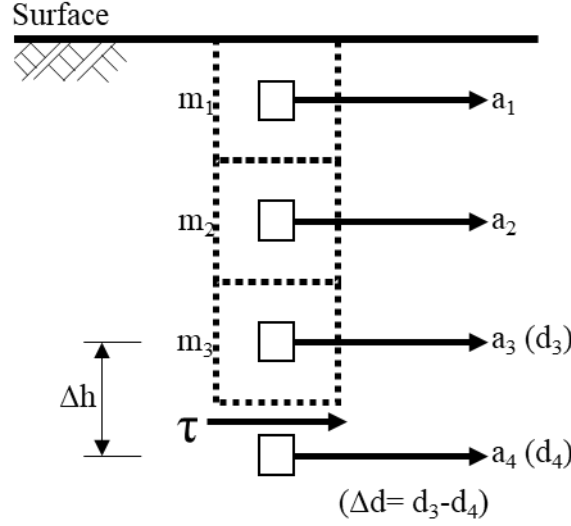


Fig. 3.6. Explanation of how to calculate shear stress and shear strain

As seen in Fig. 3.6, dynamic shear stress on a horizontal plane at a given depth could be roughly estimated by integrating the inertial force from the surface to that depth, as expressed in Eq.(3.4). This calculation assumed that the normal stresses acting on both sides of the soil column (as shown in Fig. 3.6) are equal:

$$\tau_n = \sum_{i=1}^n m_i a_i \quad (3.5)$$

In which m represents the mass of the column, and a is the response acceleration. Similarly, using a comparable method, shear stress within the embankment can also be determined. As depicted in Fig. 3.7, data recorded by three accelerometers installed along the centerline (locations 'a,' 'd,' and 'e' in Fig. 3.1) were utilized to calculate shear stress. This calculation takes into account the wet density (ρ_t), the installation distance (h) between the accelerometer, and the measured value (a) recorded by the accelerometer. The shear stress was computed as follows:

$$\tau_1 = \sum ma = \rho_t \cdot \left(\frac{h_2}{2} \cdot a_3 + \frac{h_1 + h_2}{2} \cdot a_2 + \frac{h_1}{2} \cdot a_3 \right) \quad (3.6)$$

$$\tau_2 = \sum ma = \rho_t \cdot \frac{h_2}{2} (a_2 + a_3) \quad (3.7)$$

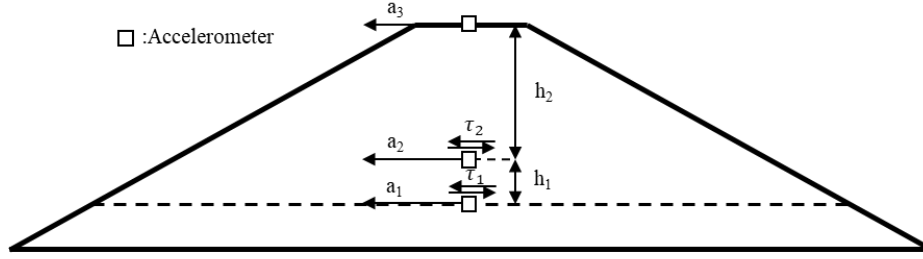


Fig. 3.7. Calculation of shear stress and shear strain in embankment based on acceleration

3.3.2. Reliability verification

Two methods have been used to determine shear stress: the three-directional earth pressure sensor and accelerometers. Similarly, horizontal displacement at the top of the embankment has been assessed using two approaches: the laser displacement sensor and computation through acceleration. Consequently, it's important to initially evaluate the reliability of these calculations.

Fig. 3.8 provides an illustrative example comparing shear stress at location 'a' in Case 1. The shear stress was obtained by the three-directional earth pressure sensor was smaller than the result obtained by accelerometers, and the baseline drift was also observed. This phenomenon could potentially be explained by the rotation of sensor during dynamic loading. The unavoidable rotation can be filtered by data analysis algorithm. The results of two methods showed a correlation so the shear stress obtained by the accelerometer will be used in analyzing.

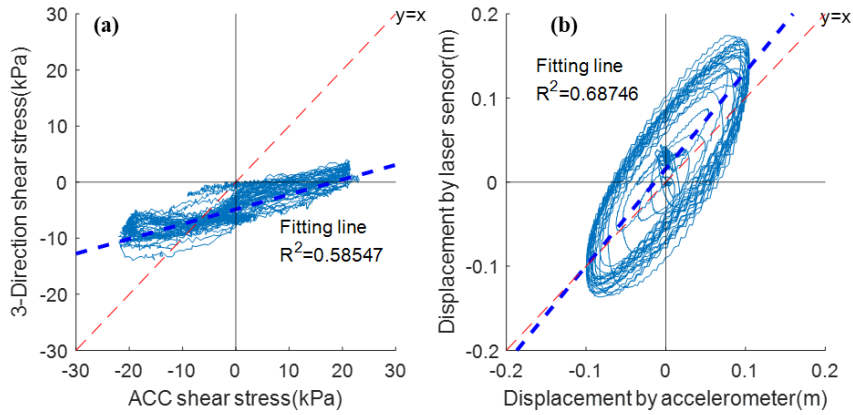


Fig. 3.8. Reliability of calculation (a) Comparison between ACC and three-directional shear stress; (b) Comparison between the horizontal displacement measured by laser displacement sensor and accelerometer

Furthermore, a comparison of the horizontal displacement measured by the laser displacement sensor and the accelerometer located at the top of the embankment is also presented in Fig. 3.8. The horizontal displacement recorded by the laser displacement sensor was approximately 1.5 times greater

than that obtained from the accelerometers. While factors such as vibrations affecting the embankment's target and potential instability in the laser displacement sensor's support frame may have influenced these measurements, we have decided to employ the values recorded by the accelerometers to calculate shear strain within the embankment.

3.4. Test results and discussions

The input and response accelerations of the selected case are displayed in Fig. 3.9. Notably, the measured input acceleration obtained from the accelerometer attached to the soil chamber was closely aligns with the theoretical input acceleration, affirming its reliability. Furthermore, the acceleration exhibits an upward trend with increasing height, indicating the propagation of the input wave from the bottom to the top of the embankment. To evaluate the dynamic behavior of the embankment as it responds to the applied load, further analysis of shear stress and shear strain data obtained through accelerometers is necessary, following the method detailed earlier.

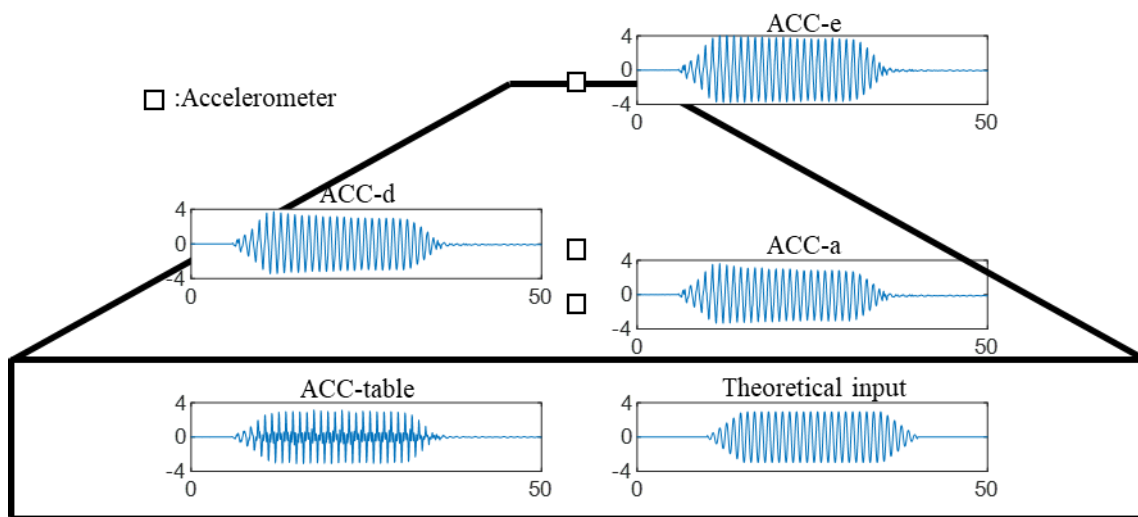


Fig. 3.9. Input wave and dynamic response

3.4.1. Deformation of the embankment

Fig. 3.10 provides a visual representation of the embankment's state, Fig. 3.10(a) to (d) showed the embankment shape before subjecting it to dynamic loads. During the centrifugal loading process, the embankment model experienced base settlement due to the soft base's deformation, the top settlement can be observed caused by the base settlement and consolidation.

To mitigate any inconsistencies in density and stress resulting from the compaction process, white noise was applied. Notably, a close examination of the recorded video revealed that, in addition to top settlement, there was also basal settlement. This finding suggests that white noise may have an impact on the soft base as well. It is plausible to assume that, prior to the application of white noise, the deformation of the Urethane base was constrained by friction between the Urethane and the surface of the soil chamber. The introduction of white noise may have reduced this friction, leading to basal settlement.

Following this, the embankment model was supplied with liquid. Once again, basal settlement was observed. This settlement could be attributed to the increased weight of the embankment model resulting from the liquid supplement.

The top settlement, which was measured using a laser sensor attached to the top of the soil chamber, is illustrated in Fig. 3.11 (a), the top settlement was normalized by the embankment height. These results highlight a clear correlation between construction water content and top settlement. Since the weight differences among the cases were minimal, it is reasonable to assume that the elastic deformation of the Urethane base was consistent. Therefore, the height of the embankment can be inferred from the top settlement. Notably, the case with 10% water content displayed the highest top settlement prior to the application of dynamic loads, while cases with lower water content exhibited less settlement. The greatest top settlement was observed in the case with water content closest to the optimum level (11.2%).

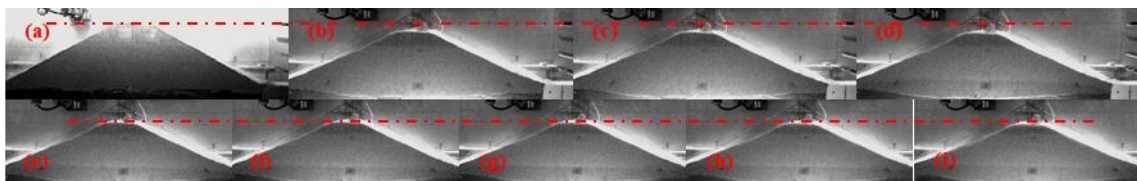


Fig. 3.10. Shape of embankment model(a) Initial shape; (b) After consolidation; (c) After white noise; (d) After liquid supplement; (e) After dynamic load-1; (f) After dynamic load-2; (g) After dynamic load-3; (h) After dynamic load-4; (i) After dynamic load-5

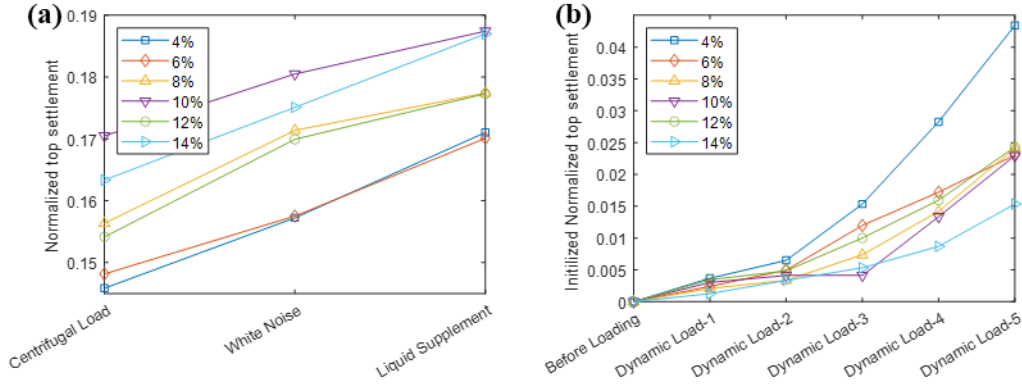


Fig. 3.11. Normalized top settlement of embankment(a) Before dynamic loading; (b) Relative top settlement during dynamic loading process

Fig. 3.10(e) to (i) depict the changing shape of the embankment throughout the dynamic loading process. As basal settlement remained minimal during this process, it is reasonable to assume that the soft base did not deform due to the dynamic load. In this context, the top settlement serves as an indicator of the embankment's response to dynamic loads. Since the embankment was well-consolidated before dynamic loading, top settlement effectively reflects its stability under dynamic loading.

Fig. 3.11 (b) illustrates the relative settlement during the dynamic loading process, with all settlements initiated at zero and normalized by the embankment height. The dynamic loads, numbered 1 to 5, represent different input wave amplitudes (0.40 m/s², 0.70 m/s², 1.10 m/s², 1.85 m/s², and 3.35 m/s², respectively). Interestingly, in contrast to the top settlement before dynamic loading, the top settlement during the dynamic loading process decreases with higher water content. The highest top settlement occurred in the case with 4% water content, while the lowest was observed in the case with 14% water content. It is important to note that top settlement exhibited no correlation with the optimum water content, but rather, the deformation of the embankment consistently increased with decreasing construction water content. This implies that dynamic stability is inversely proportional to water content.

3.4.2. Permeation behavior in the liquid supplement process

Fig. 3.12 shows the images from the high-speed camera during liquid supplement process, summarized for the three stages of initial supply, peak water level, and steady state. The volume of liquid supplement was controlled by a water pressure sensor installed at the bottom of the supply fluid tank. After the centrifugal loading reached 50G and white noise was applied, the supply valve of the

tank installed outside of the soil chamber was opened, and Metolose was supplied by the fluid supply pipe inside the embankment model. The fluid supply was performed until the value of the pore water pressure gauge placed in the tank decreased by about 5 kPa. The volume of liquid supplement and the supplement rate were showed in Table 3.2 in prototype scale.

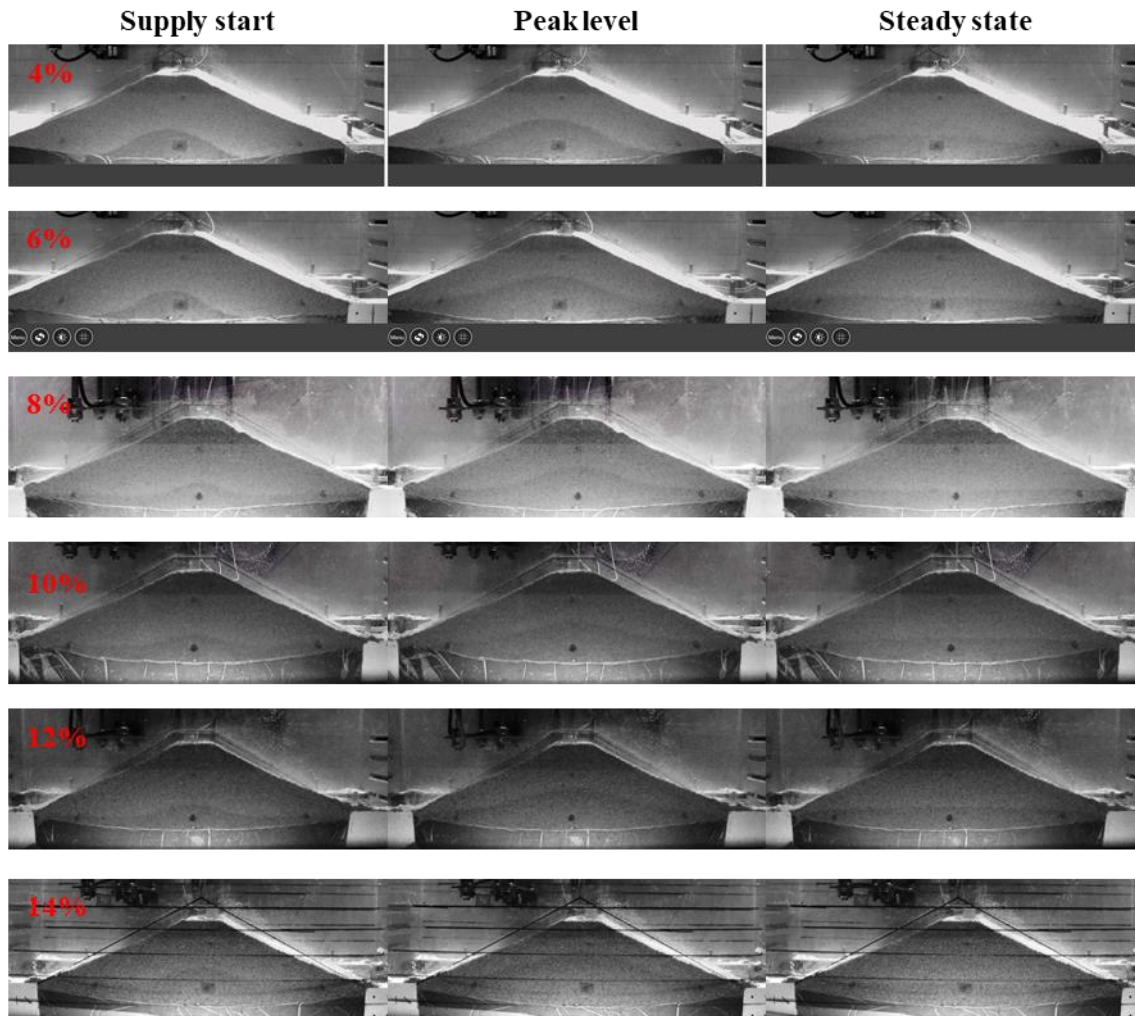


Fig. 3.12. Front image of embankment in liquid supplement process

Table 3.2. Liquid supplement condition

Case Number (water content)	Liquid volume (m ³)	Liquid supply rate (m ³ /min)
Case 1 (4%)	65.0	1.78
Case 2 (6%)	64.8	1.68
Case 3 (8%)	67.2	4.30
Case 4 (10%)	63.1	3.28
Case 5 (12%)	65.3	3.73
Case 6 (14%)	69.4	4.55

During the centrifuge test, the pore fluid, which was used for construction water content adjustment, moves towards the bottom of the embankment due to the strong centrifugal acceleration. As a result, post-centrifuge suction is different from initial suction, making the measurement of suction a challenging task. However, since the water amount within the embankment remains constant before and after centrifugation, it is currently assumed that the correlation between water content and suction also exist in post-centrifugation.

From Fig. 3.12 it can be seen that in the initial supply phase, when comparing the front view images of the embankment for Cases 1 to 6, it becomes evident that lower construction water content results in more significant vertical infiltration. This phenomenon is believed to be a result of the heightened suction effect, which is more significantly at lower water content. This increased suction effect actively draws the fluid upward, leading to a more noticeable change in the infiltration behavior, particularly in the vertical direction.

When comparing images of the embankment at its peak water level, where the water level within the embankment reaches its maximum due to the supply, a noticeable trend emerges. In Cases 1 to 3, the highest point reached by the water in the central part of the embankment is relatively lower. This observation can be attributed to the influence of the suction effect, which was discussed earlier. However, for Cases 4 to 6, the opposite phenomenon occurs. In these cases, the highest points reached by the water are relatively higher. This is believed to result from an increased degree of saturation causing a reduction in air content within the voids. As construction water content rises, the space occupied by pore water in the voids expands, thereby reducing the area available for air or fluids. Consequently, more soil volume is required to retain the same amount of fluid, leading to an expansion of the infiltration area.

For the steady-state of water level, Cases 3 to 6 exhibit a relatively flat-water surface profile. In contrast, Cases 1 and 2 maintain a comparatively higher water level with a convex shape. This pattern can also be attributed, as previously discussed, to the significant suction effect in the embankment constructed on the dry side, impacting the vertical water retention characteristics.

After the comparison of liquid supplement rate, which was calculated from the values of the pressure gauges installed on the bottom of the tank. Since the supply process in this experiment does not follow a steady-state seepage flow, it's inappropriate to discuss it using Darcy's law. Nonetheless, we conducted the experiment with the assumption that there might be a correlation between permeability and supply rate under conditions of a constant hydraulic gradient.

However, when the relative magnitudes of permeability were simply calculated from the supply rate in this experiment based on Darcy's law, the results suggest that the permeabilities follow this order: 6%, 4%, 10%, 12%, 8%, 14%. This differs from the results of permeability tests conducted on materials similar to Hiroshima sand by Takeuchi, 2018, who indicated that the permeability will decrease with the increasing of water content. To investigate the changes in pore water during the liquid supplement process, we used data from pore water pressure gauges installed at point a to assess the impact of liquid supplement. Fig. 3.13(a) have showed the pore water pressure change during the liquid supplement process.

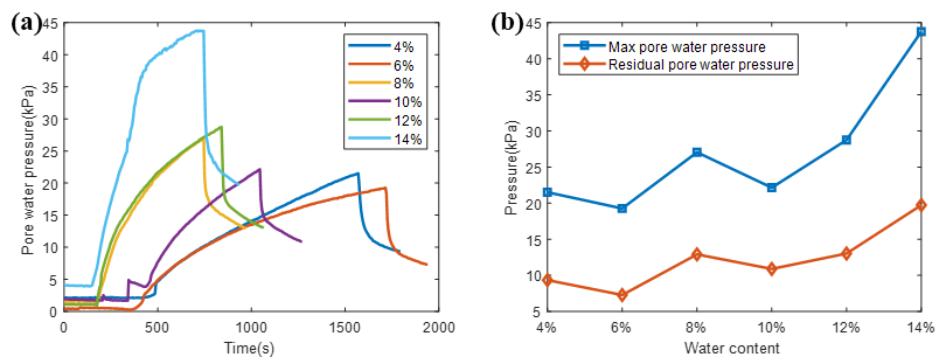


Fig. 3.13. Pore water pressure during liquid supplement(a) Time history of pore water pressure; (b) Max and residual pore water pressure in different water content

From the Fig. 3.13(a), it is clear that within the embankment, the pore water pressure gradually increased as Metolose was injected once the liquid supplement began. The maximum pressure and the rate of increase were related to the construction water content. Upon cessation of the liquid supplement, the pore water pressure immediately decreased. To explore the connection between permeation behavior and construction water content, we recorded the peak pore water pressure and the pore water pressure at steady state, as displayed in Fig. 3.13(b). Since the data recording was stopped at different times after the liquid supplement for each case, it's challenging to determine precisely when the steady state was reached. Therefore, we used the pore water pressure after 2250 seconds as the pore water pressure at steady state. Fig. 3.13(b) illustrates the relationship between construction water content and both maximum and residual pore water pressure. Despite the uniform volume of liquid supplement in each case, both maximum and residual pore water pressure increased with higher construction water content. This phenomenon suggests that factors other than material permeability significantly influence these results.

3.4.3. Vertical earth pressure distribution

As showed in Fig. 3.1, three-directional earth pressure sensors were installed at three locations at the bottom of the embankment. The vertical, horizontal and shear stress can be calculated from the three-directional earth pressure sensor. Especially, the value of vertical earth pressure from three locations can reflect the vertical earth pressure distribution on the bottom of the embankment. Due to the restricted number of earth pressure sensors (only three) within the model, the utilization of heatmap figure was embraced to depict alterations in the vertical earth pressure distribution. This approach enabled us to comprehensively analyze soil behaviors. The centrifuge experiment was structured with three phases: the centrifugal loading process, the liquid supplement process, and the dynamic loading process. By tracking transformations in vertical earth pressure, we were able to scrutinize the implications of basal settlement, saturation, and dynamic loads on soil arching.

Directly measuring basal settlement can pose challenges, prompting us to employ centrifuge acceleration to illustrate the vertical earth pressure distribution during the centrifugal loading phase. Fig. 3.14 provides a graphical depiction of the correlation between vertical earth pressures and centrifuge acceleration at the embankment's center for all cases. The construction water content was listed at the corner of the figure. The x-axis signifies alterations in centrifuge acceleration, the y-axis corresponds to the geometric position, and the color scale indicates the corresponding vertical earth pressure values. Specially, caused by the recording problem, case 5 (water content 12%) did not started from 1G, but it will not influence the rest recording.

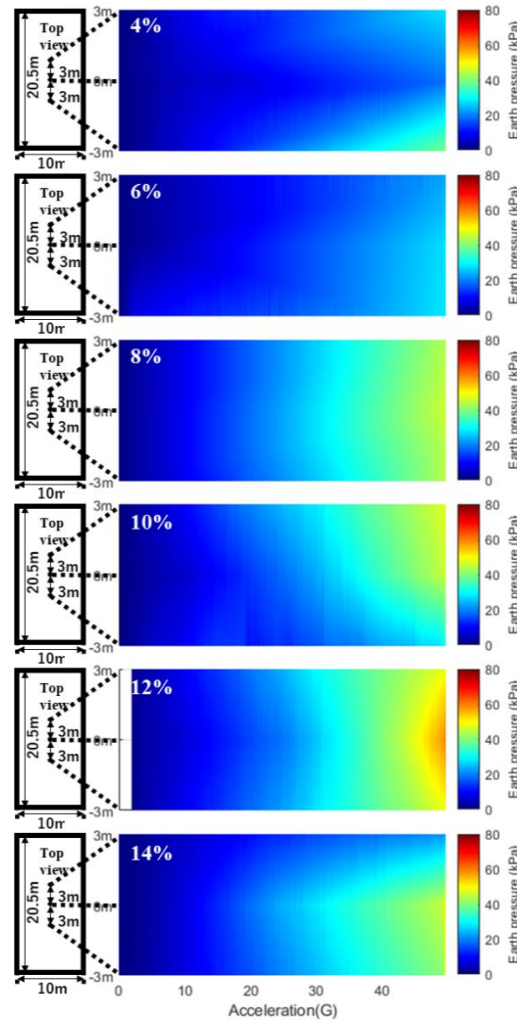


Fig. 3.14. Vertical earth pressure distribution of centrifugal loading process

As observed in Fig. 3.14 , it is evident that the vertical earth pressure at the base of the embankment increases with the rising centrifugal acceleration. Initially, vertical earth pressure increased almost linearly for all cases during the centrifugal loading process. While basal settlement occurred during this process, no significant redistribution of vertical earth pressure was noted. However, noticeable variations in earth pressure values were observed across different construction water content levels.

Specifically, when considering the central earth pressure at a centrifugal acceleration of 50G, the case with the lowest construction water content (4%) exhibited the smallest pressure, registering at 18.76 kPa. In contrast, the case with 12% construction water content recorded the highest earth pressure at 60.27 kPa. Across all six cases, with construction water content ranging from 4% to 14% in 2% increments, central vertical earth pressures were as follows: 18.76 kPa, 27.44 kPa, 45.92 kPa, 44.72 kPa, 60.27 kPa, and 49.84 kPa, respectively. There was a general trend of increasing earth

pressure with higher water content, with the maximum vertical earth pressure occurring at the case closest to the optimum water content.

Notably, in all cases, assuming no changes in water content or density during the centrifugal loading process, the vertical earth pressure was consistently lower than the theoretical earth pressure. Several factors could contribute to this, such as friction between the soil and the soil chamber, density and stress variations resulting from compaction, or non-uniform water content distribution due to centrifugal loading. The local soil arch near the sensors was considered as an important reason too. Cases with 10% and 14% water content displayed asymmetric vertical earth pressure distributions, potentially due to sensor placement errors or uneven compaction.

Across all cases, the central vertical earth pressure was higher than the pressure on the sides, except for Case 1 with 4% water content. In Case 1, the central vertical earth pressure at the end of the loading process was 18.76 kPa, while it measured 27.73 kPa and 38.92 kPa on the sides. This case exhibited limited evidence of soil-arching effects. In all cases, the change in vertical earth pressure displayed a near-linear pattern, indicating that the soil-arching effect was not significantly pronounced.

Upon reaching a centrifugal acceleration of 50G, white noise was introduced to mitigate non-uniform density and stress induced by compaction. The heatmap displaying the distribution of vertical earth pressure can be found in Fig. 3.15, with the period of white noise application indicated by a dotted line. Notably, Case 3 (8% water content) experienced two instances of white noise application. The first occurrence had an input amplitude of 0.5 times, while the second was the standard input of white noise.

An examination of the graphs before and after white noise revealed a consistent pattern: in each case, the vertical earth pressure increased as a result of white noise. However, the extent of pressure change varied with construction water content. Similar to the vertical earth pressure values before white noise, the lowest pressure was observed in the 4% water content case, measuring 24.64 kPa. Conversely, the case with 12% construction water content registered the highest earth pressure at 79.21 kPa.

Given the relatively small amplitude of the input white noise, it can be inferred that the redistribution of vertical earth pressure was not significantly affected. Instead, the primary impact of the white noise was the alleviation of uneven stress and non-uniform density. Across all six cases, spanning construction water content levels from 4% to 14% in 2% increments, central vertical earth pressures were as follows: 24.64 kPa, 33.32 kPa, 54.24 kPa, 55.20 kPa, 79.21 kPa, and 57.12 kPa, respectively.

Notably, in cases with low water content (4% and 6%), the increase in vertical earth pressure was not substantial. In contrast, cases with high water content demonstrated a significant increase in earth pressure, especially those closer to the optimum water content (12%). For instance, in Case 1, central earth pressure remained lower than the pressure at the two sides, with central and side measurements of 24.64 kPa, 33.96 kPa, and 44.71 kPa, respectively. The difference between central and side pressure values was not substantial, making it difficult to definitively attribute it to soil-arching effects or simply to the typical non-uniform distribution of vertical earth pressure.

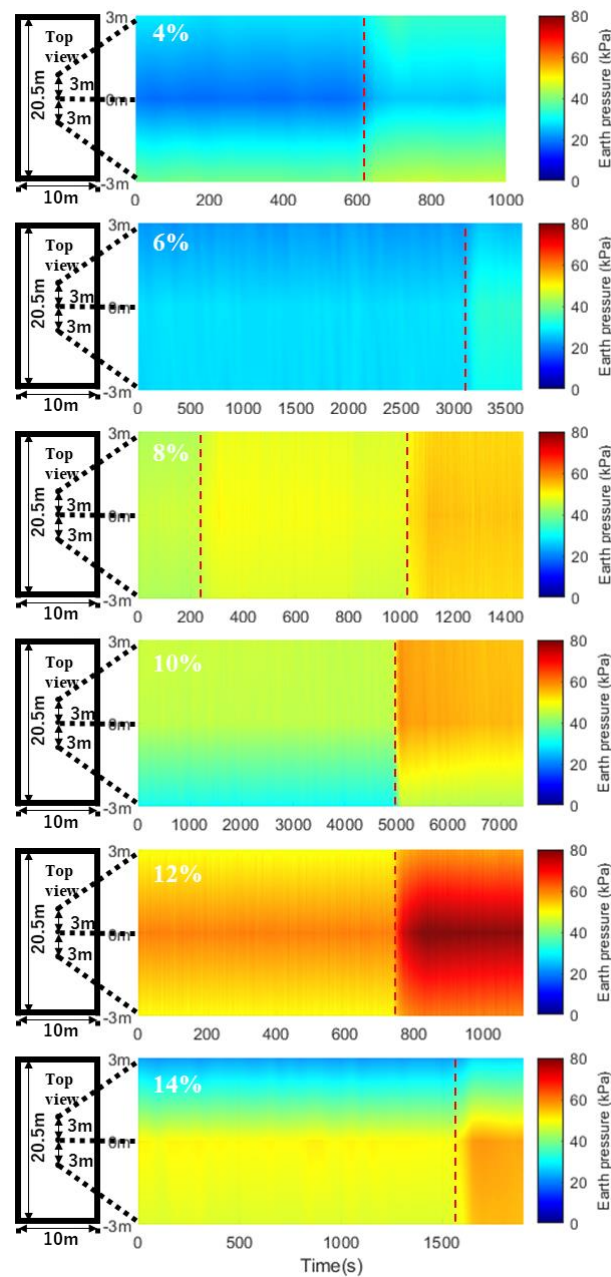


Fig. 3.15. Vertical earth pressure distribution of white noise applying process

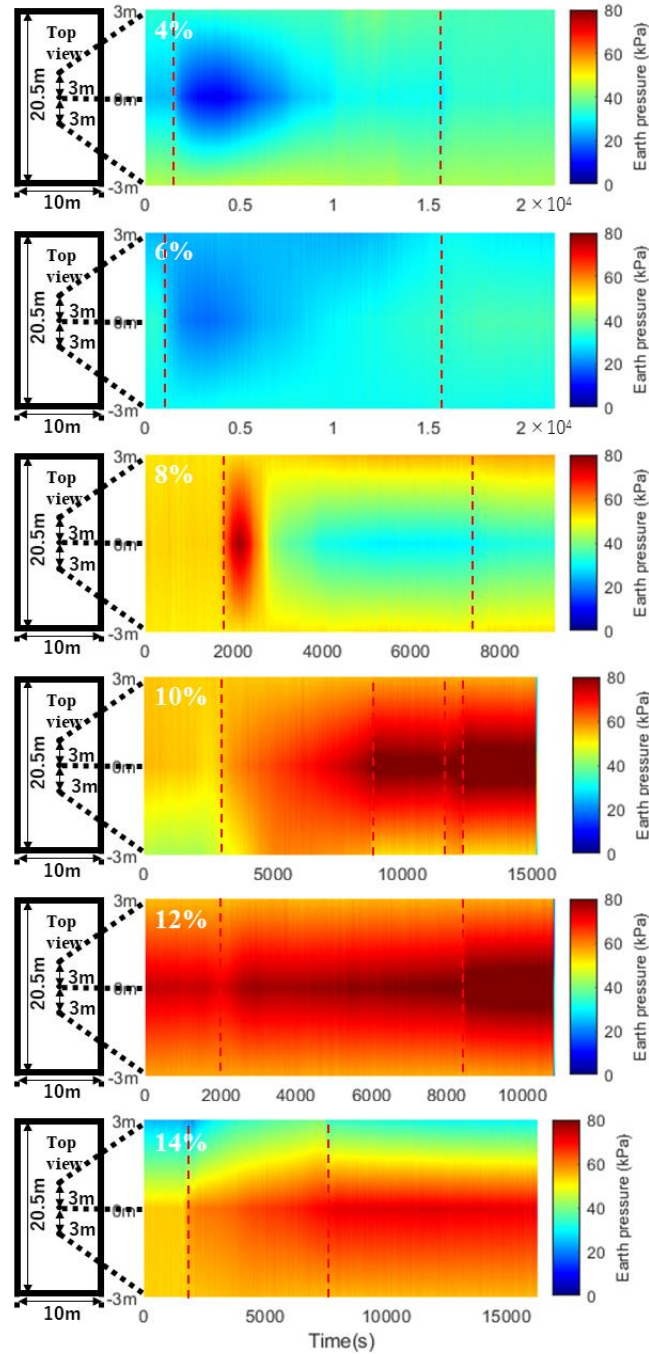


Fig. 3.16. Vertical earth pressure distribution of liquid supplement process

Following the application of white noise, Metolose was introduced into the embankment as the pore liquid. Fig. 3.16 illustrates the variation in vertical earth pressure during the liquid supplement process, with the liquid supplement period's start and end point marked by dotted lines. In Case 4, four lines are depicted because it was necessary to open the valve again after the first liquid supplement, as it was evident that the initial quantity of liquid was insufficient.

While the quantity of liquid supplied to the embankment remained consistent in each case, the supplement duration varied with the construction water content, a point we'll delve into shortly. The graph underscores the redistribution of vertical earth pressure and how it changed in response to water content fluctuations.

In cases with low water content (4% and 6%), a distinct pattern emerged. The central vertical earth pressure decreased immediately upon the commencement of supplementing, but with continued liquid input, it gradually increased once more. Following the conclusion of liquid supplement, a minor additional rise was observed. In particular, the pressures for the 4% and 6% cases were 33.04 kPa and 35 kPa, respectively.

Conversely, the sides of the embankment showed no clear alteration in vertical earth pressure with liquid supplementing. In the 4% water content case, the earth pressure on both sides barely changed, measuring 34.81 kPa and 41.95 kPa. This indicated that the region affected within the embankment was primarily the central part.

The 8% water content case exhibited a unique pattern. Here, vertical earth pressure underwent a significant immediate increase upon starting the liquid supplement, followed by a rapid decrease. The central vertical earth pressure was notably lower than before the supplement, and did not recover even after the liquid supplement ceased. Post-supplementing, the central vertical earth pressure measured 33.01 kPa, which was lower than pre-supplement levels. This phenomenon was distinct due to the high fluid supply rate, influencing soil density within the embankment's saturated zone and leading to the manifestation of soil-arching effects.

On the other hand, in cases with higher water content (10%, 12%, and 14%), the vertical earth pressure exhibited a straightforward tendency, increasing with liquid supplementing and did not revert after supplementation. The highest vertical earth pressure was recorded in the 10% water content case. This phenomenon can be attributed to a higher degree of saturation, which reduces the air content within the voids. Increasing construction water content causes the portion occupied by pore water in the voids to expand, diminishing the area available for air or fluids. Simultaneously, higher water content results in lower permeability. Therefore, liquid introduced into the embankment exerts pressure on the soil, leading to an increase in earth pressure.

While Fig. 3.14 to Fig. 3.16 provided insights into the evolution of vertical earth pressure during centrifugal loading, white noise, and the liquid supplement process, a more comprehensive representation of earth pressure distribution is needed. Following the liquid supplement, dynamic loading was introduced into the embankment. The brief duration of the loading period made it

unsuitable for a heatmap to illustrate the redistribution of vertical earth pressure. Thus, Fig. 3.17 presents the vertical earth pressure distribution throughout the entire experiment. The theoretical earth pressure was calculated by the density and depth of the sensors(4m). The vertical earth pressure after each experiment process was marked by point. Specially, in 10% case the dynamic load 3 has not be applied, so in this case there are only 4 dynamic loads included.

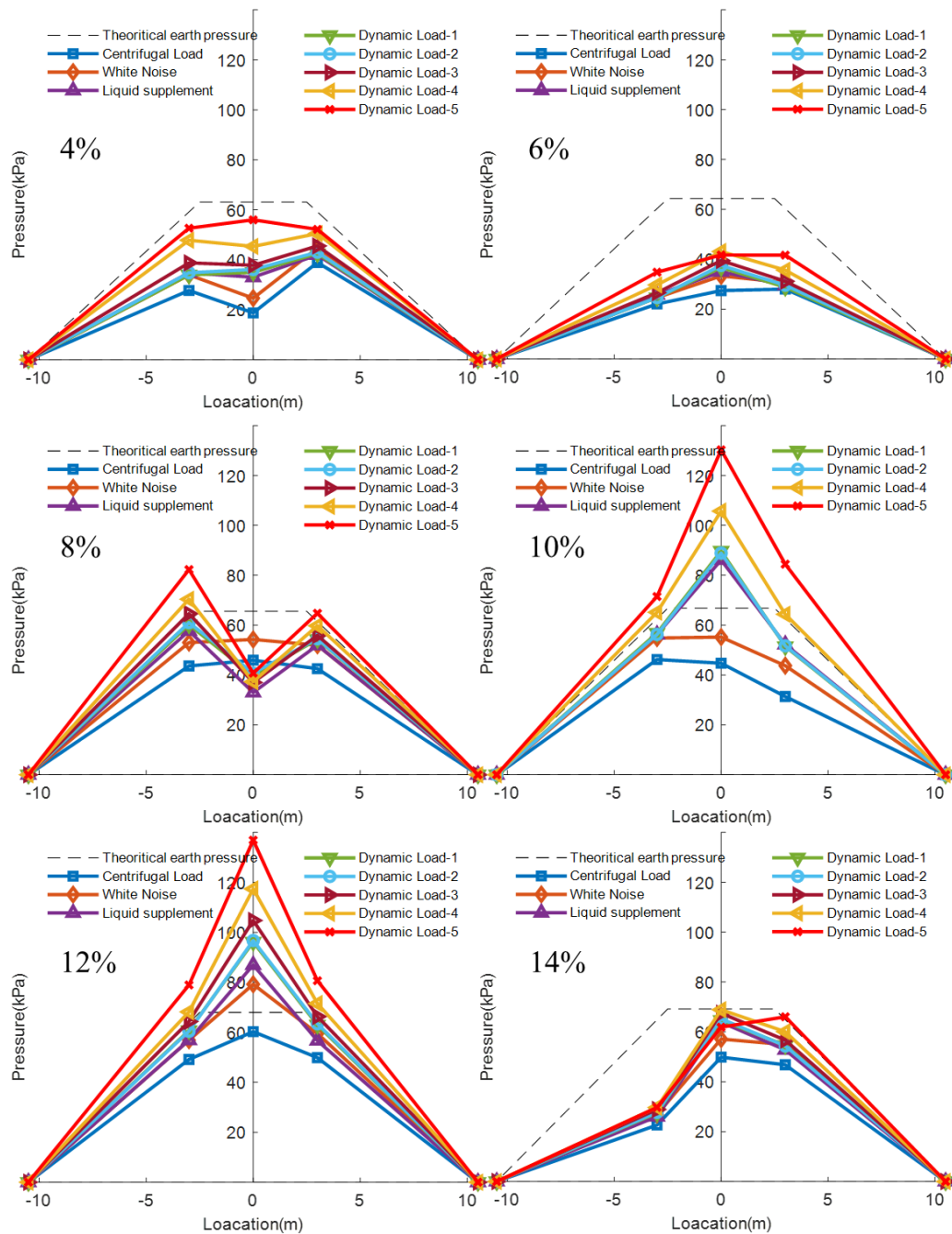


Fig. 3.17. Vertical earth pressure distribution of entire experiment

A discernible trend in the distribution of vertical earth pressure is evident from Fig. 3.17. Firstly, the vertical earth pressure following centrifugal loading in each case appeared lower than the theoretical earth pressure. This phenomenon might be attributed to material irregularities within the embankment, as mentioned earlier.

The distribution of vertical earth pressure after centrifugal loading exhibited an inclination to increase with water content. After the application of white noise, vertical earth pressure increased across all cases. The small amplitude of the white noise made embankment failure unlikely, and it significantly mitigated density and stress irregularities resulting from compaction. Importantly, the vertical earth pressure did not reach the theoretical earth pressure, except in the case with 12% water content. This indicates that construction water content influences consolidation.

Then, the liquid was supplied into the embankment, and from this process a big difference appeared for different water content. In cases with low water content (4% or 6%), vertical earth pressure increased, but not substantially. In contrast, cases with water content near the optimum level experienced a dramatic increase after the liquid supplement, especially in the central area, where the earth pressure exceeded the theoretical earth pressure. However, if the construction water content was increased to 14%, the vertical earth pressure distribution showed distinct characteristics compared to other cases, with an asymmetric tendency. This might result from sensor errors or asymmetric deformation of the soft base. When considering the earth pressure at the center of the embankment, it's apparent that the increase in earth pressure was not significant. On the other hand, the redistribution of vertical earth pressure in the 8% water content case was unique. Before the liquid supplement, the central vertical earth pressure was higher than at the sides, but after the liquid supplement, the vertical earth pressure remained constant at the sides, while the center's pressure was significantly affected. It exhibited a substantial decrease, suggesting the occurrence of soil-arching effects.

Then, the dynamic loading was applied into embankment five times. In cases with low water content, after each dynamic loading, the central vertical earth pressure increased slightly. For the 4% water content case, after five dynamic loadings, the central earth pressure increased from 32.48 kPa to 56.00 kPa. The 6% water content case exhibited a lower increase, from 34.44 kPa to 41.72 kPa.

Conversely, the cases with 10% to 12% water content experienced a substantial vertical earth pressure increase, with the central embankment pressure rising significantly. For the 10% water content case, the pressure increased from 86.66 kPa to 130.27 kPa. For the 12% water content case, it increased from 88.11 kPa to 136.90 kPa. In these instances, the vertical earth pressure exceeded the theoretical earth pressure by a significant margin.

In contrast, the 8% water content case-maintained soil-arching after dynamic loading was applied. The vertical earth pressure increased, but the redistribution of vertical earth pressure was not observed. The case with the highest water content (14%) showed almost no change in vertical earth pressure after dynamic loading, with the central embankment pressure even decreasing slightly after the most intense dynamic loading.

In summary, vertical earth pressure generally increased across all cases, except for the 8% water content case. In this case, post-liquid supplement, the vertical earth pressure was lower than at the sides. It is challenging to attribute this soil-arching effect to basal settlement. The water content will influence the properties of soil arch, high water contents can weaken the pressure transmission in the arching structure, and the high arch height caused by low water content may influence the development of soil arch(Zhang et al., 2023). The soil-arching effect persisted after dynamic loading, and the magnitude of vertical earth pressure increase was influenced by water content, with the most substantial increase occurring in the case closest to the optimum water content. This can be explained by density changes resulting from dynamic loading.

3.4.4. Liquefaction evaluation

To assess liquefaction, both vertical earth pressure and pore water pressure were employed. Liquefaction occurrence depended on a key indicator: during the dynamic loading process, liquefaction was identified when the pore water pressure surpassed the vertical earth pressure. This was quantified using the Excess Pore Water Pressure Ratio (EPWPR), defined as the ratio of the excess pore water pressure generated during an earthquake to the initial effective confining pressure of the soil. This ratio is expressed in Eq. (10), in which Δu : Excess pore water pressure and σ_{vi}' : Initial effective confining pressure. To record these pressures, both earth pressure and pore water pressure gauges were positioned at the center of the embankment's base.

$$EPWPR = \frac{\Delta u}{\sigma_{vi}'} \quad (3.8)$$

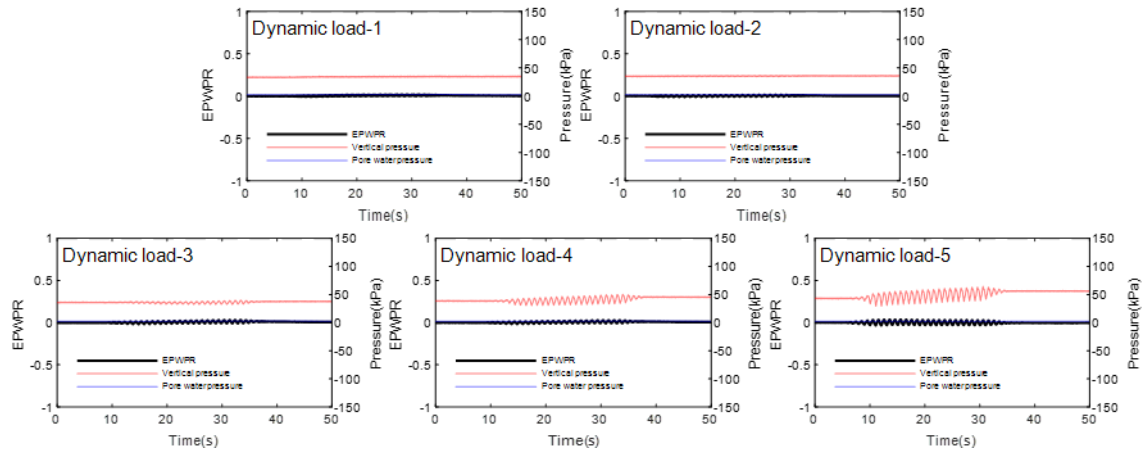


Fig. 3.18. Pressure condition and excess pore water pressure ratio of case 1

Fig. 3.18 displays the pressure conditions and EPWPR for Case 1. The vertical earth pressure and pore water pressure are represented by the red and blue lines, using the right coordinate axes. Simultaneously, the black line illustrates the EPWPR, using the left coordinate axes. The figure reveals minimal fluctuations in vertical earth pressure and pore water pressure, resulting in an exceptionally low EPWPR. This indicates that liquefaction did not occur in this case.

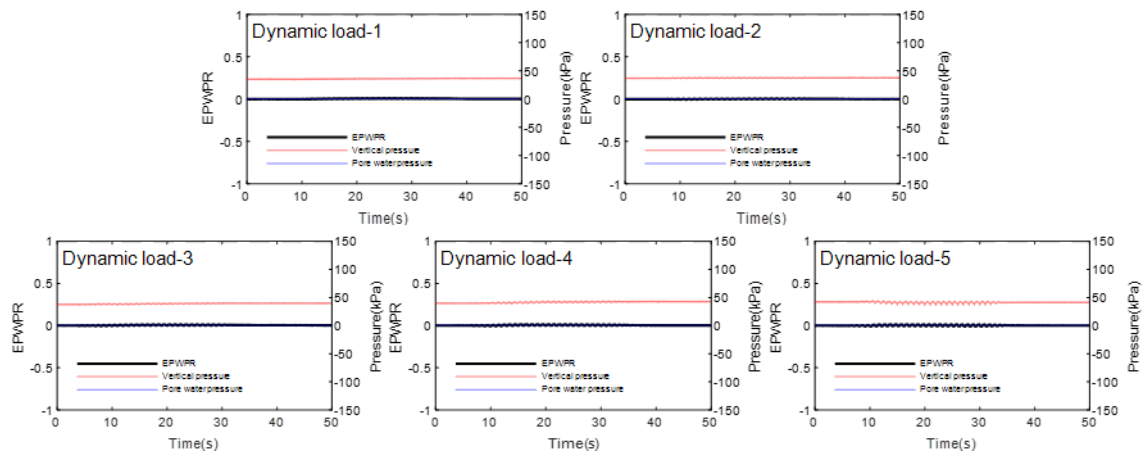


Fig. 3.19. Pressure condition and excess pore water pressure ratio of case 2

Moving on to Fig. 3.19, it provides a similar representation for Case 2. As detailed in a previous section, it was observed that the case with a 6% water content experienced the least increase in vertical earth pressure. The figure demonstrates that the fluctuations in vertical earth pressure during dynamic

loading were also modest. In both Case 1 and Case 2, the EPWPR remained significantly below 1, signifying stable pressure conditions and the absence of liquefaction.

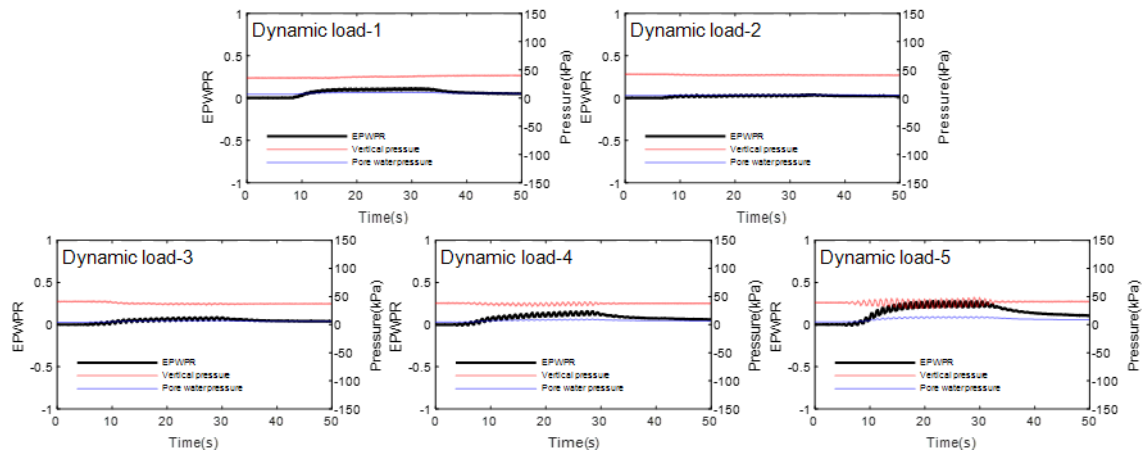


Fig. 3.20. Pressure condition and excess pore water pressure ratio of case 3

For Case 3, depicted in Fig. 3.20, the scenario is somewhat different. As previously mentioned, the soil-arching effect occurred after the liquid supplement, leading to a reduction in vertical earth pressure at the center of the embankment. The figure indicates small yet observable fluctuations in the EPWPR due to the increasing pore water pressure. However, the EPWPR remained below 0.5 due to the overall low excess pore water pressure, proofing the embankment's stability and its distance from liquefaction.

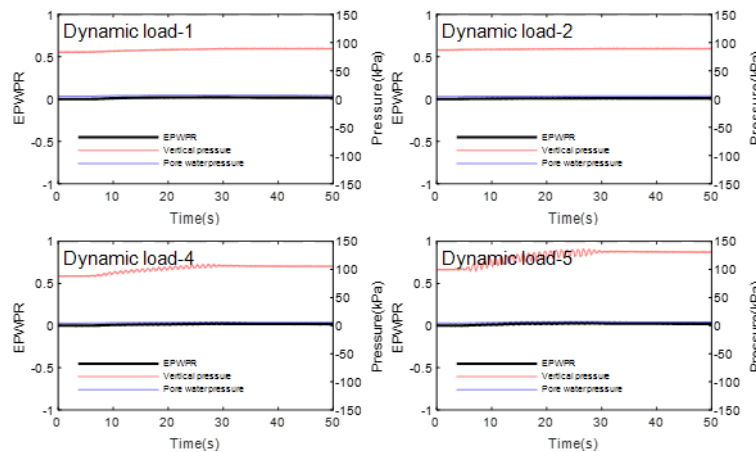


Fig. 3.21. Pressure condition and excess pore water pressure ratio of case 4

Moving to Fig. 3.21 which pertains to Case 4, the dynamic loading was only applied four times, resulting in four data points. Here, while the vertical earth pressure exhibited relatively small fluctuations, it increased significantly after dynamic loading. Nevertheless, the pore water pressure remained nearly constant throughout the dynamic loading process, and consequently, the EPWPR did not vary significantly. This serves as an important reminder that a substantial increase in vertical earth pressure does not necessarily correlate with a heightened liquefaction risk.

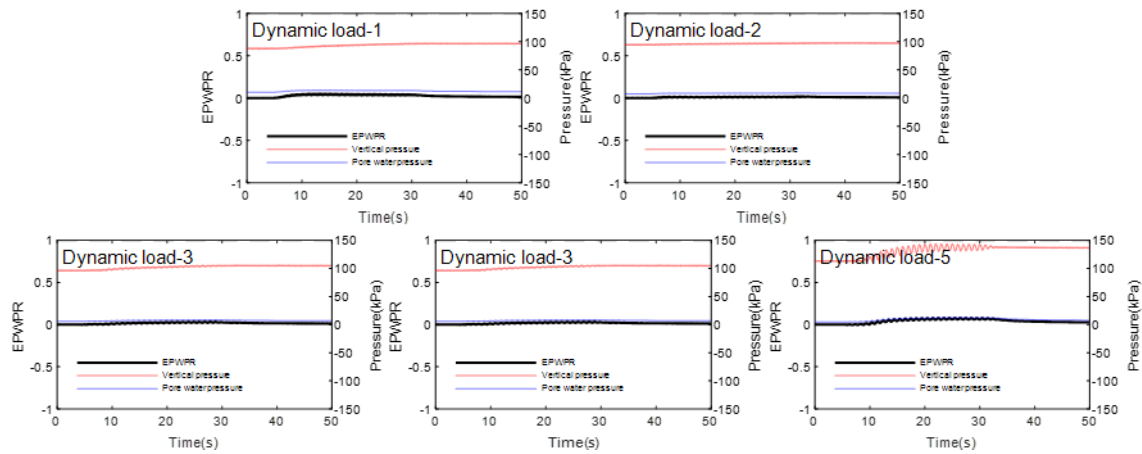


Fig. 3.22. Pressure condition and excess pore water pressure ratio of case 5

Fig. 3.22 , focusing on Case 5, mirrors the pattern observed in Case 4. Despite a notably high vertical earth pressure, the fluctuation in both vertical earth pressure and pore water pressure remains modest.

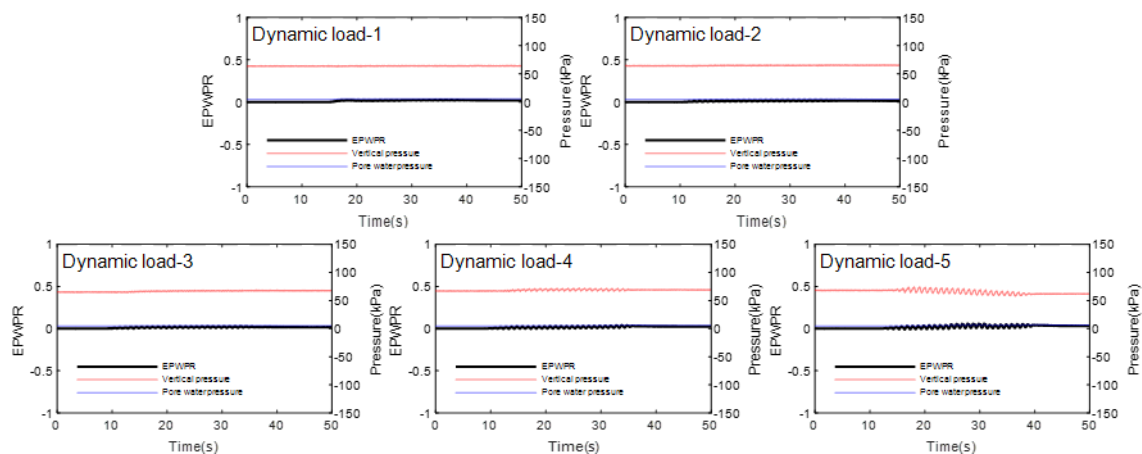


Fig. 3.23. Pressure condition and excess pore water pressure ratio of case 6

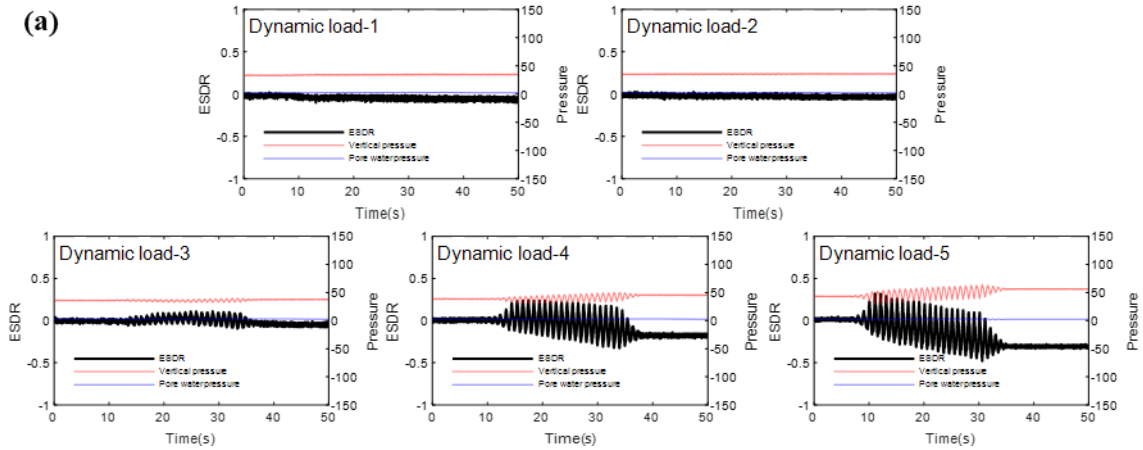
Lastly, Fig. 3.23 presents the scenario in Case 6. Here, the fluctuations in both earth pressure and pore water pressure are small, accompanied by minimal changes in the EPWPR.

In summary, the analysis of vertical earth pressure and pore water pressure, along with the EPWPR, provides valuable insights into liquefaction. Cases 1 through 6 all exhibit characteristics suggesting low liquefaction risks. Nonetheless, it's crucial to understand that a considerable increase in vertical earth pressure, by itself, does not necessarily indicate a high likelihood of liquefaction. These findings underscore the intricacies of liquefaction assessments, which consider multiple variables and measurements. For future assessments geared toward triggering liquefaction, test conditions may need refinement.

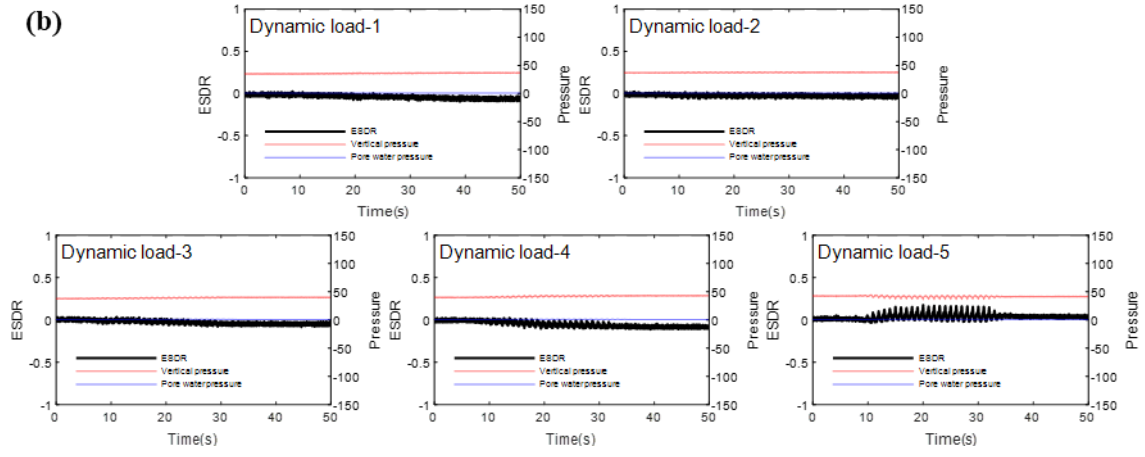
Moreover, it's worth mentioning that the EPWPR relies solely on excess pore water pressure relative to the initial effective confining pressure in its calculations. In our study, as the vertical earth pressure changes after dynamic loading, this might impact precision. Therefore, we've opted for a more precise metric known as Effective Stress Decrease Ratio (ESDR), as proposed by Lu (Lu et al., 2008). The Equivalent Static Stress Ratio (ESDR) quantifies the reduction in effective stress relative to the initial effective stress caused by loading. Its equation is outlined below:

$$ESDR = 1 - \frac{\sigma'_v}{\sigma'_{vi}} \quad (3.9)$$

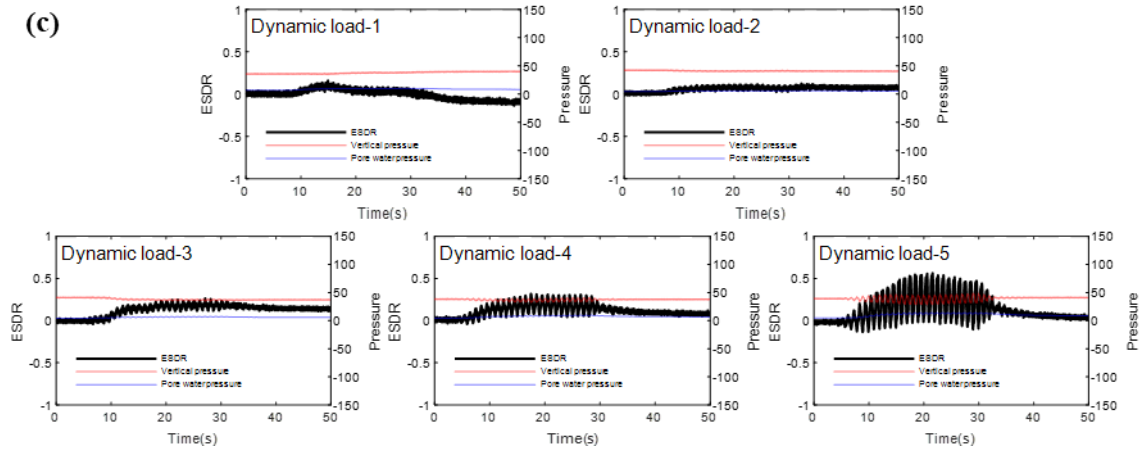
Where σ' represents the effective earth pressure, and σ'_{vi} denotes the initial effective earth pressure.



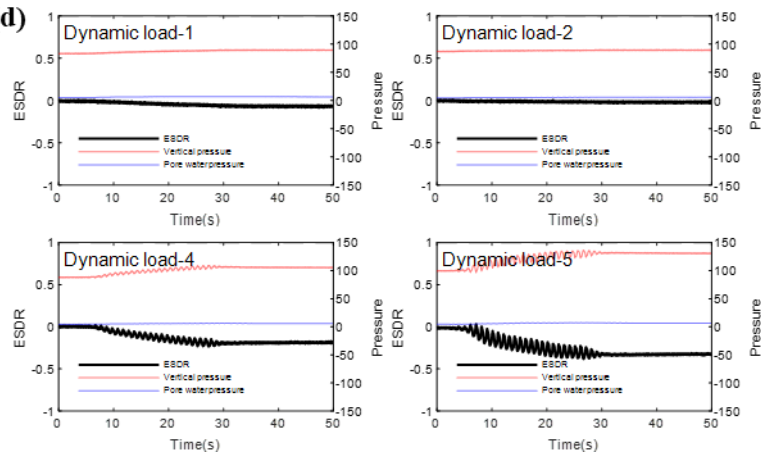
(b)



(c)



(d)



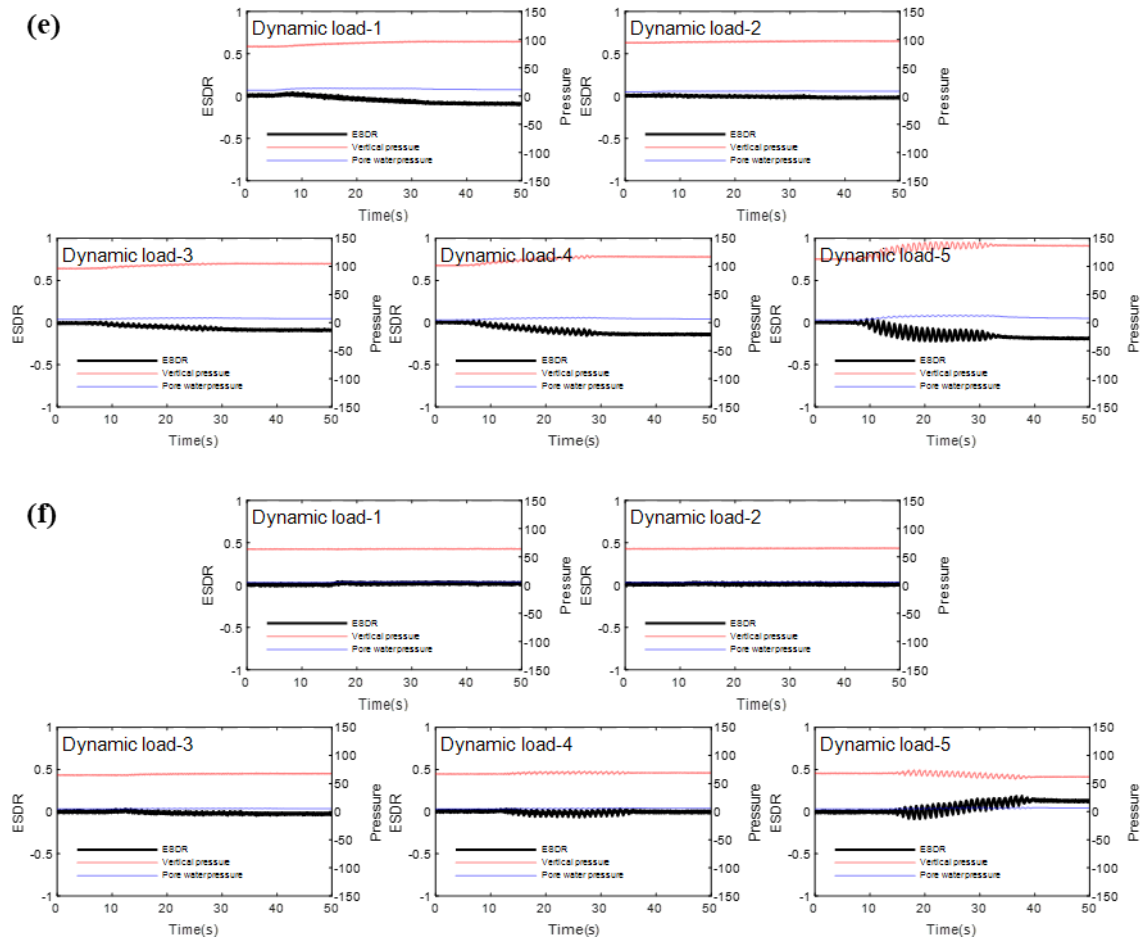


Fig. 3.24. Pressure condition and effective stress decrease ratio (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6

In this section, the evaluation of liquefaction potential is further examined through Fig. 3.24, which illustrates the pressure conditions and Effective Stress Decrease Ratios (ESDR) for all cases using the same representation method employed for EPWPR. This approach allows for a more refined assessment of the variations in earth pressure dynamics when compared to the Excess Pore Water Pressure Ratio.

However, it's essential to note the underlying dynamics of ESDR. The definition of ESDR indicates that when the excess pore water pressure is relatively low, it can cause the effective earth pressure to surpass the initial effective earth pressure, leading to a decrease in ESDR, potentially to negative values. Therefore, ESDR's fluctuations are particularly sensitive to shifts in effective earth pressure.

As evident from figure, certain cases such as case 1, case 4, and case 5 display a scenario where the vertical earth pressure increases while the pore water pressure remains relatively stable. This dynamic results in the ESDR declining, and in some instances, even falling below zero.

Of particular significance is case 3, where the soil-arching effect is observed. In this scenario, the vertical earth pressure at the center of the embankment decreases, and the ESDR value, while still relatively small, provides an insight into the potential for liquefaction, making it a valuable indicator in assessing the stability of the embankment under dynamic loading conditions.

3.4.5. Dynamic shear stress ratio

The stress ratio, defined as the ratio of the maximum shear stress to the initial consolidation stress, was used as an indicator for assessing the stress conditions during dynamic loading. Fig. 3.25 illustrates the relationship between the shear stress ratio and water content at point a (as shown in Fig. 3.1). The initial effective consolidation stress σ'_{v0} was computed by:

$$\sigma'_{v0} = \sigma_{v0} - u_0 \quad (3.10)$$

Where σ_{v0} is the initial vertical stress and u_0 is the initial pore water pressure.

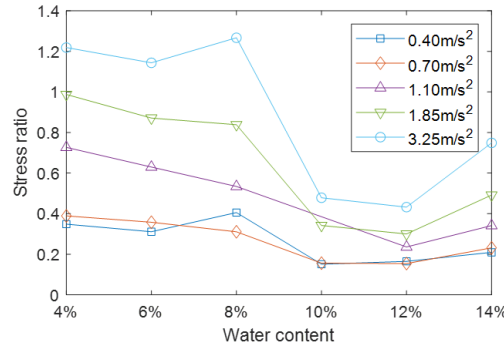


Fig. 3.25. Relationship between Stress ratio and water content at point a

The results show the correlation between shear stress ratio and construction water content of embankment in different input dynamic wave. The amplitude of input wave was gradually increased, from 0.40 m/s² to 3.25 m/s², and the dynamic shear stress ratio have the tendency to increase with the input amplitude. In a low-level dynamic load input, the shear stress ratio in different water content did not show a significantly difference. With the increasing of the amplitude, from the third dynamic load (1.10 m/s²), the shear stress ratio starts to change with the water content. The largest shear stress ratio appears in water content at 8%, and the case with a water content 10% to 12% showed a less shear stress ratio, which is close to optimum water content (11.2%).

3.4.6. Maximum dynamic shear strain

As showed in Fig. 3.1, accelerometers were installed at various heights within the embankment. To analyze the dynamic shear strain characteristics across different layers of the embankment, we specifically focused on the central line accelerometers (ACC table, ACC a, d, and e) to compute shear strain using Eq. (3.4).

The shear strain at the bottom part of the embankment was calculated using the acceleration data from the shaking table and ACC-a. The maximum shear strain, calculated as the double amplitude across all cases, is illustrated in Fig. 3.26. This particular section was saturated with metolose liquid, and the settlement of the soft base resulted in a decrease in soil density. The region in direct contact with the base experienced relative displacement between the soil and the base, leading to significant strain.

In cases with low-level input amplitudes, the maximum shear strain did not exhibit significant variations across different water content levels. However, in high-level input cases, the highest maximum shear strain occurred in the case with 4% water content. As the water content increased, the maximum shear strain decreased until it reached 8% water content. Cases with water content ranging from 8% to 14% displayed similar results, indicating that the maximum shear strain remains relatively stable with varying water content. Notably, at extremely low water content levels, the shear strain was observed to be higher.

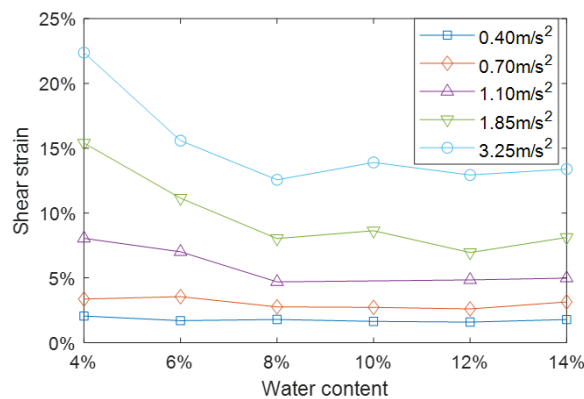


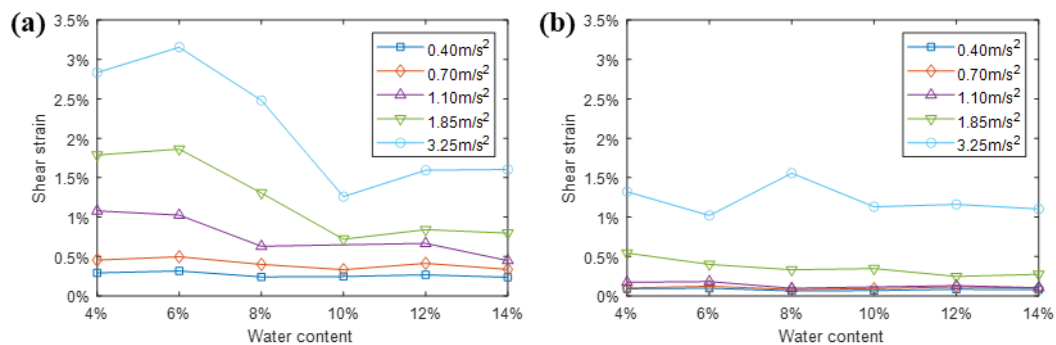
Fig. 3.26. Relationship between shear strain and water content at bottom of embankment

The maximum shear strain in other layers is illustrated in Fig. 3.27(a) and (b). Notably, compared to the shear strain in the bottom layer, the shear strain in these upper layers is significantly smaller due

to the attenuation of the wave as it propagates. Therefore, the shear strain above ACC-a was used to analyze the deformation mode of the embankment. The results reveal significant differences when comparing shear strain in the middle layer (between point a and d) and the top layer (between point d and e).

Similar to the patterns observed in shear stress ratios, it can be observed that the maximum shear strain in middle layer increased with the input wave amplitude and displayed variations with the water content. At low-level amplitudes, minor differences in shear strain were observed. However, under high input amplitudes, the maximum shear strain exhibited a clear trend of variation with water content. Notably, lower water content corresponded to higher shear strain values, with the highest maximum shear strain occurring at 6% water content, and lower values associated with cases close to the optimum water content (11.2%). In the upper layers, the highest maximum shear strain was observed at 8% water content. As the construction water content increased to higher levels, there was a slight tendency for the maximum shear strain to decrease, although this change was not significant.

Fig. 3.27(c) provides an overview of the maximum shear strain at different depths within the embankment. This figure outlines the geometrical characteristics of the embankment and the corresponding maximum shear strain values within specific depths. These observations underscore that the embankment does not behave like a rigid, solid structure. Consistently, the bottom layer displayed higher strain values compared to the middle layer across all cases.



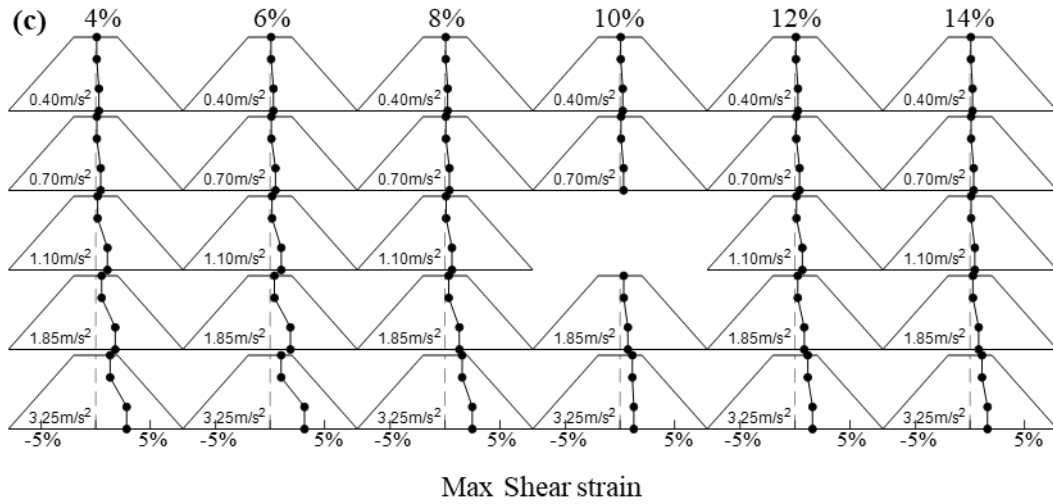


Fig. 3.27. Max shear strain condition (a) max shear strain in bottom layer; (b) max shear strain in top layer; (c) embankment outline and max shear strain along different depth

3.5. Summary

In conclusion, this study conducted dynamic centrifuge tests under varying construction water content conditions, implementing a liquid supply system to investigate the behavior of the embankment. The experiment apparatus was designed to trigger the soil-arching effect and liquefaction. The data analysis algorithm was developed to obtain the stress and strain condition. The influence of construction water content was investigated, and the properties of embankment under dynamic load were found.

Chapter 4. Experiment modification

As showed in last chapter, the soil-arching effect was not significantly through all cases, and the liquefaction did not occur too. In order to trigger the arching effect and liquefaction in laboratory experiment, the model preparation and experiment process should be modified. So, in this chapter, we delve into the adjustment of model preparation, improvement of measurement sensor, and the improvement of stress-based liquefaction evaluation method.

4.1. Modification of experiment model and process

In Chapter 3, solid Urethane was used to replicate soft ground conditions, but soil arch action did not occur during the centrifugal process in any case, indicating insufficient basal settlement. To address this issue, a new type of Urethane base was introduced, featuring several centrally drilled holes (as shown in Fig. 4.1). This design aims to induce larger basal settlement and promote soil arch action.

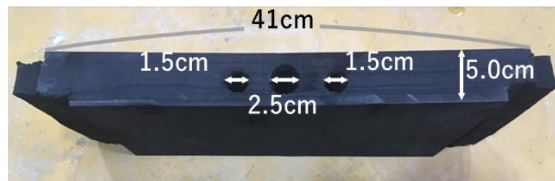


Fig. 4.1 Drilled Urethane base

The experimental process also required modification. Previously, after the liquid supplement, the model was maintained at 50G until the water level stabilized. However, as demonstrated in Section 3.4.2, liquid leakage from the embankment model caused a gradual decrease in the water level inside the embankment, reducing the saturated area and subsequently the liquefaction potential. In the revised procedure, the dynamic load is applied immediately after the liquid supplement is stopped to prevent this reduction in the saturated zone.

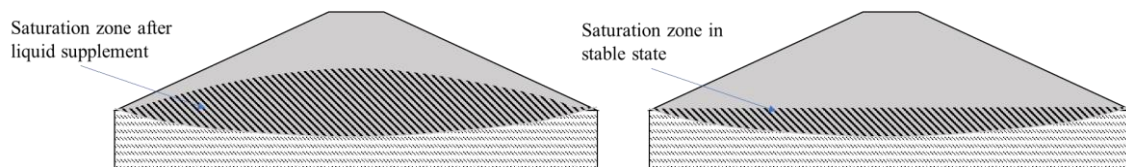


Fig. 4.2 Reduction of saturated zone

In the earlier experiments, dynamic loads were applied five times per case, with amplitudes increasing from 0.40 m/s^2 to 3.35 m/s^2 . This approach influenced the embankment's liquefaction

resistance properties, and the load amplitudes were insufficient to trigger liquefaction. Therefore, in subsequent experiments, a direct application of dynamic load with an amplitude of 4 m/s² was implemented.

4.2. Sensor Improvement and Reliability Assessment

In the previous chapter, we employed the three-directional earth pressure sensor to measure earth pressure. However, the accuracy of this sensor, especially in measuring horizontal earth pressure and shear stress, has raised concerns. It has shown errors such as initial stress errors and baseline drift. These issues necessitate a reevaluation of the sensor's reliability. So, in this section, we delve into the reliability assessment of a new three-directional earth pressure sensor and its subsequent improvement. The evaluation was conducted under varied experimental conditions to validate the sensor's reliability and improve its precision.

To address these concerns, we sought to improve the sensor design. The limitations of the existing sensor, particularly concerning its shape and rotational instability, were carefully scrutinized. In a bid to augment its precision, we leveraged a 3D printing technique to craft an innovative sensor holder, as depicted in Fig. 4.3(a). This design allowed for the installation of accelerometers at the center of the holder, while earth pressure sensors attached to its surface. This modification aimed to address the drawbacks of the previous sensor design. The two-dimensional stress tensor formulation for stress decomposition is as showed in Eq. (3.1) and Eq. (3.2), the components of σ_1 , σ_2 , and σ_3 are shown in Fig. 4.3 (b). The photograph of the 3D-printed triangular prism with sensors was showed in Fig. 4.3 (c).

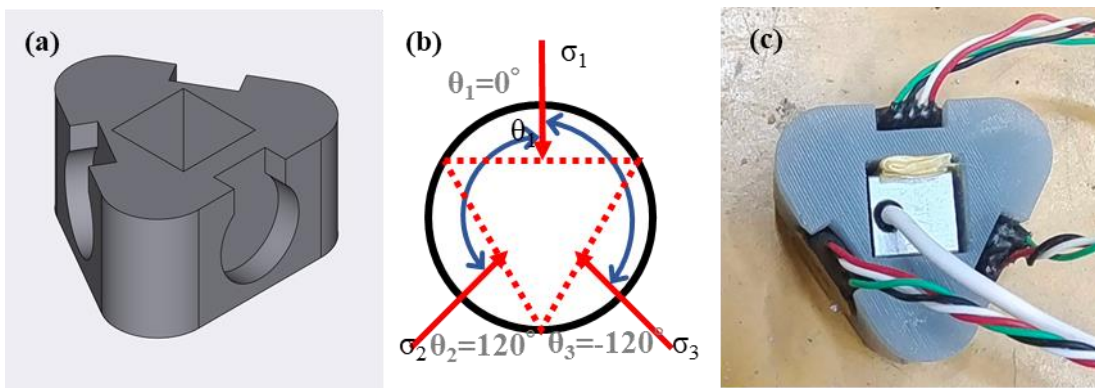


Fig. 4.3. New three-directional earth pressure sensor holder(a) 3D model of new holder; (b) Stress component; (c) Picture of TEP with sensors

4.2.1. Centrifuge test condition

To evaluate the reliability of new three-directional earth pressure sensor, centrifuge experiment was conducted using the new sensors.

4.2.1.1. Model preparation

Fig. 4.4 illustrates the model configuration and the positions of measuring instruments at a model scale. Dry Hiroshima sand was utilized, and the model was prepared using the dry deposition method with a dropping height of 54 cm.

About the measurement instruments, a total of 6 accelerometers (ACC) were utilized. Among these, 4 accelerometers were positioned at varying depths within the sand, while the remaining 2 were installed in the new Three-Directional Earth Pressure Sensor (TEP), as illustrated in Fig. 4.3(c). Additionally, 4 Three-Directional Earth Pressure Gauges (TEP) were situated in two distinct layers. In each layer, one new type TEP and one old type TEP were incorporated. To provide a reference earth pressure, several standard earth pressure sensors were also installed.

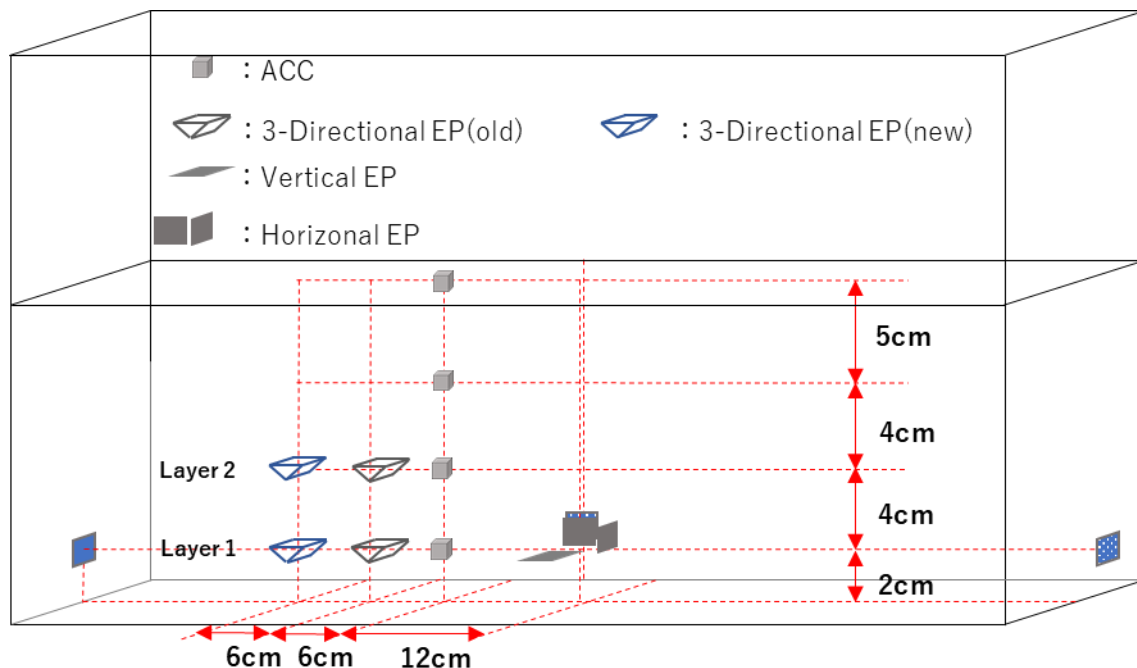


Fig. 4.4. Diagram of model

Fig. 4.5 the locations of the measuring instruments in layer 1. Additionally, to gauge the sand density following the dry deposition, three density pans were strategically positioned on this layer. The

volume of each density pan was pre-measured. Subsequent to the experiment, the weight of the sand within each density pan will be measured, allowing for the determination of the sand density.

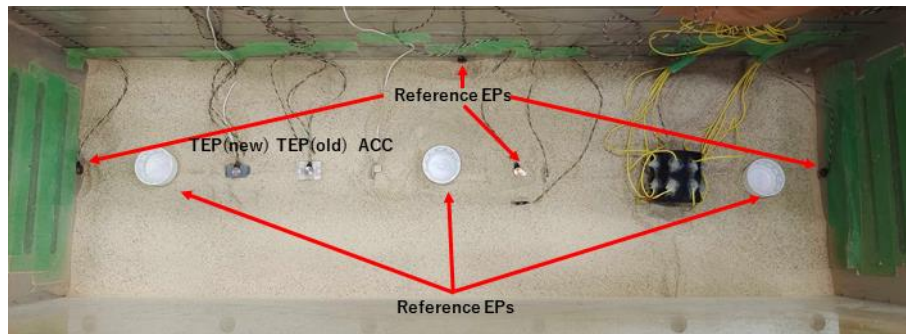


Fig. 4.5. Installation of measuring instruments in layer 1

4.2.1.2. Experiment program

After the model preparation, the soil chamber was set in the centrifuge machine, the dynamic centrifuge test is comprised of two main steps: centrifugal loading process and dynamic loading process. In the centrifugal loading process, the centrifugal acceleration gradually increased from 1G to 80G in 10G increments, then decreasing to 50G with 10G increments. Subsequently, the model was maintained at 50G, and three sine waves, each with varying amplitudes and frequencies, were independently applied. The sine wave was precisely controlled by displacement, with the specific amplitude and frequency of each load detailed in Table 4.1.

Table 4.1. Dynamic loading condition

Shaking No.	Amplitude (mm)	Frequency (Hz)	Number of cycles
1	5	1	5
2	1	50	30
3	2	50	30

4.2.2. Test result and discussion

4.2.2.1. Vertical and horizontal earth pressure

During the centrifugal loading process, the acceleration increased from 1G to 80G in 10G increments, causing a simultaneous rise in vertical and horizontal earth pressure, and then returned back to 50G. The vertical and horizontal earth pressure during the centrifugal loading process was showed in Fig. 4.6, the stress was measured by new TEP, old TEP and reference EP respectively.

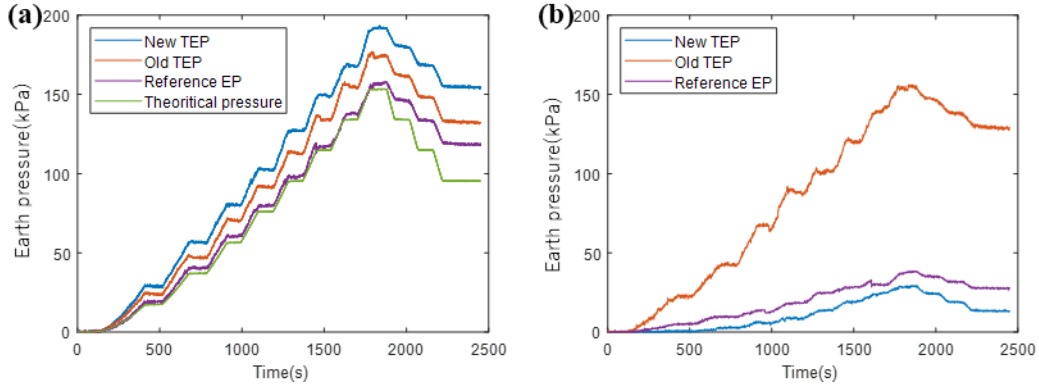


Fig. 4.6. Earth pressure in centrifugal loading process(a) Vertical earth pressure; (b) Horizontal earth pressure

Fig. 4.6(a) displays the vertical earth pressure measured by new TEP, old TEP, reference EP, and the theoretical vertical earth pressure. The correlation is evident, with reference EP being closest to the theoretical earth pressure. However, the rotation of the sensor in the embankment model after deformation can affect accuracy. TEP measurements, whether new or old, consistently registered higher values than the theoretical earth pressure. This discrepancy might be attributed to the size of the holder influencing readings from deeper sand layers. Nonetheless, both reference EP and TEP values demonstrated a strong correlation with theoretical earth pressure. Notably, sensor measurements depicted distinct earth pressure characteristics during loading and unloading processes, prompting a focused study on earth pressure during loading in this research.

The horizontal earth pressure was showed in Fig. 4.6(b), with values from old TEP unusually high, possibly due to the instability caused by the TEP holder's shape. Conversely, new TEP values displayed a strong correlation with reference EP, albeit consistently higher.

In summary, the vertical and horizontal earth pressure measured by the new TEP exhibited a strong correlation with reference EP, confirming the reliability of the new holder.

4.2.2.2. *Shear stress*

After reducing the centrifugal acceleration to 50G and holding it constant, three sets of dynamic loads were applied. Dynamic shear stress was measured by new TEP, old TEP, and calculated using acceleration values. Results for dynamic load-1 are presented in Fig. 4.7 , showing minimal changes in dynamic shear stress during shaking. The new TEP exhibited a small initial shear stress error, while the old TEP had a higher initial error.

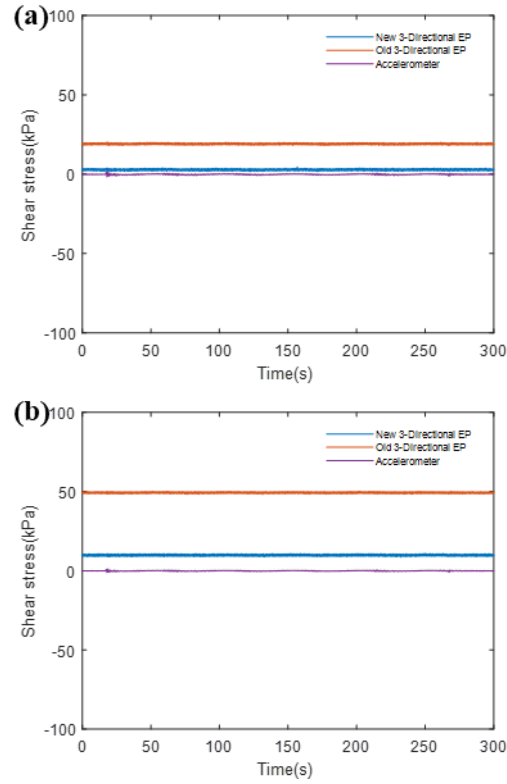


Fig. 4.7. Dynamic shear stress in dynamic load-1 (a) Stress in layer 1; (b) Stress in layer 2

The results of dynamic-2 which was showed in Fig. 4.8. The results showed an interesting tendency that no matter what kinds of measurement instrument was used, the amplitude of shear stress tends to be same. Fig. 4.8 (c) and (d) showed the comparison graph between two different measurement method. The strong correlation can be found, but the similar with dynamic load-1 result, the new TEP have a less initial error, and the less baseline drift after shaking.

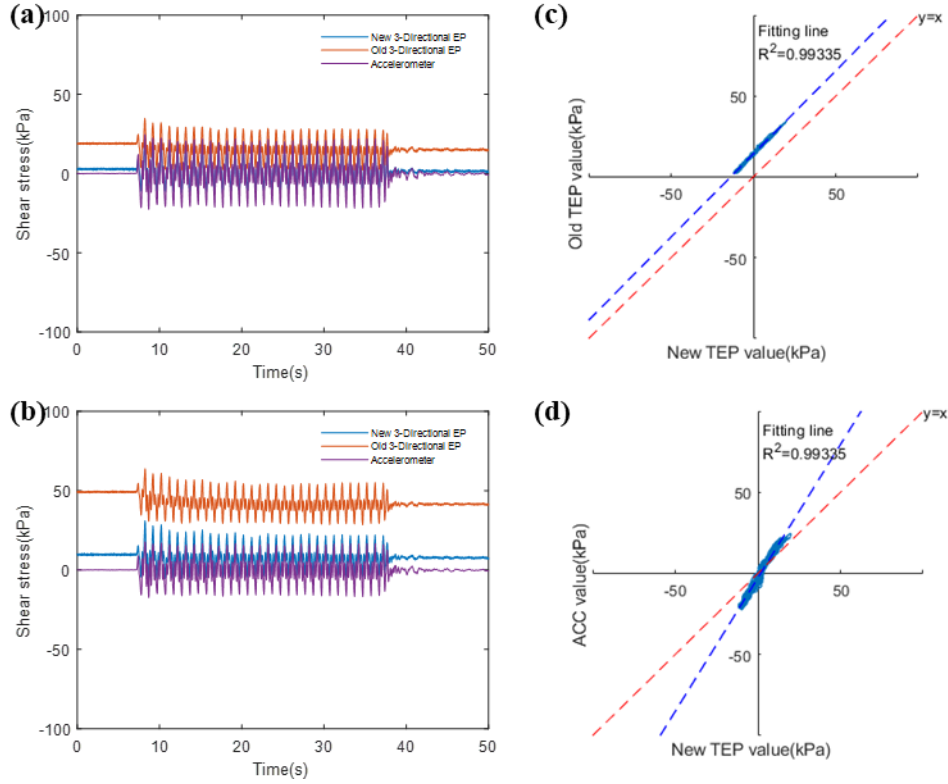


Fig. 4.8. Dynamic shear stress in dynamic load-2 (a) Stress in layer 1; (b) Stress in layer 2 (c) Comparison between new and old TEP stress value; (d) Comparison between new TEP and ACC stress value

The results of dynamic-3 was showed in Fig. 4.9. The figure showed larger shear stress due to higher input amplitude, despite similar shear stress amplitudes across different measurement methods, old TEP displayed more significant initial error and baseline drift. The new TEP demonstrated greater accuracy and less drift, highlighting its reliability compared to the old TEP.

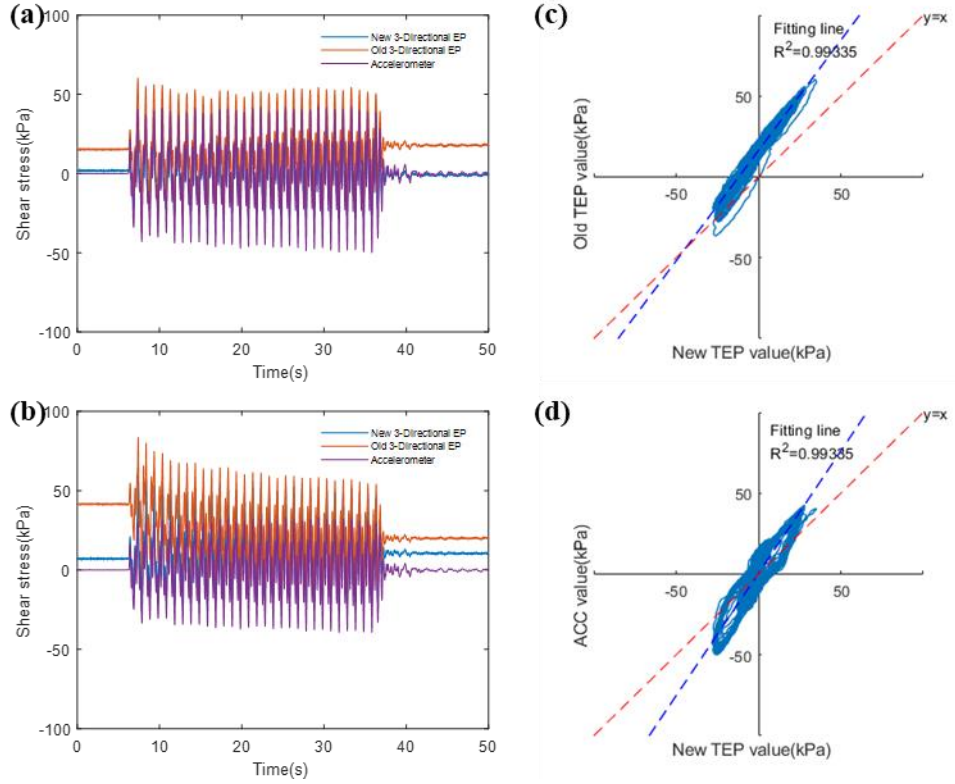


Fig. 4.9. Dynamic shear stress in dynamic load-3 (a) Stress in layer 1; (b) Stress in layer 2 (c) Comparison between new and old TEP stress value; (c) Comparison between new TEP and ACC stress value

4.3. Summary

The new three-directional earth pressure sensor design offers significant improvements in accuracy and reliability compared to the old sensor. These results validate the enhanced sensor's precision in measuring earth pressure and shear stress. Consequently, the new sensor holds promise for future research involving embankment analysis, liquefaction studies, and structural stability assessments. However, the study also emphasizes the importance of proper sensor design, calibration, and ongoing monitoring to ensure reliable data in geotechnical experiments.

Chapter 5. Behavior of embankment affected by height

5.1. Introduction

In order to investigate the arch action caused by embankment basal settlement, dynamic centrifuge model tests in variations of height placing on the flexible base were conducted with a water supply system. The use of a flexible base simulates the presence of soft ground underneath, resulting in a basal settlement, which can trigger the soil arch action(Miyata et al., 2014). To investigate the effects of liquid supplementation on arch action, a liquid supply system was utilized to deliver liquid inside the embankment following centrifugal loading. Furthermore, soil liquefaction was induced through dynamic load, to observe the relation between soil arch action and liquefaction. This study focuses on the changes in pressure change caused by the basal settlement due to centrifugal acceleration, and the earth pressure distribution on the bottom of the embankment model can be used to evaluate the arching effect.

5.2. Centrifuge tests conditions

5.2.1. Model preparation

The embankment model was built using Hiroshima sand, with a constant water content of 8% maintained across all cases. In Chapter 3, solid Urethane was employed to replicate soft ground conditions, yet soil arch action did not occur during the centrifugal process in any case. This suggests that the basal settlement was insufficient. Therefore, a new type of Urethane base was introduced in this chapter. To induce a larger basal settlement and trigger soil arch action, several holes were drilled at the center of the Urethane base, as depicted in Fig. 5.1. Additionally, a non-drainage boundary was emulated using vinyl coverings over the Urethane.

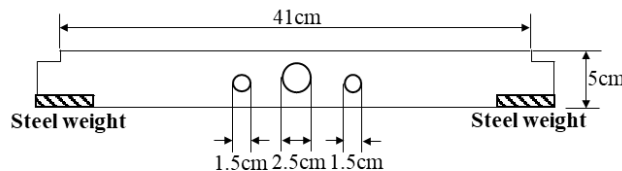


Fig. 5.1 Drilled Urethane base

To model the dynamic response of an embankment experiencing basal settlement, a specialized method was employed for model preparation, which involved the following steps:

(1) A urethane base was positioned at the bottom of the soil chamber to simulate soft ground conditions. A vinyl sheet was placed over the urethane to maintain an undrained condition.

(2) A pipe was affixed above the urethane base to introduce liquid into the embankment model during centrifuge testing.

(3) In some cases, to simulate basal settlement, a wooden rod was attached along the centerline on top of the base. One end of the rod moved with the settlement of the urethane base, while the other end protruded outside the embankment. Basal settlement was measured using a laser displacement sensor installed at the top of the embankment.

(4) Constructing the embankment model using the following standardized procedure as described in Section 2.4. The top of the Urethane base was showed in Fig. 5.2.

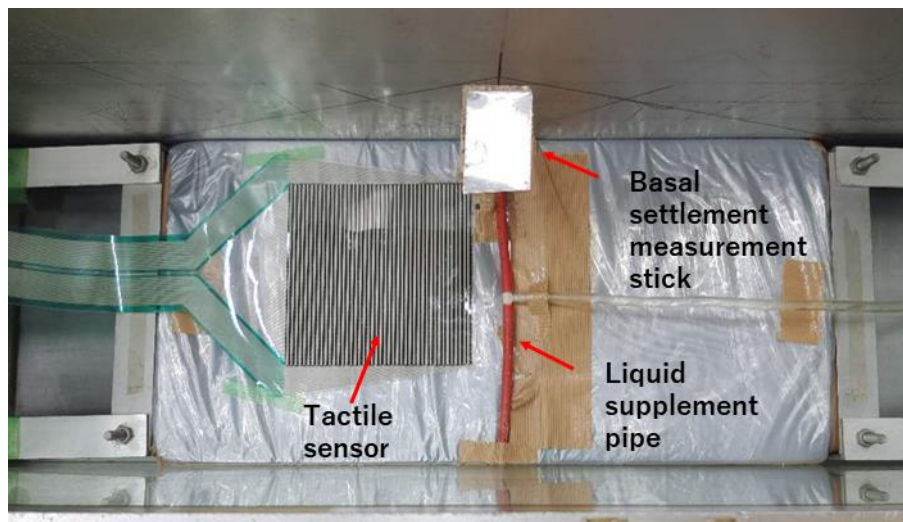


Fig. 5.2 Top of the Urethane base

5.2.2. Experiment condition

Three cases were conducted under different experimental conditions, as indicated in Table 5.1. The configuration of the embankment model remained the same in each case, although the embankment height varied in some instances, as illustrated in Fig. 5.3. The sensor condition is presented in Fig. 5.4.

The measurement instrument condition in each case was different. In total, 3 to 4 three-directional earth pressure gauges (TEP) were installed at various locations inside the embankment model to measure vertical pressure, horizontal pressure, and shear stress. In all cases, the new-type TEP was

employed. The number of accelerometers ranged from 2 to 7, installed to detect acceleration. If an ACC shared the location with the new-type TEP, it was installed within the TEP to indirectly calculate TEP rotation. Otherwise, it was installed alone. Specifically, ACC7 denotes the ACC installed at the top of the embankment, regardless of the embankment height. Four to six pore pressure transducers (PP) were installed to detect pore water pressure, and one pore pressure transducer was placed inside the water tank to monitor water levels. The conditions of the measuring instruments inside the embankment model are listed in Table 5.2. Additionally, two laser displacement sensors were positioned at the top of the soil chamber to track settlements and horizontal displacements of the embankment crest. In some cases, a wooden stick was attached to the Urethane base, so an additional laser displacement sensor was added to measure basal settlement.

Table 5.1. Experiment condition

Case No.	Test ID	Base settlement measure	Tactile sensor	White noise	Input wave type	Embankment height
1	DB-1-8-N-2	○	○	×	Ramped sine wave	100%
2	DB-0.75-8-N	×	○	×	Sine wave	75%
3	DB-0.5-8-N	○	○	×	Sine wave	50%

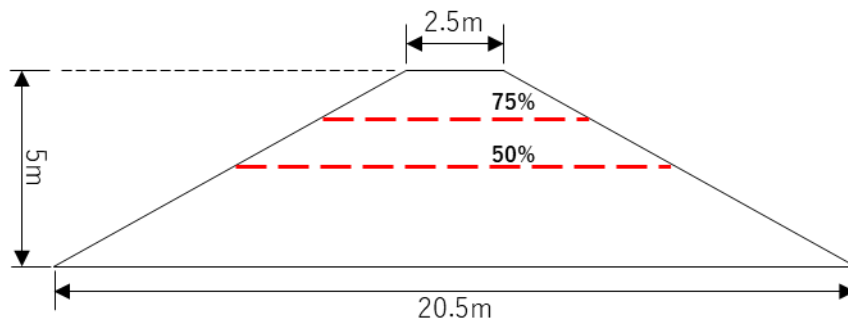


Fig. 5.3 Schematic diagram of test model

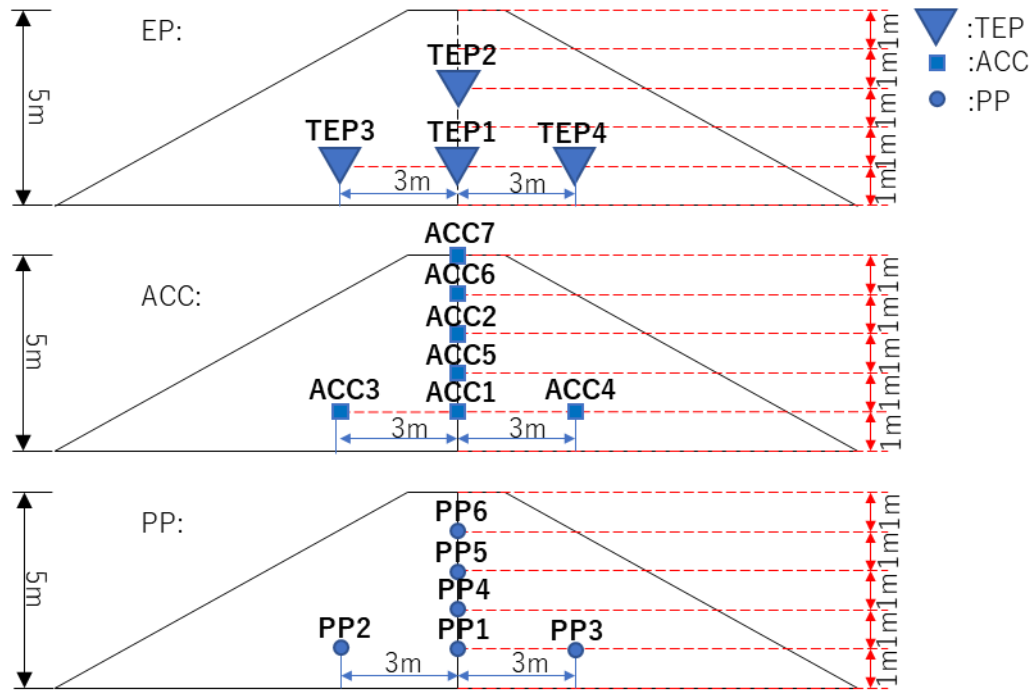


Fig. 5.4 Measurement instrument condition

Table 5.2. Measurement instrument condition

Case	TEP				ACC							PP					
No.	1	2	3	4	1	2	3	4	5	6	7	1	2	3	4	5	6
1	○	○	○	○	○	○	×	×	×	×	○	○	○	○	○	○	×
2	○	○	○	○	○	○	○	○	○	×	○	○	○	○	○	○	×
3	○	×	○	○	○	×	×	×	×	×	○	○	○	○	○	×	×

5.3. Stress closure based on Fixed Principal Axes (FPA)

In 1997, researchers investigated the central stress minimum phenomenon in a sand pile. In this study (Wittmer et al., 1997; Wittmer et al., 1996), Wittmer proposed the Fix Principal Axis (FPA) hypothesis, suggesting that the principal stress axes in a material element maintain a constant orientation fixed at the time of burial. Subsequently, in 2010, a closed-form solution for three stress components was determined (Pipatpongsa, 2010). The typical geometries of sand heap of constant unit weight γ lying at the angle of repose ϕ with the horizontal are illustrated in Fig. 5.6. The angle Ψ represents the orientation of major compression during sand avalanching and settling along the slope

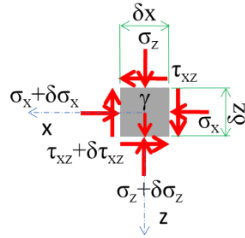
surface. Notably, this angle remains constant even as the sand heap grows by pouring more sand onto the apex. For loose sand, where the repose angle ϕ is equivalent to the friction angle, Ψ can be determined using Mohr's circle, as expressed in Eq. (5.1)

$$\Psi = \frac{\pi}{4} + \frac{\phi}{2} \quad (5.1)$$

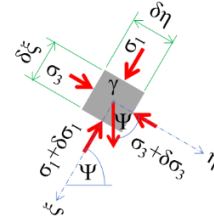
All stresses must satisfy the equilibrium condition, as illustrated in the Fig. 5.5(a). The equilibrium equation is presented in Eq. (5.2) and (5.3).

$$(\sigma_x + \delta\sigma_x)\delta z - \sigma_x\delta z + (\tau_{xz} + \delta\tau_{xz})\delta x - \tau_{xz}\delta x = 0 \quad (5.2)$$

$$(\sigma_z + \delta\sigma_z)\delta x - \sigma_z\delta x + (\tau_{xz} + \delta\tau_{xz})\delta z - \tau_{xz}\delta z = 0 \quad (5.3)$$



(a) Equilibrium condition of a differential slice in a general stress system under self-weight



(b) Equilibrium condition of a differential slice in the principal stress system under self-weight

Fig. 5.5. Equilibrium condition of a differential slice (a) Equilibrium condition of a differential slice in a general stress system under self-weight; (b) Equilibrium condition of a differential slice in the principal stress system under self-weight

In addition, Fig. 5.5(b) depicts the balance of minute elements shown in Fig. 5.6. From the Lamé-Maxwell balance equation, the balance equation in the ξ direction in Fig. 5.6 is expressed as Eq. (5.4) and (5.5).

$$(\sigma_1 + \delta\sigma_1)\delta\eta - \sigma_1\delta\eta = \gamma\delta\xi\delta\eta \sin \Psi \quad (5.4)$$

$$(\sigma_3 + \delta\sigma_3)\delta\xi - \sigma_3\delta\xi = \gamma\delta\xi\delta\eta \cos \Psi \quad (5.5)$$

Here, the transformation matrix from the x-z coordinate to the ξ - η coordinate is provided below.

$$\begin{Bmatrix} \xi \\ \eta \end{Bmatrix} = \begin{bmatrix} \cos \Psi & \sin \Psi \\ -\sin \Psi & \cos \Psi \end{bmatrix} \cdot \begin{Bmatrix} x \\ z \end{Bmatrix} \quad (5.6)$$

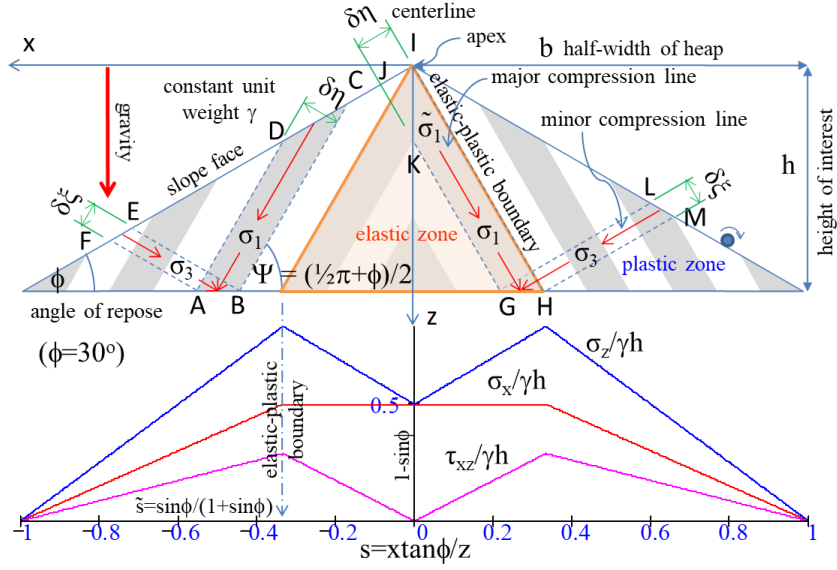


Fig. 5.6. Scaling stress distribution across the relative width of a symmetrical planar sand heap formed at its angle of repose with a height of interest measured from the apex (Pipatpongsa, 2010)

The fixed angle introduces an additional equation to the two equilibrium equations, enabling the analysis of three unknowns in stress variables. This research focused on obtaining stresses in triangular sand heaps made of non-compacted sand. However, in geotechnical engineering, where many earthen structures are commonly built in trapezoidal shapes, this method gains relevance.

Hirakawa (2021) developed and applied this method to trapezoidal embankments, incorporating base deflection to activate arch action. Fig. 5.7 illustrates the left half of a trapezoidal river embankment, with the right half calculated symmetrically.

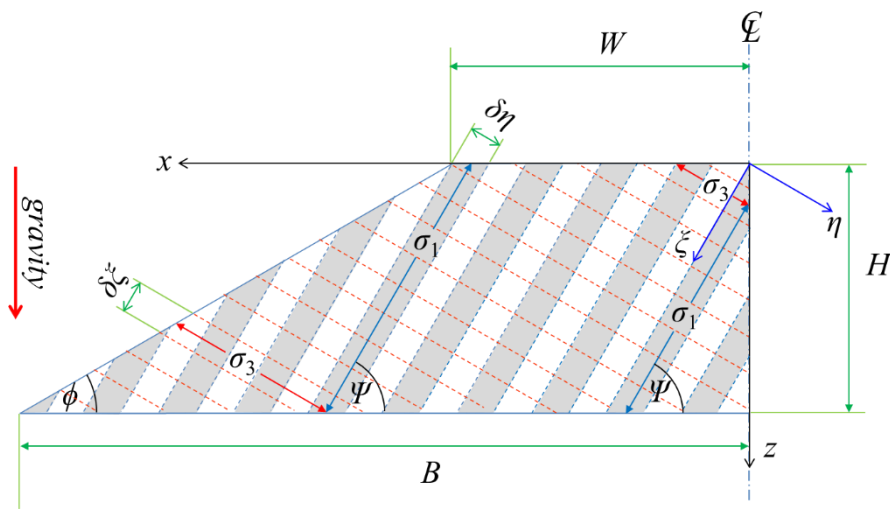


Fig. 5.7. Principal stress direction considered in FPA

Using the previously mentioned equilibrium equations and transformation matrix, stress analysis for trapezoidal embankments can be conducted. The embankment is divided into six regions, as shown in Fig. 5.8, adhering to required boundary conditions:

Regions 456: Areas subjected to the principal stresses σ_1 and σ_3 from the trapezoidal slope.

Regions 679: Areas subjected to the principal stresses σ_1 and σ_3 from the top surface of the trapezoid.

Regions 3469: Areas subjected to the principal stress σ_1 from the top surface of the trapezoid and the principal stress σ_3 from the trapezoidal slope.

Regions 789: Areas symmetrically divided at the center, where the right side receives the principal stress σ_1 from the top surface of the trapezoid, and the left side receives the principal stress σ_3 from the top surface of the trapezoid.

Regions 2398: Areas symmetrically divided at the center, where the right side receives the principal stress σ_1 from the top surface of the trapezoid, and the left side receives the principal stress σ_3 from the trapezoidal slope.

Regions 128: Areas symmetrically divided at the center, where the right side receives the principal stresses σ_1 and σ_3 from the trapezoidal slope.

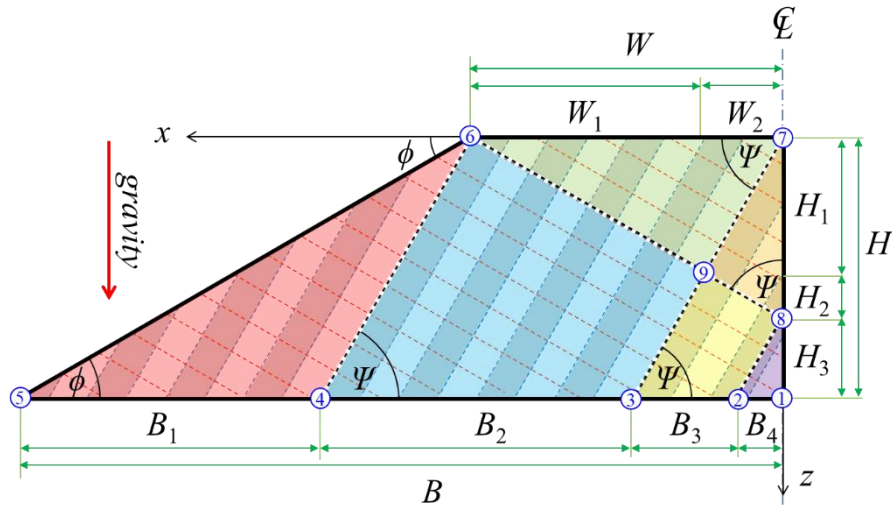


Fig. 5.8. Cross section of a trapezoidal embankment divided into six parts along boundary conditions

$$f_{56} = z - \frac{H}{B - W}(x - W) \quad (5.7)$$

$$f_{15} = z - H \quad (5.8)$$

$$f_{67} = z \quad (5.9)$$

$$f_{46} = z - \frac{H}{(B - B_1) - W} (x - W) \quad (5.10)$$

$$f_{68} = z - \frac{H}{W_2 - W} (x - W) \quad (5.11)$$

$$f_{37} = z - \frac{H_1}{W_2} x \quad (5.12)$$

$$f_{17} = -x \quad (5.13)$$

$$f_{28} = (z - H) + \frac{(H_1 + H_2) - H}{B_4} (x - B_4) \quad (5.14)$$

In the calculation process, the above boundary conditions are used and the results are summarized as shown in Fig. 5.9. Also, each region is expressed by Eq. (5.15) to (5.20).

$$F_{456} = (f_{56} > 0)(f_{46} \leq 0)(f_{15} \leq 0) \quad (5.15)$$

$$F_{679} = (f_{67} > 0)(f_{37} \leq 0)(f_{68} \leq 0) \quad (5.16)$$

$$F_{3469} = (f_{46} > 0)(f_{68} > 0)(f_{37} \leq 0)(f_{15} \leq 0) \quad (5.17)$$

$$F_{789} = (f_{67} > 0)(f_{68} \leq 0)(f_{17} \leq 0) \quad (5.18)$$

$$F_{2398} = (f_{37} > 0)(f_{68} > 0)(f_{28} \leq 0)(f_{15} \leq 0) \quad (5.19)$$

$$F_{128} = (f_{28} > 0)(f_{15} \leq 0)(f_{17} \leq 0) \quad (5.20)$$

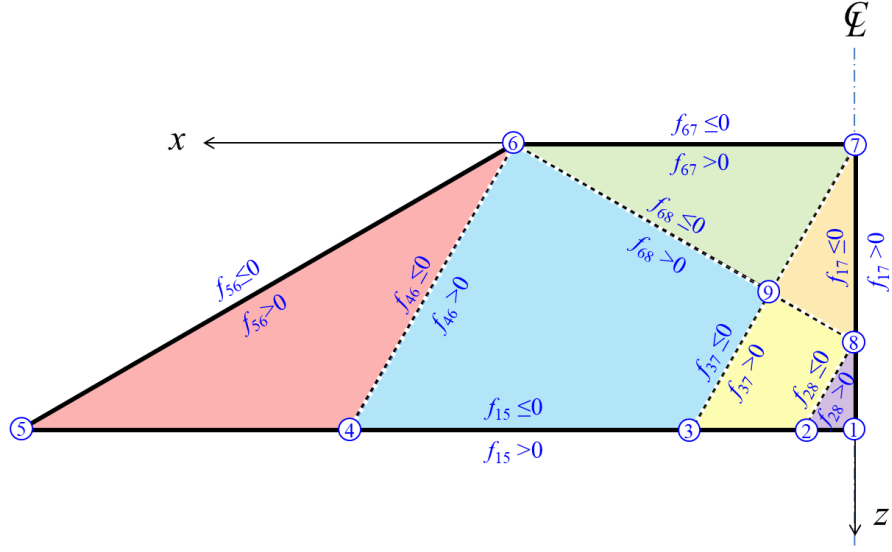


Fig. 5.9. Summary of boundary conditions

The stress boundary condition on the slope and top surface is expressed by Eq. (5.21).

$$\sigma_1 = \sigma_3 = 0 \quad (5.21)$$

By substituting each equation using Eq. (5.21) and performing an integral calculation, it can be rearranged as Eq. (5.22) to (5.25). Here, the subscript of σ_z indicates the vertical earth pressure within the regions shown in Fig. 5.9.

$$\frac{\sigma_{z_{128}}}{\gamma H_{max}} = \left(1 - \frac{1 - \sin \phi}{1 + \sin \phi} \cdot \frac{H - z}{B \tan \phi} - \frac{2 \sin \phi}{1 + \sin \phi} \left(1 - \frac{x}{B} \right) \right) (1 + \sin \phi) \quad (5.22)$$

$$\frac{\sigma_{z_{2398}}}{\gamma H_{max}} = \frac{\sigma_{z_{3469}}}{\gamma H_{max}} = \frac{1}{2} \left(\left(\left(1 - \frac{x}{B} \right) - \frac{H - z}{B \tan \phi} \right) \frac{(1 - \sin \phi)^2}{(1 + \sin \phi)} + \frac{z}{B \tan \phi} \right) (1 + \sin \phi) \quad (5.23)$$

$$\frac{\sigma_{z_{456}}}{\gamma H_{max}} = \left(\left(1 - \frac{x}{B} \right) - \left(\frac{H - z}{B \tan \phi} \right) \right) (1 + \sin^2 \phi) \quad (5.24)$$

$$\frac{\sigma_{z_{789}}}{\gamma H_{max}} = \frac{\sigma_{z_{679}}}{\gamma H_{max}} = \frac{z}{B \tan \phi} \quad (5.25)$$

Then, the stress closure based on Fixed Principal Axes (FPA) was used to obtain the stress solution. The schematic outline of trapezoidal embankment was showed in Fig. 5.10. In calculation, the left half of a trapezoidal embankment was used with the right half calculated symmetrically. The friction angle of the material is $\theta=31.3^\circ$ and cohesion is $c=7.4$ kPa. According to the FPA stress closure, $\Psi=\pi/4+\theta/2$ is adopted in the theoretical analysis based on the Mohr-Coulomb failure criterion. The incline angle

of the embankment slope φ and the half-base width B were showed in the figure. The maximum height of embankment appeared when the embankment was triangular shape, which can be expressed as:

$$H_{max} = B \tan \varphi \quad (5.26)$$

In this research, the embankment height varies from 100% height to 50% height, the relative height ratio H/H_{max} can be used to represent the embankment height, the relative height ratio for the 100%, 75% and 50% height were 0.88, 0.66 and 0.44 respectively.

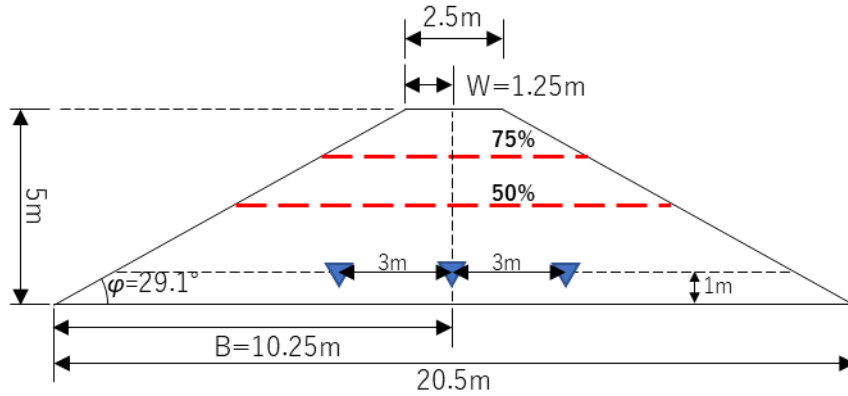


Fig. 5.10. Outline of embankment model

The vertical earth pressure of the stress solution was showed in Fig. 5.11. The results showed the soil arch action in embankment by the earth pressure dropping in center part. However, the pressure dropping is depending on the height of embankment, it will not occur if the embankment height is not sufficient, it means if the embankment height is not higher than the crown of arch, the soil arch action will not occur.

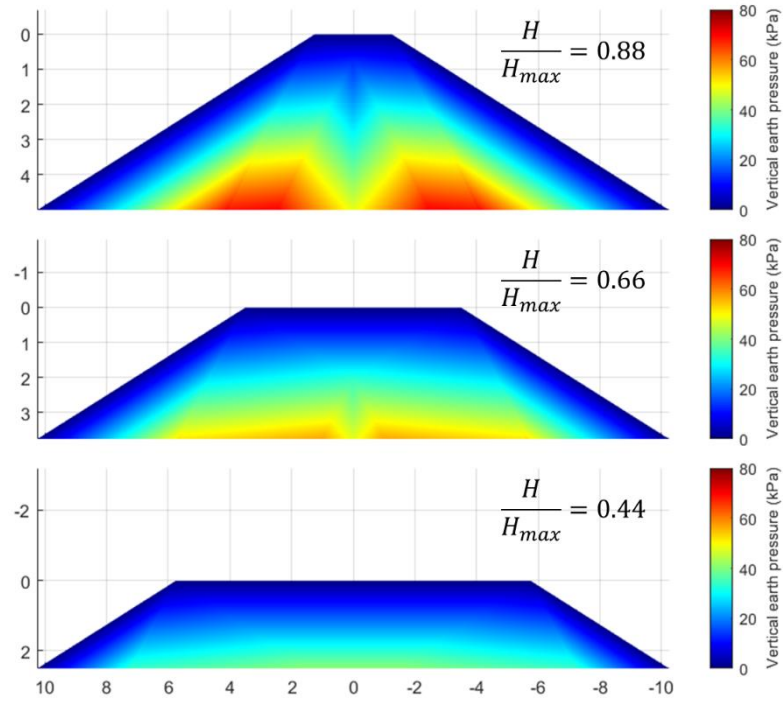


Fig. 5.11. Vertical earth pressure distribution

The vertical earth pressure distribution obtained by FPA stress solution on the bottom of embankment was showed in Fig. 5.12 to compare with the theoretical overburden pressure. It can be observed that when the embankment height reduced, the vertical earth pressure distribution tends to be more uniform and closer to overburden pressure.

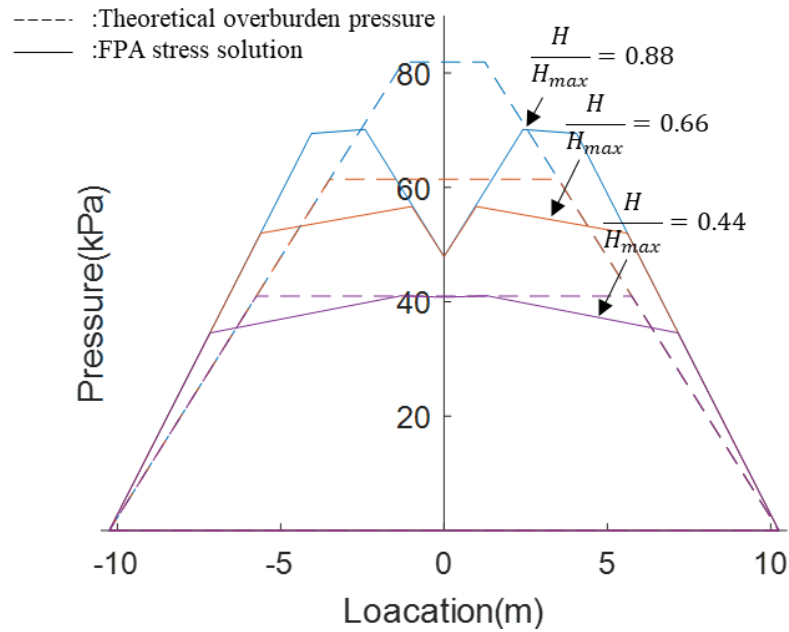
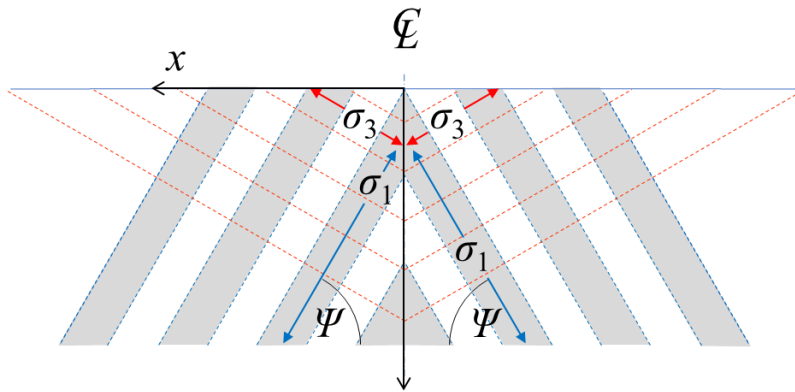


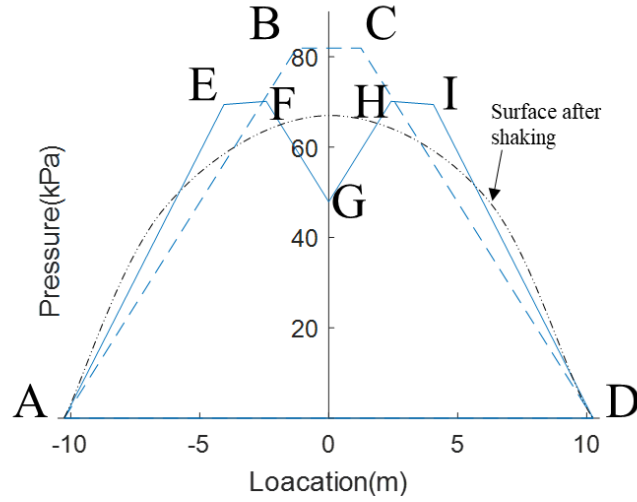
Fig. 5.12. Vertical earth pressure solution and theoretical overburden pressure

A symmetrical trapezoidal embankment was showed in Fig. 5.13 (a). Upon applying centrifugal acceleration to the embankment, basal settlement occurs, triggering arch action. The soil's weight is transferred linearly towards the base. In the Fixed Principal Axis (FPA) model, the arch's direction aligns with the major principal stress axis, forming an angle $\Psi = \pi/4 + \theta/2$, where θ is the friction angle. Fig. 5.13 (b) illustrates the theoretical overburden pressure as trapezoid ABCD and the vertical stress obtained using the FPA method as AEF GHID.

It's crucial for the areas of ABCD and AEF GHID to be equal, representing the embankment's weight. However, due to soil arching, the FPA model results in smaller vertical stress at the center. Subsequently, after applying dynamic load, embankment deformation occurs, as shown in the Fig. 5.13 (b). Lateral acceleration in the soil breaks the soil arch and redistributes stress inside the embankment. The vertical earth pressure should accurately describe the embankment's shape, with the pressure curve closely matching the outline of the embankment. The earth pressure in the center will increase because the arch collapsed and the soil weight transfers to the bottom, and on the two foots of arch the stress will decrease. In such cases, settlement on the embankment crest and bulging on the sides may occur, causing the embankment to lose its trapezoidal shape. This deformation in earthen structures can lead to significant disasters.



(a) Fixed Principal Axis (FPA) model



(b) Vertical stress distribution

Fig. 5.13. Arch action and stress distribution of embankment (a) Arch action in FPA model; (b) vertical earth pressure distribution

5.4. Test results and discussions

5.4.1. Vertical earth pressure distribution

As showed in Fig. 5.4, regardless the embankment height, earth pressure sensors were installed at three locations at the bottom of the embankment. These sensors, located at three distinct positions, facilitated the assessment of vertical earth pressure distribution. The heatmap figure was used to visualize the vertical earth pressure distribution change due to basal settlement, and the earth pressure between two earth pressure sensors was calculated using linear interpolation method. However, measuring basal settlement directly can be challenging. Nevertheless, since basal settlement correlates with centrifugal acceleration, so the centrifugal acceleration was used in heatmap figure instead.

In this chapter, the white noise did not be applied before the dynamic load, so only the influence of centrifugal acceleration, liquid supplement and dynamic loading were considered. By monitoring changes in vertical earth pressure distribution, we can explore how embankment height influences soil-arch action during these processes.

Fig. 5.14 provides a graphical representation of the correlation between vertical earth pressure distribution and centrifuge acceleration during the centrifugal loading process. The relative height of the embankment is indicated at the figure's corner. As explained in Chapter 3, the x-axis, y-axis and

color represent the centrifuge acceleration, geometric position and vertical earth pressure value respectively.

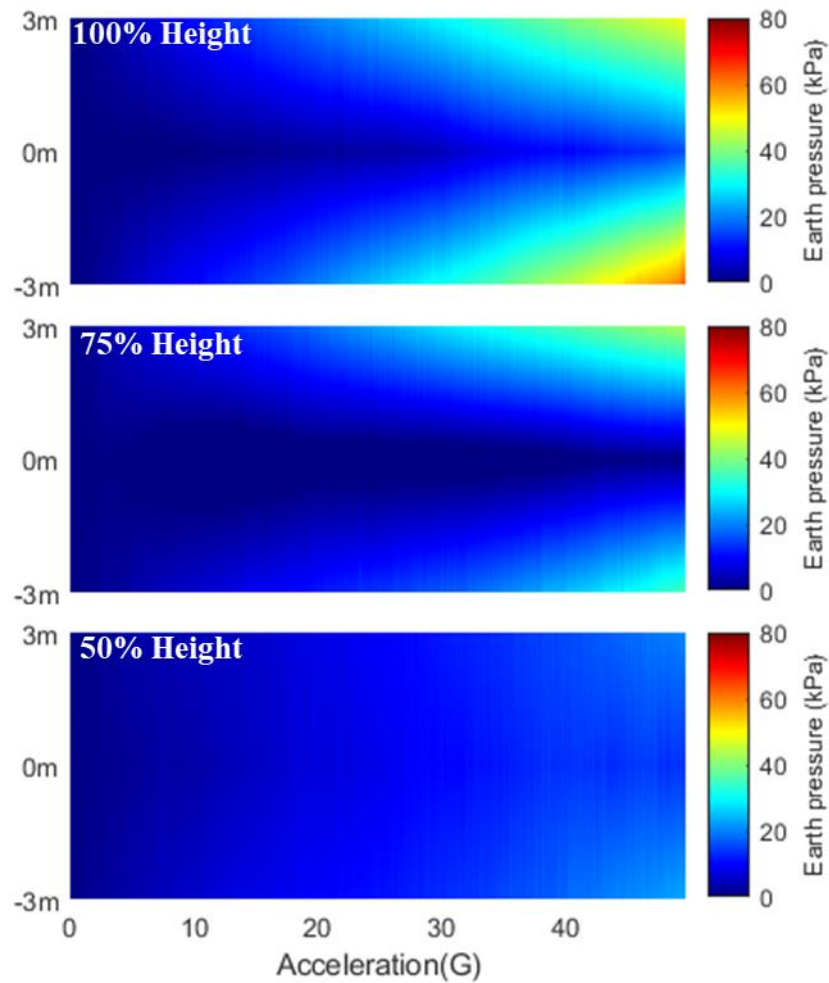


Fig. 5.14. Vertical earth pressure distribution of centrifugal loading process

As observed in Fig. 5.14, the vertical earth pressure at the base of the embankment increases during the loading process due to basal settlement induced by centrifugal acceleration. However, the extent of this increase varies across different cases. In case 1, the vertical earth pressure at the center is significantly smaller than on the sides, measuring 13.85 kPa at the center, and 46.44 and 61.02 kPa on the sides. Experimental limitations, such as measurement installation errors, compaction issues, or asymmetric settlement, make achieving complete symmetry in stress distribution challenging. Nevertheless, the occurrence of soil-arch action is evident. Similarly, in case 2, at a relative embankment height of 75%, the vertical earth pressure displays the same tendency as case 1, being smaller at the center and larger on the sides, signifying soil-arch action. However, the pressure values in case 2 are significantly smaller than in case 1, measuring 1.11 kPa at the center, and 41.04 and 34.29

kPa on the sides. This reduction is attributed to the decreased embankment height, resulting in decreased overburden pressure. In contrast, case 3, with a relative height of 50%, exhibits a linear increase in vertical earth pressure at all three locations during the centrifugal loading process. After loading, the vertical earth pressure tends to be similar across these locations. This can be attributed to the diminished height of the embankment model, leading to a weaker arch action, as the formation of soil arching requires sufficient height (Miyata et al., 2014)

Upon reaching a centrifugal acceleration of 50G, Metolose was introduced into the embankment model as pore liquid. Illustrated in Fig. 5.15 is the variation in vertical earth pressure during the liquid supplement process, with the start and end points of the liquid supplement period marked by dotted lines. Although the record lengths varied among cases, the quantity of liquid supplied to the embankment remained consistent in each case. This similarity in supplement duration across cases indicated that the supplement speed was uniform. This consistency resulted from using the same construction water content in all cases, suggesting that the supplement speed is unaffected by embankment height.

In Case 1, after the liquid supplement commenced, the vertical earth pressure slightly increased at the sides, while in the center, it initially decreased slightly and then rose. Upon reaching the target liquid volume, the supplementation was halted, and the vertical earth pressure at the center peaked at 32.13 kPa. However, after the liquid supplement ceased, the vertical earth pressure at the center decreased, while the pressure on the sides slightly increased. This phenomenon can be explained by the influence of excess pore water pressure caused by the liquid supplement on the earth pressure sensor during the supplementation. After supplementation ceased, the pressure induced by the liquid supplement dissipated, resulting in the recovery of earth pressure.

Case 2 exhibited a different behavior. Unlike Case 1, there was no initial decrease after the liquid was introduced into the embankment. During the liquid supplement process, the vertical earth pressure in the center was significant. After reaching the target liquid volume and stopping the supplement, the vertical earth pressure in the center did not recover. Post-liquid supplement, the vertical earth pressure in the three locations became 48.475 kPa in the center, 48.33 kPa, and 48.87 kPa on the sides. The vertical earth pressure distribution at the bottom of the embankment tended to be uniform, indicating the collapse of the soil-arch. Soil-arch action occurred due to basal settlement, but the stability of the arch was insufficient, leading to its disruption by the liquid supplement.

In Case 3, the change in vertical earth pressure distribution was similar to Case 1. Initial decreasing was observed, followed by an increase. However, in Case 3, the vertical earth pressure in the center

only slightly increased and then remained constant until the supplement was stopped. This phenomenon can be explained by the limited water level inside the embankment due to insufficient height, restricting the excess water pressure caused by the liquid supplement. After the supplement stopped, the earth pressure in the center recovered due to the dissipation of excess water pressure.

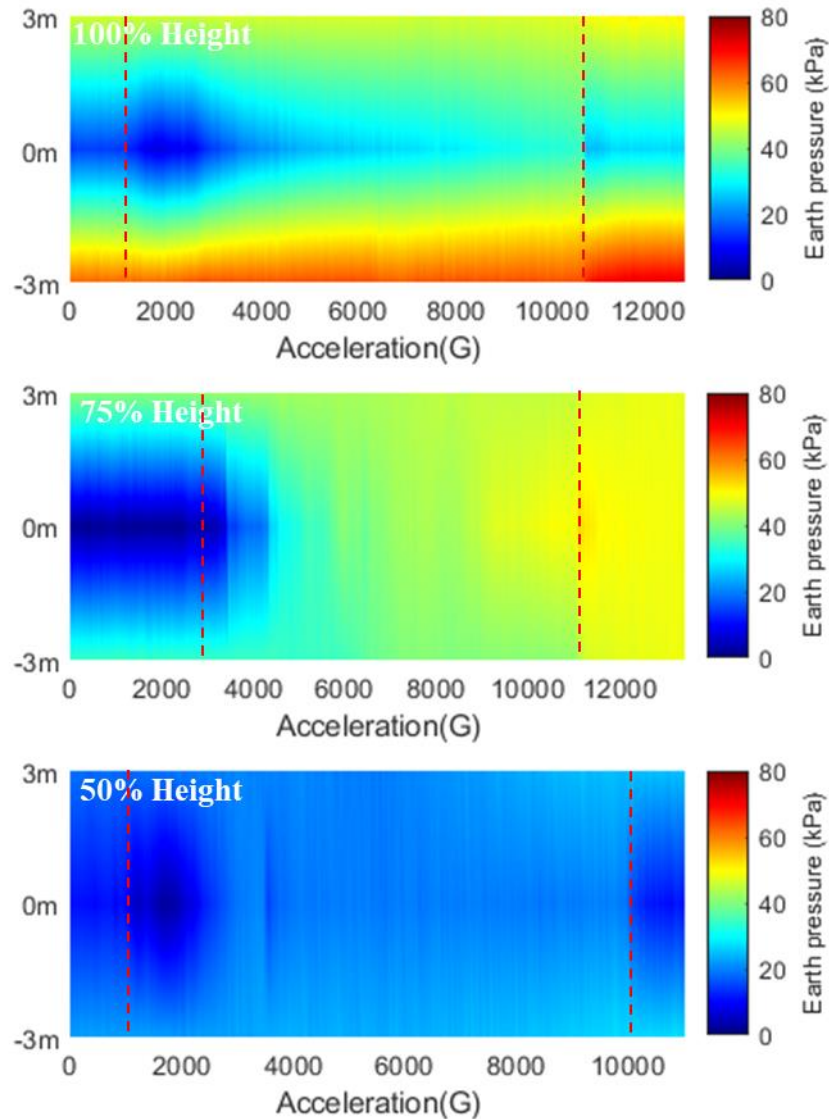


Fig. 5.15. Vertical earth pressure distribution of liquid supplement process

5.4.2. Deformation of the embankment

The normalized top settlement, measured by a laser sensor at the soil chamber's top, is presented in Fig. 5.16. This settlement metric was normalized based on the embankment height in each case. An evident correlation between embankment height and top settlement was identified. Throughout the

experimental process, a consistent inverse relationship between normalized top settlement and embankment height was observed. Despite variations in the weight of the embankment model across different heights—from Case 1 to Case 3—there was an increase in top settlement as the embankment height decreased.

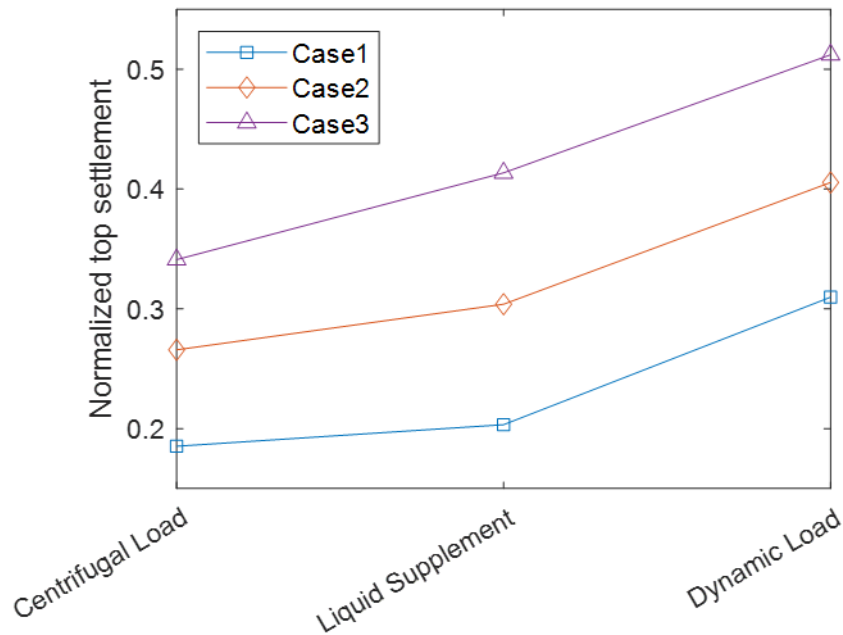


Fig. 5.16. Normalized top settlement of embankment

The top settlement is influenced by both basal settlement and changes in embankment thickness. To detect thickness changes during centrifuge testing, alternative measurement methods are necessary. In certain instances, a wooden stick was employed to measure basal settlement. Combining basal settlement and top settlement data enables us to evaluate changes in embankment height.

Fig. 5.17 illustrates the variation in embankment height throughout the entire experiments of Case 1 and Case 3. The height of the embankment exhibited a markedly different trend corresponding to the embankment height. In Case 1, after the application of centrifugal load, the embankment height increased. This occurrence can be attributed to soil-arch action. During the loading process, soil-arch action took place, transferring the load above the arch to the base. Deformation and density reduction primarily occurred below the soil-arch, with limited influence on the part above the arch. Despite basal settlement, the deformation of the embankment crest was not substantial, and the embankment height increased due to soil expansion beneath the arch.

In contrast, in Case 3, the embankment lacked sufficient height for soil-arch occurrence and development. Consequently, basal settlement impacted the entire embankment, resulting in significant deformation, central settlement, and a reduction in embankment height. Furthermore, the impact of liquid supplementation provided additional evidence supporting this observation. In Case 1, the liquid only influenced the soil below the arch, resulting in observed height increase. However, in Case 3, the entire embankment was influenced, causing a decrease in embankment height due to embankment deformation. Following the application of dynamic loading, significant damage occurred in both cases, leading to a substantial reduction in embankment height.

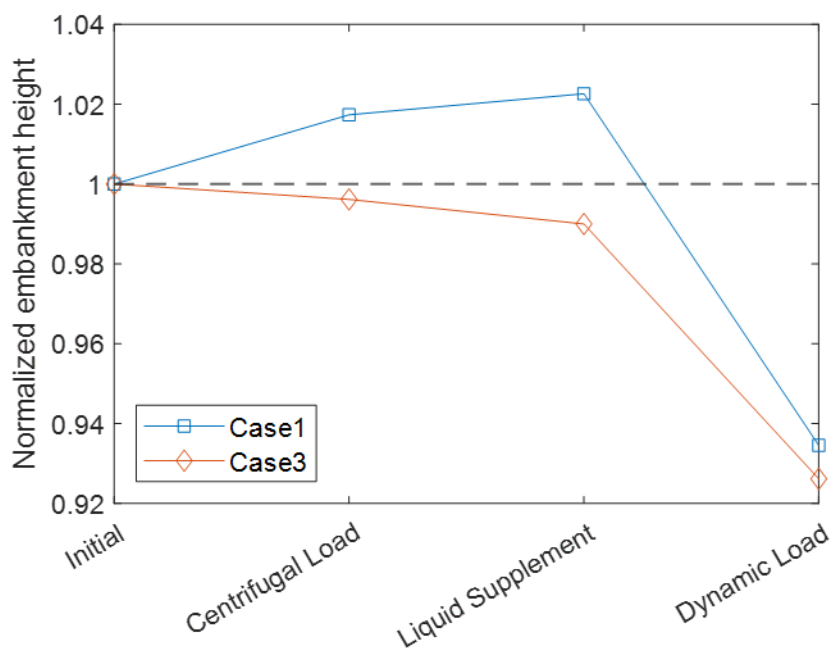


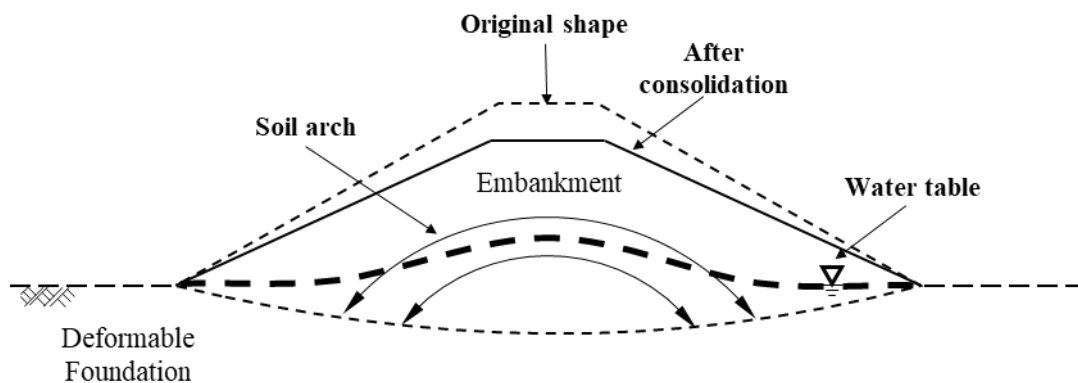
Fig. 5.17. Normalized embankment height

In summary, embankment height significantly influences the deformation pattern. In Cases 1 and 2, soil-arch action was triggered, restricting deformation primarily to the region below the soil-arch. However, in Case 3, the entire embankment was affected. The normalized top settlement of the embankment exhibits an inverse relationship with embankment height; as height decreases, settlement increases, despite a reduction in the weight of the embankment. In static conditions, embankment height increased in cases with sufficient height due to limited deformation below the soil-arch. Conversely, when height was small, substantial deformation occurred, resulting in a decrease in embankment height.

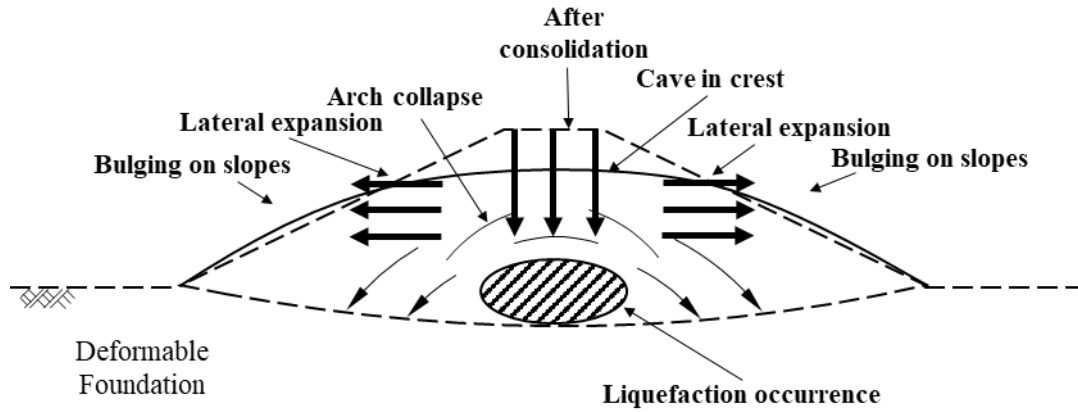
5.4.3. Interaction between arching effect and liquefaction in terms of stability of embankments

In this chapter, impact of embankment height was investigated. The interaction between arching effect and liquefaction in terms of stability of embankments was discussed in this section.

Case 1 has showed a typical example of arch collapse failure mode of embankment. After the centrifugal loading application, the deformation of embankment will happen due to basal settlement, the loose zone will appear on the bottom of embankment. However, the soil arch action will occur, the load above the soil arch will be transferred laterally, so the outline shape of embankment crest did not change a lot. Then, the liquid supplement will create a saturated zone, the pore water will increase the risk of liquefaction occurrence. However, the liquid supplement will not cause the collapse of soil arch, the shape of embankment holds constant after the liquid supplement, the deformation mode of embankment in static procedure was showed in Fig. 5.18(a). Then, the dynamic load applied in the embankment will cause the liquefaction in saturated loose zone beneath the soil arch, lead to the collapse of soil arch, the collapse of the soil arch transpires, leading to the upper part caving into the liquefied zone and causing damage to the embankment. Observable effects include cave-ins on the crest and bulging on the slopes.



(a) Static procedure



(b) Dynamic procedure

Fig. 5.18. Arch collapse mechanism (a) Static procedure; (b) Dynamic procedure

The visual representation of embankment deformation of case 1 is presented in Fig. 5.21, showcasing the outline shape in the initial state, after centrifugal loading, post-liquid supplementation, and after dynamic loading. The embankment's outline can be perceived as a trapezoid in initial state, with a slight basal settlement at the bottom. Upon reaching a centrifugal acceleration of 50G, substantial deformation was observed in the soft base, influencing the overall embankment, with a sufficiently high embankment, deformation primarily occurred at the bottom, preserving the overall crest shape. Post-application of dynamic loading, significant damage was observed in case 1, including a significantly cave-in in crest and bulging on slopes.

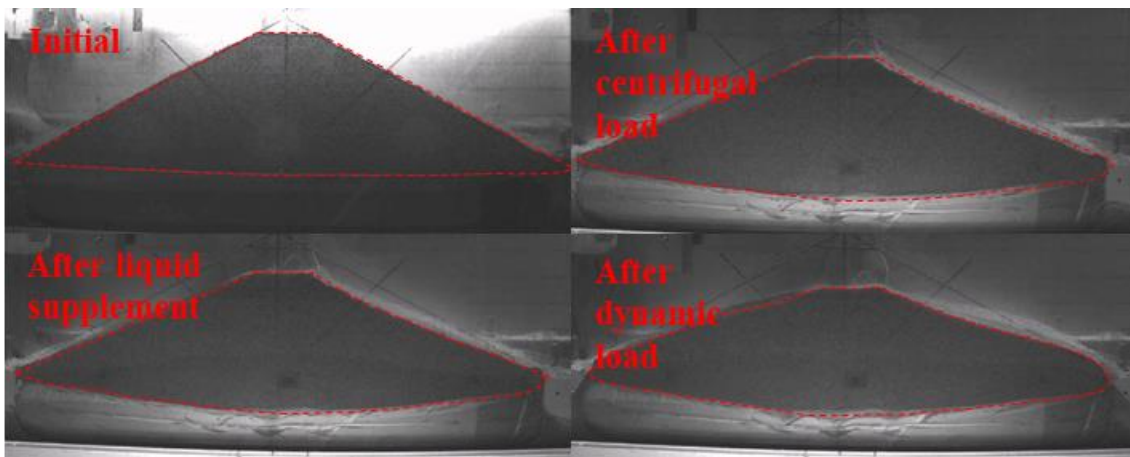


Fig. 5.19. Shape of embankment model

For dynamic process, to examine the influence of embankment height on liquefaction, a liquefaction evaluation was carried out. The pressures measured by TEP1 and PP1 (as depicted in Fig.

5.4) were utilized to assess the liquefaction conditions in each case. The Effective Stress Decrease Ratio (ESDR), as introduced by Lu(Lu et al., 2008), was employed to evaluate the occurrence of liquefaction. The ESDR is illustrated in Eq. (3.9).

Fig. 5.20 illustrates the pressure conditions and Effective Stress Decrease Ratio (ESDR) for case 1. The vertical earth pressure and pore water pressure are represented by the red and blue lines, respectively, using the right coordinate axes. Concurrently, the black line depicts the ESDR, utilizing the left coordinate axes, providing a comprehensive depiction of vertical earth pressure, pore water pressure, and ESDR in a single graph.

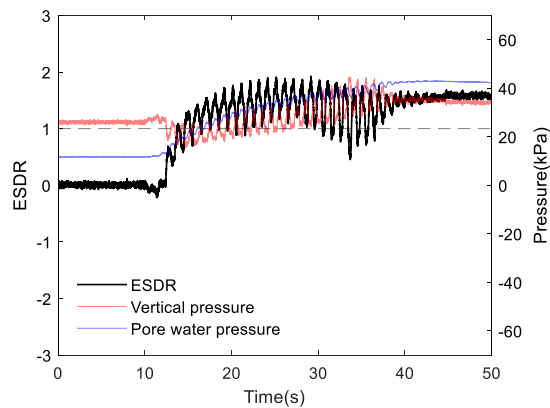


Fig. 5.20. Pressure condition and Effective Stress Decrease Ratio (ESDR) of case 1

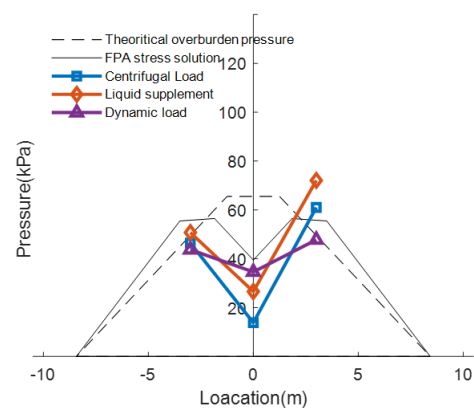


Fig. 5.21. Vertical earth pressure distribution change in case 1

The test results clearly indicate liquefaction occurrence. Pore water pressure increased immediately upon the application of dynamic loading, and ESDR swiftly reached 1. Liquefaction transpired irrespective of embankment height and the presence or absence of soil-arch action.

In Case 1, the initial earth pressure was relatively small, attributable to soil-arch action. Post-dynamic loading, pore water pressure increased, leading to liquefaction. Following liquefaction, earth pressure rose due to soil-arch collapse.

The vertical earth pressure during the centrifugal loading and liquid supplement processes was illustrated earlier, and in-depth discussions on the occurrence and development of soil-arch action were conducted. However, to achieve a more comprehensive representation of earth pressure distribution, additional insights are essential. Subsequent to the liquid supplement phase, dynamic loading was introduced into the embankment. Due to the short duration of the input wave, it was deemed unsuitable for a heatmap figure. Hence, Fig. 5.21 provides a depiction of the vertical earth pressure distribution throughout the entire experiment. The theoretical earth pressure was calculated based on the density and depth of the sensors, and it varied with different embankment heights. The result of FPA solution

was showed in figure too. Each point on the figure corresponds to the vertical earth pressure after a specific experimental process.

Initially, the vertical earth pressure in the center was smaller than on the two sides, indicating a clear soil-arch action. Post-liquid supplement, the soil-arch action remained unchanged, but the pressure values slightly increased. Following the application of dynamic loading, the vertical earth pressure decreased on the two sides, while it increased in the center, leading to a more uniform distribution of vertical earth pressure. This change indicates the collapse of the soil arch.

However, in Cases 2 and 3, deformation was noticeable not similar as case 1, as showed in Fig. 5.22. In static procedure, after the application of centrifugal load, the deformation will happen on the crest of embankment too, the settlement of embankment crest will match the settlement of soft base. Then, after dynamic loading, the liquefaction occurrence will lead to a larger collapse in center of the embankment.

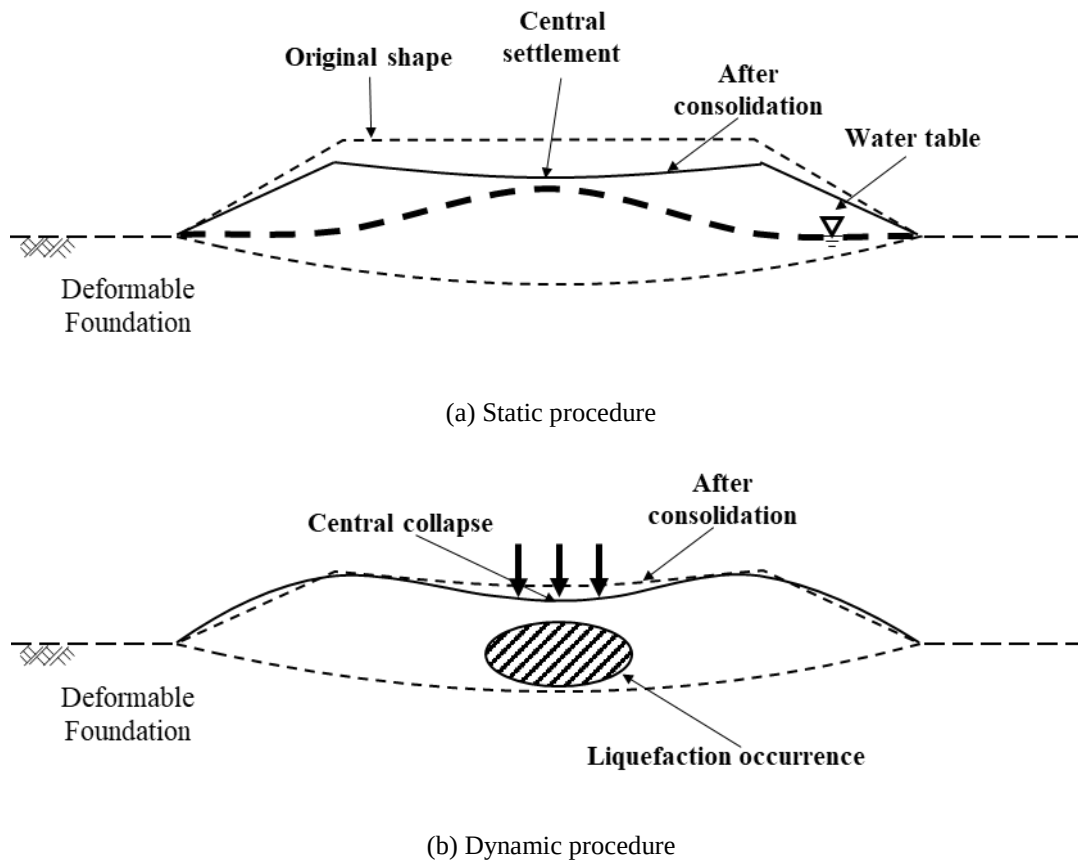
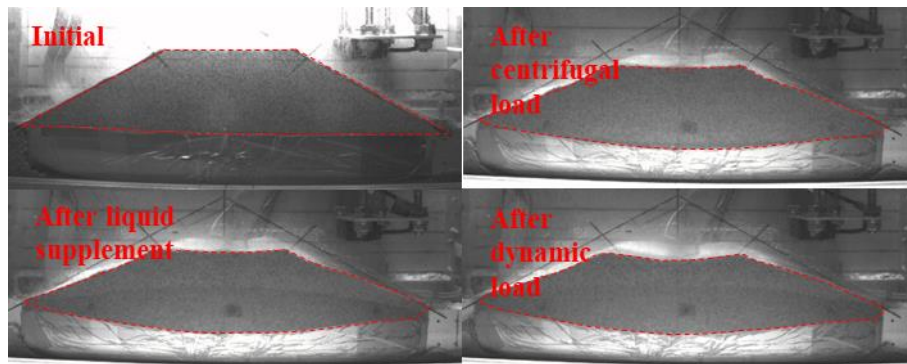


Fig. 5.22. Central collapse of embankment(a) Static procedure; (b) Dynamic procedure

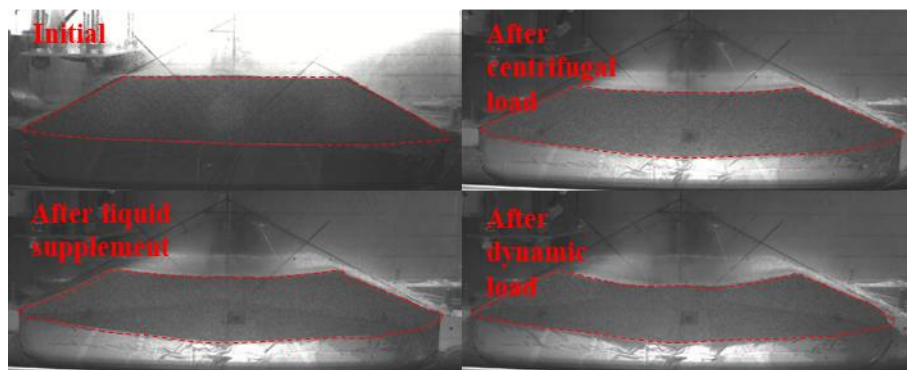
The picture of embankment deformation was showed in Fig. 5.23. After application of centrifugal load, the settlement not only at the bottom but also in the embankment crest. Case 3 exhibited larger crest deformation, attributed to the absence of soil-arch action. Subsequent to the liquid

supplementation, Cases 2 and 3 exhibited slight deformations, particularly in the center part, with Case 3 showing more settlement. Notably, Case 3 had a higher water level in the center, and liquid was observed on the embankment's top, indicating that insufficient embankment height affects permeation behavior.

Post-application of dynamic loading, significant damage was observed in both cases. Cases 2 and 3 experienced considerable deformation on the embankment crest, with the center settling more than the sides.



(a) Case 2



(b) Case 3

Fig. 5.23. Shape of embankment model(a) Case 2; (b) Case 3

The pressure conditions and Effective Stress Decrease Ratio (ESDR) for case 2 and case 3 were showed in Fig. 5.24. In Case 2, the initial earth pressure was much higher than in Case 1, as the soil-arch had already been disrupted by liquid supplementation. Upon application of dynamic loading, vertical earth pressure significantly decreased immediately, and excess pore water pressure dramatically increased, causing immediate liquefaction. This phenomenon may be attributed to significant deformation in the saturated zone resulting from pressure redistribution. In Case 3, the

change in vertical earth pressure was relatively small compared to other cases, but the increase in pore water pressure still triggered immediate liquefaction.

In summary, the change in embankment height has a minimal impact on liquefaction occurrence. While earth pressure conditions may vary with different relative embankment heights, excess pore water pressure increases rapidly, leading to liquefaction within the initial cycles. However, when the embankment height is sufficient to prevent soil-arch collapse due to liquid supplementation or small enough to avoid soil-arch action, earth pressure stabilizes.

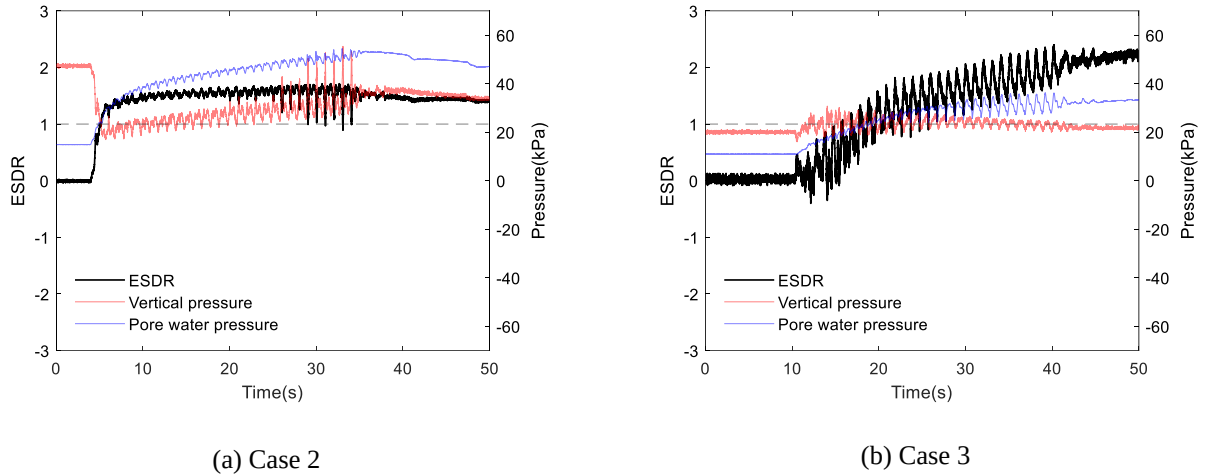


Fig. 5.24. Pressure condition and Effective Stress Decrease Ratio (ESDR) (a) Case 2; (b) Case 3

The change of vertical earth pressure of case 2 and case 3 were showed in Fig. 5.25. Case 2 exhibits a distinct trend in pressure redistribution. Similar to Case 1, significant soil-arch action occurred after the application of centrifugal loading. Nevertheless, upon the introduction of liquid, the soil arch collapsed, leading to a uniform distribution of vertical earth pressure across all three locations. Following the dynamic loading, there was a slight decrease in vertical earth pressure at the center.

In Case 3, following the application of centrifugal acceleration, the vertical earth pressure in the three locations was closely aligned. The pressure at the center was slightly smaller than on the two sides, a discrepancy attributable to the deformation and thickness changes in the embankment. It is challenging to definitively state whether soil-arch action occurred. The addition of liquid did not affect the distribution of vertical earth pressure; rather, the pressure values experienced a slight increase due to the liquid volume. Post-dynamic loading, the vertical earth pressure distribution remained largely unaffected, with only a marginal increase.

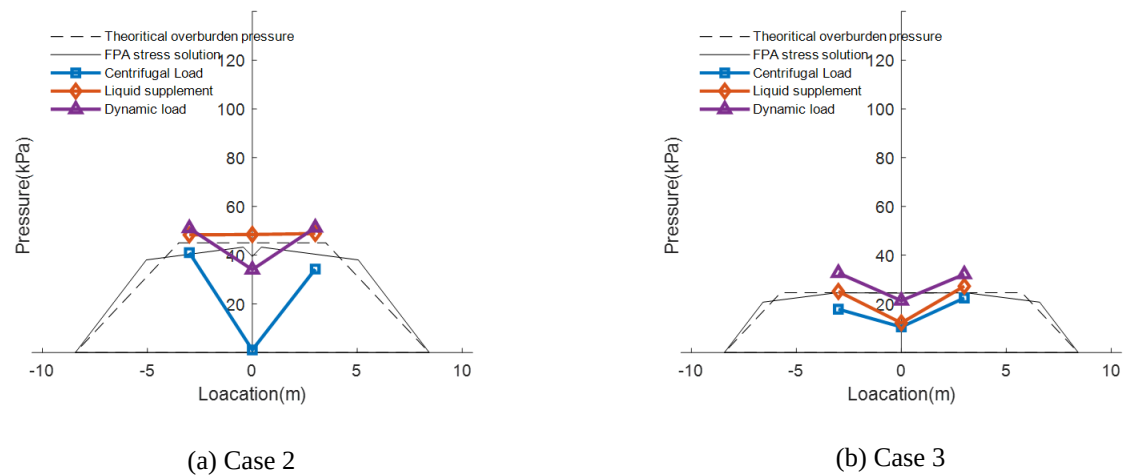


Fig. 5.25. Vertical earth pressure distribution change (a) Case 2; (b) Case 3

In all three cases, the center earth pressure was lower than predicted. This occurrence may be attributed to substantial basal settlement, causing extensive deformation at the embankment's base and a subsequent decrease in density. While the Fixed Principal Axis (FPA) method assumes the soil arch triggered by basal settlement, it doesn't account for shape changes in the embankment. Additionally, the FPA method assumes a fixed direction of the major principal stress, potentially introducing errors. Despite this, the advantage of the small center pressure is its clarity in observing soil arch action resulting from basal settlement.

In summary, the height of the embankment has a significant influence on the occurrence and development of soil-arch action. Soil-arch action occurred in the 100% height and 75% height cases but did not happen in the 50% height case. The development of soil-arch requires a sufficient height. In Case 3, during the centrifugal loading process, the arch continued to develop but reached the border of the embankment, causing the soil-arch to collapse.

Additionally, the height of the embankment affects the arch stability. In the liquid supplement process, soil-arch existed in Case 1 but collapsed in Case 2, indicating that the reduction in height has an impact on soil-arch stability.

5.5. Summary

This chapter focused on the impact of embankment height to the behavior of embankment. The behavior of the embankment was meticulously examined, resulting in the following key findings:

(1) The Fixed Principal Axes (FPA) was used to obtain the earth pressure distribution of embankment, the result of simulation showed the relationship between embankment height and soil-arching effect. The solution shows the soil-arching will be influenced by embankment height.

(2) In centrifugal loading process, the occurrence of soil-arch action will be influenced by the embankment height. The occurrence of soil-arch required a sufficient height, the reduction of height will cause no soil-arch occurrence. On the other hand, the reduction of height will lead a weaker soil-arch, although the soil-arch action occurred, the arch may collapse by the liquid supplement in 75% height case.

(3) The change in embankment height has a minimal impact on liquefaction occurrence. While earth pressure conditions may vary with different relative embankment heights, excess pore water pressure increases rapidly, leading to liquefaction within the initial cycles.

(4) The deformation of the embankment exhibited variations based on embankment height. Soil-arch action limited deformation in Cases 1 and 2 to the region below the arch, while Case 3, lacking sufficient height, experienced widespread deformation. The normalized top settlement demonstrated an inverse relationship with embankment height, emphasizing the influence of height on settlement. Embankment height changes were assessed by combining basal and top settlement data, revealing trends consistent with soil-arch action and deformation.

Chapter 6. The liquefaction evaluation and soil-arch action of soil nail reinforced embankment

6.1. Introduction

Japan, situated at the convergence of tectonic plates, witnesses about 10% of global earthquakes. In areas like Osaka Bay, with thick clay layers in the ground, construction on soft soil often leads to compaction settlement, altering earthquake dynamics and potentially increasing damage. The 2011 Great Tohoku Earthquake revealed significant embankment damage due to liquefaction (Ministry of Land, 2011). Understanding how embankments on compacted soil respond to liquefaction and accurately evaluating their resistance can refine seismic embankment design, particularly in vulnerable regions. Moreover, exploring efficient methods to reinforce embankments constructed on soft ground and enhance their resistance to liquefaction is a valuable avenue for research.

Okamura et al demonstrated through centrifuge model experiments that lateral elongation deformation occurs at the bottom of embankments due to ground settlement, leading to a decrease in density at the central bottom and a reduction in lateral confinement pressure (Okamura et al., 2013). Furthermore, some researchers suggested through centrifuge experiments that stress within embankments is distributed in an arch shape (referred to as arching) as a result of concentrated subsidence in the central part of the foundation soil.

However, this occurrence, reminiscent of the New Austrian Tunneling Method (NATM) utilized in tunnel construction, has raised the possibility of reinforcing this arching within embankments as a means to enhance stability.

This occurrence, reminiscent of the New Austrian Tunneling Method (NATM) utilized in tunnel construction, has raised the possibility of reinforcing this arching within embankments to enhance stability. Developed in Austria during the 1960s (Rabcewicz, 1964), NATM emphasizes real-time support decisions based on ground behavior, differing from conventional methods. It has revolutionized tunneling practices by providing a versatile and cost-effective approach to tunnel construction, particularly in soft ground conditions.

NATM's core principle is to utilize the surrounding ground to provide primary support, allowing controlled deformation and mobilization of the ground's inherent strength. This method employs a flexible support system, including shotcrete (sprayed concrete), rock bolts, and steel ribs, to stabilize the tunnel as excavation progresses. Continuous monitoring and adjustment of the support system are integral to NATM, ensuring the stability and safety of the tunnel. In NATM, the shotcrete and

reinforcement systems work together to create a stable arch that resists the forces exerted by the surrounding ground. The initial shotcrete lining, applied immediately after excavation, adheres to the tunnel's surface, preventing loose materials from falling and providing immediate support. Rock bolts and steel ribs further reinforce this primary lining, distributing loads and enhancing the arching effect that stabilizes the tunnel. As showed in Fig. 6.1, this graph illustrates how shotcrete, rock bolts, and steel ribs are used in combination to form a stable arch within the tunnel, resisting the forces from the surrounding ground and ensuring structural integrity.

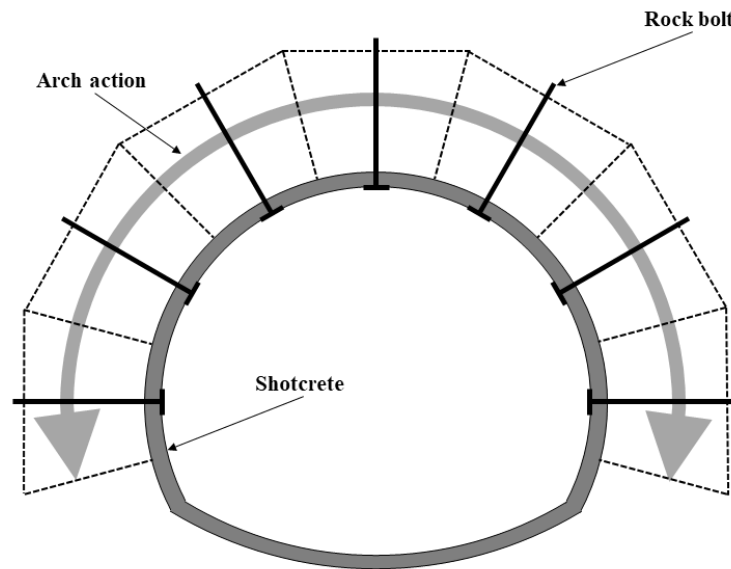
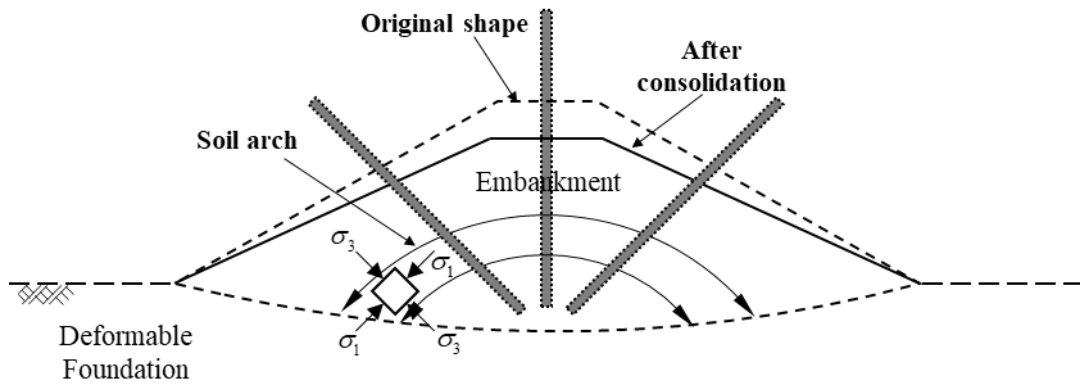


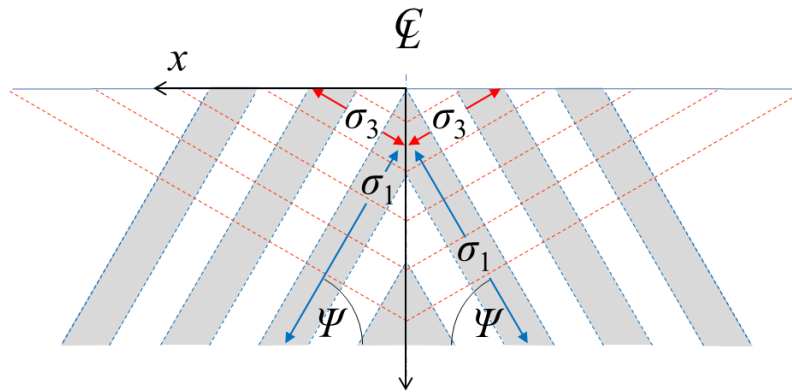
Fig. 6.1 Schematic diagram of NATM tunneling

Inspired by the principles of NATM, this study explores the application of a similar reinforcement mechanism in embankment engineering. Specifically, the concept of reinforcing soil arches formed by basal settlement in embankments using soil nails is investigated. Soil arching is a phenomenon where the load from the overlying soil is transferred laterally around a weaker zone, creating a stable arch. This arching action can significantly influence the stability and deformation behavior of embankments, particularly under conditions of differential settlement or liquefaction. In the context of embankments subjected to basal settlement, depicted in Fig. 6.2(a), soil arching emerges due to the settlement, as showcased in the figure's principal stress condition. The Fixed Principal Axis (FPA) model mentioned in the previous chapter also provides a rough estimation of soil arch action, as shown in Fig. 6.2(b). This arching yields advantages for the embankment, enabling enhanced load-bearing and deformation resistance. However, the soil arch's strength diminishes and it becomes vulnerable under seismic loads.

The collapse of the soil arch and the subsequent redistribution of earth pressure pose significant threats to the embankment's stability. To mitigate this risk, soil nails are strategically inserted into the embankment along the σ_3 direction, as shown in Fig. 6.2 (b). Such reinforcement, similar to the rock bolts used in NATM construction, can increase the stability of the soil arch and the deformation resistance of the embankment under seismic loads.



(a) Schematic figure of soil arch



(b) Fixed Principal Axis (FPA) model

Fig. 6.2 Soil arching above basal settlement

This chapter aims to contribute to the understanding of soil nail reinforcement's efficacy performance in embankments subjected to basal settlement and dynamic loading. The research delves into the arching effect, liquefaction resistance, and deformation behavior within the context of soil nail reinforcement. By conducting dynamic centrifuge tests, the study seeks to provide insights into the behavior of shear-reinforced embankments under varying conditions. Importantly, the advantage of soil nails lies in their post-construction applicability, offering a convenient method for reinforcing

embankments even after their initial formation. This reinforces the adaptability and convenience of using soil nails, both during and after the construction phase.

6.2. Centrifuge tests conditions

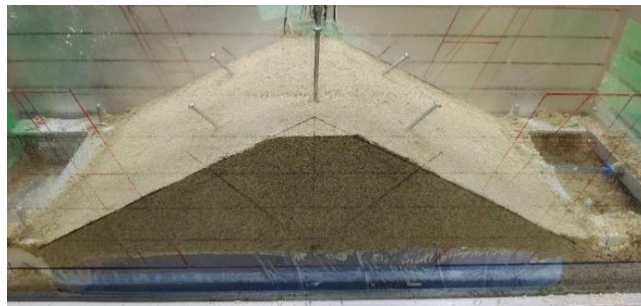
6.2.1. Model preparation

In this chapter, the construction water content of the model hold at 8%, the construction water content did not change in all cases. To simulate soft ground conditions, Urethane base was drilled holes at its center, as shown in Fig. 5.1. A non-drainage boundary was replicated by vinyl coverings over the Urethane.

The construction method of embankment model was same with the last chapter. In this chapter, the embankment height holds constant, which was 10cm in model scale. After construction, soil nails were employed as reinforcement. A piercer was used to pre-drill a hole in the embankment model, and M4 bolts (diameter 4mm, length 120mm) were inserted, as illustrated in Fig. 6.3.



(a) The pre-drill



(b) The embankment after soil nail insertion

Fig. 6.3 Soil nail installation (a) The piercer was used to pre-drill the hole; (b) The embankment after soil nail installation

6.2.2. Experiment condition

8 cases were conducted with the different reinforcement condition, as indicated in Table 6.1. The model configuration of each case was showed in Fig. 6.4, the number and position of soil nail were showed, but it should be noticed that and two rows of soil nails was used, so the number of soil nail was doubled from the figure. Fig. 6.5 shows the condition of sensors. If the soil nails are insert into the embankment through σ_3 direction as showed in Fig. 5.13(a), the confining pressure in arch structure can be reinforced and increase the strength of arch. To prove this, some control cases with the same number of nails but did not insert through the σ_3 direction were conducted.

The measurement instrument condition in each case was different. In total, 4 new-type three-directional earth pressure gauges (TEP) was installed on different location inside the embankment model, to measure the vertical pressure, horizontal pressure and shear stress. The number of accelerometers ranged from 2 to 7, installed to detect acceleration. If an ACC shared the location with TEP, it was installed within the TEP to indirectly calculate TEP rotation. Otherwise, if the ACC was located separately from TEP, it was installed alone. 4 to 6 pore pressure transducers (PP) was installed to detect the pore water pressure, and one pore pressure transducer was placed inside the water tank to monitor water levels. The measurement instrument condition inside the embankment model was listed in Table 6.2. Additionally, two laser displacement sensors were positioned at the top of the soil chamber to track settlements and horizontal displacements of the embankment crest, and in some case wood stick was attached on the Urethane base, so an addition laser displacement sensor was attached to measure the basal settlement.

Table 6.1. Experiment condition

Case No.	Test ID	Base settlement measure	Tactile sensor	White noise	Input wave type	Reinforcement condition
1	DB-1-8-N-2	○	○	×	Ramped sine wave	No nail
2	DB-1-8-R6-1	×	×	×	Sine wave	6 nails symmetric
3	DB-1-8-R6-2	×	○	○	Ramped sine wave	6 nails symmetric
4	DB-1-8-R6-3	○	○	×	Ramped sine wave	6 nails symmetric
5	DB-1-8-R4-1	×	×	×	Sine wave	4 nails asymmetric
6	DB-1-8-R4-2	×	○	×	Ramped sine wave	4 nails symmetric
7	DB-1-8-R6-4	○	○	×	Ramped sine wave	6 nails vertically
8	DB-1-8-R4-3	○	○	×	Ramped sine wave	4 nails vertically

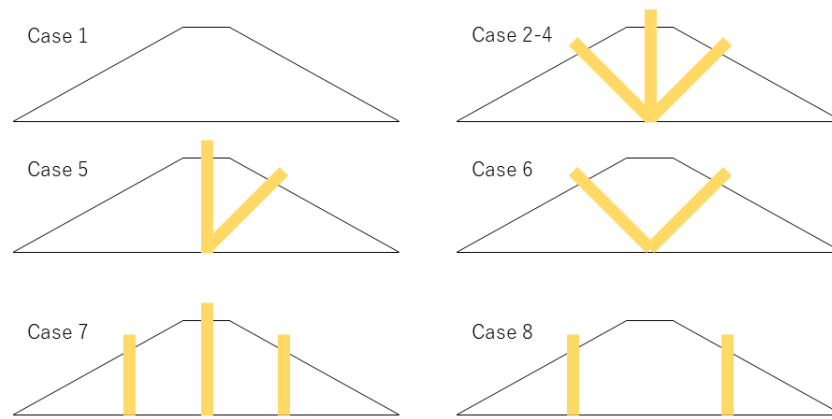


Fig. 6.4 Soil nail reinforcement condition

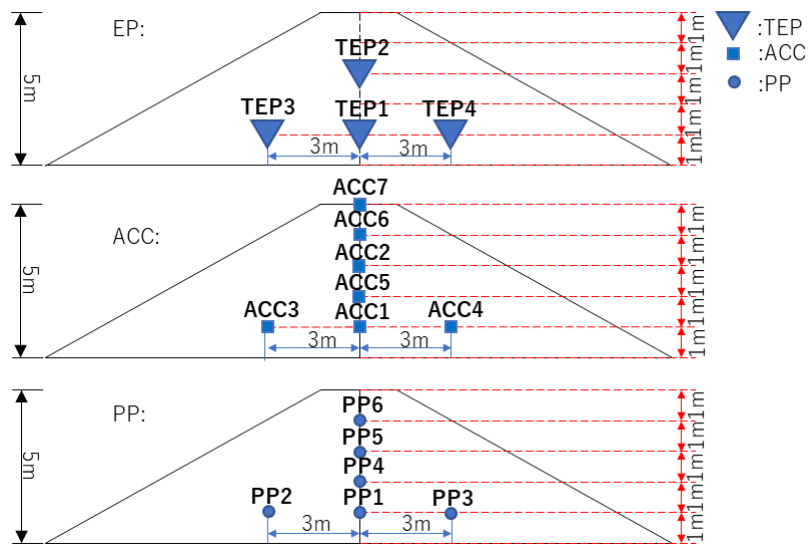


Fig. 6.5 Measurement instrument condition

Table 6.2. Measurement instrument condition

Case No.	TEP				ACC							PP					
	1	2	3	4	1	2	3	4	5	6	7	1	2	3	4	5	6
1	○	×	○	○	○	×	×	×	×	×	○	○	○	○	○	○	×
2	○	○	○	○	○	○	○	○	○	○	○	○	×	×	○	○	○
3	○	○	○	○	○	○	○	○	○	×	○	○	○	○	○	○	×
4	○	○	○	○	○	○	○	○	○	×	○	○	○	○	○	○	×
5	○	○	○	○	○	○	○	○	○	○	○	○	×	×	○	○	○
6	○	○	○	○	○	○	×	×	×	×	○	○	○	○	○	○	×
7	○	○	○	○	○	×	×	×	×	×	○	○	○	○	○	○	×
8	○	○	○	○	○	×	×	×	×	×	○	○	○	○	○	○	×

6.2.3. Experimental program

Similar with the former chapter, the dynamic centrifuge test is comprised of three main steps: centrifugal loading step, liquid supplement step and dynamic loading step. Firstly, the centrifugal acceleration gradually increased from 1G to 50G. While white noise was considered for alleviating density and stress unevenness induced by compaction, its implementation was restricted to case 4 as a control measure. The use of white noise can impact the embankment's strength, necessitating a cautious approach.

Following this, a controlled volume of Metolose-based liquid was supplied. This liquid was stored in a water tank attached on the back side of the soil chamber, and the liquid flow was controlled by the solenoids, once the solenoids opened, the liquid will flow into the model through the water pipe above the urethane base. Precise control of liquid volume was achieved by monitoring the pore pressure transducer within the water tank, with water level measurements derived from water pressure readings. The fluid was supplied until the pore water pressure gauge in the tank decreased around 7 kPa.

Following the liquid supplement, as it rapidly leaked from the embankment, the dynamic load was promptly applied. In all cases, the input dynamic load maintained a consistent amplitude of 4 m/s^2 in prototype scale, with a frequency of 1Hz and a total of 30 cycles. Two types of input waves were employed in this research: the sine wave and the ramped sine wave. Fig. 6.6 showed the input acceleration of ramped sine wave and sine wave used in this Chapter.

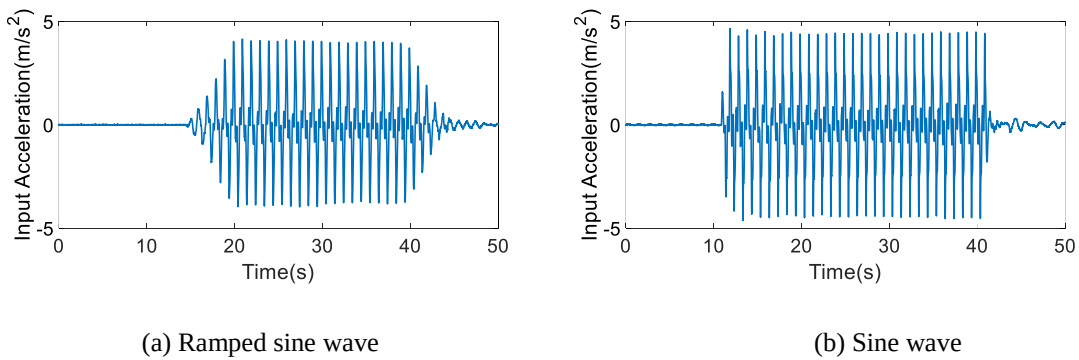


Fig. 6.6. Input acceleration time histories (a) Ramped sine wave; (b) Sine wave

6.3. Test results and discussions

6.3.1. Deformation of the embankment

Fig. 6.7 have showed a visual representation of the no-reinforcement embankment's shape change through all experiment process. The initial shape of all cases was same, although the bottom of the embankment has a little basal settlement, the outline shape of embankment can be seen as a trapezoid. And during the centrifugal loading process, the embankment model experienced base settlement due to the soft base's deformation, the deformation mainly happened on the bottom of the embankment, it can be observed that although the basal settlement happened and the bottom part of embankment have a large deformation, the outline shape of the embankment crest did not change a lot. Similarly, the supplement of liquid caused a slightly deformation of embankment, and again, the alteration in the outline shape of the embankment crest was small. However, the pivotal phase occurred with the application of dynamic loading, resulting in substantial damage to the embankment. It becomes important to assess the embankment's condition to unravel the relationships between soil arch, liquefaction, and the consequential deformation of the embankment. This evaluation is crucial for comprehending the nuanced dynamics at play during the experimental processes.

The top settlement, which was measured using a laser sensor attached to the top of the soil chamber, is illustrated in Fig. 6.8, the top settlement was normalized by the embankment height. To investigate the settlement caused by the dynamic load, the top settlement before and after dynamic load was used to calculate the normalized relative top settlement, as showed in Table 6.3. It's noteworthy that due to issues such as a leakage problem in Case 5 and sensor malfunctions in Case 8, only results from six cases are presented in this figure.

The outcomes have highlighted the strong correlation between soil nail reinforcement and top settlement. Initially, post-centrifugal top settlement did not exhibit a high correlation with reinforcement. The case with the highest top settlement was Case 4 (utilizing 6 soil nails toward the center), while the smallest top settlement occurred in Case 2 (also with 6 soil nails toward the center). However, when considering final settlements, Case 1, which lacked soil nail reinforcement, recorded the highest top settlement.

Further differences were found when inspecting Cases 6 and 7, where soil nails were not oriented toward the center of the embankment's bottom, resulting in large top settlements. Conversely, Cases 2 to 4, all utilizing 6 soil nails directed toward the center of the embankment's bottom, displayed varied top settlement dynamics. Case 4 exhibited significant top settlement, whereas Cases 2 and 3

experienced relatively modest top settlements. Interestingly, Case 3, incorporating white noise, showcased that this additional input dynamic load did not significantly influence top settlement dynamics.

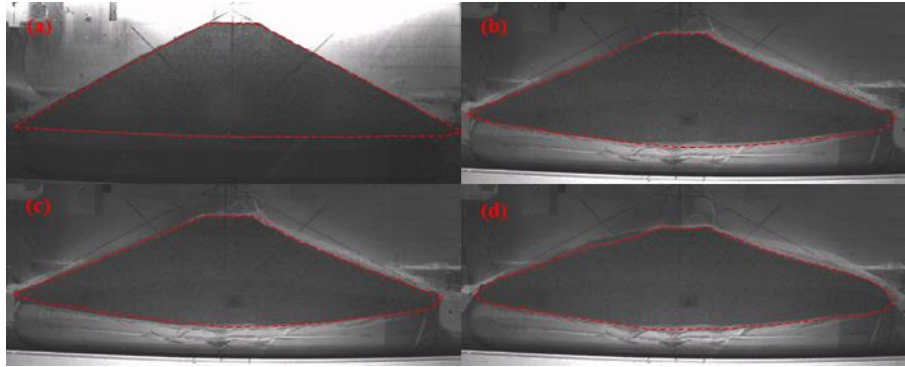


Fig. 6.7. Shape of embankment model(a) Initial shape; (b) After liquid supplement; (c) After white noise; (d) After dynamic load;

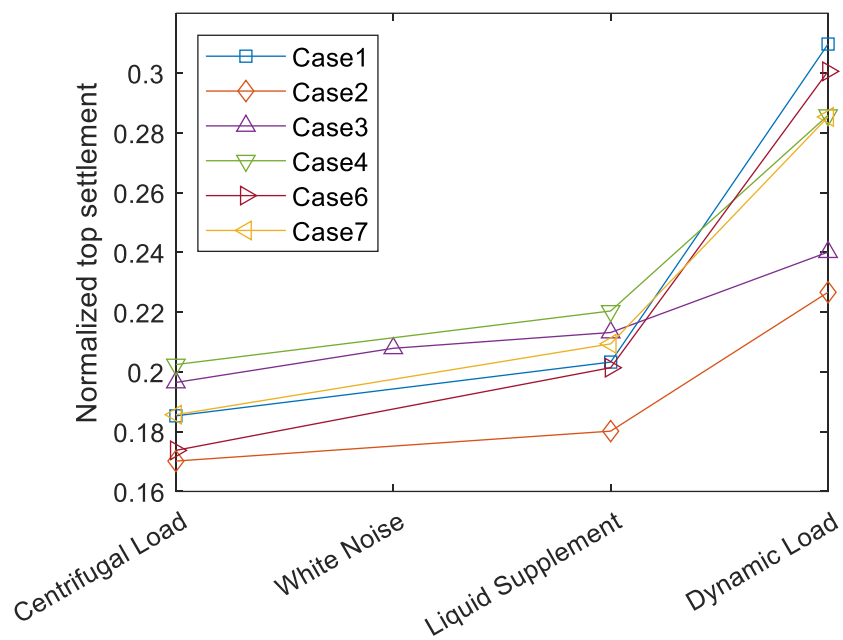


Fig. 6.8. Normalized top settlement of embankment

Table 6.3 Normalized relative top settlement after dynamic loading

Case No.	Normalized top settlement caused by dynamic load
1	0.106
2	0.047
3	0.027
4	0.066
6	0.100
7	0.076

From Table 6.3, a clear correlation between reinforcement method and settlement was showed. The highest relative top settlement after dynamic loading happened in case 1, which have no reinforcement. Then, the settlement in case 6 and case 7 was large too, which the soil nail was used but did not towards the center of embankment's bottom. The top settlement of case 2 to case 4 were small.

In some instances, a wooden stick was utilized to measure basal settlement. By combining basal settlement and top settlement data, we can assess changes in embankment height. Fig. 6.9 illustrates the variation in embankment height throughout the entire experiment. Notably, after the application of the centrifuge load, the embankment height exceeded its initial state. This increase may be attributed to a density decrease resulting from basal settlement at the bottom of the embankment.

In case 4, where soil nail reinforcement was directed toward the center of the embankment's bottom, the change in embankment height was minimal. This indicates that soil nail reinforcement in this configuration improves the deformation resistance of the embankment. On the other hand, Case 7 employed vertical reinforcement, yet the height of the embankment remained high. This suggests that vertical reinforcement alone may not significantly enhance deformation resistance. In Case 1, lacking soil nail reinforcement, the change in height after dynamic loading was the most significant. This underscores the substantial impact of the absence of soil nail reinforcement on embankment height dynamics.

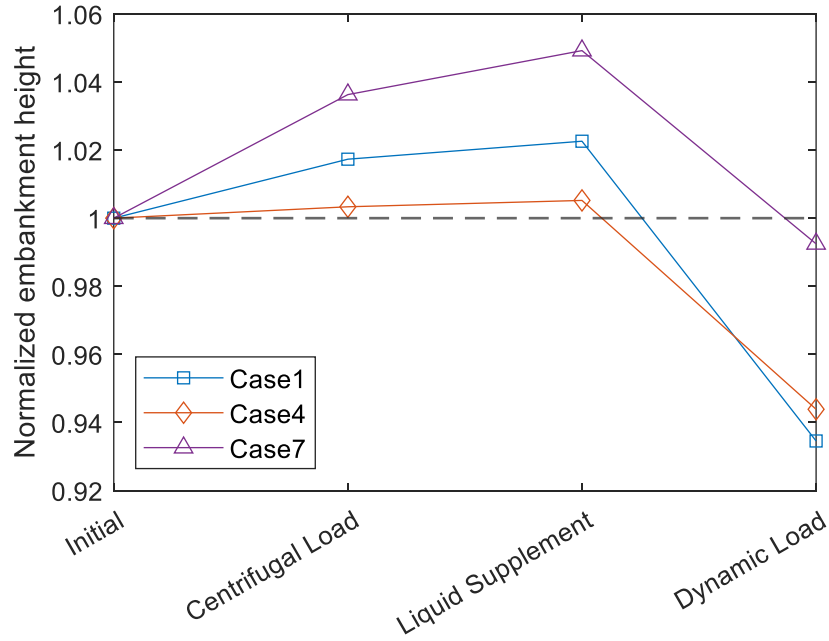


Fig. 6.9. Normalized embankment height

In Case 7, 6 soil nails were used vertically. In both Case 6 and Case 7, it can be observed that although the deformation of the embankment was smaller than the unreinforced case, the effectiveness of reinforcement was lower than in Case 2 to Case 4. Case 8, with vertical soil nail reinforcement and reduced number, experienced substantial deformation, similar to Case 1.

In summary, the settlement of the embankment in the static process is not influenced by soil nail reinforcement. However, when the dynamic load was applied, settlement in the reinforced case was small, and reinforcement also limited the change in embankment height. During dynamic loading, reinforcement significantly improved the deformation resistance of the embankment. The effectiveness of reinforcement was influenced by the method used. The most crucial factor is the direction of the soil nail, with soil nails towards the center of the embankment's bottom proving most effective. Additionally, a sufficient number of soil nails was important for optimal performance.

6.3.2. Vertical earth pressure distribution

Although the measurement instrument condition in each case was different, the number and installation location of TEP still same. The TEPs installed at the bottom of the embankment were used to obtain the vertical, horizontal and shear stress. In this section, the vertical earth pressure measured by TEP1, TEP3 and TEP4 can reflect the vertical earth pressure distribution on the bottom of the

embankment. Heatmap figure was used to express the vertical earth pressure distribution using three TEPs. Due to the restricted number of earth pressure sensors (only three) within the model, the linear interpolation method was used to estimate the vertical earth pressure between TEPs. By tracking transformations in vertical earth pressure, we were able to scrutinize the implications of basal settlement, saturation, and dynamic loads on soil arching.

6.3.2.1. Basal settlement and arch-action development

During the centrifugal loading process, the centrifugal acceleration will gradually be increased from 1G to 50G. The vertical earth pressure inside the embankment will increase due to centrifugal acceleration. On the other hand, deformation occurs in the soft Urethane base, leading to basal settlement. This settlement triggers lateral elongation deformation at the bottom of the embankment, resulting in a reduction in density at the central bottom and a decrease in lateral confinement pressure. Consequently, the vertical earth pressure on the bottom of the embankment is influenced, causing a decrease in vertical earth pressure at the center of the embankment.

To establish a baseline for comparison, a centrifuge test without soil nail reinforcement, designated as case 1, was conducted as a control case. During the centrifugal loading process of case 1, the basal settlement was measured using the wood stick attached on the top of the Urethane base, and the time history of centrifugal acceleration and basal settlement was showed in Fig. 6.10(a), in model scale. The y-axis in left side represent the acceleration, while the right side represent the basal settlement.

From the figure it can be seen that the centrifugal acceleration did not increase linearly; rather, the rate of increase in centrifugal acceleration became progressively faster during the loading process. Conversely, the rate of basal settlement increased rapidly initially, but as the acceleration reached higher levels (over 30G), it began to decrease. Notably, when the centrifugal acceleration reached 50G, it immediately ceased increasing; however, basal settlement continued to increase. This phenomenon, where consolidation does not occur in the Urethane base, may be attributed to the friction between the Urethane base and the soil chamber. Fig. 6.10(b) further depicts the relationship between centrifugal acceleration and basal settlement. Despite the absence of a linear correlation in this relationship, a discernible correlation is evident. Throughout the experiments conducted in this chapter, some cases incorporated basal settlement measurement sticks, while others did not. Therefore, in the ensuing discussion, centrifugal acceleration will be utilized to illustrate the vertical earth pressure distribution during the centrifugal loading process.

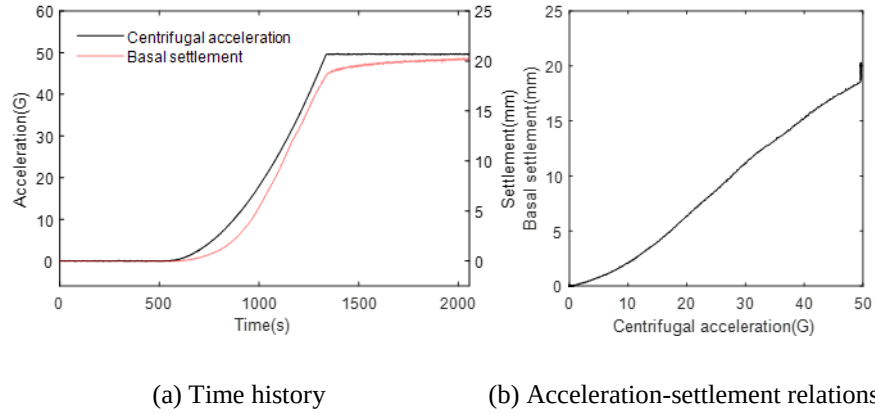


Fig. 6.10. The basal settlement during centrifugal loading process(a) Time histories of centrifugal acceleration and basal settlement; (b) Relationship between centrifugal acceleration and basal settlement

The vertical earth pressure at three locations on the embankment bottom is depicted in Fig. 6.11. The theoretical earth pressure, calculated based on soil density and sensor depth, is also presented and follows an upward trend with increasing centrifugal acceleration. Notably, the vertical earth pressure is higher on the two sides compared to the middle section. On the right side, the vertical earth pressure is slightly below the theoretical earth pressure, possibly due to the geometric shape of the embankment where the sensor depth on both sides is slightly less than theoretical. Conversely, the vertical earth pressure on the left side is lower than both the theoretical value and the right-side pressure. This discrepancy might be attributed to TEP rotation or non-uniform deformation. Despite these variations, both vertical earth pressures exhibit a robust correlation with increasing centrifugal acceleration.

Conversely, the vertical earth pressure in the middle of the embankment is notably lower, and its correlation with centrifugal acceleration is weak. In the initial period of centrifugal loading, as basal settlement commences with increasing acceleration, the vertical earth pressure in the middle of the embankment remains stagnant, even decreasing and becoming smaller than 0. This observation strongly suggests the occurrence of soil-arching action during the centrifugal loading process.

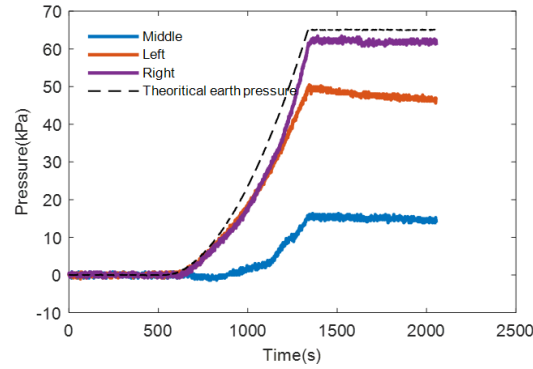


Fig. 6.11. Vertical earth pressure on the bottom of embankment

6.3.2.2. Occurrence and development of soil arch in static process

The centrifuge test is divided into three processes: centrifugal loading, liquid supplement, and dynamic loading. The soil-arching action evolves during centrifugal loading and liquid supplement, akin to static processes, forming the focus of this section. Fig. 6.12 presents a heatmap of vertical earth pressure distribution for all cases. The x-axis depicts changes in centrifuge acceleration, the y-axis corresponds to the geometric position, and the color scale represents vertical earth pressure values.

The influence of soil nail reinforcement on vertical earth pressure distribution is initially explored using Case 1 to Case 4. In case 1 to case 4, as the centrifugal acceleration increases, the vertical earth pressure on the two sides of the embankment is larger than at the center, indicating the occurrence of a soil arching effect. Case 1, without reinforcement, exhibits vertical earth pressure at the center, measuring 13.85 kPa, compared to 46.44 and 61.02 kPa on two sides. Due to experimental constraints, ensuring complete symmetry in stress distribution is challenging. Consequently, pressures in Case 2 are smaller but not perfectly symmetrical (11.12 kPa at the center, 31.70 and 45.45 kPa on the sides). Case 3 records values of 19.11 kPa at the center and 48.87 and 63.72 kPa on the sides. Case 4, with values of 4.99 kPa at the center and 51.57 and 52.99 kPa on the sides, shows no significant change in vertical earth pressure distribution.

Then, the cases with varies soil nail reinforcement method was used to discuss the vertical earth pressure distribution. In case 5, where only 4 soil nails were utilized, and they were inserted asymmetrically into the embankment, a notable observation emerged. Contrary to expectations, the vertical earth pressure distribution in Case 5 did not exhibit asymmetry. Instead, the vertical earth pressure at all three locations increased simultaneously. When the centrifugal acceleration reached

50G, the vertical earth pressure was 45.04 kPa at the center and 39.90 and 43.61 kPa on the sides, the vertical earth pressure on the bottom of the embankment were almost same, this suggests that soil-arching action did not occur. The fact that during the centrifugal loading process, soil-arching action typically arises with basal settlement. Asymmetric reinforcement in Case 5, where one side of the arch was reinforced and the other was not, changed the occurrence and development of the soil arch. This change may have impacted the location and rotation of the soil arch. Considering the geometry shape of embankment, if the soil arch developed and touch the surface of embankment, the development of soil arch will stop. However, detecting such phenomenon is challenging due to limited earth pressure sensors inside the embankment. Except case 5, other cases featured symmetric reinforcement, leading to the occurrence of soil-arching action. In Case 6 to Case 8, soil-arching action was evident, indicating that symmetrical reinforcement does not deter the occurrence of soil-arching action.

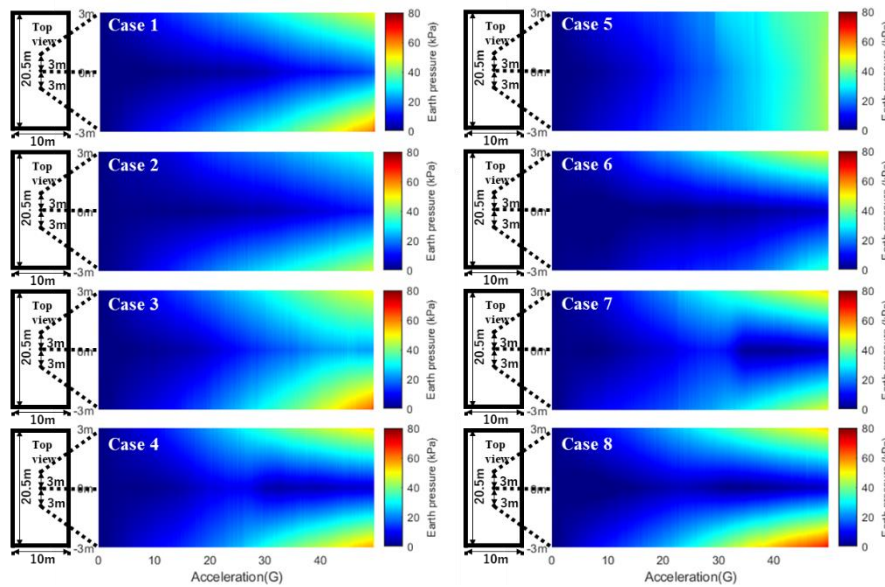


Fig. 6.12. Vertical earth pressure distribution in centrifugal loading process

In particular, Case 3 involved the application of white noise. Fig. 6.13 illustrates the Vertical earth pressure distribution before and after the application of white noise. Following the application of white noise, there was an increase in the vertical earth pressure at all three locations. The vertical earth pressure at the center rose to 20.78 kPa, while measuring 53.19 and 75.33 kPa on the lateral sides. However, it's noteworthy that the soil-arching action remained unchanged; the vertical earth pressure at the center remained significantly smaller than on the two sides.

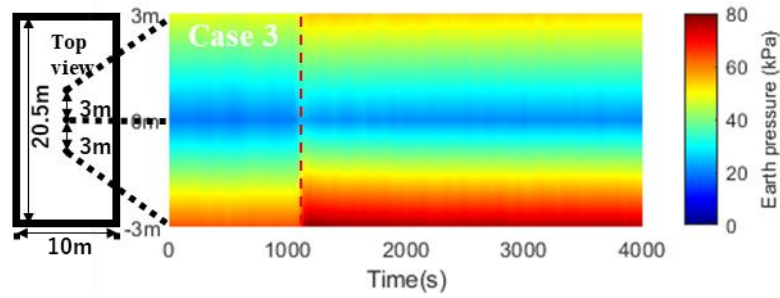


Fig. 6.13. Vertical earth pressure distribution in white noise process (Case 3)

Then, Metolose was introduced into the embankment as the pore liquid. Fig. 6.14 illustrates the variation in vertical earth pressure during the liquid supplement process, with the liquid supplement period's start and end point marked by dotted lines. Specially, in case 5, a sealing problem occurred on the vinyl sheet covered in Urethane base, as a result, during the liquid supplement process, Metolose leaked into the base but was not supplied to the embankment. Consequently, the data from Case 5 had to be excluded from the analysis.

The results of case 1 showed a tendency that after the liquid supplement started, the vertical earth pressure slightly rises at the sides. However, at the central location, an initial decrease in vertical earth pressure is evident as the liquid supplement begins. This occurrence can be attributed to an additional arch action, arising from the immediate load transfer from the saturated and weaker section to the unsaturated and stronger part upon liquid supply. Nevertheless, the vertical earth pressure at the middle spot gradually ascends due to the liquid's weight. Upon reaching the target volume of liquid supplement, the supplementation was stopped, and the vertical earth pressure at the center reached the peak peak at 32.13 kPa. Subsequently, after the liquid supplement, the vertical earth pressure at the center decreases, while the earth pressure on the two sides slightly increases. One possible explanation for this phenomenon is that during the liquid supplement process, in addition to the increase in earth pressure, the excess pore water pressure caused by liquid supplement affects the earth pressure sensor. After the supplementation ceases, the pressure induced by liquid supplement dissipates, leading to the recovery of earth pressure, and the load transfers to the base of the soil arch. Cases 2 to 4 utilized 6 soil nails to symmetrically reinforce the embankment load, but with the different experiment condition, and in comparison, with the unreinforced case, Cases 2 to 4 exhibited greater stability, with a smaller variation in earth pressure compared to Case 1. However, the reinforcement did not impact the soil arch in this process.

On the other hand, case 6 to case 8 used varies reinforcement method, in case 6 the soil nail was inserted towards the center of embankment's bottom, but the number of soil nail was reduced into 4. The results of Case 6 were particularly noteworthy; following the liquid supply into the embankment, the central vertical earth pressure increased significantly and eventually surpassed that of the two sides. Specifically, the central earth pressure reached 50 kPa, compared to 46.98 kPa and 27.27 kPa on the two sides. This suggests that the soil-arching action disappeared due to the liquid supplement. In case 7, the vertical earth pressure exhibited a similar trend to Case 1, with a temporary increase in central earth pressure. The peak central earth pressure reached 40 kPa, and after the liquid supplement, the vertical earth pressure measured 31.86 kPa at the center and 54.54 kPa and 58.86 kPa on the sides. In Case 8, where no soil nail was inserted through the center of the embankment's bottom, the vertical earth pressure increased in all three locations. After the liquid supplement, the vertical earth pressure measured 30.19 kPa at the center and 58.32 kPa and 81 kPa on the sides.

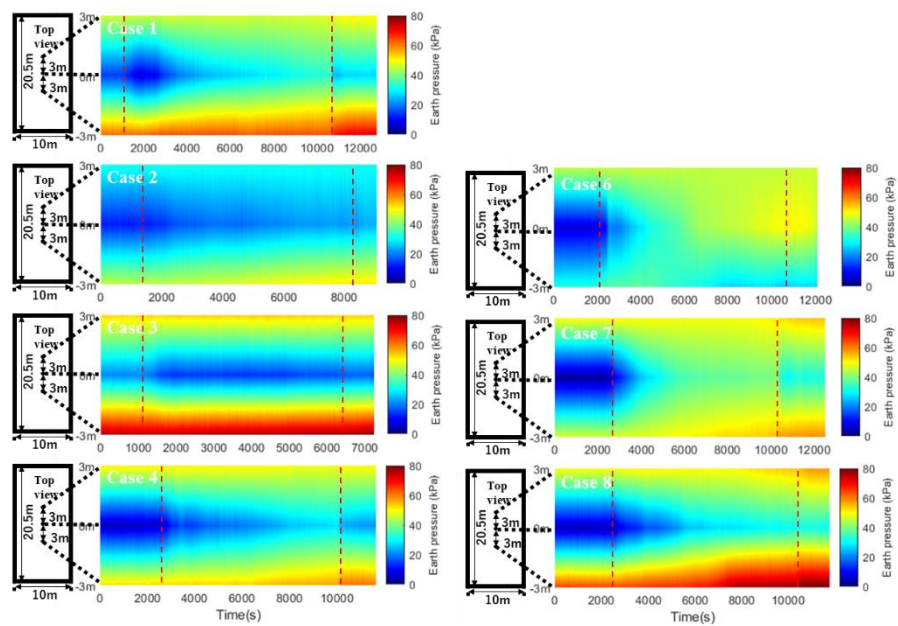


Fig. 6.14. Vertical earth pressure distribution in liquid supplement process

In conclusion, symmetric reinforcement does not affect the initiation and progression of soil-arching action. However, when soil nails are inserted asymmetrically, it disrupts the location and development of the soil arch, influencing the occurrence of soil-arching action. Conversely, if soil nails are inserted toward the center of the embankment's bottom and in sufficient numbers, as case 2 to case 4, the stability of the soil arch increases. Conversely, insufficient soil nails lead to a decrease in soil

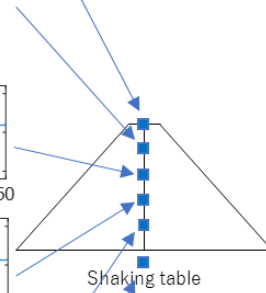
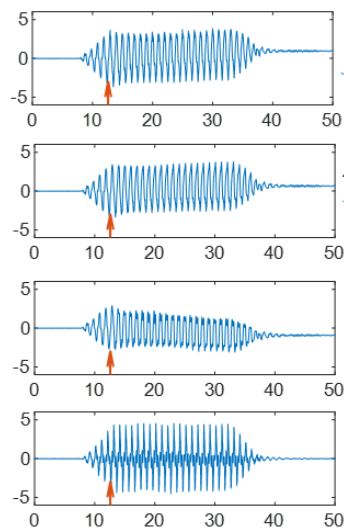
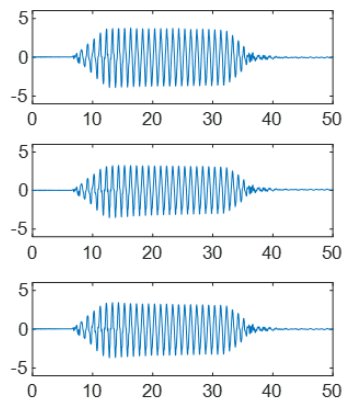
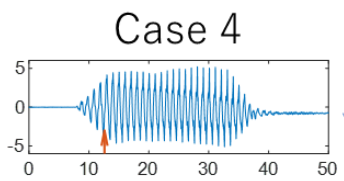
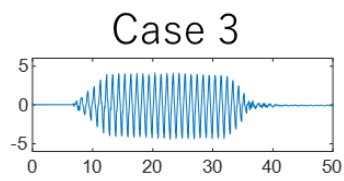
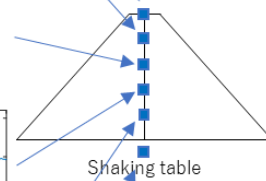
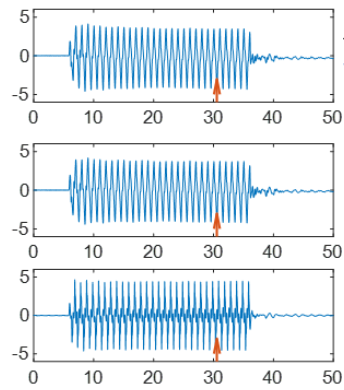
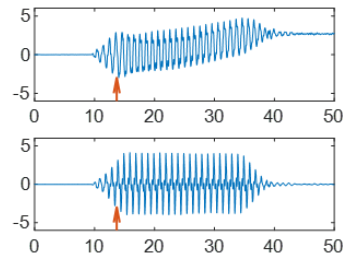
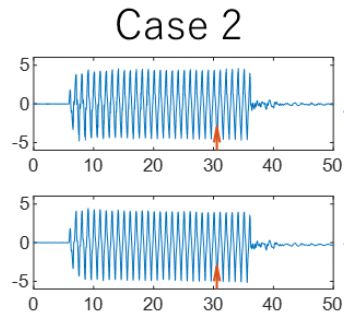
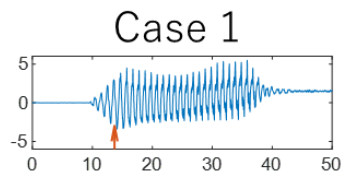
arch stability and may even result in the disappearance of the soil arch, as the results of case 6 to case 8.

6.3.3. Dynamic response

Fig. 6.15 shows the time history of response in different depth of embankment model in prototype scale, and the time that liquefaction was occurred was showed by red arrow. However, due to variations in measurement instrument conditions, some graphs exhibit gaps. Notably, an accelerometer on the shaking table was employed to measure the table's acceleration, revealing that despite the utilization of two different input waves, the amplitude of the shaking table's acceleration remained consistent at approximately 4 m/s^2 .

As the acceleration propagates through the embankment model, a significant attenuation is observed. In the bottom of embankment model, the ACC1 as showed in Fig. 6.5, the amplitude of acceleration was smaller than the input amplitude. However, the acceleration attenuation can reflect the liquefaction, in cases where liquefaction is delayed or does not occur, as seen in cases 2, 3, and 5, the attenuation is less pronounced. In contrast, cases with liquefaction events exhibit a substantially reduced amplitude of acceleration post-liquefaction occurrence. This rapid decay is attributed to the significant reduction in effective stress and shear stiffness within the soil after liquefaction transpires.

After the input acceleration propagated into embankment model, it is evident that the amplitude increases from the bottom to the top. ACC7, positioned at the top of the embankment, registers the highest acceleration. This variance is attributed to the geometric shape of the embankment, where the width narrows towards the top, leading to more pronounced shaking at the higher parts.



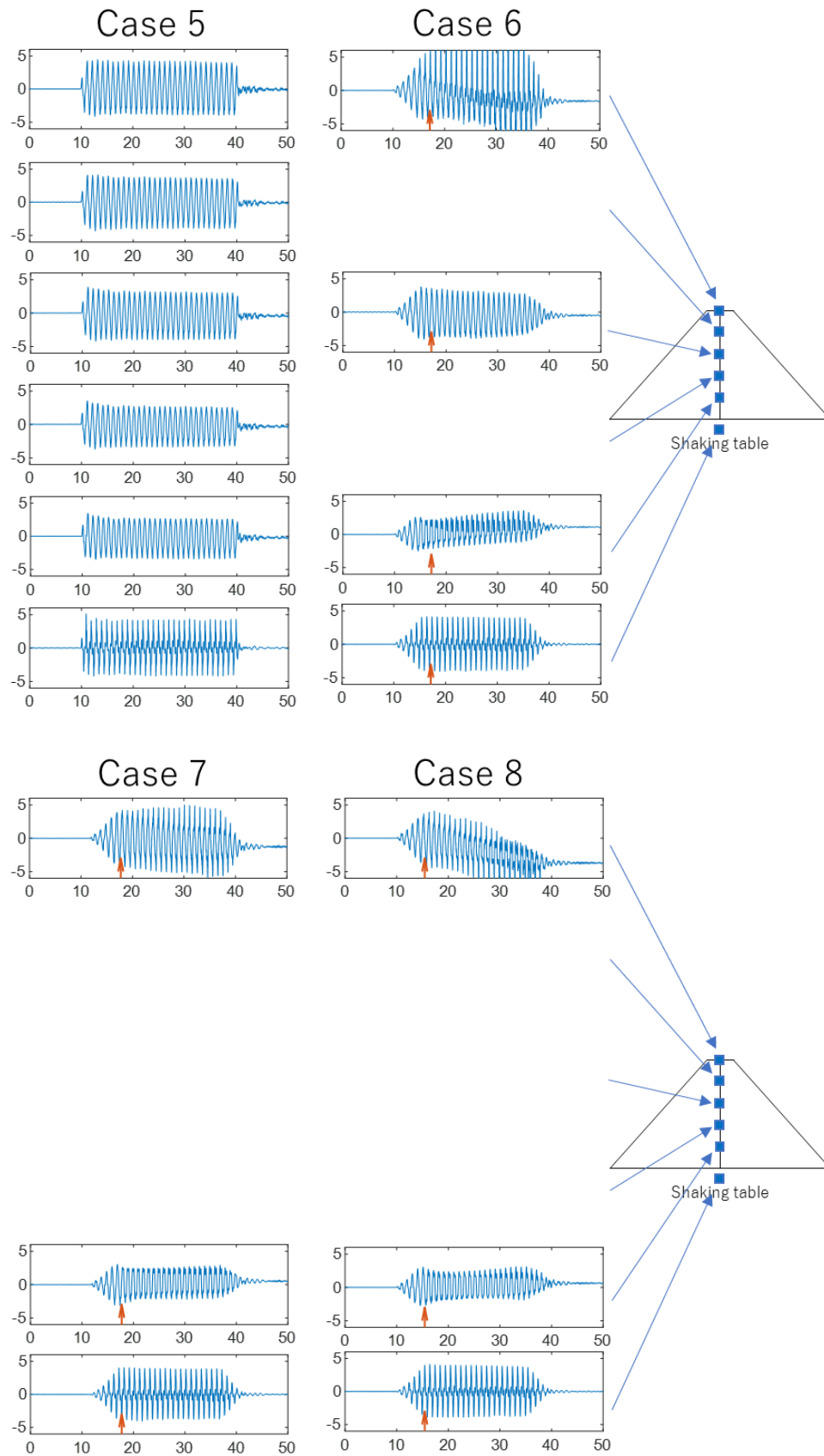


Fig. 6.15. Time history of response acceleration

On the other hand, after the liquefaction occurrence, another phenomenon was observed, that is the baseline drift of acceleration record. After the liquefaction occurrence, deformation have happened and that cause the rotation of accelerometer, so the baseline drift will happen. To solve this problem, in further study, the high-pass filter was used to correct baseline drift.

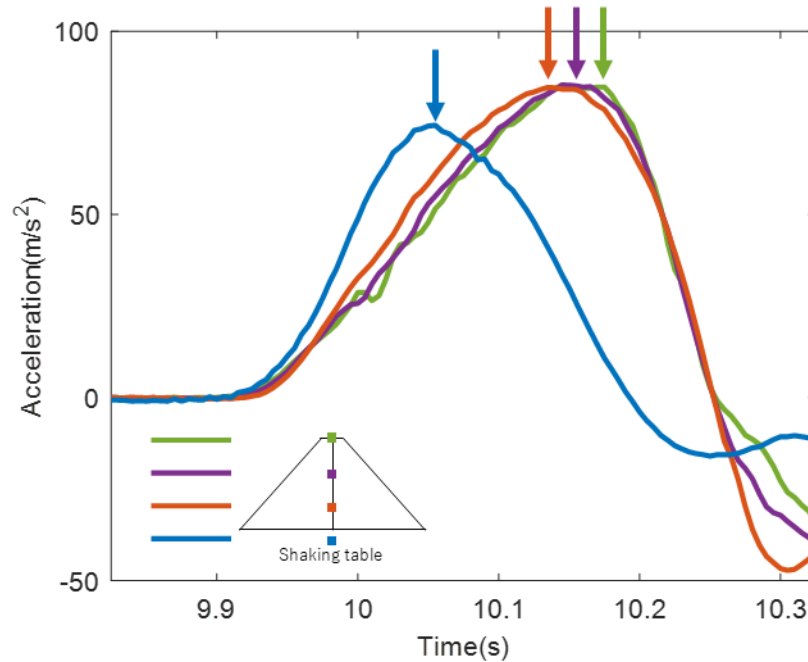


Fig. 6.16. First peak of acceleration in different depth

Fig. 6.16 shows the acceleration record of first peak, the acceleration of shaking table and inside the embankment model was showed, from the bottom to top of embankment, ACC in shaking table, ACC1, ACC2 and ACC7 were used. The time that acceleration reach peak showed the propagation of acceleration, the acceleration will happen in the shaking table first, and then propagated from bottom to top of the embankment. A larger delay was observed from shaking table to ACC1, it can be explained that although soil chamber was fixed with the shaking table, it's still needs time to let the input wave propagated through the soft base. These findings provide an insight of acceleration propagation, the difference can be observed though different cases, the acceleration will be used to make the further calculation, to investigate properties of embankment model under different reinforcement condition.

6.3.3.1. *Dynamic shear stress*

The dynamic shear stress was computed based on the response acceleration, as detailed in Section 3.3.1. In this study, at a specific depth, shear stress was determined by accumulating the shear stress

above that depth. Therefore, only the shear stress at the location of ACC1 was used to investigate shear stress properties. To mitigate the baseline drift problem, a high-pass filter was initially applied to remove baseline drift, ensuring that baseline drift did not occur in any of the cases. Fig. 6.17 illustrates the time history of shear stress in all cases, with red arrows indicating the time of liquefaction occurrence. First and foremost, attention should be given to the amplitude of shear stress. Similar to the response acceleration, cases without liquefaction occurrence exhibited larger shear stress. It can be observed that in cases 2, 3, and 5, the shear stress amplitude was more stable. In contrast, in other cases, after the occurrence of liquefaction, fluctuations in shear stress amplitude happened, possibly due to the reduction of shear stiffness within the soil after liquefaction occurrence. The results shows that the soil nail reinforcement will not impact the shear stress, but the occurrence of liquefaction have impact the dynamic stress behavior of embankment. The decreasing and baseline drift were observed after the liquefaction occurrence.

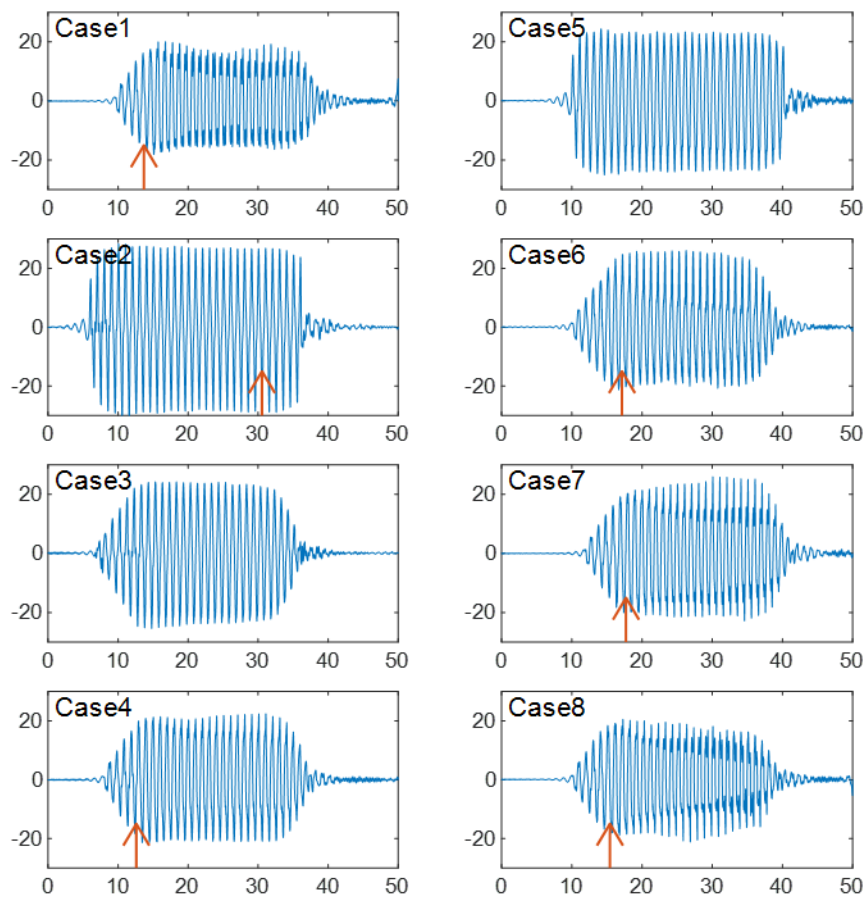


Fig. 6.17. Time history of shear stress at bottom of embankment

6.3.3.2. Max dynamic shear strain

As showed in Fig. 6.5, accelerometers were installed at various heights within the embankment. To analyze the dynamic shear strain characteristics across different layers of the embankment, the distribution of maximum shear strain along embankment depths is presented in Fig. 6.18. The number of shear strain values varied between cases due to differences in measurement instrument conditions. A high-pass filter was applied to remove baseline drift. Subsequently, the Fast Fourier Transform (FFT) method was employed, transforming acceleration into the frequency domain and double integrating in the frequency domain to avoid error accumulation. The obtained displacement was then used to compute shear strain using Eq. (3.4).

Firstly, the shear strain at the bottom part of the embankment was calculated using the acceleration data from the shaking table and ACC1. The input wave propagated from the shaking table to the soil chamber, then to the soft base, finally reaching the embankment. While the soil chamber can be considered fixed with the shaking table, it's not necessary to consider the relative displacement between shaking table and soil chamber. However, the deformation of the Urethane soft base cannot be ignored. Therefore, in the calculations, the thickness of the soft base was considered, and the bottom shear strain represents the shear strain of both soil and soft base. Notably, in case 2, the acceleration of the shaking table was not measured. For other cases, the bottom shear strain was stable, with the double amplitude strain changing from 4.2% to 5.2%. This suggests that with a constant input wave applied to the embankment model, the shear strain on the soft base and bottom of the embankment remained constant.

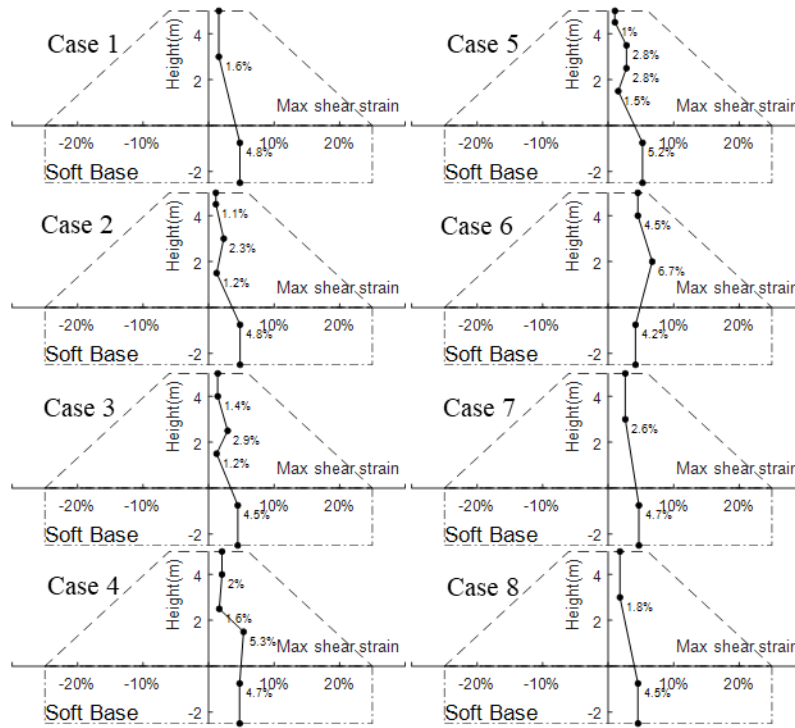


Fig. 6.18. Shear strain distribution in vertical direction

In cases 1, 7, and 8, only 2 accelerometers were used, with one installed 1m above the embankment bottom and another at the top of the embankment. Therefore, only the shear strain between these two ACCs could be obtained. The results indicate that the shear strain inside the embankment was smaller than at the bottom part. In other cases where more accelerometers were used, the distribution of shear strain along depths of the embankment could be observed. The results show that the maximum shear strain at different heights was not the same, indicating that the embankment did not behave like a rigid solid. Specifically, in cases 2, 3, and 5 where liquefaction did not occur, the max shear strain was small compared with the base strain, and the largest max shear strain appeared in the height range of 2m to 4m. On the other hand, cases 4 and 6 experienced liquefaction, and in these instances, a large strain was observed in the saturation zone, with a height ranging from 1m to 2m. Case 6 exhibited an extremely large shear strain.

6.3.4. Interaction between arching effect and liquefaction in terms of stability of embankments

As described in Section 5.4.3, arch collapse failure will happen due to dynamic load. In this chapter, soil nail was used to reinforce the embankment, protect the soil arch under dynamic load. The failure

mode of embankment can be divided in two types, as showed in Fig. 6.19. In this section, depends on the reinforcement method, both of these two deformation patterns were observed. Case 1 was already showed in Section 5.4.3 as an example of Arch collapse failure, then, case 2 to case 6 have showed a compression mode, prevented a large damage of embankment. In case 7 and case 8, due to insufficient reinforcement, the embankment shows Arch collapse failure mode again. These two deformation patterns were showed in this section.

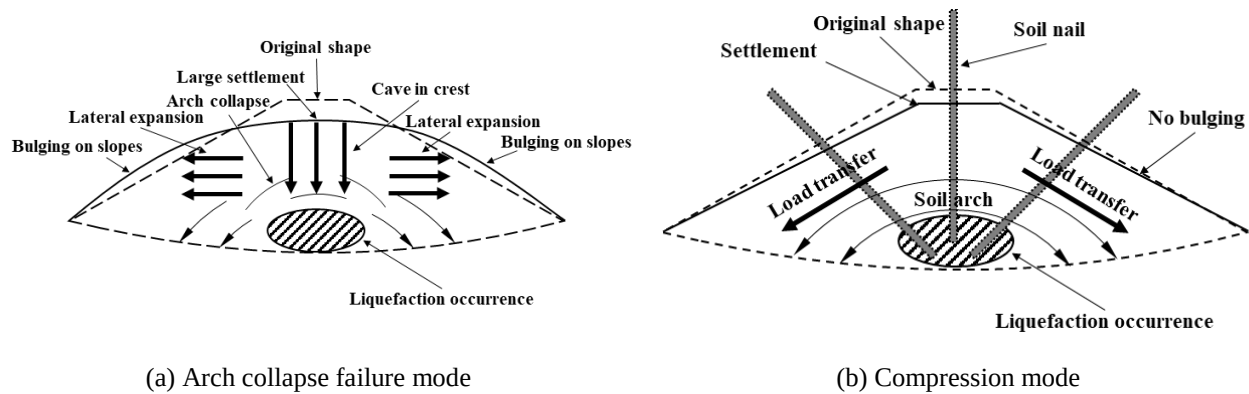
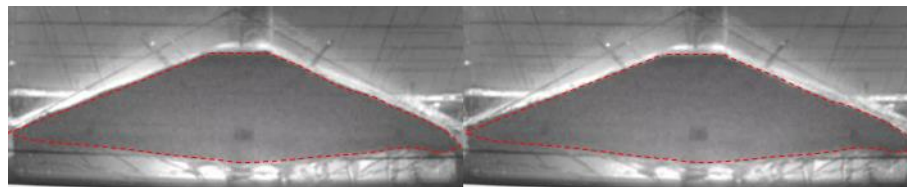
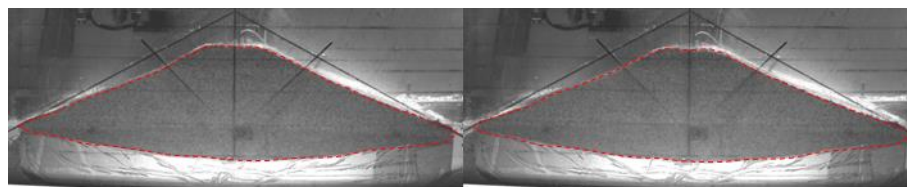


Fig. 6.19. Failure mode of embankment(a) Arch collapse failure; (b) Compression mode

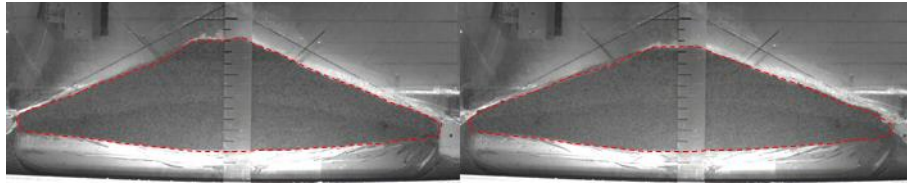
Various reinforcement conditions were used in this research, in case 2 to case 6, soil nails were inserted to the center of embankment bottom, and in case 7 and case 8, the soil nails were inserted vertically. The results of case 1 was already showed in Section 5.4.3, so firstly, the outline shape of embankment in case 2 to 6 was showed in Fig. 6.20. Case 5 was removed due to leakage problem.



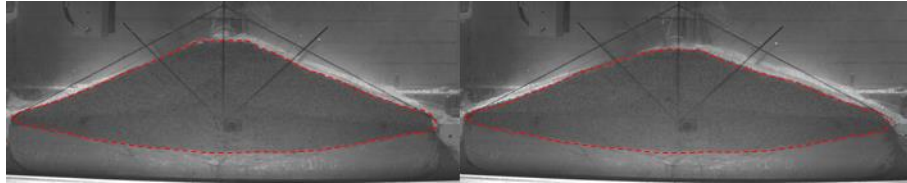
(a) Case 2



(b) Case 3



(c) Case 4



(d) Case 6

Fig. 6.20. Outline shape of embankment before and after dynamic load(a) Case 2; (b) Case 3; (c) Case 4; (d) Case 6

It can be observed from the picture that, in Case 2 to Case 6, although top settlement can be observed, the shape of the embankment did not collapse after the dynamic load, the deformation of embankment did not act like arch collapse failure. In order to exclude the possibility that the embankment damage is small because the liquefaction did not occur due to reinforcement, liquefaction evaluation was then conducted, as showed in Fig. 6.21.

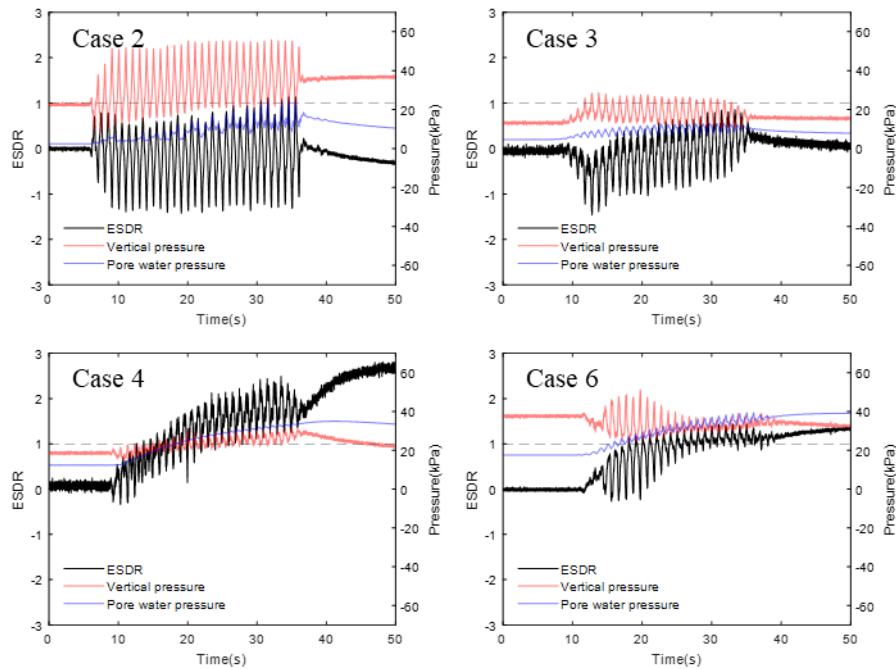


Fig. 6.21. Pressure condition and Effective Stress Decrease Ratio (ESDR)

Case 2 to case 4 have used same reinforcement method (6 nails towards center) and different test condition, and the results was showed in the figure. Case 2 employing a sine wave as the dynamic load instead of a ramped sine wave, exhibited a larger amplitude in vertical earth pressure fluctuation compared to Case 1. However, pore water pressure did not rise immediately, and ESDR experienced significant changes in the initial cycles but did not reach 1. With continued sine wave application, pore water pressure gradually increased, and ESDR surpassed 1 only in the later cycles. In case 3, white noise has been applied before the liquid supplement, which was believed that enhanced the strength of the soil arch. The change of vertical earth pressure after the dynamic load was small, and during the dynamic loading the amplitude of vertical earth pressure fluctuation was quite small, the increasing of ESDR was not significantly and never reached 1 until the end of dynamic loading. Case 4 used a same test condition with case 1 except soil nail reinforcement, and different with the expectation, the result was really similar with case 1. The excess pore water pressure increased immediately and ESDR have reach 1 at the first few cycles. The impact of various reinforcement methods on liquefaction was further explored through Case 6, this case has used the same input wave as Case 1 (ramped sine wave), exhibited similar results. Excess pore water pressure increased immediately, and ESDR reached 1 in the first few cycles. The liquefaction occurred in each case.

Fig. 6.22 presents the vertical earth pressure distribution throughout the entire experiment of case 2 to case 6. The theoretical earth pressure was calculated by the soil density and depth of the sensors(4m), the vertical earth pressure after each experiment process was marked by point. Specially, in case 5 the leakage problem has happened, so this case was removed.

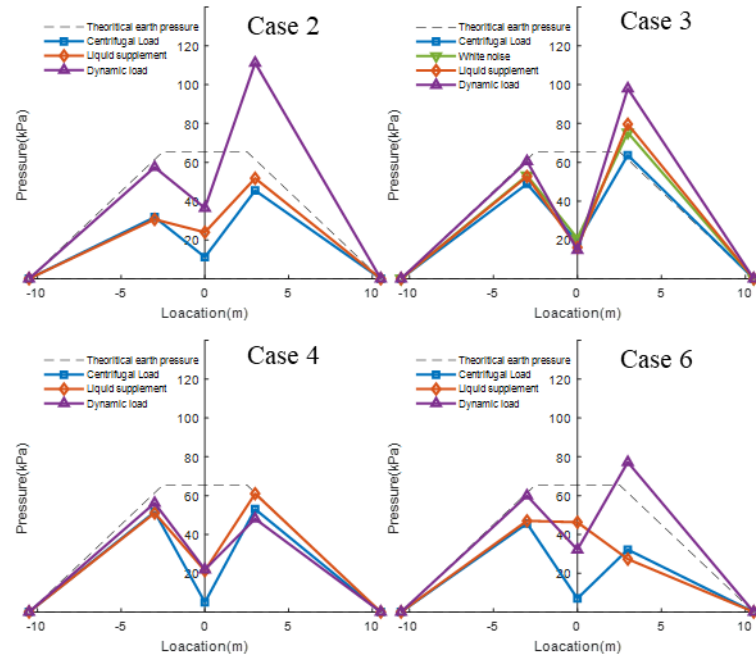


Fig. 6.22. Change of vertical earth pressure distribution of entire experiment

In Cases 2 to 4, six nails were strategically inserted into the embankment, all directed towards the center of the embankment's bottom. This method, involving a sufficient number of soil nails, appeared to be the most effective means of reinforcing the soil arch itself. In vertical direction, the interaction between soil and soil nails was facilitated by the friction between them. Despite the limited cross-section and number of soil nails, the soil arch did not happen in centrifugal loading process.

However, the application of dynamic loading yielded different results for soil nail-reinforced embankments compared to unreinforced ones. In Case 2, the vertical earth pressure before dynamic loading was 23.91 kPa in the center, 30.56 and 51.77 kPa on the sides. After dynamic loading, these values increased significantly to 36.42 kPa in the center, 57.45 and 111.254 kPa on the sides. The vertical earth pressure on all three locations experienced a substantial increase, with the right side showing an extremely high increase, even surpassing the theoretical vertical earth pressure. In Case 3, before dynamic loading, the vertical earth pressure was 16.07 kPa in the center, 52.65 and 79.65 kPa on the sides. Post dynamic loading, it became 14.68 kPa in the center, 60.48 and 98.01 kPa on the sides. While the center's vertical earth pressure slightly decreased, the sides witnessed a significant increase. Similarly, in Case 4, before dynamic loading, the vertical earth pressure was 21.33 kPa in the center, 51.03 and 61.00 kPa on the sides. After dynamic loading, it became 21.88 kPa in the center, 56.16 and 48.02 kPa on the sides. Here, the vertical earth pressure on the center slightly decreased, but

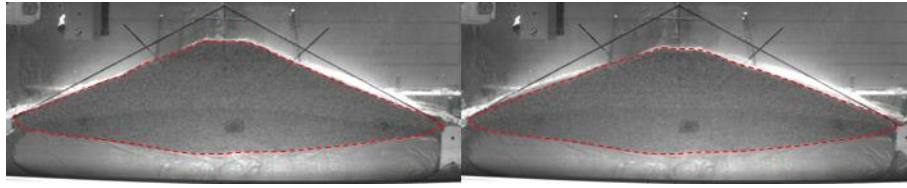
not significantly, whereas the sides showed a significant increase. In this case, the vertical earth pressure distribution remained almost unchanged after dynamic loading.

Compare with case 3 and case 4, case 2 exhibited a larger increase in vertical earth pressure, possibly due to the different input wave. Cases 3 and 4 both utilized a ramped sine wave, gradually increasing and decreasing in amplitude over the initial and final cycles, respectively. In contrast, Case 2 used a sine wave, with the same amplitude but higher energy. In Case 3, the vertical earth pressure on the sides increased, but the center's vertical earth pressure remained largely unchanged. This might be attributed to the use of white noise, enhancing the strength of the soil arch. Thus, the center's vertical earth pressure remained stable. While there were some differences in the changes of vertical earth pressure distribution among Cases 2 to 4, a consistent observation was the persistence of the soil arch after the application of dynamic loading, substantiating the reinforcing effect of soil nails.

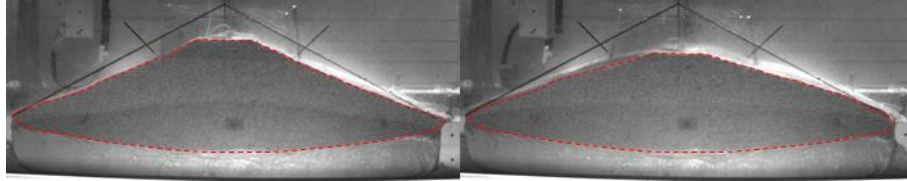
In case 6, soil nails were inserted towards center of embankment's bottom, but the number was reduced into 4. From the figure it can be observed that in case 6, before the liquid supplement, the vertical earth pressure was 6.92 kPa in center, 45.63 kPa and 32.13 kPa on sides, a clear soil arch action was occurred. But after the liquid supplement, the vertical earth pressure changed to 46.26 kPa on center, 46.98 and 27.27 kPa on sides suggesting the potential disappearance of the soil arch due to liquid supplementation. However, after dynamic loading, the vertical earth pressure on the sides increased while the center decreased, resulting in 32.13 kPa at the center, 59.94 kPa, and 77.22 kPa on the sides, reaffirming the presence of the soil arch. Two assumptions are proposed to explain this phenomenon: firstly, liquid supplementation has a temporary impact, affecting earth pressure only by flow rate or temporary liquid accumulation, and secondly, the soil arch, once disappeared due to liquid supplementation, reoccurs during dynamic loading.

Although in some case the center earth pressure increased after dynamic loading, the earth pressure on two sides increased too, a clear soil arch exists even after the dynamic loading. In these cases, the reinforcement has prevented the collapse of soil arch under liquefaction occurrence, and have prevented the arch collapse failure.

Then, the outline shape of embankment in case 2 to 6 was showed in Fig. 6.23. In case 7 and case 8, with vertical soil nail reinforcement, the embankment experienced substantial deformation, similar as the arch collapse failure showed in Section 5.4.3.



(a) Case 7



(b) Case 8

Fig. 6.23. Outline shape of embankment before and after dynamic load(a) Case 7; (b) Case 8

The impact of vertical reinforcement methods on liquefaction was further explored through Case 7 and Case 8. Despite different reinforcement methods, all using the same input wave as Case 1 (ramped sine wave), exhibited similar results. Excess pore water pressure increased immediately, and ESDR reached 1 in the first few cycles, as showed in Fig. 6.24.

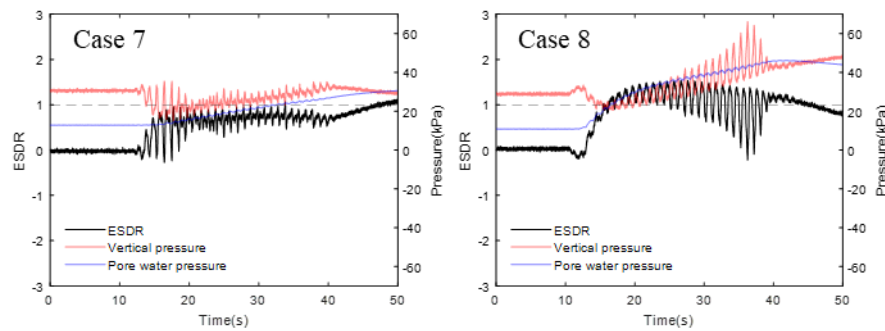


Fig. 6.24. Pressure condition and Effective Stress Decrease Ratio (ESDR)

The vertical earth pressure distribution of entire experiment of case 7 and case 8 was illustrated by Fig. 6.25. In case 7, with 6 soil nails inserted vertically into the embankment, the soil arch action still occurred in the static process. Before the dynamic load the vertical earth pressure stood at 31.86 kPa on center, 54.54 and 58.86 kPa on sides. However, after dynamic load, left side earth pressure have decreased a lot, the vertical earth pressure became 27.98 kPa on center, 35.37 and 59.67 kPa on sides. However, post dynamic loading, the left side's earth pressure decreased significantly, resulting in 27.98 kPa at the center, 35.37 kPa, and 59.67 kPa on the sides. Although the vertical earth pressure in the

center remained less than that on the sides, the soil arch was affected, suggesting a potential shift in location or direction.

In case 8, employing 4 soil nails without any inserted at the center, the vertical earth pressure before dynamic loading was 30.19 kPa at the center, 58.32 kPa, and 81.00 kPa on the sides. After dynamic loading, the vertical earth pressure changed to 47.37 kPa at the center, 49.41 kPa, and 75.33 kPa on the sides. Notably, the earth pressure on the sides decreased while the center increased, indicative of soil arch collapse.

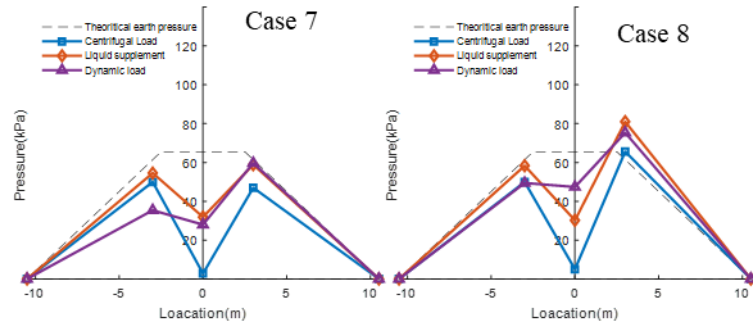


Fig. 6.25. Change of vertical earth pressure distribution of entire experiment

6.3.5. Analysis of Soil Arch Collapse and Embankment Deformation Under Dynamic Loading

To evaluate the relationship between soil arch collapse and embankment damage, we summarized the results from all cases used in Chapters 6 and 7.

Firstly, the emerging and vanishing of arch action are qualitatively evaluated using the vertical earth pressure ratio, r_{VEP} , calculated as follows:

$$r_{VEP} = \frac{2 \cdot EP_{middle}}{EP_{left} + EP_{right}} \quad (6.1)$$

Where EP_{middle} represent the vertical earth pressure in middle of embankment, and EP_{left} and EP_{right} represent the vertical earth pressure on the left and right sides, respectively. The value and change of r_{VEP} can reflect the load transmission due to soil arching. When no arching like overburden pressure, r_{VEP} is 1; however, if the load is transferred horizontally, r_{VEP} is less than 1.

Next, to assess embankment damage more precisely, we employed image analysis to compare the embankment's outline shape before and after dynamic loading. Fig. 6.26 illustrates two typical failure modes using cases DB-1-8-N-2 (unreinforced) and DB-1-8-R6-3 (reinforced). illustrates two typical failure modes using cases DB-1-8-N-2 (unreinforced) and DB-1-8-R6-3 (reinforced). Before dynamic loading, both embankments maintained a trapezoidal shape. However, after dynamic loads, the

unreinforced embankment (DB-1-8-N-2) showed significant crest settlement and bulging, resulting in large deformation and loss of its trapezoidal shape, indicating severe damage. In contrast, the reinforced embankment (DB-1-8-R6-3) exhibited only minor shape changes, suggesting reduced damage, which highlights the effectiveness of soil nail reinforcement in maintaining embankment stability during dynamic loading.

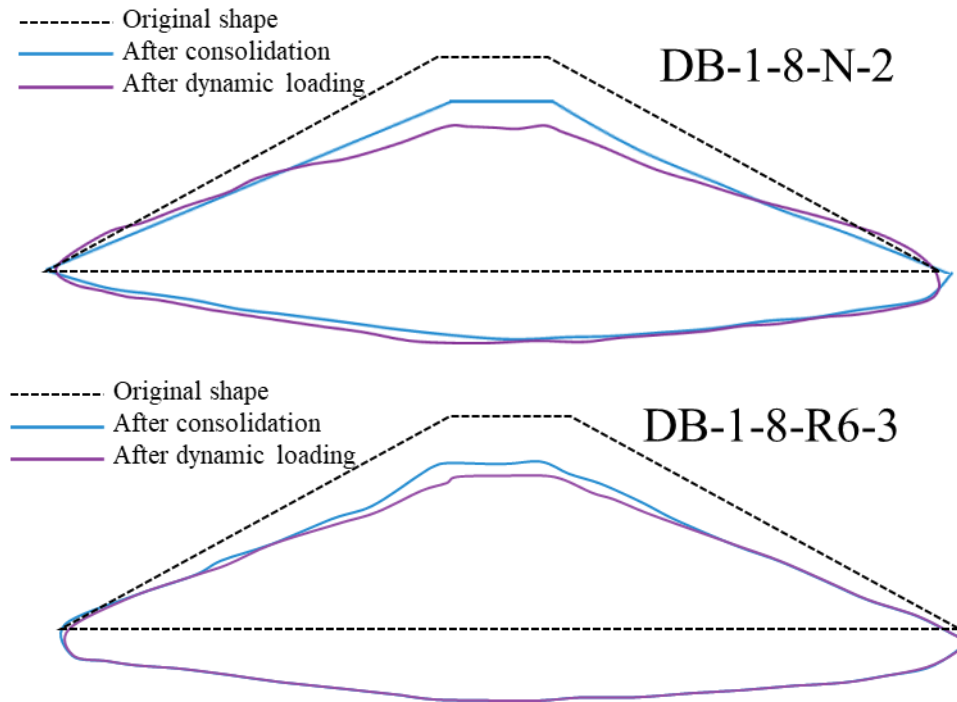


Fig. 6.26. Deformation mode of embankment

The crest settlement and the embankment height were already analysed in earlier section. However, only one laser sensor was used to detect the crest settlement of embankment, it is hard to describe the damage of embankment by only one sensor. To numerically evaluate the damage, we introduced the intersection area difference, as marked in red in Fig. 6.27. This area represents the accumulated vertical displacement of the embankment surface and was normalized by the embankment's width and height.

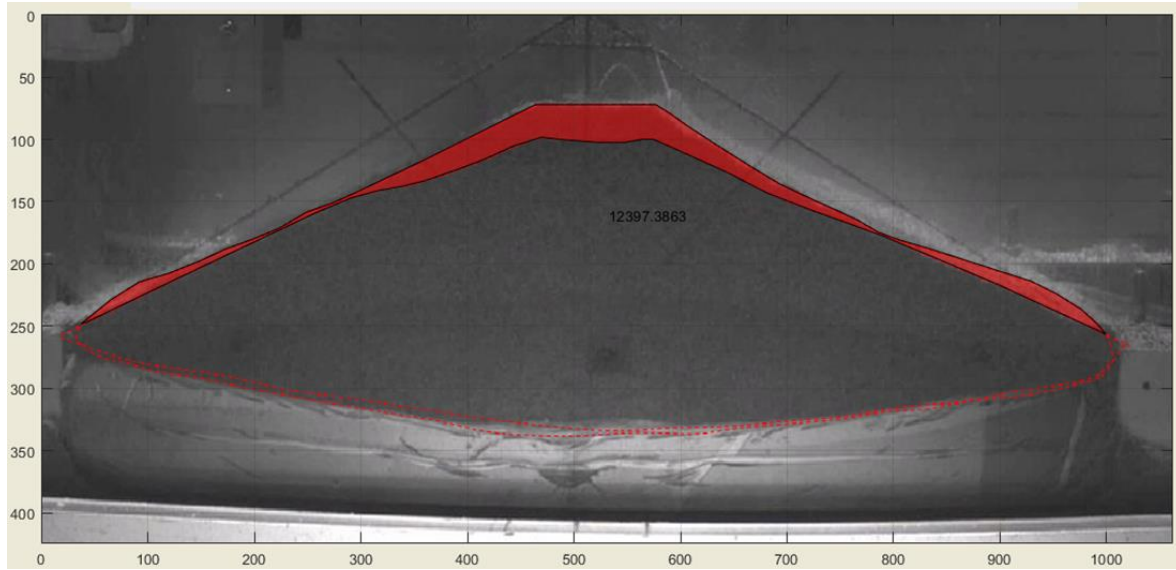


Fig. 6.27. Area difference before and after dynamic loads

For comparison, some cases appeared in Fig. 6.4 were chosen and renumbered. The experimental results are summarized in Table 6.4, including embankment height reduction cases (Nos. 8 and 9) and soil nail reinforcement cases (Nos. 2 to 7). Case DB-1-8-R4-1 was excluded due to leakage of Metolose during the experiment.

In summary, embankment height reduction or soil nail reinforcement exhibited minimal influence on liquefaction occurrence. Excess pore water pressure increased rapidly, leading to liquefaction within the first few cycles. However, test condition changes, such as shifting from a ramped sine wave to a sine wave, significantly influenced liquefaction occurrence by increasing the amplitude of vertical earth pressure fluctuations and delaying liquefaction. Additionally, the application of white noise enhanced the liquefaction resistance of the embankment.

Soil nail reinforcement did not impact the vertical earth pressure distribution during the static process, with the soil arch action occurring irrespective of soil nail usage. However, due to complex stress conditions within the embankment, the r_{VEP} values varied significantly across different cases. In Nos. 1 through 7, r_{VEP} remained below 0.6, indicating the presence of soil arch action. In contrast, Nos. 8 and 9 exhibited higher r_{VEP} values before shaking (0.983 and 0.753, respectively), suggesting that soil arching did not occur.

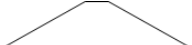
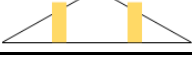
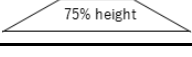
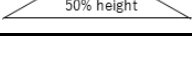
During the dynamic process, the existence of the soil arch was influenced by soil nail reinforcement. In Nos. 2 through 6, the r_{VEP} values changed but remained below 0.6, indicating the continued presence of the soil arch even after dynamic loading. The effectiveness of soil nail reinforcement was significantly influenced by the method of insertion, particularly the direction toward the center of the

embankment's base. Moreover, an optimal number of soil nails was important, as increased soil nail use correlated with enhanced reinforcement effectiveness. The shape changes in these cases were relatively low.

In contrast, in Nos. 1 and 7, r_{VEP} significantly increased after shaking, and the normalized deformation area was much higher. For Nos. 8 and 9, r_{VEP} values were already high before shaking, indicating that soil arching did not occur. However, these values slightly decreased after shaking, possibly due to embankment deformation leading to a reduction in thickness. The normalized deformation area for these cases was extremely high, indicating severe embankment damage.

The results of No. 1 are taken as the baseline, with r_{VEP} before and after shaking as well as a normalized deformation area. For the 3-nailing scheme, the relative reinforcement performance for Nos. 2-4, using radial alignment toward the center, or technically along the minor principal stress, was noticeably better than No. 6, which used vertical alignment. Even with the 2-nailing scheme, the same tendency was observed when comparing No.5 with No. 7 because r_{VEP} after shaking and the normalized deformation area remained lower than that of the baseline. Therefore, the centrifugal models validated the use of soil nails to enhance soil arch strength by increasing confining pressure, aiming to prevent collapse during liquefaction-induced loading as described in Fig. 6.2.

Table 6.4 Summary of centrifuge experiment

No.	Test ID	Experiment condition	Liquefaction	Vertical earth pressure ratio		Normalized Deformation Area
				Before shaking	After shaking	
1	DB-1-8-N-1		○	0.426	0.757	0.052
2	DB-1-8-R6-1		○	0.559	0.431	0.006
3	DB-1-8-R6-2		○	0.193	0.185	0.010
4	DB-1-8-R6-3		○	0.345	0.420	0.026
5	DB-1-8-R4-2		○	0.192	0.468	0.029
6	DB-1-8-R6-4		○	0.537	0.588	0.029
7	DB-1-8-R4-3		○	0.425	0.759	0.050
8	DB-0.75-8-N		○	0.983	0.666	0.048
9	DB-0.5-8-N		○	0.753	0.658	0.103

6.4. Summary

This chapter focused on the utilization of soil nails as reinforcement in dynamic centrifuge tests under various conditions. A detailed investigation revealed that soil nail reinforcement does not impact the formation of soil arching or the occurrence of liquefaction. This is the main conclusion of this chapter. However, after the liquefaction happened, the soil nail can reinforce the soil arch. The deformation of embankment in centrifugal loading process and liquid supplement process is not influenced by soil nail reinforcement. However, during dynamic loading, reinforcement significantly improved the deformation resistance of the embankment. The effectiveness of reinforcement was influenced by the method used. The direction of the soil nail and a sufficient number of soil nails was important for optimal performance.

Chapter 7. Energy-based liquefaction evaluation

7.1. Introduction

The assessment of liquefaction resistance in geotechnical engineering often hinges on the cyclic resistance ratio (CRR) derived from laboratory tests employing sine wave motion (Townsend, 1978). However, seismic waves in actual earthquakes deviate from such harmonic motions, prompting the need for further research to refine the stress-based method (SBM) for liquefaction resistance evaluation. Additionally, various liquefaction evaluation criteria, such as excess pore water pressure ratio, pore water pressure ratio, and effective stress decreasing ratio, have been utilized (Lu et al., 2008), underscoring the complexity of the liquefaction phenomenon.

In contrast to SBM, the energy-based method (EBM) has emerged as a promising alternative, grounded in the assumption of seismic energy dissipation per unit volume of soil (Kokusho & Kaneko, 2018; Kokusho & Mimori, 2015; Kokusho, 2017, 2021; Kokusho & Tanimoto, 2021). Kokusho (2018) emphasized the importance of cumulative dissipated energy ($\Sigma\Delta W$) based on shear work in elucidating liquefaction behaviors, despite the constraints of cyclic triaxial tests. The number of cycles for the initial liquefaction based on energy method were agreed well with those based on stress method through results obtained from a series of undrained cyclic triaxial tests. (Pipatpongsa, 2022) Moreover, for the accurate delineation of cycle counts in the energy-based method, the cycles separation technique assumes paramount significance.

This study delves into the assessment of liquefaction resistance using undrained cyclic triaxial tests, with a focus on energy dissipation per unit volume. By employing two distinct methods—namely, the angular method and the derivative zero-crossing method—we aim to elucidate the intricacies of liquefaction resistance. Through a series of experiments conducted on reconstituted sand, we analyze the characteristics of pore water pressure buildup, double amplitude axial strain, and stress ratio across varying densities and cyclic stress ratios (CSR). Our comprehensive analysis seeks to enrich our understanding of liquefaction resistance and contribute to the ongoing refinement of design concepts in geotechnical engineering.

7.2. Undrain triaxial test conditions

7.2.1. Apparatus

For this experiment, the triaxial compression testing apparatus installed at Kyoto University's Katsura Campus was utilized. The performance overview of the triaxial compression testing apparatus is shown in Table 7.1.

Table 7.1 Summary of Triaxial Testing apparatus Performance

Load Capacity	0.98 kN, 4.9 kN
Load Frequency	0.001~10 Hz
Pressure Capacity	(Cell Pressure) 490 Pa (Back Pressure) 490 Pa
Specimen Dimensions	Diameter 50 mm, Height 100 mm
Volume Measurement Method	Burette (25 cm ³)
Displacement Measurement Method	Axial Displacement Gauge (50 mm) Gap Sensor (2 mm)
Test Conditions	Monotonic Loading, Cyclic Loading
Loading Method	(Monotonic Loading) Motor (Cyclic Loading) Pneumatic Servo

The load capacity during monotonic loading is 0.98 kN, while for cyclic loading facilitated by pneumatic servo, it stands at 4.9 kN. Load frequencies ranging from 0.01 to 10 Hz are attainable, with pressure endurance reaching 490 Pa for both cell and back pressures. Specimens with dimensions of 50 mm in diameter and 100 mm in height are accommodated, and volume measurement is achieved through employment of a burette. Displacement is quantified utilizing axial displacement gauges alongside gap sensors. Experimental conditions encompass both monotonic and cyclic loading regimes, the former driven by a motor and the latter by a pneumatic servo.

The cyclic undrained triaxial testing apparatus is fundamentally similar to conventional triaxial testing equipment but is designed to apply repeated axial loading using hydraulic or pneumatic systems. Since the amplitude and frequency of actual seismic waves are irregular, replicating them accurately in laboratory tests is challenging. Therefore, specimens, which are isotopically consolidated, are subjected to cyclic axial loading with a constant cell pressure under undrained conditions. This loading

involves the repetitive application of compressive and tensile axial loads with a constant sinusoidal waveform and amplitude. The cyclic axial deviator stress (σ_d) is calculated by dividing the average of these loads by the cross-sectional area of the specimen. During testing, axial strain (ϵ_a) and excess pore water pressure (Δu) are measured.

7.2.2. Materials

The soil used for specimen was sieved Hiroshima sand, as showed in Chapter 2.

7.2.3. Specimen Preparation

When employing traditional specimen preparation methods and compacting Hiroshima sand into the summit mold, void typically form between soil particles, limiting saturation to a maximum of approximately 85%. So, in this experiment, a saturation enhancement technique was implemented. It involved replacing the air within the void with water using a vacuum chamber. Additionally, the specimen is frozen in a freezer for one day so that the specimen can keep its shape after the removal of the summit mold in the undrained cyclic triaxial test, then, after the specimen was installed into triaxial machine, it will be left for half a day to let the frozen specimen melt. This approach successfully increased saturation without significantly altering the structure of the soil particles. The saturation level of the specimens prepared in this experiment averaged over 95%. The specimen was prepared using the following standardized procedure:

- (1). Adjust the water content of the Hiroshima sand to 8%, consistent with centrifuge model experiments.
- (2). Measure the weight of the summit mold and proceed to fill it with Hiroshima sand, ensuring consistent density by compacting layers of 1 cm thickness. The appearance of the summit is depicted in Figure 2.4.
- (3). After compacting all 10 layers, re-measure the weight. Consistent monitoring of the summit's weight and the combined weight of the summit and soil in each experiment allows for error detection, particularly when comparing soil weights across experiments with the same Dr.
- (4). Cover the top of the sample with filter paper and a porous stone, sealing it with vinyl tape pierced with ventilation holes (see Fig. 7.1(A)).
- (5). Place the sample in a desiccator filled with distilled water (see Fig. 7.1(B)). Perform successive cycles of deaeration by air suction and exposure to atmospheric pressure for approximately 5 minutes each, repeating until the mass of the summit mold containing the specimen stabilizes.

(6). Transfer the summit to a refrigerator and allow it to freeze for a minimum of one day.

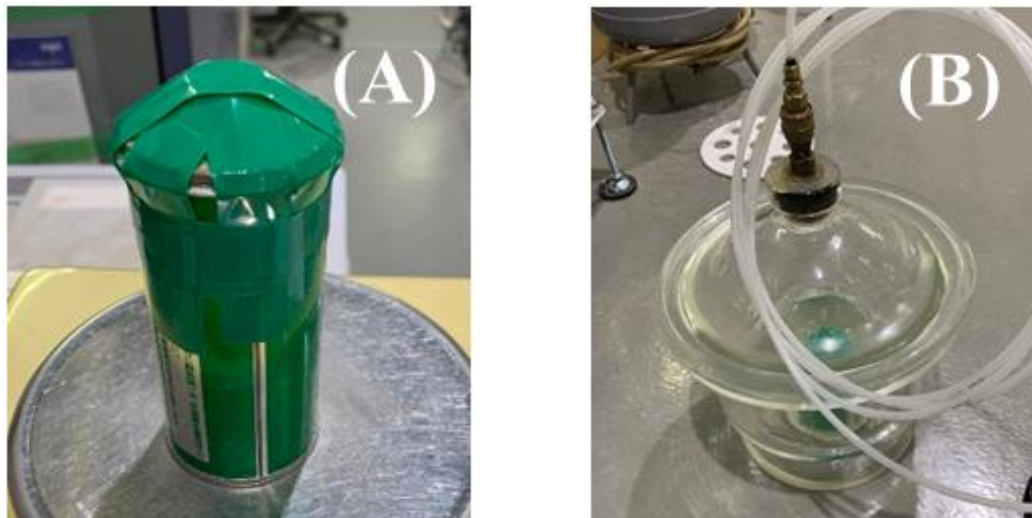


Fig. 7.1. Specimen of undrain triaxial test (A) Frozen sample (B) Desiccator

7.2.4. Experimental Procedure

After install the specimen into the triaxial machine, the triaxial test will be conducted through the following procedure:

- (1). Gradually increase both vacuum pressure and cell pressure while maintaining a 10 kPa difference between them. By doing so, evacuate air while maintaining a confinement pressure of 10 kPa.
- (2). Allow the specimen to keep under a confinement pressure of 10 kPa for half a day to one day.
- (3). Measure the compaction of degree, as well as the saturation degree.
- (4). Apply cyclic loading under undrained conditions.
- (5). Stop loading when double amplitude axial strain reaches 10%. In this experiment, loading continues until the specimen undergoes a vertical displacement of 2 cm.

7.2.5. Experimental cases condition

Nine tests were conducted to investigate liquefaction resistance using specimens prepared at three different relative densities (D_r) of 45%, 55%, and 65%, along with a specific range of cyclic stress ratios (CSR). Test conditions are outlined in Table 7.1, where groups A, B, and C correspond to D_r values of 45%, 55%, and 65%, respectively. Within each group, designated as 1, 2, and 3, three

different cyclic stress ratios (CSR) ranging from 0.13 to 0.20 were applied, resulting in the labeling of tests A1 to A3, B1 to B3, and C1 to C3.

Table 7.2. Summary of cyclic stress ratio (CSR) specified to each test with pre-consolidation pressure (σ'_c) shown in brackets

Condition		Cyclic stress ratio (CSR)		
Relative density (D_r)	Test	1	2	3
45%	A	0.13 (83)	0.15 (72)	0.18 (80)
55%	B	0.15 (87)	0.18 (96)	0.20 (81)
65%	C	0.15 (94)	0.18 (97)	0.20 (89)

7.2.6. Test results

As mentioned in Table 7.3, 9 cases of undrain triaxial test was conducted, and the stress deviator (σ_d), cyclic stress ratio(CSR), exceed pore water pressure(Δu) and axial strain (ϵ_a) was showed in Fig. 7.2. which were obtained by following equations:

$$\sigma_a = \sigma_d + \sigma_r \quad (7.1)$$

$$p = \frac{(\sigma_a + 2\sigma_r)}{3} = \frac{(\sigma_d + 3\sigma_r)}{3} \quad (7.2)$$

$$\sigma'_c = p_i - u_i \quad (7.3)$$

$$\tau_d = \frac{\sigma_d}{2} \quad (7.4)$$

$$\tau_d = \frac{\sigma_d}{2} \quad (7.5)$$

$$CSR = \frac{\tau_d}{\sigma'_c} = \frac{\sigma_d}{2\sigma'_c} \quad (7.6)$$

$$\Delta u = u - u_i \quad (7.7)$$

Where: σ_a represent axial stress, σ_r represent radial stress, p represent mean total stress, σ'_c represent pre-consolidation pressure, p_i represent initial p after consolidation, u_i represent Initial u after consolidation, and τ_d represent shear stress.

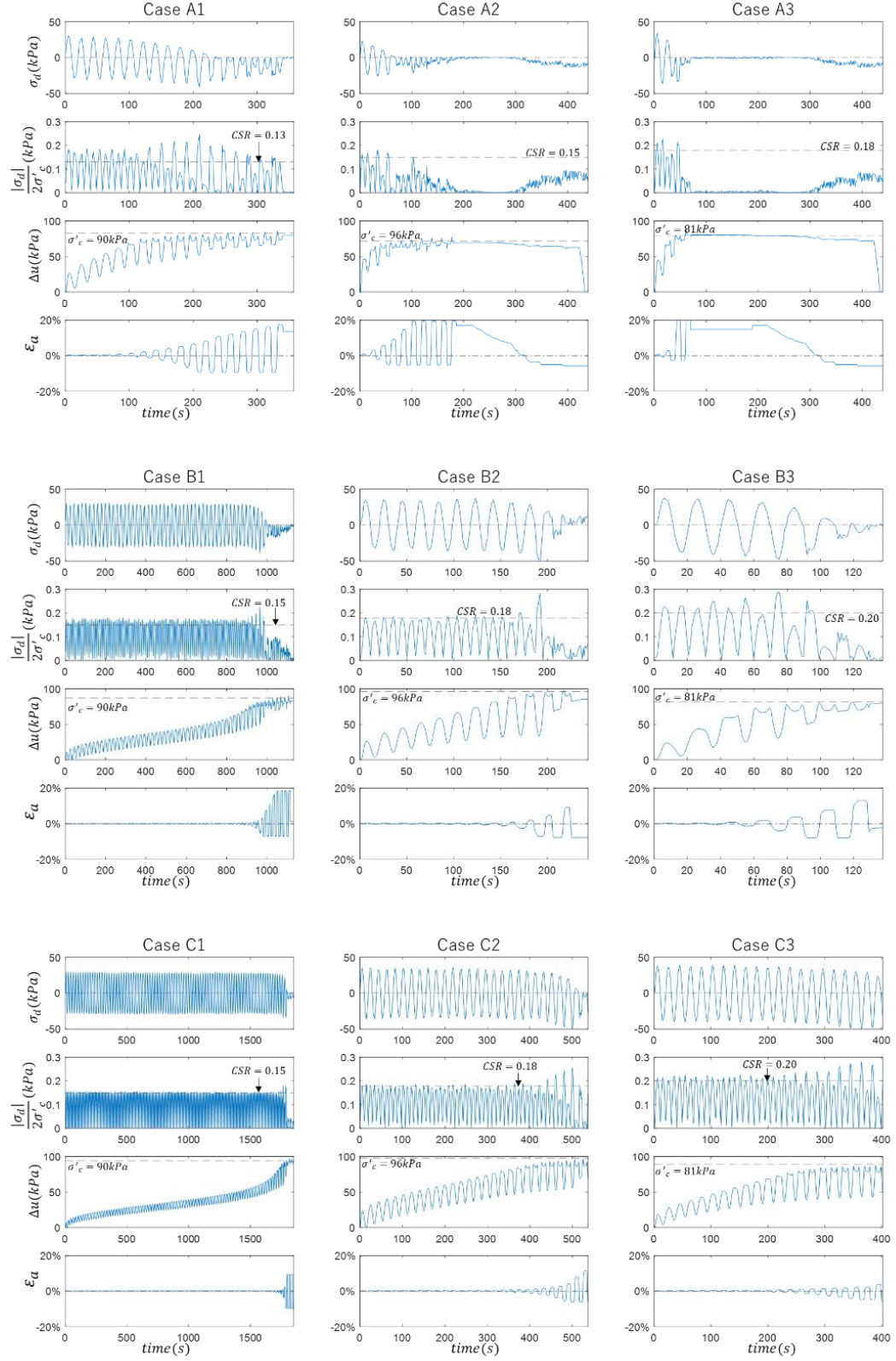


Fig. 7.2. Undrain triaxial test result

7.2.7. Analysis Methodology for Results

7.2.7.1. Cycles separation

As demonstrated in the preceding section, cyclic loading was applied to the specimen until liquefaction occurred. Throughout the testing period, the axial strain ε_a steadily increased. In this study, determining the number of cycles (N_c) to initial liquefaction was essential for evaluating liquefaction onset, and the energy method required separate the stress and strain data by cycle. Hence, an objective algorithm was indispensable for accurately delineating the cycle counting. In this research, two different methods were utilized and discussed for delineating the cycle count.

Angular method

In this method, the relationship between stress deviator (σ_d) and axial strain (ε_a) was used to delineate the cycle count. Case A1 serves as an illustrative example to demonstrate the angular method. Fig. 7.3 depicts the stress deviator and axial strain data for Case A1. Additionally, Fig. 7.4 presents a schematic diagram illustrating the stress-strain relationship within one cycle. In this method, the accumulated angle is employed to separate the cycles. As depicted in Fig. 7.4, the data point $(\varepsilon_{dk}, \sigma_{dk})$ can be represented as a vector $\vec{v}_k$, originating from the origin point and rotating with the development of stress and strain. In common, the angular difference $\Delta\theta_k$ can be obtained by the following equation:

$$\cos(\Delta\theta_k) = \frac{v_k \cdot v_{k+1}}{|v_k||v_{k+1}|} \quad (7.8)$$

However, the value range of cos function is $[0 \pi]$, due to the fluctuation of stress and strain, the angular difference between vectors may vary in both clockwise and counterclockwise directions, as illustrated in Fig. 7.4. Therefore, it is necessary to consider both positive and negative angles in the angle calculation. The angular difference was computed using the following equations:

$$\sin(\Delta\theta_k) = \frac{v_k \times v_{k+1}}{|v_k||v_{k+1}|} \quad (7.9)$$

$$\tan(\Delta\theta_k) = \frac{\sin(\Delta\theta_k)}{\cos(\Delta\theta_k)} = \frac{v_k \times v_{k+1}}{v_k \cdot v_{k+1}} \quad (7.10)$$

Here, the positive and negative of angular difference can be obtained. However, the value range of tan function is $[-\frac{\pi}{2} \frac{\pi}{2}]$, so the atan2 function of Matlab(Mathworks, 2024) was used to returns the four-quadrant inverse tangent ($\tan^{-1}$), as expressed in the equation:

$$\Delta\theta_k = \text{atan2}(v_k \times v_{k+1}, v_k \cdot v_{k+1}) \quad (7.11)$$

Unlike $\arctan$, the atan2 function has a value range of $[-\pi, \pi]$ (excluding $-\pi$). Then, the accumulated angular difference $\sum \theta_k$ can be used to delineate the cycle count, the range that accumulated angular difference varies from 0 to 2π was counted as one cycle.

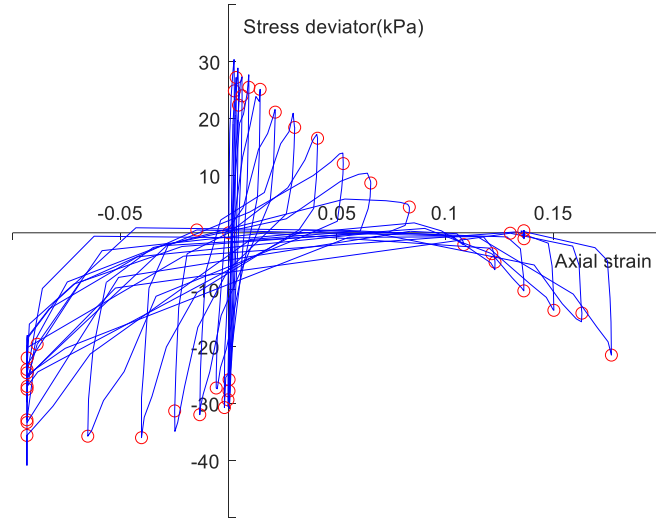


Fig. 7.3. Strain-stress relationship of case A1

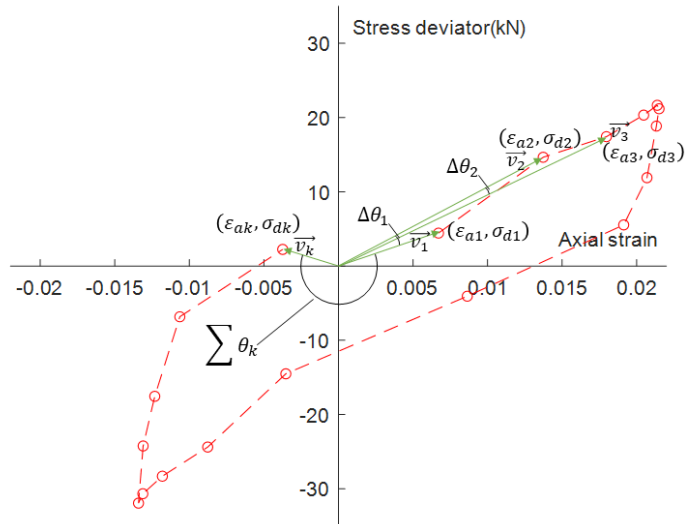


Fig. 7.4. Strain-stress relationship in single loop

Another issue to consider in the angular method is the potential for baseline drift. While cycles can be separated by accumulated angular differences, this method relies on the precondition that the origin point must be situated within the closed area formed by the stress-strain curve. As depicted in Fig. 7.5, the baseline of stress gradually deviates from the zero-axis due to specimen deformation, and there is an initial drift in strain due to measurement error. To address this challenge, curve modification techniques must be employed to correct the baseline drift. In this research, High-pass filter was used to remove the baseline drift.

Fig. 7.5(a) and (b) present the results of the high-pass filter applied in the time and frequency domains, respectively. In both stress and strain, the power spectrum peak appeared at around 50 mHz, corresponding to the frequency of the input load, with unexpected peaks observed at frequencies lower than 50 mHz. Consequently, the high-pass frequency was temporarily set at 50 mHz to eliminate the baseline drift. As evident from the filtered data, the baseline drift has been successfully removed.

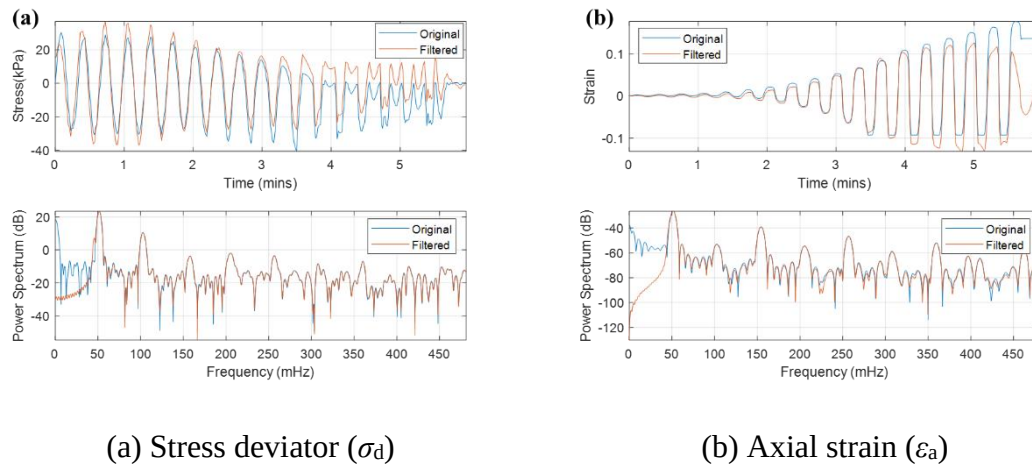


Fig. 7.5. High-pass filter results (a) Stress deviator (b) Axial strain

Subsequently, the stress and strain values of Case A1 were segmented into individual cycles by integrating the accumulated angular differences for cycle counting and applying a high-pass filter, as depicted in Fig. 7.6. Notably, the last portion of data was excluded from analysis since loading had ceased.

It is important to acknowledge that curve modification can impact the experimental data values. Therefore, it is employed solely for cycle number counting purposes. In the data analysis process, the original data will be utilized.

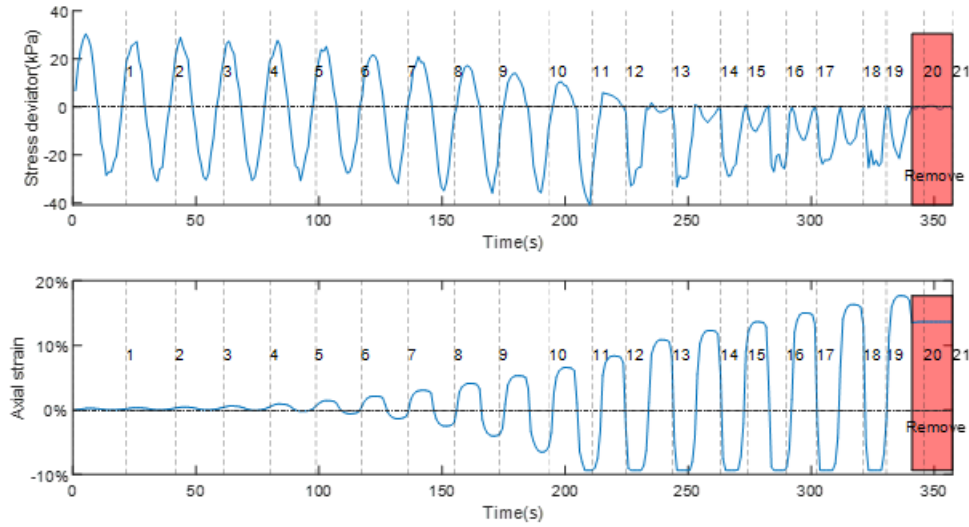


Fig. 7.6. Cycle number counting of case A1

In summary, the angular method, utilizing both stress and strain for cycle counting, offers enhanced accuracy, maintaining stability even under conditions of substantial deformation. However, this method necessitates data stability and is sensitive to baseline drift. Conversely, the high-pass filter requires knowledge of the input load frequency and sampling frequency, making it challenging to objectively determine the high-pass frequency. Moreover, when the input load frequency is unclear, such as in the case of natural earthquake loads, objectively determining the high-pass frequency becomes problematic.

Derivative zero-crossing method

A zero-crossing is identified where a mathematical function transitions between positive and negative values, marked by crossing the horizontal axis at zero on its graph. This concept is prevalent in various disciplines including electronics, mathematics, acoustics, and image processing. However, due to the baseline drift, detecting the intercept between data curve and zero value is challenged, so derivative was used in this method to avoid the influence came from baseline drift. In this algorithm, the stress deviator was used in cycle number counting, the derivative of data was obtained, zero-crossings of the derivative often correspond to local maxima/minima of the original data, which can be more robust for non-symmetric signals.

By this algorithm, the influence of baseline drift can be avoiding. However, in this method, the noise of signal may cause the local peak in derivative data, so a smoothing method was required to avoid the local peak. So, the kernel smoothing was used in this research. Kernel smoothing, also known as kernel density estimation, is a non-parametric technique used to estimate the probability density

function (PDF) of a random variable from a set of observed data points(Wand & Jones, 1994). As showed in Fig. 7.7, given a random sample $X_1, \dots, X_n$ with a continuous, univariate density f . The kernel density estimator is:

$$\hat{f}_h(x, h) = \frac{1}{n} \sum_{i=1}^n K_h(x - X_i) = \frac{1}{nh} \sum_{i=1}^n K\left(\frac{x - X_i}{h}\right) \quad (7.12)$$

With kernel K and bandwidth h . Under mild conditions (h must decrease with increasing n) the kernel estimate converges in probability to the true density. It involves assigning a weight to each data point based on its distance from a given point of interest, typically using a kernel function. The kernel function specifies the shape of the weight assigned to each data point, in this research, Gaussian (normal) kernel was used. The bandwidth h controls how wide the probability mass is spread around a point, bandwidth selection bears danger of under- or oversmoothing. The selection of bandwidth is related to the interval of data sampling. In the calculating we set h as two times of the interval:

$$h = \frac{2}{f} \quad (7.13)$$

In which f is sampling frequency.

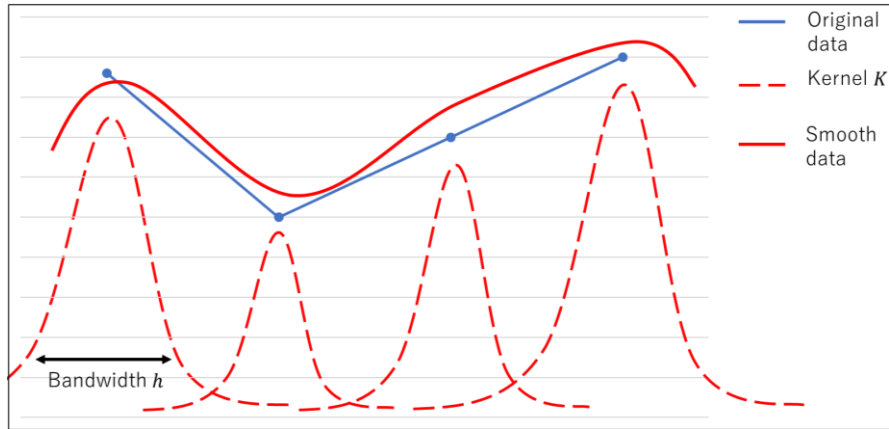


Fig. 7.7. Schematic diagram of kernel smoothing

The result after kernel smoothing and the cycle number counting was showed in Fig. 7.8. The original and smoothed data were showed in upper graph. Again, curve modification can impact the experimental data values, the smoothed data was used only for cycle separation.

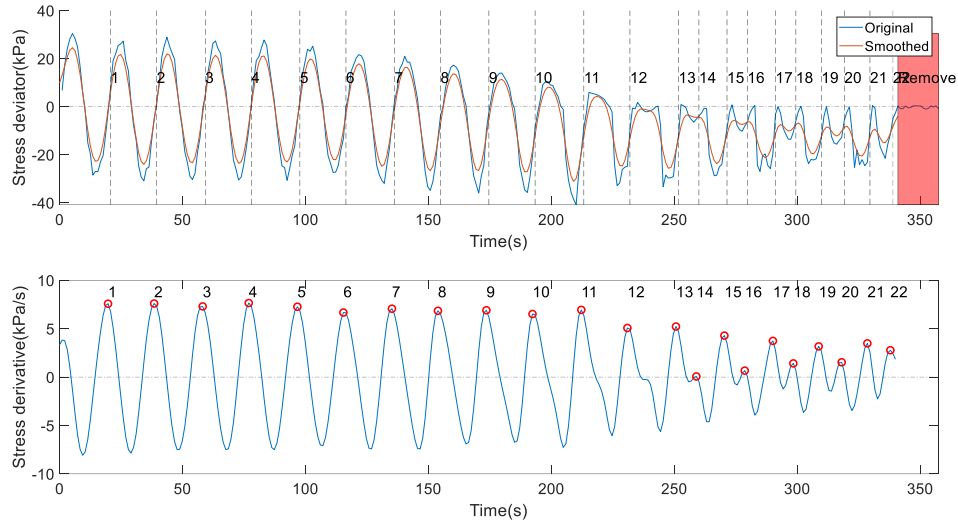


Fig. 7.8. Smoothing and cycle number counting

From the figure, it is evident that although a local peak is present in the last part of the data, affecting the stability of cycle counting, the method effectively separates the data in the earlier sections. In summary, this approach is suitable for datasets with less symmetry. Moreover, the method's reliance solely on the sampling frequency for smoothing enhances its objectivity. In this research, the derivative zero-crossing method was utilized for cycle separation.

7.2.7.2. *Liquefaction evaluation*

The following methods were used to evaluate the liquefaction. While the conventional method, known as the excess pore water pressure ratio (EPWPR), is primarily used for liquefaction evaluation in triaxial tests, it may not always be suitable for other scenarios. Particularly in centrifuge experiments, it is common for σ'_c to change significantly before and after liquefaction. Therefore, additional methods such as the pore water pressure ratio (PWPR) and effective stress decrease ratio (ESDR) were incorporated in attempts to evaluate liquefaction:

$$EPWPR = \frac{\Delta u}{\sigma'_c} \quad (7.14)$$

$$PWPR = \frac{u}{p} \quad (7.15)$$

$$ESDR = 1 - \frac{p'}{\sigma'_c} \quad (7.16)$$

Specially, p' represent the effective mean total stress, which id obtained by following equation:

$$p' = p - u \quad (7.17)$$

During the undrained triaxial test, excess pore water pressure increases, resulting in an increase in all the parameters listed above. Liquefaction is deemed to have occurred when these parameters reach a value of 1. However, due to measurement limitations and constraints imposed by the apparatus, both parameter values of 0.95 and 1 were utilized to evaluate liquefaction in this research.

Additionally, axial strain is employed as a criterion for assessing liquefaction occurrence. The double amplitude strain(ε_{DA}) was introduced in liquefaction evaluation:

$$\varepsilon_{DA} = 2|\varepsilon_a| \quad (7.18)$$

According to this criterion, liquefaction is observed when the axial strain reaches 5% double amplitude strain ($\pm 2.5\%$ axial strain)(Ishihara, 1993). However, the 5% double amplitude strain criterion was originally developed for natural soils, where limited cyclic mobility is observed until initial liquefaction (or shortly before), followed by excessive deformation (flow liquefaction). This raises concerns regarding the applicability of the 5% double amplitude strain criterion, as the dynamic response of soil may differ from that of traditional soils (El Mohtar, 2009). To further investigate liquefaction with greater precision, double amplitudes of 2.5%, 5%, and 10% were all taken into consideration.

7.2.7.3. *Energy-based method*

The soil was characterized as viscoelastic, indicating that when subjected to cyclic loading, the resulting strain shows a phase shift relative to the applied stress. As shown in Fig. 7.9 , this phase shift δ results in a closed-loop strain-stress relationship. The enclosed elliptical loop, represented by the area ΔW , denotes the energy dissipated during a single loading cycle (Kokusho, 2017). Conversely, the maximum elastic energy stored during cyclic loading occurs at the peak stress or strain, corresponding to the areas of triangles OAB or OA'B'.

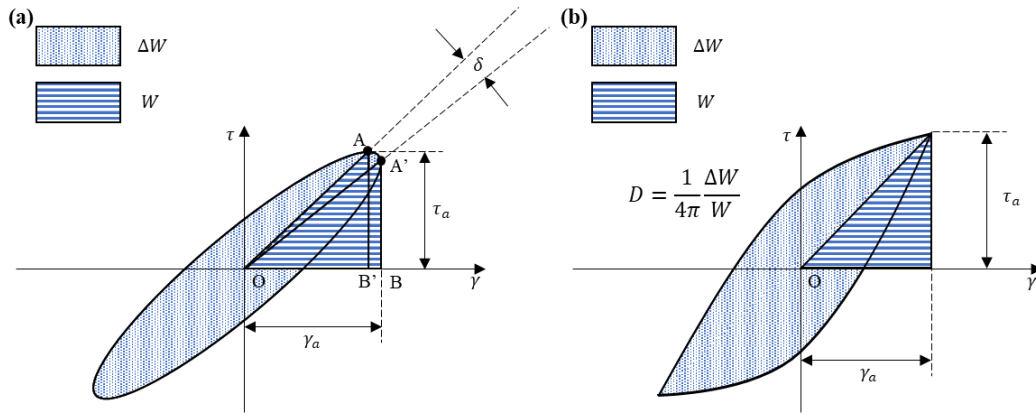


Fig. 7.9. General stress-strain relationship of viscoelastic material (a) Linear stress-strain model; (b) Nonlinear stress-strain model

The above method can be used most conveniently. However, in this research, shear strain levels are significant, leading to a nonlinear hysteresis loop within the cyclic stress-strain relationship, as depicted in Fig. 7.9 (b). The same example has used in section 4.1.1 was showed in Fig. 7.10. It should be noticed that due to the asymmetric shape of the close area, the dissipated energy was obtained by the average of area A1 and A2.

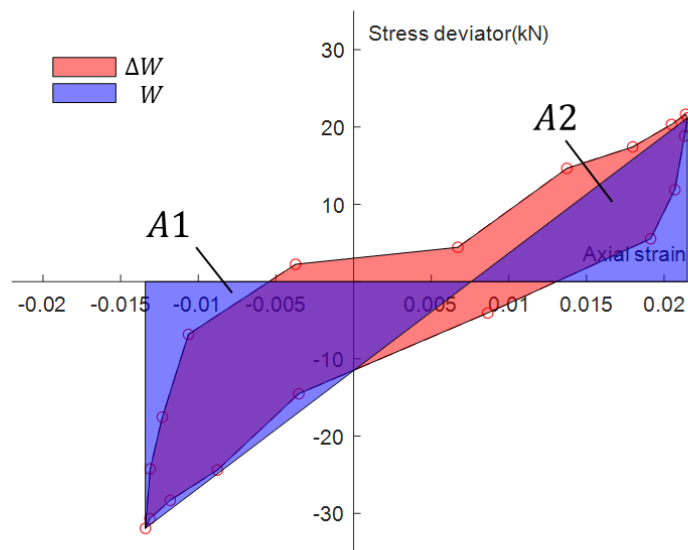


Fig. 7.10. Dissipated energy and strain energy in single cycle.

7.2.8. Discussion of liquefaction evaluation

Using the derivative zero-crossing method, the results of undrained triaxial tests were analyzed and translated into the domain of cycle numbers (N_c), the liquefaction evaluation criterions (EPWPR, PWPR and ESDR), the double amplitude strain (ε_{DA}) and the energy (ΔW and W) were showed in Fig. 7.11. The number of cyclic loads for initial liquefaction (N_l) can be obtained by these criterions, as showed in Table 7.3.

Table 7.3 Number of cycles for initial liquefaction (N_l) by each criterion

Criterion	EPWPR		PWPR		ESDR		ε_{DA}		
Limit	0.95	1.00	0.95	1.00	0.95	1.00	2.5%	5%	10%
N_l (A1)	5.5	9.6	8.1	13.3	8.1	9.6	5.2	7.2	9.2
N_l (A2)	1.1	2.6	3.1	6.3	1.6	2.6	1.2	1.2	2.2
N_l (A3)	1.1	2.1	2.2	4.1	2.1	2.1	1.2	1.2	2.3
N_l (B1)	46.5	52.6	51.1	53.3	49.0	52.6	47.7	48.7	49.7
N_l (B2)	8.1	-	9.1	11.2	10.5	-	8.6	8.7	9.6
N_l (B3)	3.0	5.1	4.1	6.1	4.1	5.1	2.7	3.7	4.6
N_l (C1)	88.5	92.6	90.1	92.1	90.5	92.6	89.6	90.6	91.6
N_l (C2)	19.0	24.6	23.0	-	22.5	24.6	21.2	23.6	24.7
N_l (C3)	12.5	18.6	18.1	-	15.6	18.6	13.7	15.8	20.9

If power regression is used to relate the cyclic stress ratio (CSR) and the number of cyclic loads for initial liquefaction (N_l) obtained from double amplitude strain (ε_{DA}), the simple and its logarithmic form could be considered below (Pipatpongsa, 2022).

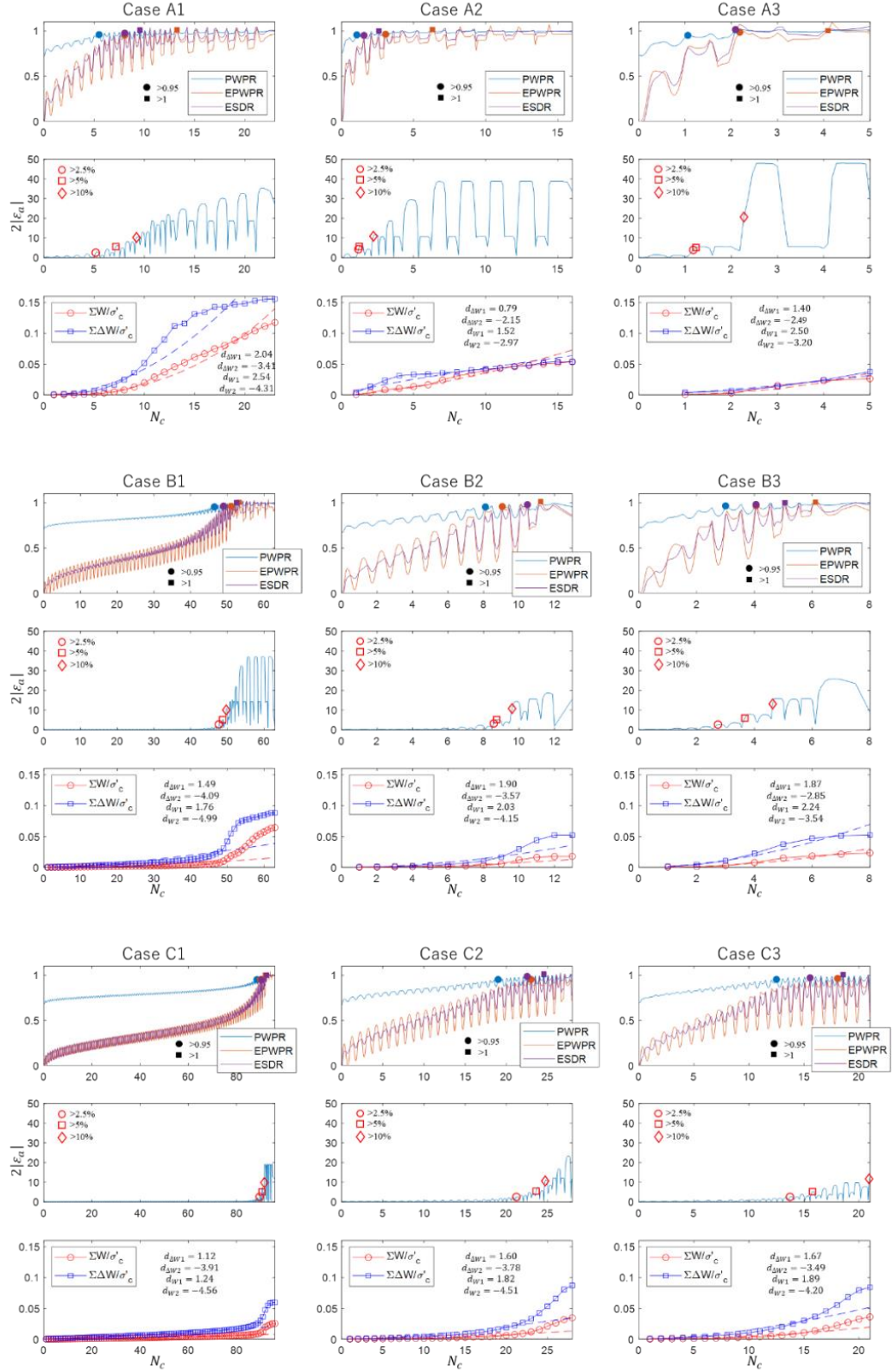


Fig. 7.11. Liquefaction evaluation and energy-based analysis

$$\frac{\sigma_d}{2\sigma'_c} = 10^{\alpha} N_c^{\beta}, N_c = \left(\frac{1}{10^{\alpha}} \frac{\sigma_d}{2\sigma'_c} \right)^{\frac{1}{\beta}} \quad (7.19)$$

where α and β are assumed to be approximated by Eq. (7.20) and Eq. (7.21), respectively,

$$\alpha = a_1 + a_2 D_r \quad (7.20)$$

$$\beta = b_1 + b_2 D_r \quad (7.21)$$

In this research, considering the set of $D_r = 45\%$, 55% and 65% , the coefficients a_1 , a_2 , as well as b_1 , b_2 , can be determined by the N_l data listed in Table 7.3. The N_l when the liquefaction evaluation criterions reach 0.95 or 1, as well as ϵ_{DA} reach 2.5%, 5% and 10% were considered and the liquefaction strength curve was showed in Fig. 7.12.

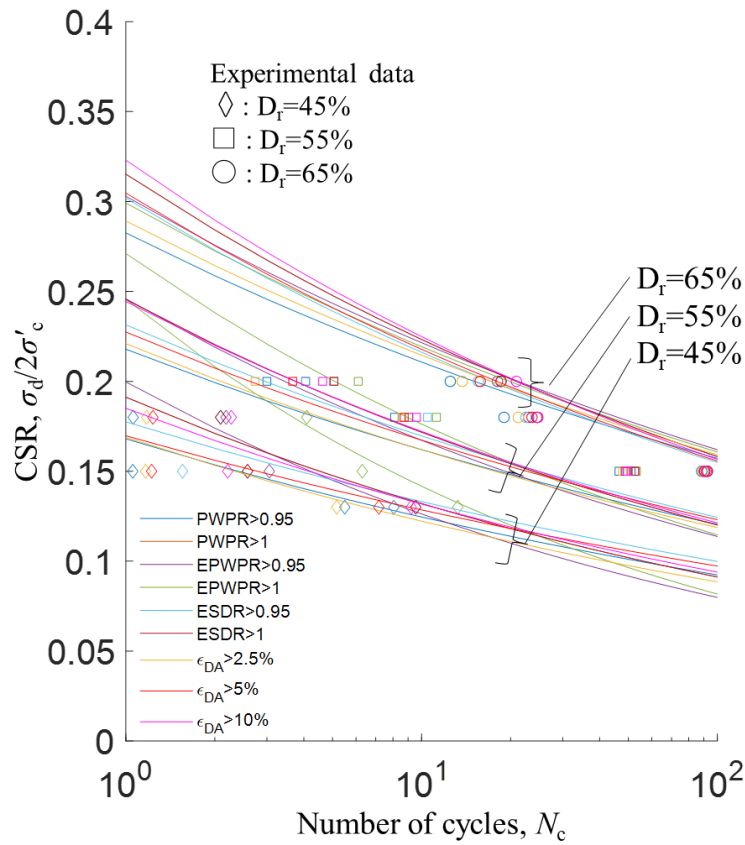


Fig. 7.12. Liquefaction strength curve

By the regression curve, the cyclic resistance ratio (CRR) can be determined from Eq. (7.19) using $N_l = 20$ for a given D_r as listed in Table 7.4.

Table 7.4 Cyclic resistance ratio (CRR) at $N_l = 20$

Criterion	EPWPR		PWPR		ESDR		ε_{DA}		
Limit	0.95	1.00	0.95	1.00	0.95	1.00	2.5%	5%	10%
A(45%)	0.114	0.118	0.110	0.120	0.122	0.118	0.111	0.118	0.119
B(55%)	0.148	0.154	0.149	0.155	0.155	0.154	0.147	0.153	0.155
C(65%)	0.193	0.201	0.202	0.200	0.196	0.201	0.196	0.197	0.202

The results of liquefaction evaluation are discussed in Fig. 7.13 illustrating unique relationships regardless of relative densities. It should be noted that energy can only be obtained for the entire cycle. To simplify calculations and improve accuracy, the maximum values of liquefaction evaluation criteria and ε_{DA} in one cycle were used in fitting. The relations between ε_{DA} and liquefaction evaluation criteria (EPWPR, PWPR and ESDR), as well as relations between $\Sigma\Delta W/\sigma'_c$ and liquefaction evaluation criteria, were fitted with hyperbolic forms shown in Eq. (7.22). Here, y represents the liquefaction evaluation criteria, while x represents ε_{DA} or $\Sigma\Delta W/\sigma'_c$. The value of parameters was listed in Table 7.5.

$$y = \frac{c_2 x}{c_1 + x} + c_3 \quad (7.22)$$

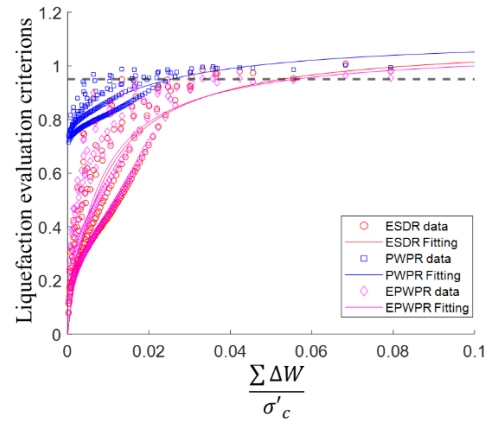
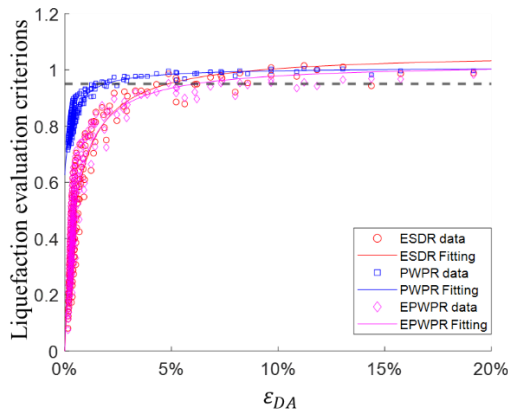
(a) ε_{DA} and liquefaction evaluation criteria(b) $\Sigma\Delta W/\sigma'_c$ and liquefaction evaluation criteria

Fig. 7.13. Unique relationships obtained by curve fitting functions

Table 7.5 Parameters of parabolic fitting

	ε_{DA} – EPWPR	ε_{DA} – PWPR	ε_{DA} – ESDR	$\Sigma\Delta W/\sigma'_c$ – EPWPR	$\Sigma\Delta W/\sigma'_c$ – PWPR	$\Sigma\Delta W/\sigma'_c$ – ESDR
c_1	0.487	0.350	0.568	0.007	0.021	0.008
c_2	1.026	0.383	1.061	1.071	0.366	1.125
c_3	-	0.626	-	-	0.749	-

In Fig. 7.13, the relationship between ε_{DA} and the hyperbolic curve is clearly evident. Additionally, the fitting lines for *EPWPR* and *ESDR* closely align. However, there is a slight deviation in the results of *PWPR* compared to the other two criteria. This difference arises from the fact that the *EPWPR* do not account for changes in effective stress. Conversely, the initial value of *PWPR* does not start from 0, and in different conditions, it may vary. Therefore, *EPWPR* and *ESDR* are recommended for analysis purposes.

By using hyperbolic fitting functions, the corresponding ε_{DA} or $\Sigma\Delta W/\sigma'_c$ values for certain liquefaction evaluation criteria can be calculated, as shown in Table 7.6. The ε_{DA} values obtained from the fitting functions fall within the range of 1.92% to 18.73%. This range is consistent with the ε_{DA} range associated with liquefaction (2.5% to 10%), indicating the reliability of the hyperbolic fitting approach. Then, the corresponding $\Sigma\Delta W/\sigma'_c$ values were calculated and listed in Table 7.6.

Table 7.6 ε_{DA} and $\Sigma\Delta W/\sigma'_c$ based on hyperbolic fitting function

Criterion	<i>EPWPR</i>		<i>PWPR</i>		<i>ESDR</i>	
Limit	0.95	1.0	0.95	1.0	0.95	1.0
$\varepsilon_{DA}(\%)$	6.09	18.73	1.92	14.54	4.86	9.31
$\Sigma\Delta W/\sigma'_c$	0.056	0.101	0.026	0.045	0.045	0.067

The $\Sigma\Delta W/\sigma'_c$ and $\Sigma W/\sigma'_c$ in N_c domain was showed in Fig. 7.11 and can be fitted by power regression shown in Eq. (7.23) and (7.24). The fitting parameters was listed in Fig. 7.11 and Table 7.7.

$$\frac{\Sigma\Delta W}{\sigma'_c} = 10^{d_{\Delta W2}} \cdot N_c^{d_{\Delta W1}}, N_c = \left(\frac{1}{10^{d_{\Delta W2}}} \frac{\Sigma\Delta W}{\sigma'_c} \right)^{\frac{1}{d_{\Delta W1}}} \quad (7.23)$$

$$\frac{\Sigma W}{\sigma'_c} = 10^{d_{W2}} \cdot N_c^{d_{W1}}, N_c = \left(\frac{1}{10^{d_{W2}}} \frac{\Sigma W}{\sigma'_c} \right)^{\frac{1}{d_{W1}}} \quad (7.24)$$

Table 7.7 Parameters of power regression of energy

	$\frac{\Sigma\Delta W}{\sigma'_c}$		$\frac{\Sigma W}{\sigma'_c}$	
Parameter	$d_{\Delta W1}$	$d_{\Delta W2}$	d_{W1}	d_{W2}
N_l (A1)	2.04	-3.41	2.54	-4.31
N_l (A2)	0.79	-2.15	1.52	-2.97
N_l (A3)	1.40	-2.49	2.50	-3.20
N_l (B1)	1.49	-4.09	1.77	-4.99
N_l (B2)	1.91	-3.57	2.03	-4.15
N_l (B3)	1.87	-2.85	2.24	-3.54
N_l (C1)	1.12	-3.91	1.24	-4.56

N_l (C2)	1.60	-3.78	1.82	-4.51
N_l (C3)	1.67	-3.49	1.89	-4.20

In order to compare with the traditional liquefaction evaluation criterion showed in Eq. (7.19) to (7.21), a similar method was used, as showed below:

$$N_l = \left(\frac{1}{10^\lambda} \frac{\sum \Delta W}{\sigma'_c} \right)^{\frac{1}{\mu}} \quad (7.25)$$

$$\lambda = l_1 + l_2 D_r + l_3 CSR + l_4 D_r CSR \quad (7.26)$$

$$\mu = m_1 + m_2 D_r + m_3 CSR + m_4 D_r CSR \quad (7.27)$$

The parameters l and m were obtained from the linear regression using the parameters of Table 7.7. By using the Eq. (7.25), the number of cyclic loads for initial liquefaction (N_l) can be obtained by the certain $\sum \Delta W / \sigma'_c$ value. Here, the value listed in Table 7.6 were used to obtain the N_l . The N_l obtained from the stress-based method and energy-based method were showed in Fig. 7.14.

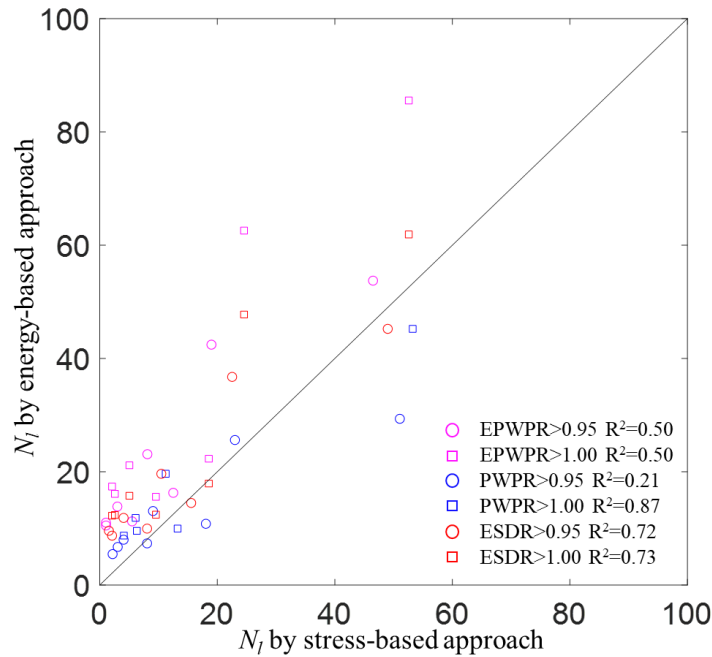


Fig. 7.14. N_l corresponding to liquefaction evaluation criterions

In conclusion, the number of cyclic loads for initial liquefaction (N_l) determined using the energy-based approach exhibits a correlation with those obtained through the stress-based method. Notably, the N_l results obtained via the excess pore water pressure ratio (EPWPR) method exhibited a lower R^2

value. As discussed earlier, this discrepancy may be attributed to the EPWPR method's failure to account for changes in effective stress during dynamic loading. On the other hand, the pore water pressure ratio (PWPR) method, which relies on a simple ratio between pore water pressure and mean stress, exhibited higher instability prior to the pore water pressure reaching the mean stress threshold. Consequently, the liquefaction evaluation based on PWPR showed a higher correlation with a limitation of 1.0. Conversely, the effective stress decrease ratio (ESDR) method demonstrated greater stability and accounted for changes in effective mean stress. Consequently, the results obtained from the ESDR method exhibited a strong correlation with those obtained from the stress-based approach.

The evaluation of liquefaction resistance through undrained cyclic triaxial tests using an energy-based method represents a significant advancement in geotechnical engineering. By introducing the concept of energy dissipation per unit volume of soil, this method offers a more comprehensive understanding of soil behavior under cyclic loading conditions, surpassing the limitations of stress-based methods.

One significant aspect of our analysis lies in the cycles separation process, where we delineate individual cycles within the test data. This approach allows us to isolate the effects of different loading cycles, facilitating a more granular understanding of soil behavior under varying stress conditions.

Both the angular method and the derivative zero-crossing method offer distinct advantages and disadvantages for cycles separation in liquefaction assessment. The derivative zero-crossing method is straightforward and intuitive but sensitive to noise and measurement errors. On the other hand, the angular method provides robustness against noise but requires expertise in signal processing and may introduce subjectivity in threshold selection. The choice between these methods depends on factors like data complexity and available expertise. Further research is needed to refine their application in practical engineering contexts.

The hyperbolic fitting functions utilized in the analysis provided a robust framework for understanding the relationships between energy and liquefaction evaluation criteria. Furthermore, by using this hyperbolic fitting functions and energy characteristics, the number of cyclic loads for initial liquefaction (N_i) can be predicted by the energy approach.

Overall, the findings of this study contribute significantly to the ongoing development of methodologies for evaluating liquefaction resistance in geotechnical engineering. By incorporating energy-based approaches alongside traditional stress-based methods, engineers can obtain a more comprehensive understanding of soil behavior under seismic loading conditions, thereby enhancing the reliability and effectiveness of liquefaction assessment techniques in geotechnical practice.

7.3. Energy-based liquefaction evaluation of centrifuge test

By using the cycle separation method introduced in this chapter earlier, the dynamic shear stress and strain of centrifuge embankment model test were dissected into individual cycles, which allowed us to compute both the dissipated energy (ΔW) and the strain energy (W) within each cycle.

7.3.1. Stress-based liquefaction evaluation criterions in centrifuge test

In triaxial test, both of the stress-based method and energy-based method was used in liquefaction evaluation discussion. However, the stress and strain condition inside embankment was much more complex, the soil arching effect and saturation will influence the stress and strain distribution, as mentioned in earlier Chapters, the stress and strain inside embankment was not uniform. Hence, the stress-based liquefaction evaluation needs to be discussed in this analysis.

Thus far in this thesis, the earth pressure used in liquefaction evaluation was the vertical earth pressure measured by the three-directional earth pressure sensor. The pore water pressure ratio (PWPR), excess pore water pressure ratio (EPWPR) and effective stress decrease ratio (ESDR) are the parameters usually used in liquefaction evaluation was showed below, in which: u represents the pore water pressure; Δu represent the excess pore water pressure; σ_v represents the vertical earth pressure; the σ_v' and σ_{vi}' represent the effective vertical earth pressure and initial effective vertical earth pressure respectively.

$$PWPR = \frac{u}{\sigma_v} \quad (7.28)$$

$$EPWPR = \frac{\Delta u}{\sigma_{vi}'} \quad (7.29)$$

$$ESDR = 1 - \frac{\sigma_v'}{\sigma_{vi}'} \quad (7.30)$$

The PWPR was the simplest method to evaluate the liquefaction occurrence, the liquefaction happened when the effective earth pressure become zero, which means the pore water pressure was large than earth pressure. The EPWPR only considered initial effective vertical earth for evaluation, and the ESDR used both effective vertical earth and the initial effective vertical earth. The relationship between EPWPR and ESDR can be derivate by the equations below:

$$\begin{aligned}
ESDR &= 1 - \frac{\sigma_v'}{\sigma_{vi}'} = \frac{\sigma_{vi}' - \sigma_v'}{\sigma_{vi}'} = \frac{\sigma_{vi} - u_i - \sigma_v + u}{\sigma_{vi}'} \\
&= \frac{(u - u_i) - (\sigma_v - \sigma_{vi})}{\sigma_{vi}'} = \frac{\Delta u - \Delta \sigma_v}{\sigma_{vi}'} \\
&= \frac{\Delta u}{\sigma_{vi}'} - \frac{\Delta \sigma_v}{\sigma_{vi}'} \\
&= EPWPR - \frac{\Delta \sigma_v}{\sigma_{vi}'}
\end{aligned} \tag{7.31}$$

Thus, when the $\Delta \sigma_v$ equals zero, the EPWPR and ESDR was equal. However, in dynamic centrifuge test, the value of vertical earth pressure changed a lot during dynamic loading. The EPWPR did not considered about the change of vertical earth pressure, so the EPWPR did not suitable to embankment used dynamic centrifuge test.

Due to the initial sensor installation error, as well as the rotation of sensors, the value of earth pressure may be influenced. On the other hand, the stress condition inside the embankment under dynamic loading was complex, the reliability of the earth pressure measurement and liquefaction should be discussed.

In triaxial test, the confining pressure can be hold constantly, and the stress was applied cyclically. As showed in Fig. 7.2, The initial excess pore water pressure was zero, and with the cyclic loading apply it increased until reach pre-consolidation pressure. The liquefaction was evaluated by the pore water pressure ratio u/p and excess pore water pressure ratio $\Delta u / \sigma'_c$. The occurrence of liquefaction evaluated by the two parameters shows differences, but both two parameters will increase and converging to 1, and did not larger than 1.

Then, based on a series of undrained cyclic triaxial tests, the liquefaction and energy were investigated. The accumulated dissipated energy per unit volume (ΔW) can be obtained and normalized by initial effective confining pressure, the normalized accumulated dissipated energy per unit volume ($\Delta W / \sigma'_c$) showed a clear range when liquefaction occurred irrespectively of relative density and fine contents of sands. As showed in Fig. 7.13(b).

In centrifuge test, the dissipated energy can be also obtained. However, the stress condition in centrifuge experiment was different with the elementary test. So, the liquefaction evaluation by stress-based method should be discussed first. The pressure condition and liquefaction evaluation were already described in earlier chapter, as an example, the stress condition of case DB-1-8-R6-3 was showed in Fig. 7.15. In centrifuge experiment, the initial effective earth pressure cannot be controlled

in each case, and was quite different in each case because of the arch action. Similarly, the initial pore water pressure was quite different in each case too. Both the vertical earth pressure and pore water pressure experienced a substantial change during dynamic loading, and the pore water pressure increased significantly, in some cases it's even quite larger than the earth pressure. As results, the ESDR have a large increase too, and after liquefaction it still keep increasing and far larger than 1.

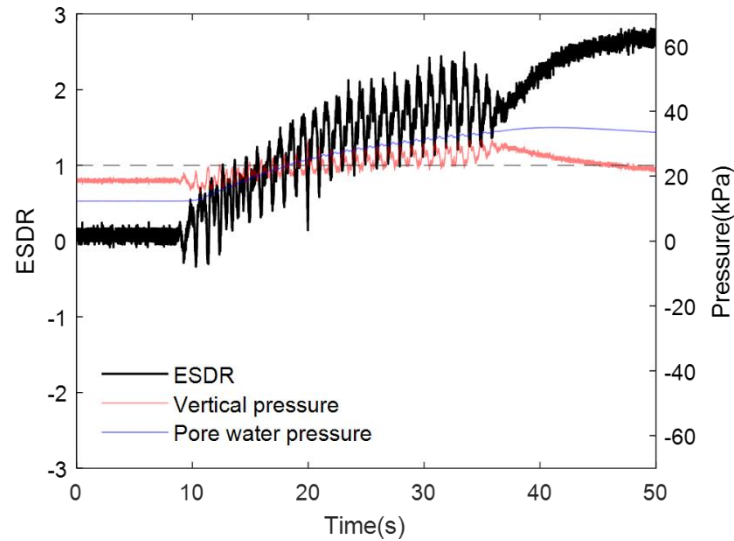


Fig. 7.15. Pressure condition and Effective Stress Decrease Ratio (ESDR) of case DB-1-8-R6-3

To solve this problem, another data analysis method was purposed. In this research, three-directional earth pressure sensor was used, and the improvement was conducted. The shear stress, vertical earth pressure, and horizontal earth pressure can be calculated by decomposing the measured values, as showed in Eq. (3.1). Hence, the mean stress σ_c can be calculated by the equation below:

$$\sigma_c = \frac{\sigma_y + 2\sigma_x}{3} \quad (7.32)$$

The σ_y and σ_x represent the vertical and horizontal earth pressure respectively. The experimental data are analyzed assuming a plane strain condition, where the horizontal and longitudinal earth pressures are considered equal. The pressure distribution, based on the mean earth pressure for the same scenario, is illustrated in Fig. 7.16. While there is still a significant variation in mean stress and pore water pressure, the latter is not notably larger than the mean stress this time. Additionally, the Equivalent Stress Deviation Ratio (ESDR) is relatively small, just slightly exceeding 1.

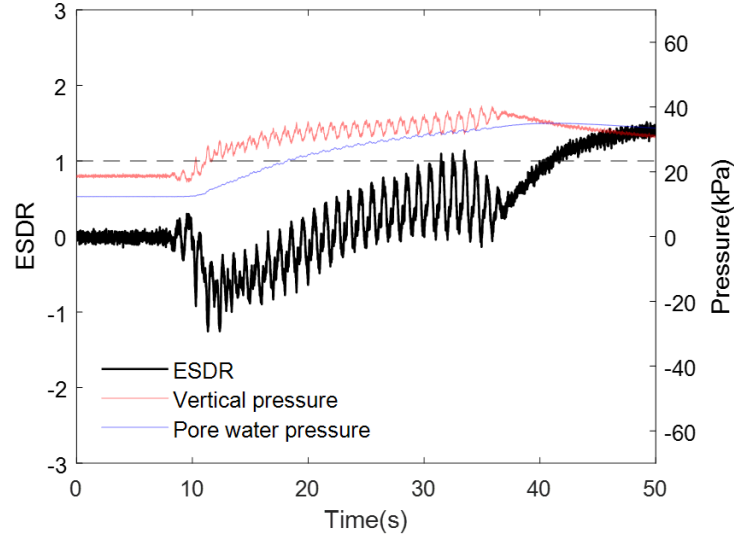


Fig. 7.16. Pressure condition based on mean stress of case DB-1-8-R6-3

7.3.2. Energy-based analysis

After analyzing liquefaction potential using the stress-based method (SBM), the energy-based method (EBM) was proposed to evaluate liquefaction potential and associated large liquefaction-induced soil settlement. The EBM focuses on the dissipated energy, where the cumulative dissipated energy ($\Sigma\Delta W$) based on shear stress and shear strain represents liquefaction behavior.

As discussed in Section 7.2.8, the relationship between the number of cyclic loads for initial liquefaction (N_i) and normalized dissipated cumulative energy ($\Sigma\Delta W/\sigma'_c$) is described by Eq. (7.25). In the centrifuge experiment, however, the dynamic load is applied to the embankment via the movement of a shaking table. Therefore, the method to predict the energy at which liquefaction occurs should be carefully examined.

Dief developed a regression analysis method that predicts the energy per unit volume dissipated in generating liquefaction (δw) in centrifuge tests using relative density (D_r) and initial effective earth pressure (σ'_c) (Dief, 2000). Additionally, strain energy has been utilized in predicting liquefaction (Baziar et al., 2011). More recently, the cyclic resistance ratio (CRR) at $N_i=20$ or 15, obtained from elementary tests, has been employed for liquefaction evaluation by Kokusho (Kokusho & Kaneko, 2018; Kokusho & Mimori, 2015; Kokusho, 2017, 2021; Kokusho & Tanimoto, 2021). Regression analysis was used to establish the relationship between $\Sigma\Delta W/\sigma'_c$ and CRR.

In this research, a simplified method developed from Kokusho's work was employed. The regression function obtained from triaxial tests was used to predict the energy in centrifuge

embankment tests. In Eq. (7.25), the $\Sigma\Delta W/\sigma'_c$ have been obtained by the number of cyclic loads for initial liquefaction (N_l), the cyclic stress ratio (CSR) and relative density (D_r). To obtain the $\Sigma\Delta W/\sigma'_c$ that can trigger the liquefaction, the cyclic resistance ratio at $N_l=20$ (CRR₂₀) was used in calculation using the Eq. (7.19). The D_r was obtained by following equations, using the properties of Table 2.4, the max and min void ratio $e_{max} = 1.003$, $e_{min} = 0.441$, Soil particle density $\rho_s = 2.633g/cm^3$, maximum dry density $\rho_{dmax} = 1.75g/cm^3$, and the degree of compaction $D_c = 90\%$. The relative density in centrifuge test was around 58.9%.

$$\rho_d = D_c \cdot \rho_{dmax} \quad (7.33)$$

$$e = \frac{\rho_s}{\rho_d} - 1 \quad (7.34)$$

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% \quad (7.35)$$

Then, the CRR₂₀ of a relative density 58.9% was obtained by Eq. (7.19), The result was listed in Table 7.8. Then, the D_r , CRR₂₀ and $N_l=20$ have been substitute into the Eq. (7.25), the final result shows that the liquefaction will occur when the $\Sigma\Delta W/\sigma'_c$ reach 0.064 to 0.065.

Table 7.8 Number of cycles for initial liquefaction (N_l) by each criterion

Criterion	EPWPR		PWPR		ESDR		ε_{DA}			
Limit	0.95	1.00	0.95	1.00	0.95	1.00	2.5%	5%	10%	
CRR ₂₀	0.164	0.171	0.168	0.171	0.169	0.171	0.165	0.169	0.172	
$\Sigma\Delta W/\sigma'_c$	0.065	0.064	0.065	0.064	0.065		0.064	0.065	0.065	0.064

The schematic diagram of the experiment model is shown in Fig. 7.17. In this research, the bottom of the embankment model was saturated. Based on pore water pressure readings and image observations, the water level after liquid supplementation was between ACC4 and ACC2. Shear stress was derived from acceleration data, and the shear stress at the bottom layer was calculated based on the cumulative shear stress from the crest. Consequently, the variation in shear stress across different layers is minimal, with shear stress gradually accumulating from the crest to the bottom. Therefore, the shear stress in the ACC1 layer was used in the calculations.

Shear strain, on the other hand, was calculated using the differential displacement between two adjacent accelerometers, resulting in significant variations across different layers. Previous results indicated that the shear strain at the bottom of the embankment was larger than that at the upper layers.

Considering the distribution of shear strain and the saturated zone, the cumulative dissipated energy of the embankment was calculated separately for different layers. Shear strains between ACC1 and ACC5(γ_1) and between ACC1 and ACC3(γ_2) was used in the calculations.

It is important to note that the embankment height and sensor conditions varied in each case. Regardless of the experimental conditions, the accelerometers located 1 meter above the soft base (ACC1) and at the crest (ACC5) were consistently used in calculations. To ensure consistency and comparability, the energy was normalized to obtain a dimensionless result. In triaxial tests, normalization is typically done using the initial effective mean stress. However, due to the complex stress conditions within an embankment, normalizing with the initial effective consolidation stress (σ_{v0}') may not be reliable. Therefore, to compare the accumulated dissipated energy across different cases, the theoretical overburden pressure at the depth of ACC1 was used for normalization.

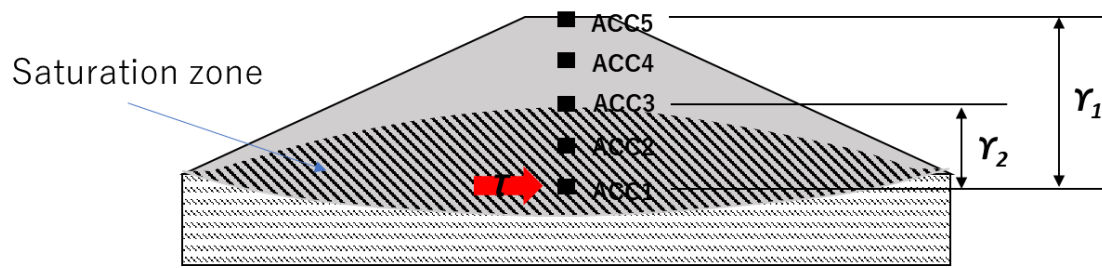


Fig. 7.17. Shear stress and strain condition

Nine cases were used in this analysis, the experiment conditions were showed in Table 7.9.

Table 7.9. Experiment conditions

No.	Test ID	White noise	Height	Input wave	Reinforcement
1	DB-1-8-N-1	○	100%	Ramped sine wave	None
2	DB-1-8-R6-1	×	100%	Sine wave	6 nails
3	DB-1-8-R6-2	○	100%	Ramped sine wave	6 nails
4	DB-1-8-R6-3	×	100%	Ramped sine wave	6 nails
5	DB-1-8-R4-1	×	100%	Sine wave	4 nails
6	DB-1-8-R4-2	×	100%	Ramped sine wave	4 nails
7	DB-1-8-R6-4	×	100%	Ramped sine wave	6 nails
8	DB-1-8-R4-3	×	100%	Ramped sine wave	4 nails
9	DB-0.75-8-N	×	75%	Sine wave	None

10	DB-0.5-8-N	×	50%	Sine wave	None
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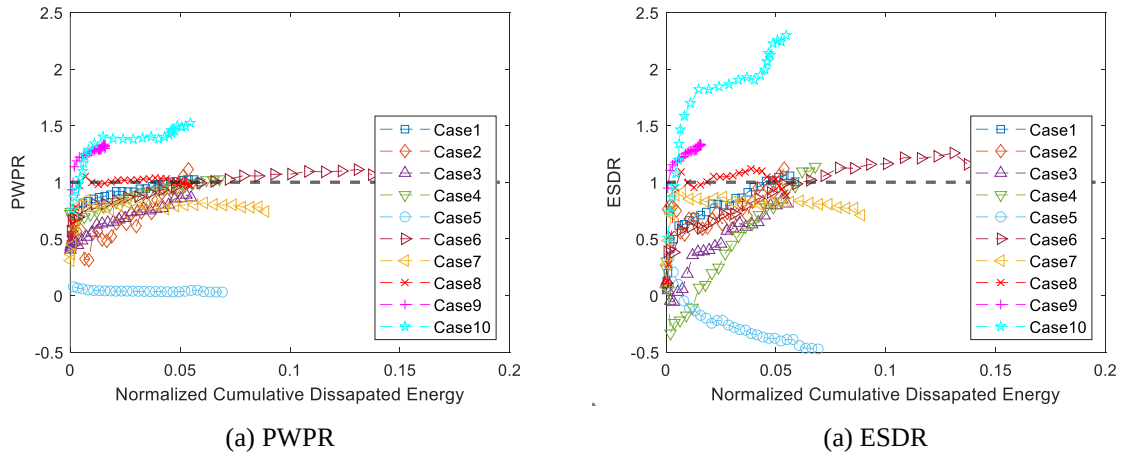


Fig. 7.18. Comparison between normalized cumulative dissipated energy and liquefaction evaluation criterions using γ_1
(a) PWPR; (b) ESDR

The results of the energy-based analysis of the embankment are presented in Fig. 7.18. The pore water pressure ratio (PWPR) and effective stress decrease ratio (ESDR) were utilized to evaluate the occurrence of liquefaction. It is important to note that in Case 5, a leakage issue resulted in an insufficient amount of liquid inside the embankment, leading to extremely low liquefaction evaluation criteria for this case.

From the figure, unlike the triaxial test results, the relationship between energy and liquefaction evaluation criteria does not exhibit any noticeable trend. This discrepancy may be attributed to the complex stress conditions within the embankment. As previously discussed, the shear strain inside the embankment varies with depth, which implies that the shear strain of the entire embankment may not accurately reflect the liquefaction properties of the saturated zone.

So, to investigate the energy properties in saturated zone of embankment, shear strain between ACC1 and ACC3(γ_2) was used. However, the sensor condition in each case was different, so the cases using in energy analysis should satisfy the requirement that the ACC3 was installed. The case 2, case 5 and case 7 was removed because the sensor limitation. The relationship between cumulative dissipated energy and liquefaction evaluation parameters was showed in Fig. 7.19. It should be noticed that along different cases, due to soil arch action and reduction of embankment height, the initial earth pressure was quite different. Similarly, after the liquid supplement was stopped, the liquid will immediately leak from the embankment model so unavoidably, the initial pore water pressure was

quite different too. So, the dissipated energy was normalized by the theoretical overburden pressure at the depth of ACC1.

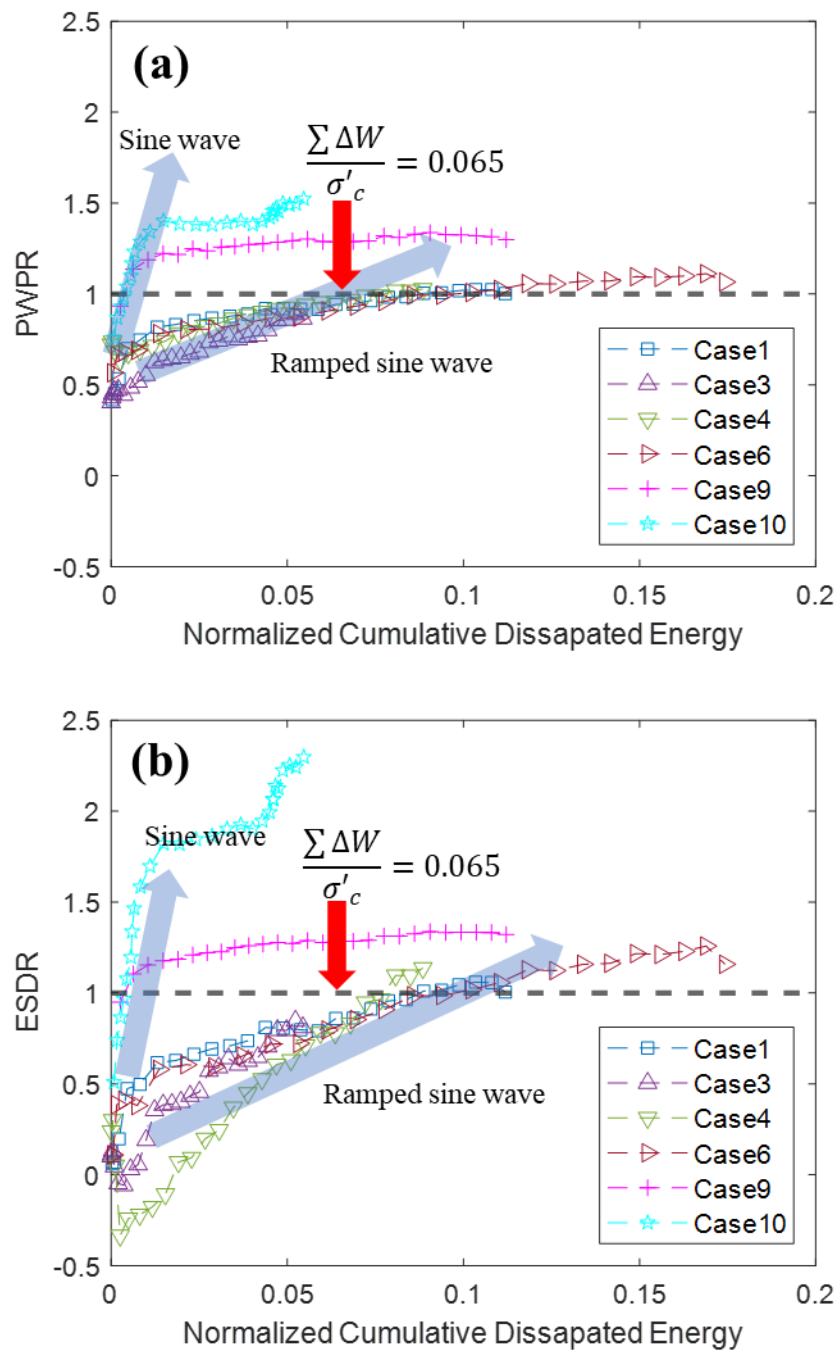


Fig. 7.19. Comparison between normalized cumulative dissipated energy and liquefaction evaluation criteria using Y_2 (a) PWPR; (b) ESDR

The results of the effective stress decrease ratio (ESDR) and pore water pressure ratio (PWPR) showed similar trends. Firstly, the type of input wave significantly influenced the liquefaction

evaluation criteria. When using the sine wave, the liquefaction evaluation criteria increased rapidly, reaching 1 within several cycles. However, with the ramped sine wave, despite differences in white noise usage, input wave, embankment height, and reinforcement conditions, the normalized cumulative dissipated energy at liquefaction (when the criteria reach 1) consistently ranged between 0.05 and 0.1. In Case 3, although liquefaction did not occur, the trend indicated that with more input energy, liquefaction would likely occur similarly to other cases.

The liquefaction evaluation criteria also caused some variations in results. In some cases, ESDR exhibited a large decrease due to a sudden increase in earth pressure, resulting in the effective mean stress exceeding the initial effective mean stress and causing the ESDR to become negative. However, this did not affect the cumulative dissipated energy value.

In summary, comparing the stress-based liquefaction evaluation with the energy-based method revealed a relationship between energy and liquefaction. Liquefaction occurred when the normalized cumulative dissipated energy reaches the range around 0.065.

7.4. Conclusion

In this chapter, algorithms for data analysis were developed and verified through elementary experiments. Two algorithms were devised to objectively separate cycles and determine the dissipated energy. The results of the triaxial tests illustrate the relationship between liquefaction and energy. Hyperbolic fitting functions were employed to validate the reliability of the energy-based method.

Subsequently, the energy-based method was applied to centrifuge tests on embankments. Initially, to compare with the energy-based approach, the stress-based method was refined. Given the complex stress conditions in the embankment centrifuge tests and the difficulty in measurement, mean stress was derived using three-directional earth pressure sensors instead of vertical earth pressure to calculate liquefaction evaluation criteria. The results of the centrifuge tests were used to obtain normalized cumulative dissipated energy. Upon comparing this with liquefaction evaluation criteria, it was observed that the energy of the entire embankment did not exhibit a clear relationship with liquefaction. However, there was a notable correlation between liquefaction and energy in the saturated zone. This correlation was found to be influenced by the type of input wave. For instance, when a sine wave was applied, significant initial acceleration was imposed on the embankment, causing large stress fluctuations and immediate liquefaction. Conversely, when a ramped sine wave was applied, dissipated energy accumulated gradually, leading to liquefaction once the normalized cumulative dissipated energy reached a certain threshold.

Chapter 8. Conclusions and future work

8.1. Conclusion

This thesis explores the occurrence and progression of soil-arch action in embankments affected by basal settlement, examining liquefaction behavior and damage induced by dynamic loads. The key findings are summarized as follows:

8.1.1. Validation experiment and experiment procedure improvement

Dynamic centrifuge tests were conducted under varying construction water content conditions, implementing a liquid supply system to investigate the behavior of the embankment. Key findings are summarized as follows:

(1) **Stress measurement:** The three-directional earth pressure sensor effectively detected stress, but its accuracy, particularly in measuring horizontal earth pressure and shear stress, raised concerns. Dynamic shear stress was accurately measured using acceleration values, proving to be a reliable method. Displacement was calculated by double integration of acceleration values, and shear strain was derived from the differential between displacements. To avoid drift, a high-pass filter and frequency-domain integration method were used to improve accuracy.

(2) **Soil-Arching and liquefaction:** Earth pressure sensors at the bottom of the embankment assessed soil-arching effects due to soft base deformation. Soft base deformation was insufficient to trigger significant soil-arching, but changes in vertical earth pressure distribution at the bottom of the embankment were observed. Liquefaction assessment was carried out through the analysis of both the Excess Pore Water Pressure Ratio and the Effective Stress Decrease Ratio. Future research should refine experimental conditions to better trigger soil-arching effects and understand liquefaction.

(3) **Dynamic response strain:** During dynamic loading, response acceleration inside the embankment gradually increased from bottom to top, but shear strain in different layers decreased from bottom to top. The embankment model did not behave as a rigid solid under dynamic loading, with larger shear strain in the lower part. Maximum shear strain was influenced by water content.

(4) **Experimental Procedure Modifications:** To better trigger soil-arching and liquefaction in laboratory centrifuge tests, procedures were modified. The soft base was changed to drilled urethane for larger deformation, dynamic load was applied immediately after liquid supplementation to ensure saturation, and the amplitude of the input wave was increased to induce liquefaction.

(5) **Improved Earth Pressure Sensors:** A new three-directional earth pressure sensor design offered significant improvements in accuracy and reliability. These results validate the enhanced sensor's precision in measuring earth pressure and shear stress. Consequently, the new sensor holds promise for future research involving embankment analysis, liquefaction studies, and structural stability assessments. Proper sensor design, calibration, and ongoing monitoring are crucial for reliable data in geotechnical experiments.

8.1.2. Soil Arching Action and Liquefaction Occurrence Under Various Conditions

A series of embankment experiments were conducted under different conditions to investigate soil arch development, liquefaction properties, and deformation patterns, yielding the following conclusions:

8.1.2.1. Soil arching action

(1) **Occurrence of Soil Arching:** Soil arching action can be triggered under conditions of significant basal settlement. However, if the basal settlement is insufficient to induce soil arching, the introduction of pore fluid may trigger soil arching. This occurrence is influenced by the construction water content, with soil arching specifically observed when the initial construction water content is less than optimum water content.

(2) **Height Requirement:** The occurrence and development of soil arching require a sufficient embankment height. The trigger height of embankment was related to the geometric shape of embankment. In this study, the soil arching action was not significant when the relative height ratio H/H_{max} was 0.44, and when the H/H_{max} was 0.88, a significantly arch was observed.

(3) **Dynamic Loading Effects:** Dynamic loading influences soil-arch existence. When dynamic loading is applied, liquefaction occurs beneath the soil arch, reducing confining pressure and causing the arch to collapse. Soil nail reinforcement does not impact soil arch occurrence but influences its stability, ensuring persistence even after dynamic loading.

8.1.2.2. Liquefaction

After saturation, dynamic loads were applied to the embankment model, and liquefaction evaluation ensued, yielding the following conclusions:

(1) **Saturation Influence:** Liquefaction occurrence is influenced by the saturated condition, with insufficient liquid content preventing liquefaction under dynamic loading.

(2) **Loading History:** Loading history affects embankment liquefaction resistance. When white noise was applied before the main input wave, vertical earth pressure fluctuations were minimal, and excess pore water pressure was low, influencing liquefaction occurrence.

(3) **Reinforcement Impact:** The rise of excess pore water pressure was not influenced by reinforcement, as the decrease of effective stress always occurred regardless of soil nail use. Soil nail reinforcement does not impact liquefaction occurrence.

8.1.2.3. *Embankment damage*

(1) **Deformation Patterns:** Deformation during the static process is not influenced by construction water content or soil nail reinforcement. However, relative deformation induced by dynamic loading consistently increased with decreasing construction water content, and reinforcement reduced embankment deformation.

(2) **Primary Cause of Damage:** Liquefaction is not the primary cause of embankment damage. Instead, soil-arch collapse under dynamic loading leads to extensive damage. Soil nail reinforcement strengthens the soil arch, mitigating damage if the soil arch withstands dynamic loading.

(3) **Failure Mode:** Unreinforced embankments with arch action occurrence under dynamic loading demonstrated a failure mode consistent with the arch collapse mechanism. Soil nail-reinforced embankments experienced minimal deformation, with the shape largely unchanged.

8.1.3. *Energy-based method*

The energy-based method was used to evaluate liquefaction occurrence. Key findings are as follows:

(1) **Data Analysis Algorithms:** Two data analysis algorithms were developed for the energy method: the angular method and the derivative zero-crossing method. The choice between these methods depends on data complexity and available expertise, with further research needed for practical engineering applications.

(2) **Triaxial Test Results:** Triaxial test results were analyzed using stress-based and energy-based methods. A correlation between liquefaction and energy dissipation per unit volume of soil was established. A simplified liquefaction evaluation prediction method was developed through regression analysis.

(3) **Normalized Cumulative Dissipated Energy:** Given the complex stress conditions in embankment centrifuge tests, mean stress was derived using three-directional earth pressure sensors to calculate liquefaction evaluation criteria. The results showed that the energy of the entire embankment did not exhibit a clear relationship with liquefaction, but a notable correlation was observed between liquefaction and energy in the saturated zone. This correlation was influenced by the type of input wave, with liquefaction occurring when normalized cumulative dissipated energy reached a certain value calculated from triaxial test results.

8.2. Limitation and Future work

Based on the findings of this study, several recommendations for future work have been identified to enhance the robustness, accuracy, and applicability of experimental setups, ensuring a more thorough exploration of factors influencing soil-arch behavior and embankment response under dynamic loading conditions.

(1) Improvement of Experimental Measurement Apparatus:

In this study, a three-directional earth pressure sensor was used to measure the earth pressure, and accelerometers were used to calculate shear stress and shear strain. However, there were reliability issues, as shown in Fig. 8.1. Due to the deformation of the embankment, a loose zone can appear at the bottom, leading to local arching or gaps around the sensor. Additionally, the rotation of accelerometers affects the accuracy of calculations. Therefore, the measurement apparatus should be improved:

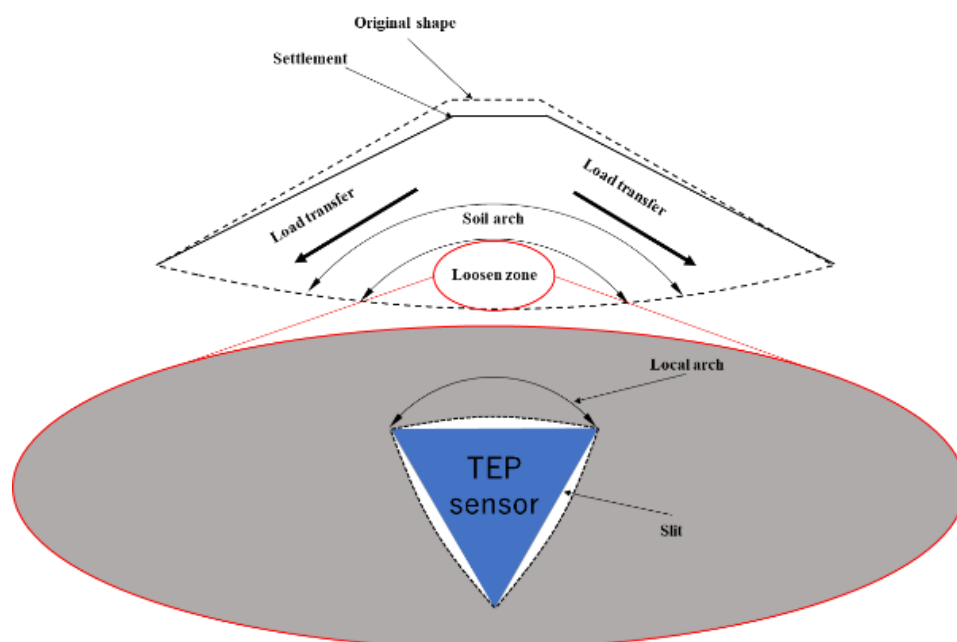


Fig. 8.1. Reliability problem of three-directional earth pressure sensor

- **Shear Strain and Stress Measurement:**

- Address error accumulation in shear strain and shear stress obtained by accelerometers.
- Mitigate baseline drift and noise issues through enhanced installation and data analysis methods.

- **Pressure Measurement Accuracy:**

- Enhance the reliability of pressure measurement apparatus to achieve higher accuracy in earth pressure measurements.

(2) Utilization of Realistic Ground Materials:

In this study, urethane was used as the soft ground material. However, due to the elastic behavior of urethane, it may not deform adequately when unloaded compared to actual soft ground. In centrifugal model experiments, the sinking of the foundation ground is influenced by increasing centrifugal acceleration, complicating the assessment of stress changes during loading. To address this, future experiments should employ materials more representative of real ground conditions or devise a mechanism using multiple jacks to control the sinking shape post-centrifugal loading, allowing for controlled sinking without deformation during acceleration.

(3) Utilization of numerical simulation:

Although FPA analysis was utilized in this study, the response of embankments under dynamic load has not been thoroughly investigated. The necessary basal settlement that can trigger soil arching has not been explored either. In the future, a modeling simulation method for arch development and collapse should be developed to predict soil arching occurrence and compare it with experimental results.

(4) Exploration of varied reinforcement conditions:

It was found that the number of soil nails or the reinforcement condition influences the behavior of embankments. However, the research was insufficient to explore more interactions between soil nail reinforcement and embankment damage. The following improvements should be conducted in future studies to deeply study the reinforcement properties:

- **Reinforcement Material Impact:**

- Investigate the impact of different reinforcing materials and explore complex reinforcement conditions beyond changes in number or inset angle.

- **Soil nail Mechanism:**

- Delve into the mechanism of soil nail reinforcement, examining the interaction between soil nails and soil for a more comprehensive understanding.

(5) Utilization of Image analysis:

Captured images were used in this study to investigate the failure mode of embankments. However, this method lacked objectivity. It is necessary to use image analysis to investigate the deformation model and failure patterns of embankments.

(6) Energy-based method improvement:

Although energy-based analysis was used to predict liquefaction evaluation, the number of elementary tests was limited, and the energy properties of soil have not been sufficiently investigated. Moreover, the liquefaction prediction method used in this thesis was rough. The following work should be conducted in the future to improve energy-based analysis in embankments:

- **Database Development:**

- Conduct more elementary experiments, such as triaxial tests and hollow torsional tests, to build a comprehensive database for the energy-based method. Enhance the accuracy of parameters for regression calculations.

- **Real Seismic Load Analysis:**

- Improve data analysis methods to account for real seismic loads rather than just harmonic motion. This advancement will facilitate the application of the energy-based method in real-case analyses in the future.

(7) Practical Engineering Applications of Soil Nailing:

In this study, soil nails have been utilized primarily to reinforce embankments subjected to basal settlement and dynamic loading. The main objective of soil nail reinforcement is not to prevent liquefaction itself but to minimize large-scale deformation and structural damage to the embankment. By incorporating soil nails, the stability of the soil mass is significantly enhanced, mitigating the collapse of soil arches and reducing overall embankment deformation during dynamic events such as liquefaction. The nails function by increasing the confining pressure along potential failure planes, thereby improving the embankment's resistance to large displacements and collapses rather than directly halting liquefaction.

Soil nailing has seen widespread application in numerous real-world geotechnical projects, where preventing large-scale damage, rather than halting liquefaction, is critical to maintaining structural integrity. For instance, in the construction of retaining walls for highways and rail embankments, soil nailing is used to prevent excessive deformation and slope failure during seismic activity or heavy

rainfall. Similarly, in urban excavation projects, soil nailing supports retaining walls and reduces the risk of catastrophic failure during construction. By reinforcing these structures, soil nailing helps manage the potential for significant settlement and lateral movement, ensuring that infrastructure remains safe and serviceable even under dynamic loads.

In the context of embankments, the practical application of soil nailing effectively reduces damage under complex loading conditions, including dynamic forces and settlement-induced stresses. The fast installation process, adaptability to various ground conditions, and minimal environmental impact have contributed to its popularity. Soil nails are particularly effective in controlling deformation and preventing large-scale damage, reinforcing embankments under challenging geotechnical conditions, as demonstrated in both experimental and field applications. Hence, this study potentially introduces a new design concept and provides a rational method for determining the necessary reinforcement along the minor principal axes of soil arching or confining pressure within existing embankments.

Reference

- [1]Adachi, T. (1997). Trap door experiment under centrifugal conditions. *Deformation and Progressive Failure in Geomechanics, IS-Nagoya'97*, 725-730.
- [2]Adalier, Korhan, Elgamal, Ahmed-W, & Martin, Geoffrey R. (1998). Foundation liquefaction countermeasures for earth embankments. *Journal of Geotechnical and Geoenvironmental Engineering*, 124(6), 500-517.
- [3]Adalier, Korhan, & Sharp, Michael K. (2004). Embankment dam on liquefiable foundation—Dynamic behavior and densification remediation. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(11), 1214-1224.
- [4]Adapa, Gautham, Ueda, Kyohei, & Uzuoka, Ryosuke. (2021). Seismic stability of embankments with different densities and upstream conditions related to the water level. *Soils and Foundations*, 61(1), 185-197.
- [5]Ashford, Scott A, Juirnarongrit, Teerawut, Sugano, Takahiro, & Hamada, Masanori. (2006). Soil–pile response to blast-induced lateral spreading. I: field test. *Journal of Geotechnical and Geoenvironmental Engineering*, 132(2), 152-162.
- [6]Ausilio, Ernesto, Conte, Enrico, & Dente, Giovanni. (2001). Stability analysis of slopes reinforced with piles. *Computers and Geotechnics*, 28(8), 591-611.
- [7]Baziar, Mohammad H, Jafarian, Yaser, Shahnazari, Habib, Movahed, Vahid, & Tutunchian, Mohammad Amin. (2011). Prediction of strain energy-based liquefaction resistance of sand–silt mixtures: an evolutionary approach. *Computers & Geosciences*, 37(11), 1883-1893.
- [8]Bo, Myint Win, Fabius, Mike, Arulrajah, Arul, & Horpibulsuk, Suksun. (2015). Environmentally friendly slope stabilization using a soil nail and root system in Canada. In *Ground improvement case histories* (pp. 629-654). Elsevier.
- [9]Bonazzi, D, & Colombet, G. (1984). Reajustement et entretien des ancrages de talus. RENFORCEMENT EN PLACE DES SOLS ET DES ROCHES-COMPTES RENDUS DU COLLOQUE INTERNATIONAL, PARIS, 9-11 OCTOBRE 1984,
- [10]Byrne, Rj, Cotton, D, Porterfield, J, Wolschlag, C, & Ueblacker, G. (1996). Manual for design and construction monitoring of soil nail walls.
- [11]Cai, Fei, & Ugai, Keizo. (2000). Numerical analysis of the stability of a slope reinforced with piles. *Soils and Foundations*, 40(1), 73-84.
- [12]Chen, Lt, & Poulos, Hg. (1997). Piles subjected to lateral soil movements. *Journal of Geotechnical and Geoenvironmental Engineering*, 123(9), 802-811.
- [13]Craig, Wh. (2018). Geotechnical centrifuges: past, present and future. In *Geotechnical Centrifuge Technology* (pp. 1-18). CRC Press.
- [14]Dewoolkar, Mandar M, Santichaianant, Kitidech, & Ko, Hon-Yim. (2007). Centrifuge modeling

of granular soil response over active circular trapdoors. *Soils and Foundations*, 47(5), 931-945.

- [15]Dey, Arindam. (2015). Issues and aspects of soil nailing. Conference on Challenges and Recent Advantageous in Geotechnical Engineering Research and Practice,
- [16]Dief, Hesham Moustafa. (2000). *Evaluating the liquefaction potential of soils by the energy method in the centrifuge*. Case Western Reserve University.
- [17]El Mohtar, Cs. (2009). Evaluation of the 5% double amplitude strain criterion. Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering (Volumes 1, 2, 3 and 4),
- [18]El Sawwaf, Mostafa A. (2005). Strip footing behavior on pile and sheet pile-stabilized sand slope. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(6), 705-715.
- [19]Enomoto, Tadao, & Sasaki, Tetsuya. (2015). Several factors affecting seismic behaviour of embankments in dynamic centrifuge model tests. *Soils and Foundations*, 55(4), 813-828.
- [20]Fang, Kun. (2019). Progression and onset of undercut slope failure observed by surface velocity in physical models subjected to arch action.
- [21]Figueroa, J Ludwig, & Dahisaria, Nitin. (1991). *An energy approach in defining soil liquefaction* International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.
- [22]Figueroa, J Ludwig, Saada, Adel S, Liang, Liqun, & Dahisaria, Nitin M. (1994). Evaluation of soil liquefaction by energy principles. *Journal of Geotechnical Engineering*, 120(9), 1554-1569.
- [23]Hamada, Masanori, Yasuda, Susumu, Isoyama, Ryoji, & Emoto, Katsutoshi. (1986). Observation of permanent ground displacements induced by soil liquefaction. *Doboku Gakkai Ronbunshu*, 1986(376), 211-220.
- [24]Han, Gao-Xiao, Gong, Quan-Mei, & Zhou, Shun-Hua. (2015). Soil arching in a piled embankment under dynamic load. *International Journal of Geomechanics*, 15(6), 04014094.
- [25]Handy, Richard L. (1985). The arch in soil arching. *Journal of Geotechnical Engineering*, 111(3), 302-318.
- [26]Harder, Leslie F, Kelson, Keith I, Kishida, Tadahiro, & Kayen, Robert. (2011). Preliminary observations of the Fujinuma dam failure following the March 11, 2011 Tohoku Offshore Earthquake, Japan. *Geotechnical Extreme Events Reconnaissance Report No. GEER-25e*.
- [27]Higo, Yosuke, Lee, Chung-Won, Doi, Tatsuya, Kinugawa, Teppei, Kimura, Makoto, Kimoto, Sayuri, & Oka, Fusao. (2015). Study of dynamic stability of unsaturated embankments with different water contents by centrifugal model tests. *Soils and Foundations*, 55(1), 112-126.
- [28]Hirakawa, Shouya. (2021). Collapse mechanisms and liquefaction resistance inside levee resting

- on soft ground, Master's thesis, Department of Urban Management, Kyoto University (in Japanese)
- [29]Ishac, Magdi F, & Heidebrecht, Arthur C. (1982). Energy dissipation and seismic liquefaction in sands. *Earthquake Engineering & Structural Dynamics*, 10(1), 59-68.
- [30]Ishihara, Kenji. (1993). Liquefaction and flow failure during earthquakes. *Géotechnique*, 43(3), 351-451.
- [31]Ito, Tomio, Matsui, Tamotsu, & Hong, Won Pyo. (1981). Design method for stabilizing piles against landslide—one row of piles. *Soils and Foundations*, 21(1), 21-37.
- [32]Ito, Tomio, Matsui, Tamotsu, & Hong, Won Pyo. (1982). Extended design method for multi-row stabilizing piles against landslide. *Soils and Foundations*, 22(1), 1-13.
- [33]Iwasaki, Toshio. (1978). A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan. Proc. of 2nd Int. National Conf. on Microzonation, 1978,
- [34]Iwasaki, Toshio. (1986). Soil liquefaction studies in Japan: state-of-the-art. *Soil Dynamics and Earthquake Engineering*, 5(1), 2-68.
- [35]Izawa, Jun, Doi, Tatsuya, Suzuki, Akira, & Kojima, Kenichi. (2022). Seismic Design of Embankments in Consideration of Damage Process during Earthquakes. *Quarterly Report of RTRI*, 63(1), 56-63.
- [36]Kaneko, M, Nishikawa, J, Sasaki, Y, Nagase, M, & Mamiya, K. (1995). River dike failure in Japan by earthquakes in 1993.
- [37]Kawakami, Fusayoshi, & Asada, Akie. (1966). Damage to the ground and earth structures by the Niigata earthquake of June 16, 1964. *Soils and Foundations*, 6(1), 14-30.
- [38]Kazama, Motoki, Suzuki, Takahiro, & Yanagisawa, Eiji. (1999). Evaluation of dissipation energy accumulated in surface ground and its application to liquefaction prediction. *Doboku Gakkai Ronbunshu*, 1999(631), 161-177.
- [39]Kazama, Motoki, Toyota, Hirofumi, Towhata, Ikuo, & Yanagisawa, Eiji. (1996). Stress strain relationship of sandy soils obtained from centrifuge shaking table tests. *Doboku Gakkai Ronbunshu*, 1996(535), 73-82.
- [40]Koga, Yasuyuki, & Matsuo, Osamu. (1990). Shaking table tests of embankments resting on liquefiable sandy ground. *Soils and Foundations*, 30(4), 162-174.
- [41]Koizumi, Yasunori. (1966). Changes in density of sand subsoil caused by the Niigata earthquake. *Soils and Foundations*, 6(2), 38-44.
- [42]Kokusho, T, & Kaneko, Y. (2018). Energy evaluation for liquefaction-induced strain of loose sands by harmonic and irregular loading tests. *Soil Dynamics and Earthquake Engineering*, 114, 362-377.
- [43]Kokusho, T, & Mimori, Y. (2015). Liquefaction potential evaluations by energy-based method

and stress-based method for various ground motions. *Soil Dynamics and Earthquake Engineering*, 75, 130-146.

- [44]Kokusho, Takaji. (2013). Liquefaction potential evaluations: energy-based method versus stress-based method. *Canadian Geotechnical Journal*, 50(10), 1088-1099.
- [45]Kokusho, Takaji. (2017). *Innovative earthquake soil dynamics*. CRC Press.
- [46]Kokusho, Takaji. (2021). Energy-based liquefaction evaluation for induced strain and surface settlement — evaluation steps and case studies —. *Soil Dynamics and Earthquake Engineering*, 143. <https://doi.org/10.1016/j.soildyn.2020.106552>
- [47]Kokusho, Takaji, & Tanimoto, Shunsuke. (2021). Energy Capacity versus Liquefaction Strength Investigated by Cyclic Triaxial Tests on Intact Soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 147(4). [https://doi.org/10.1061/\(asce\)gt.1943-5606.0002484](https://doi.org/10.1061/(asce)gt.1943-5606.0002484)
- [48]Law, Kum Tim, Cao, Yl, & He, Gn. (1990). An energy approach for assessing seismic liquefaction potential. *Canadian Geotechnical Journal*, 27(3), 320-329.
- [49]Lee, Ik, & Herington, Jr. (1971). *Stresses beneath granular embankments*.
- [50]Liang, Liqun. (1995). *Development of an energy method for evaluating the liquefaction potential of a soil deposit*. Case Western Reserve University.
- [51]Lu, Cw, Oka, F, & Zhang, F. (2008). Analysis of soil-pile-structure interaction in a two - layer ground during earthquakes considering liquefaction. *International Journal for Numerical and Analytical Methods in Geomechanics*, 32(8), 863-895.
- [52]Luscher, Ulrich, & Høeg, Kaare. (1964). The beneficial action of the surrounding soil on the load-carrying capacity of buried tubes. Proceedings of the Symposium on Soil-Structure Interaction September 1964.,
- [53]Mathworks. (2024). *atan2*. <https://www.mathworks.com/help/matlab/ref/atan2.html>
- [54]Michalowski, Radoslaw L, & Park, Namgyu. (2005). Arching in granular soils. In *Geomechanics: testing, modeling, and simulation* (pp. 255-268).
- [55]Ministry of Land, Infrastructure, Transport and Tourism, Tohoku Regional Development Bureau. (2011). *Report of "Technical Study Group for Rehabilitation of Levees of Kitakami River, etc.*
- [56]Miyata, Y., Pipatpongsa, T., Techawongsakorn, T., & Takemura, J. (2015). *Arch action in embankments subjected to subsidence of foundation* Proceedings of the 50th Japan National Conference on Geotechnical Engineering, Hokkaido University of Science, Hokkaido.
- [57]Miyata, Yoshito, Pipatpongsa, Thirapong, Takemura, Jiro, & Mohammad Hossein, Khosravi. (2014). *Arch action in trapezoidal embankments subjected to subsidence of foundation* JGS annual conference,

- [58]Nemat-Nasser, S, & Shokooh, A. (1979). A unified approach to densification and liquefaction of cohesionless sand in cyclic shearing. *Canadian Geotechnical Journal*, 16(4), 659-678.
- [59]Ng, Charles Ww, Li, Xiang-Song, Van Laak, Paul A, & Hou, David Yj. (2004). Centrifuge modeling of loose fill embankment subjected to uni-axial and bi-axial earthquakes. *Soil Dynamics and Earthquake Engineering*, 24(4), 305-318.
- [60]Nguyen, Tan, & Tran, Loc V. (2021). Arching effect in sand piles under base deflection using geometrically non-linear isogeometric analysis. *Geomechanics and Engineering*, 26(4), 369.
- [61]Oka, F, Tsai, P, Kimoto, S, & Kato, R. (2012a). Damage patterns of river embankments due to the 2011 off the Pacific Coast of Tohoku Earthquake and a numerical modeling of the deformation of river embankments with a clayey subsoil layer. *Soils and Foundations*, 52(5), 890-909.
- [62]Oka, F, Yoshida, N, Kai, S, Tobita, T, Higo, Y, Torii, N, Kagamihara, S, Nakanishi, N, Kimoto, S, & Yamakawa, Y. (2012b). Reconnaissance report of geotechnical damage due to the 2011 off the Pacific Coast of Tohoku Earthquake—4 Northern Area of Miyagi Prefecture. *Journal of the Geological Society*, 7(1), 37-55.
- [63]Okabe, T, Morimoto, S, Imoto, A, Yokota, Y, Date, K, & Yamamoto, T. (2014). Study on Application to Tunnel Stabilization Method Using High-friction Anchored Rockbolts. ISRM International Symposium-Asian Rock Mechanics Symposium,
- [64]Okamura, Mitsu, & Matsuo, Osamu. (2002). Effects of remedial measures for mitigating embankment settlement due to foundation liquefaction. *International Journal of Physical Modelling in Geotechnics*, 2(2), 01-12.
- [65]Okamura, Mitsu, & Tamamura, Shuji. (2011). Seismic stability of embankment on soft soil deposit. *International Journal of Physical Modelling in Geotechnics*, 11(2), 50-57.
- [66]Okamura, Mitsu, Tamamura, Shuji, & Yamamoto, Rikuto. (2013). Seismic stability of embankments subjected to pre-deformation due to foundation consolidation. *Soils and Foundations*, 53(1), 11-22.
- [67]Ovesen, N Krebs. (1979). The scaling law relationship-Panel discussion. Proc. 7th European Regional Conf. on SMFE,
- [68]Park, Young-Ho, Kim, Sung-Ryul, Kim, Sung-Hwan, & Kim, Myoung-Mo. (2000). Liquefaction of embankments on sandy soils and the optimum countermeasure against the liquefaction. Proceeding of 7th International Conference on Computer Methods and Advances in Geomechanics. Balkema, Rotterdam,
- [69]Patel, Radhika M, Jayalekshmi, Br, & Shivashankar, R. (2021). Stress distribution in basal geogrid reinforced pile-supported embankments under seismic loads. *Transportation Infrastructure Geotechnology*, 1-26.
- [70]Pipatpongsa, T, Nakamura, J, Borely, C, & Khosravi, Mh. (2015). Numerical lower bound limit analyses of sand heap subjected to basal settlement with hysteretic reversals. Computer

Methods and Recent Advances in Geomechanics: Proceedings of the 14th International Conference of International Association for Computer Methods and Recent Advances in Geomechanics, 2014 (IACMAG 2014),

- [71]Pipatpongsa, Thirapong, Aroonwattanaskul, Krit, & Fang, Kun. (2021). On the progression of slope failures using inverse velocity of surface movements in an undercut slope model. *Understanding and Reducing Landslide Disaster Risk: Volume 4 Testing, Modeling and Risk Assessment 5th*, 293-301.
- [72]Pipatpongsa, Thirapong;. (2022). *Evaluation of Liquefaction Resistance in Undrained Cyclic Triaxial Tests by Using Energy-based Method* 8th Asia Conference on Earthquake Engineering,
- [73]Pipatpongsa, Thirapong; Takeyama, Tomohide. (2010). *Stress Analysis Assuming Fixed Principal Stress Axes on a Planar Slope Formed at the Angle of Repose* 45th Japanese Geotechnical Society (in Japanese),
- [74]Pipatpongsa, Tirapong. (2019). *Experimental and theoretical diagnosis of arch collapse with liquefaction in embankment constructed on soft ground subjected to seismic loading*. <https://kaken.nii.ac.jp/ja/report/KAKENHI-PROJECT-15H04037/15H04037seika/>
- [75]Pourakbar, Mehran, Khosravi, Mohammad, Soroush, Abbas, Hung, Wen-Yi, Hoang, Kien Khai, & Nabizadeh, Ali. (2022). Dynamic Centrifuge Tests to Evaluate the Seismic Performance of an Embankment Resting on Liquefiable Ground Improved by Unreinforced and Reinforced Soil–Cement Columns. *Journal of Geotechnical and Geoenvironmental Engineering*, 148(12), 04022106.
- [76]Pramaditya, Avantio, & Fathani, Teuku Faisal. (2020). Physical Modelling of Earthquake-induced Liquefaction on Uniform Soil Deposit and Settlement of Earth Structures. *J. Civ. Eng. Forum*,
- [77]Pytlik, Andrzej. (2019). Comparative bench testing of steel arch support systems with and without rock bolt reinforcements. *Archives of Mining Sciences*, 64(4), 747-764.
- [78]Rabcewicz, Lv. (1964). The new Austrian tunnelling method. *Water power*.
- [79]Rawat, S, & Gupta, Ak. (2016). An experimental and analytical study of slope stability by soil nailing. *Electron J Geotech Eng*, 21(17), 5577-5597.
- [80]Saade, Chedid, Li, Zheng, Escoffier, Sandra, & Thorel, Luc. (2023). Centrifuge and numerical modeling of the behavior of homogeneous embankment on liquefiable soil subjected to dynamic excitation. *Soil Dynamics and Earthquake Engineering*, 172, 107999.
- [81]Sasaki, Y. (1994). Embankment failure caused by the Kushiro-Oki earthquake of January 15, 1993. Performance of Ground and Soil Structures during Earthquakes, 13th Int. Conf. on SMFE,
- [82]Sasaki, Yasushi, Towhata, Ikuo, Miyamoto, Kenya, Shirato, Masami, Narita, Akiyoshi, Sasaki, Tetsuya, & Sako, Shunsuke. (2012). Reconnaissance report on damage in and around river

levees caused by the 2011 off the Pacific coast of Tohoku earthquake. *Soils and Foundations*, 52(5), 1016-1032.

- [83]Seed, H Bolton. (1979). Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. *Journal of the geotechnical engineering division*, 105(2), 201-255.
- [84]Seed, H Bolton, & Idriss, Izzat M. (1967). Analysis of soil liquefaction: Niigata earthquake. *Journal of the Soil Mechanics and Foundations Division*, 93(3), 83-108.
- [85]Seed, H Bolton, & Idriss, Izzat M. (1971). Simplified procedure for evaluating soil liquefaction potential. *Journal of the Soil Mechanics and Foundations Division*, 97(9), 1249-1273.
- [86]Shahbodagh, B, Sadeghi, H, Kimoto, S, & Oka, F. (2020). Large deformation and failure analysis of river embankments subjected to seismic loading. *Acta Geotechnica*, 15(6), 1381-1408.
- [87]Simcock, Kevin J, Davis, Robert O, Berrill, John B, & Mullenger, George. (1983). Cyclic triaxial tests with continuous measurement of dissipated energy. *Geotechnical Testing Journal*, 6(1), 35-39.
- [88]Terzaghi, K. (1936). Stress distribution in dry and saturated sand above a yielding trap-door. *Proc. 1st Int. Conf. on Soil Mechanics*, 1, 307-311.
<https://cir.nii.ac.jp/crid/1573668924473309568>
- [89]Thompson, Mj, & White, Dj. (2006). Design of slope reinforcement with small-diameter piles. In *Advances in Earth Structures: Research to Practice* (pp. 67-73).
- [90]Tiznado, Juan Carlos, Dashti, Shideh, Ledezma, Christian, Wham, Brad P, & Badanagki, Mahir. (2020). Performance of embankments on liquefiable soils improved with dense granular columns: Observations from case histories and centrifuge experiments. *Journal of Geotechnical and Geoenvironmental Engineering*, 146(9), 04020073.
- [91]Towhata, I. (2008). History of geotechnical earthquake engineering in Japan. The 14th World Conference on Earthquake Engineering, October, Beijing, China,
- [92]Towhata, Ikuo, & Ishihara, Kenji. (1985). Shear work and pore water pressure in undrained shear. *Soils and Foundations*, 25(3), 73-84.
- [93]Townsend, Fc. (1978). A review of factors affecting cyclic triaxial tests. *Dynamic geotechnical testing*.
- [94]Trollope, David Hugh. (1956). The stability of wedges of granular materials.
- [95]Trollope, Dh, & Burman, Bc. (1980). Physical and numerical experiments with granular wedges. *Géotechnique*, 30(2), 137-157.
- [96]Wand, Matt P, & Jones, M Chris. (1994). *Kernel smoothing*. CRC press.
- [97]Wang, Liping P, & Zhang, Ga. (2014). Centrifuge model test study on pile reinforcement behavior of cohesive soil slopes under earthquake conditions. *Landslides*, 11, 213-223.

- [98]Wang, Liping, Zhang, Ga, & Zhang, Jian-Min. (2010). Nail reinforcement mechanism of cohesive soil slopes under earthquake conditions. *Soils and Foundations*, 50(4), 459-469.
- [99]Wanjala, Samuel Comfort, Pipatpongsa, Thirapong, & Nguyen, Tan. (2020). Experimental realization of incipient active failure in sand heap by seismic loading. *Granular Matter*, 22, 1-16.
- [100]Wittmer, Jp, Cates, Me, & Claudin, Ph. (1997). Stress propagation and arching in static sandpiles. *Journal de Physique I*, 7(1), 39-80.
- [101]Wittmer, Jp, Claudin, P, Cates, Me, & Bouchaud, J-P. (1996). An explanation for the central stress minimum in sand piles. *Nature*, 382(6589), 336-338.
- [102]Yamada, Goji. (1966). Damage to Earth Structures and Foundations by the Niigata Earthquake June 16, 1964, in JNR. *Soils and Foundations*, 6(1), 1-13.
- [103]Yamagami, Takuo, Jiang, Jing-Cai, & Ueno, Katsutoshi. (2000). A limit equilibrium stability analysis of slopes with stabilizing piles. In *Slope Stability 2000* (pp. 343-354).
- [104]Yamamoto, Minoru, & Ohno, Kazuaki. (1985). BEHAVIOR OF ROCKBOLTS FOUND IN THE EXPERIMENTS OF TWO LOWERING PANELS. *Doboku Gakkai Ronbunshu*, 1985(364), 143-152.
- [105]Yoshimi, Yoshiaki. (1967). An experimental study of liquefaction of saturated sands. *Soils and Foundations*, 7(2), 20-32.
- [106]Yun-Min, Chen, Wei-Ping, Cao, & Ren-Peng, Chen. (2008). An experimental investigation of soil arching within basal reinforced and unreinforced piled embankments. *Geotextiles and Geomembranes*, 26(2), 164-174.
- [107]Zeghal, Mourad, Elgamal, Ahmed-W, Zeng, Xiangwu, & Arulmoli, Kandiah. (1999). Mechanism of liquefaction response in sand-silt dynamic centrifuge tests. *Soil Dynamics and Earthquake Engineering*, 18(1), 71-85.
- [108]Zhang, Chengcheng, Liu, Guanshi, Tian, Shengkui, & Cai, Mingxuan. (2023). Passive Soil Arching Effect in Aeolian Sand Backfills for Grillage Foundation. *Sensors*, 23(19), 8098.