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LIQUEFACTION-INDUCED LARGE GROUND DISPLACEMENTS: PART 1 - MECHANISM AND GENERAL CHARACTERISTICS

Rolando P. Orense, Dr. Eng.
Research Engineer
Kiso-jiban Consultants Co., Ltd.
Tokyo, Japan

ABSTRACT

This paper deals with the nature and mechanism of liquefaction-induced large permanent ground displacements. The study is carried out by first reviewing several historical earthquakes in which permanent ground displacements induced by soils liquefaction have been documented. The information obtained is supplemented by the results of small and large scale shaking table tests, and the magnitude and pattern of ground movement are analyzed in relation to geomorphological and topographical factors. The findings indicate that liquefied ground moves from higher elevation to lower elevation suggesting the effect of gravity, while large shear distortions occur in the liquefied soil. Displacements are observed to be maximum at the top of the slope and minimum at the bottom of the slope. The patterns of lateral displacements are strongly influenced by the topography of the area. Seismic acceleration affects the permanent displacements of the liquefied soil only indirectly. Based on these observations, it is suggested that the liquefied soil undergoing large displacements behaves like liquid. Consequently, the idea can be used to predict the extent of permanent displacements of liquefied ground.

INTRODUCTION

Permanent ground displacements due to liquefaction-induced lateral spreading of loose saturated sand deposits have been identified as major hazards to lifeline systems and urban facilities. A lateral spread typically involves extensional movement of large superficial blocks of soil as a consequence of liquefaction and transient loss of strength in subsurface layer. Because this type of ground failure involves the movement of competent soil, full passive soil pressure can be mobilized against an underground structure. Pipelines, bridges, buildings and other civil engineering structures on or within the extensional zone are typically cracked or torn apart. Structures located near the margins of the lateral spread are sheared and offset by differential movement. Structures at the toe are commonly compressed or buckled. Due to the fact that they develop on slopes as gentle as 0.30°, the locations of these permanent displacements are difficult to identify. In the current earthquake resistant design of structures, permanent ground displacements are hardly taken into consideration. This can be traced to the fact that the causal relation between permanent ground displacement and damage has not been sufficiently clarified for practical application and the magnitude and spatial distribution of permanent ground displacement has been considered to be a difficult task.

However, in designing important civil engineering structures on soils vulnerable to liquefaction during earthquakes, it is inevitable to consider the effects of liquefaction on the seismic stability of the structure. Clearly, engineering measures to mitigate the effect of liquefaction must include the following: first, an evaluation of the susceptibility of the site to liquefaction; and second, an assessment of the pattern of ground movement and soil failure resulting from liquefaction. The identification of zones of potentially large ground movements and the estimation of displacement patterns can be used to limit failures in lifeline systems through improved siting and design of future facilities and the modification of existing ones.

At present, there are two research subjects on liquefaction-induced ground displacements. One is the investigation of the mechanism of the occurrence of large ground displacements of several meters, and the other is to quantify the effect of such displacements on buried structures and lifeline facilities.

Regarding the mechanism of large ground displacements, several key issues require clarifications (Dobry, 1992). The first is the relative importance of seismic inertia force on the magnitude, distribution and direction of ground deformation. Second is the importance of delayed (post-seismic) deformation occurring after the end of shaking. Third is the nature of soil in its liquefied state. And finally, it is not clear yet whether the in-situ ground displacement is generated sliding along a failure surface or is induced by distributed shear strain within the liquefied layer.

This study describes the mechanism of occurrence of liquefaction-induced large ground displacements. Several historical earthquakes where permanent ground displacements have been documented are analyzed with emphasis on the relation between the geological and geographical conditions. In addition, shaking table tests are conducted to reproduce the phenomenon and to investigate the mechanism. From the information obtained, the mechanisms of permanent ground displacements are clarified, and these observations can serve as a basis for the development of a model to predict the magnitude and distribution of liquefaction-induced ground displacements.

CASE HISTORIES OF LIQUEFACTION-INDUCED GROUND DISPLACEMENTS

Case histories of liquefaction-induced ground displacements during past earthquakes are important ingredients in understanding ground failure processes and the factors controlling these processes. Noteworthy is the fact that they also assist in the development of analytical models for predicting ground displacements and they provide sufficient data to be used in the verification of these models. In addition, the relation between the geological and soil conditions and the resulting displacements can be clearly evaluated.

For this purpose, three major earthquakes where large ground displacements have been observed are reviewed. These include the 1964 Niigata (Japan) earthquake, the 1983 Nihonkai-

Chubu (Japan) earthquake and the recent 1990 Luzon (Philippines) earthquake. Special emphasis is focused on the role of geological and topographical factors in bringing about liquefaction-induced ground displacements, and on the influence of these displacements on lifeline facilities.

1964 Niigata Earthquake

On June 16, 1964, an earthquake of magnitude 7.5 shook Niigata Prefecture, Japan. The earthquake caused extensive damage to various engineering structures such as buildings, bridges, harbors, river dikes and lifeline facilities. The maximum recorded acceleration in the area was 0.16 g, and the water table was about 1.5 m below the ground surface in the area of heaviest damage. It was clear that most of the damage in the area was not caused by the ground motion but by the liquefaction of the ground. It was reported that liquefaction occurred in reclaimed former channels of the Shinano and Tsusen rivers (Katayama et al., 1966).

Measurements of permanent ground displacements were conducted by Hamada et al. (1986) by comparing aerial photographs taken before and after the earthquake. Their studies revealed that large lateral spreading occurred in Niigata City, with displacements as large as 8.5 m in the area of Hakusan power station and 8.8 m on the left bank near Bandai Bridge. Figure 1 shows the measured permanent displacements along Shinano River, including locations of ground fissures and sand boils. It can be observed that the directions of the displacement vectors are almost perpendicular to the river.

It was also noted that along Shinano River, the location of the former river channel mostly coincided with the areas where large ground displacements occurred. Ground fissures were observed along the river, most of which were parallel to the revetments and were generally perpendicular to ground displacements. Studies also showed that the width of the river was reduced by around 7 to 23 m, and these measured contractions coincided quantitatively with the permanent ground displacements measured on the river banks (Hamada et al., 1986).

Investigations on the soil profile revealed that although the terrain was generally flat, the lower boundary of the liquefied layer was inclined towards the center of Shinano River from both banks, and that this inclination of the boundary had some effects on the occurrence of ground displacements towards the river. The estimated liquefiable layer was very thick, generally about 10 m.

In addition, it was observed that in areas where horizontal ground displacements occurred, the ground surface subsided and many ground fissures were noted due to the tensile strain in the ground. In contrast, heaving was observed in areas where the displacements terminated (Hamada, 1992).

During the said earthquake, the Showa Bridge collapsed. According to eye witnesses, the bridge collapsed a few after the quake had ended (Horii, 1968). Similar phenomenon was observed for the revetment at the bank of Showa bridge (Public Works Research Institute, 1965). These clearly show that the inertia forces associated with the earthquake motion is not the direct cause of the collapse of these structures.

Another interesting fact is the result of an investigation regarding the concrete piles of NHK building which were excavated twenty years after the earthquake (Kawamura et al., 1985). The piles, shown in Figure 2, were found to be broken in two positions, which coincided with the boundaries of the liquefied portion. This suggests that the piles were broken at the boundaries due to stress concentration resulting from liquefaction-induced ground displacements. From the damage, the horizontal deformation of the piles was estimated to be between 1.0 and 1.2 m, which is in good agreement with the measured ground displacement in the area. Moreover, the direction of the ground displacement is almost the same as that of the pile deformation. Thus, it is clear that permanent ground displacements were the cause of the damage to the piles.

1983 Nihonkai-Chubu Earthquake

The May 26, 1983 Nihonkai-Chubu Earthquake had a magnitude of 7.7 and caused extensive damage to the coastal area of the Tohoku region. The maximum horizontal ground acceleration was 200 gals in Akita City. One of the most severely damaged areas was Noshiro City, which is built on the sand dunes along Japan Sea coast and the alluvial plain of Yoneshiro River. Houses, buildings, and lifeline facilities were damaged. Numerous cracks on gentle slopes indicated the occurrence of lateral sliding due to liquefaction.

The permanent horizontal ground displacements measured in Noshiro City are illustrated in Figure 3. It can be seen that large ground displacements occurred along the gentle slopes of the sand dunes (Hamada et al., 1986). The maximum horizontal displacement exceeded 5 m and the direction of the displacement was almost parallel to that of the slope. The observed displacements become smaller near the toe of the slope where the liquefied layers become thinner and the ground surface is flat.

In addition, the upper sections of the slopes had dominant tensile strains while compressive strains were notable in the lower portion. Ground fissures were found on the upper part of the northern slope of Mae Hill, where large tensile ground strains occurred because of horizontal ground displacements. It was also observed that ground failures were concentrated in areas of large permanent ground displacements. Moreover, the cracks were almost perpendicular to the direction of the horizontal permanent ground displacements.

Figure 4 illustrates both horizontal and vertical movements in the north slope of Mae Hill. It can be seen that subsidence is predominant in the higher elevation while heaving occurred near the foot of the slope. The vertical displacement is much smaller than the horizontal component.

Subsurface investigations revealed that the liquefied soil layers with a total thickness of 2 to 5 m underlay the gently sloping ground surface where the average gradient is less than 5%. Correlations performed by Hamada et al. (1986) showed that the magnitude of permanent ground displacement is larger where the surface gradient is large and/or the liquefiable soil layer is thick.

1990 Philippines Earthquake

An earthquake of magnitude 7.8 occurred in the Philippines on July 16, 1990, and this caused widespread damage to buildings and civil engineering structures located in the central and northwestern parts of Luzon Island. Among the affected areas, Dagupan City suffered the most damage due to the liquefaction of loose saturated sand deposit induced by the strong and prolonged shaking of the ground.

Figure 5 shows the approximate extent of liquefaction in Dagupan City, as indicated by the location of sand boils and fissures. The lateral displacements indicated in the figure were not actually measured due to the poor weather condition. Values were based on estimates from field surveys and interviews with the residents.

Lateral spreading occurred along the banks of the Pantal River. Large lateral displacements and flow failures, indicated by fissures parallel to the river, accounted for the damage to several buildings and other structures built near the river. This was clearly observed in the portion of the river near Magsaysay Bridge where residents noticed that the river width decreased by as much as 10 m after the earthquake. A 3-story R/C hospital building near the river was dragged and tilted toward the river, due to the lateral spreading of the foundation ground.

Figure 6 illustrates the damage to Magsaysay Bridge which collapse as a result of the lateral movement of the opposing banks of Pantal River where the bridge was abutted. At the same time, the bridge piers sank and tilted toward the center of the river brought about by the liquefaction of the river bed. A similar compressional failure phenomenon was observed when a buried water pipe along Rizal Street buckled and was thrusted upward.

Subsurface investigations revealed that the thickness of the liquefied layer was in the order of 10 m. In addition, the unliquefied layer above it was thin, being less than 3 m deep in most areas. The existence of a thick liquefied layer and a thin non-liquefied surface layer, in addition to inadequate rigid river protection, is the major reason why significantly large displacements occurred.

As in the case of Niigata City, sedimentologic factors, such as presence of loose sands deposited in abandoned river meander, may have controlled the areal distribution of liquefaction and the associated ground displacements in Dagupan City. The lateral spreading toward the center of Pantal River is similar to that which occurred in Niigata City during the 1964 earthquake which was discussed earlier. It can therefore be concluded that the lateral spreading in Dagupan City is most likely the result of mechanism similar to those identified for the earlier earthquakes.

SHAKING TABLE TESTS

Shaking table tests have been performed to investigate liquefaction-induced permanent displacements. An advantage of these types of tests is that ground deformation can be observed during testing. However, they have their own shortcomings. One problem is the effect of boundary walls. In actual ground, boundary walls do not exist so that measurements near the walls are not very reliable. To minimize wall effects, hinge or flexible boundaries have been attempted. Another problem is the scale effect. Since the depth of the model ground is relatively shallow, the excess pore pressure developed during shaking dissipates very quickly. As a consequence, the duration time of liquefaction in the model ground is much shorter compared to that in-situ. Therefore, the model ground can not deform extensively within a short period of time. Because of this, the shaking table is known to underestimate the deformation of liquefied ground. To solve the problem, the model ground is prepared at the minimum density possible and, in some cases, the duration of shaking is made long enough to induce large deformation of the ground.

Small Scale Shaking Table Tests

Two sets of small scale shaking table tests were carried out at the University of Tokyo. The first set of tests was done in a box measuring 100 cm in length, 40 cm in width and 40 cm in depth. Water pluviation technique was used to obtain a loose saturated sand with 50% relative density. To measure the pattern of subsoil displacement, dyed sand was used to form grid lines on the sides of the transparent wall.

Figure 7 shows a model ground consisting of water-saturated deposit with dry embankment resting on one side. The initial configuration is shown by the dashed lines while that after shaking by the solid lines. A layer of coarse gravel was placed between the ground and embankment in order to keep the embankment dry. After complete liquefaction, the embankment sank, and this squeezed the liquefied soil which caused the ground surface to heave on the left portion of the box.

Figure 8 shows the test results wherein dry sand was underlain by a water-saturated sand. The ground surface was level, while the interface was inclined by about 9 degrees. The two layers were separated by a plastic sheet. Figure 8(a) shows the condition during shaking. It can be seen that although the saturated soil has already liquefied, no appreciable deformation can be observed. This phenomenon suggests that the force equilibrium was maintained in the lower layer without causing any significant soil movement. After sometime, the plastic sheet was broken near the right end of the box, and the ground water and sand started to boil, which triggered a large deformation near the place of boiling as shown in Figure 8(b). Such behavior was noted to be very similar to that of liquid.

These tests indicate that during flow of liquefied ground, substantial shear distortions developed which suggest that liquefied sand has a negligible resistance against shear. In addition, it seems that the flow is governed by the total head gradient, which is defined in terms of total overburden stress and elevation heads.

The second set of experiments were conducted by using a box 100 cm long, 20 cm wide and 40 cm high. In the first case, the model ground was prepared such that the water table was inclined by maintaining continuous water flow. Figure 9(a) illustrates the experimental set-up. This was done so as to simulate the actual condition in-situ where the water table is not generally flat.

The results of the tests are indicated in Figure 9(b). It was noted that lateral displacement was maximum near the ground surface, while negligible at the bottom. In addition, the lateral movement is continuous across the interface of the two materials, indicating that there is no slip at the level of the ground water table.

The second case involved a loose saturated deposit with flat surface and inclined bottom, as shown in Figure 10(a). To prevent sloshing, the frequency of shaking was set at 5 Hz. Figure 10(b) illustrates that the displacements of the liquefied soil were negligible in most part of the ground except near the end walls. Thus, it was concluded that the liquefied ground with level surface does not develop any lateral displacement.

Large Scale Shaking Table Tests

A series of eight large scale shaking table tests was carried out at the Public Works Research Institute (PWRI) by Sasaki et al. (1991) to study the fundamental characteristics of lateral flow of the ground. The model grounds were 6.0 m long, 0.8 m wide and the average height was 1.2 m. The slope of the ground surface and the lower boundary conditions of liquefied layer were varied. A lowermost sandy layer was compacted enough in the container so that it will not liquefy during excitation. The model grounds were shaken in four stages, with the acceleration increasing in each subsequent stage. Vertical marked lines made of white colored sands were installed behind the transparent glass windows to trace the deformation of the layers with depth. In addition, bench mark points were also set-up on the ground surface to measure the change of lateral ground displacement.

Of special interests in this study were the results of the experiments considering Model 6 and Model 8 embankments. In Model 6 test, a gently sloping gravel surcharge was placed over one half the length of the loose saturated deposit. Figure 11 shows the pattern of horizontal ground movement at the end of shaking. It is to be noted that the deformation was again negligible at the bottom and maximum at the top. The maximum lateral displacement was observed to occur in the middle portion. In addition, no slip was observed between the liquefied layer and the dry embankment. Settlement of the embankment occurred, while the toe rose slightly.

Model 8 was prepared as a three-dimensional ground model to clarify the influence of the direction of excitation on the lateral ground flow. In this model, a semi-circular liquefiable deposit of 4 m in diameter and 0.25 m in height was overlain by a cone-shaped gravel embankment with a diameter of 2 m and a height of 0.15 m at the center. Figure 12 shows the direction and the magnitude of ground displacements of the bench mark points which were set on the radial lines after the second stage of shaking. The figure shows that the surface of the semi-cone shaped embankment and the neighboring ground seem to move almost radially, i.e., in the direction of the slope.

Based on their studies, Sasaki et al. (1991) obtained the following conclusions:

1. Lateral displacement of the inclined surface began to increase remarkably when the excess pore water pressure ratio $\blacktriangle u/\delta_v'$in the underlying layer was in range of 0.8-10.

2. The lateral deformation occurred only in the liquefied layers, and the unsaturated surface layer was displaced by the deformation of the underlying liquefied layers.

3. Lateral ground flow stopped after the excitation was terminated.

4. Lateral ground flow was affected significantly by the slope of the ground surface; the slope of the lower boundary of the liquefied layer had no significant effect, except when the ground surface was also inclined.

5. The direction of excitation had no significant influence on the direction of lateral flow of the ground, and the lateral ground deformation was related to the direction of the slope of the surface.

6. The lateral flow of the ground surface was a function of the surface slope, the thickness of perfectly liquefied ground and the duration time of excitation.

GENERAL NATURE OF PERMANENT DISPLACEMENTS

The nature and fundamental characteristics of permanent displacements of liquefied ground will now be discussed in the light of the observations obtained from field experiences and shaking table tests.

Site and Earthquake Parameters

An important area relevant to the clear understanding of the nature of permanent displacement, and consequently to its accurate analytical modeling, relates to site and earthquake parameters. These parameters can be classified into three groups: seismic factors, geological and topographic factors, and material parameters.

(a) Seismic Factors

One of the key factors that needs clarification is the relative importance of gravity and inertia forces during shaking (including the duration of the latter) on the magnitude and spatial distribution of ground deformation. Actual case histories reveal that gravity-controlled driving shear stresses play an important role as the observed lateral displacements are always in the downhill direction of the ground surface or towards a free face. The results of shaking table tests conducted by PWRI clearly illustrate that permanent displacement is caused by gravity force and cyclic acceleration influences the movement only indirectly by triggering liquefaction and in determining the extent of liquefaction. This can be viewed from the fact that shear waves, which cause horizontal displacements, hardly propagate through liquefied layer when the liquefied soil behaves as liquid.

This is not to say that seismic forces do not influence displacement; obviously, there is no displacement without shaking. Ground motion must have sufficient duration and magnitude to generate excess pore water pressure and initiate liquefaction. Increased duration of ground motion results in a greater extent of soil liquefaction and prolongs the time that liquefied soil is subjected to gravitational forces. The duration of strong ground motion increases with the earthquake magnitude, and increases slightly with the distance from the zone of energy release.

(b) Geologic and Topographical Factors

Case histories also reveal that surficial geology and the depth of water table strongly influence the magnitude and distribution of permanent ground displacements. In flood plain deposits, liquefaction-induced ground failures were widespread; on the contrary, structures located in bedrock received little or no damage from lateral spread.

Based on studies made by Hamada et al. (1986), the types of permanent ground displacements caused by soil liquefaction can be summarized as shown in Figure 13. Case A depicts the type of displacements that occurred in Noshiro City, where the ground surface is slightly inclined and an unliquefied layer exists along the surface. Case B shows one where the ground terrain is flat on the surface but has an abrupt discontinuity on revetments of the river, and the lower boundary of the liquefied layer is inclined towards the river center. This type of topography was prevalent in Niigata City and Dagupan City. Case C, also observed in Niigata City, is the case where the ground surface is almost horizontal but the lower boundary of the liquefied layer is inclined.

Permanent ground displacements are also influenced by the total thickness of the liquefiable layer. This was clearly observed from shaking table tests and in the results of studies in Niigata City and Noshiro City conducted by Hamada et al. (1986). Another parameter is the slope of the ground surface. Once a layer has liquefied, the component of gravity acting parallel to the shear zone becomes a driving force for movement in the downslope direction. It is therefore apparent that there is a direct relation between the slope of the ground surface (or of the bottom of the liquefiable layer) and the resulting permanent ground displacements.

The presence of surface unliquefied layer also influences the surface manifestation of soil liquefaction and the corresponding displacement. Shaking table tests show that the surface layer overlying the liquefied subsoil does not develop significant shear deformation, and the axial compression in the lateral direction is the predominant mode of distortion. In addition, case histories also show that compression is predominant in the lower portion of the slope while tensile behavior is important in the upper portion. This tensile state is manifested in terms of surface cracks and fissures, which occurred in-situ near the top of the slope as observed in Noshiro City (see Figure 3).

The proximity of the area to a free surface and the existence of structures on the surface should also be considered. Because of lack of restraining forces, lateral displacements are generally larger near a free face (e.g., river channel) than in regions far from the said boundary as noted in Niigata City (see Figure 1). Thus, the soil movement depends not only on the slope and stratification local to the point of interest but on the overall topography of the study area.

Drainage boundary and characteristics (e.g., permeability, existence of reservoir, gravel drains) and initial hydrological conditions may also affect the resulting permanent ground displacements.

(c) Material Parameters

Studies have shown that permanent displacements are strongly influenced by soil parameters, such as shear modulus and other stress-strain parameters, void ratio and relative density, fabric and structure, permeability, viscosity, residual strength and other strength properties (for unliquefied soils).

In addition, the age of sediment, as well as the degree of consolidation and cementation affects the susceptibility of soil to liquefaction and the associated displacements. Grain size distribution apparently controls the dissipation of excess pore pressure and, consequently, the length of time the soil stays in liquefied state. Layering, such as the presence of less permeable layer above a liquefied layer retards the dissipation of pore pressure. Confinement produced by less permeable layer expedites pore pressure build-up and lengthens the time the soil remains liquefied and subjected to displacement. Analyses show that silty soils may not displace as much as clean sand (Bartlett and Youd, 1992) while clayey soils have more resistance to liquefaction because of higher plasticity (Seed et al., 1983).

Mode of Failure

It is not clear yet whether the in-situ ground displacement is generated by a discontinuous movement of soil mass along a failure surface, or is induced by a large deformation of liquefied sand with distributed shear strain within the layer. The Committee on Earthquake Engineering (1985) has recognized that the former can occur in actual case through the Mechanism B type of failure where the presence of liquid film at the interface between the impervious layer and liquefied layer can cause the deposit to loosen up so that the steady state strength becomes less than the static shear stress that must be sustained. On the other hand, the results of shaking table tests support the latter, which indicate that the flow of liquefied ground involves voluminal transfer of materials. Observations from case studies such as displacements at the top of slopes being greater than those at the foot of the slope and subsidence being predominant near the top of slope while heaving occurs in the lower portion suggest the absence of well-defined slip plane. One may speak rather of a boundary between the moving and stationary mass. Shaking table tests also reveal that the magnitude of the lateral displacement in liquefied sandy layer is minimum at the base and increases towards the surface, with no slip between the unsaturated surface layer and the liquefied subsoil (see Figures 9 and 11).

One factor which can possibly influence the mode of failure is the magnitude of permanent ground displacement. For flow type of failure, the magnitude of displacement can be as large as several hundreds of meters and, obviously, such flow over large distance is possible only if there is a slip surface through which failure can occur. On the other hand, such slip surface probably does not exist for lateral spreads where the displacements are comparatively of much smaller magnitude. Since the displacement is small, there is no relative displacement at the failure surface (which can be the boundary between the liquefied layer and unliquefied base) and therefore, slipping is not mobilized.

Nature of Liquefied Soil

At present, there are two different (and contradictory) schools of thought about how liquefied soil behave: is it a solid with reduced stiffness or a viscous fluid?

Shaking table tests seem to favor the idea that liquefied soil behaves like liquid, and the flow is governed by the total head gradient as in fluid mechanics. Moreover, in-situ experiences such as sinking of buildings and floating of underground structures such as sewage tanks and pipelines can be explained by considering the liquefied soil as liquid.

On the other hand, undrained shear tests on loose sands, except very loose ones, exhibit a residual strength at large shear distortion. In this large strain level, sand can undergo unlimited deformation under constant effective stress (Poulos, 1981); this shear resistance at large strain level is referred to as steady state strength. Laboratory tests have shown that the steady state strength is highly dependent on void ratio, as shown in Figure 14. It can be seen from this figure that a minor increase in void ratio results in dramatic decrease in the steady state strength. In fact, the in-situ ground may not be under undrained condition, and this variation in void ratio due to water permeation or seepage flow can alter the steady state strength drastically.

It can be noted from the foregoing discussion that the behavior of liquefied soil in-situ is very much different from those in laboratory tests. Not being undrained, seepage forces acts on the soil particles in vertical direction. On the other hand, there is no seepage flow observed in small specimen.

To illustrate this, consider the progress of shear event in small specimen and in in-situ ground. Figure 15(a) refers to small undrained soil sample. Initially, the sand particles are in contact with one another. When cyclic shear is applied, the sand particle contact is broken and the particles are suspended in water, causing the development of 100% pore water pressure and a hydraulic gradient τ' is generated. However, because the sample has high permeability and small size, this hydraulic gradient is dissipated quickly. Thus, the sand particles settle downward and the pore water migrates quickly toward the top of the specimen. In the process, a small sand deposit is created as the reconsolidation process continues. In this micro sand deposit, the particle contact force is small, which is equal to the buoyant weight of sand. This layer is stable and shows dilatant behavior when further sheared, and some residual strength is observed.

On the contrary, the in-situ ground is unstable after 100% development of pore water pressure due to the action of seepage force, as shown in Figure 15(b). When the particles ar suspended in water due to earthquake shaking, seepage occurs immediately. Due to the substantial thickness of the ground, longer time is required for pore water pressure to dissipate. Sand particles start to sink, and when they hit a stable deposit at the bottom, reconsolidation is achieved. This process of reconsolidation proceeds from the bottom to the surface. In the upper portion, the suspended particles are highly unstable, and until reconsolidation occurs, they are susceptible to considerable distortion without much resistance. This suspended state in the upper elevation is maintained by the vertical seepage flow of boiling water is generated by the reconsolidation volume change in sand in the bottom portion. Thus, the in-situ mixture of sand and water behaves like a liquid.

Therefore, one of the major differences between laboratory undrained tests and in-situ liquefaction phenomenon is the existence of seepage force which maintains sand particles in suspension for a sufficient period of time.

CONCLUSIONS

Based on the review of actual case histories and results of shaking table tests, the nature of permanent ground displacements of liquefied soil was clarified, and the factors influencing the magnitude and spatial distribution were identified. The characteristics of permanent ground displacements induced by liquefaction can be summarized by the following observations:

1. The permanent displacement is oriented in the downward direction of a slope, suggesting the influence of gravity. Tension cracks are detected near the top of liquefied slopes in the field after movement, and they are oriented normal to the direction of the ground displacement.

2. Lateral displacements at the top of the slopes are greater than those at the foot of the slopes. In addition, subsidence is predominant near the top of the slope while heaving is the case in the lower portion. The magnitude of the vertical displacement is much smaller than that of the horizontal one.

3. Shaking table tests show that liquefied ground behaves very similar to liquid; hence, its movement is highly affected by the total head gradient which is defined in terms of the total overburden stress and the elevation heads.

4. The magnitude of the lateral displacement in a liquefied sandy layer is minimum at the bottom and increases towards the surface. When partially saturated layer at the surface is present, the lateral displacement is continuous at the interface between this layer and the underlying liquefied layer.

5. Permanent displacement is caused by the gravity force; cyclic acceleration influences the movement only indirectly by triggering liquefaction and in determining the extent of liquefaction.

6. The magnitude and pattern of ground movement depend not only on the slope and stratification local to the point of concern but on the overall topography of the study area.

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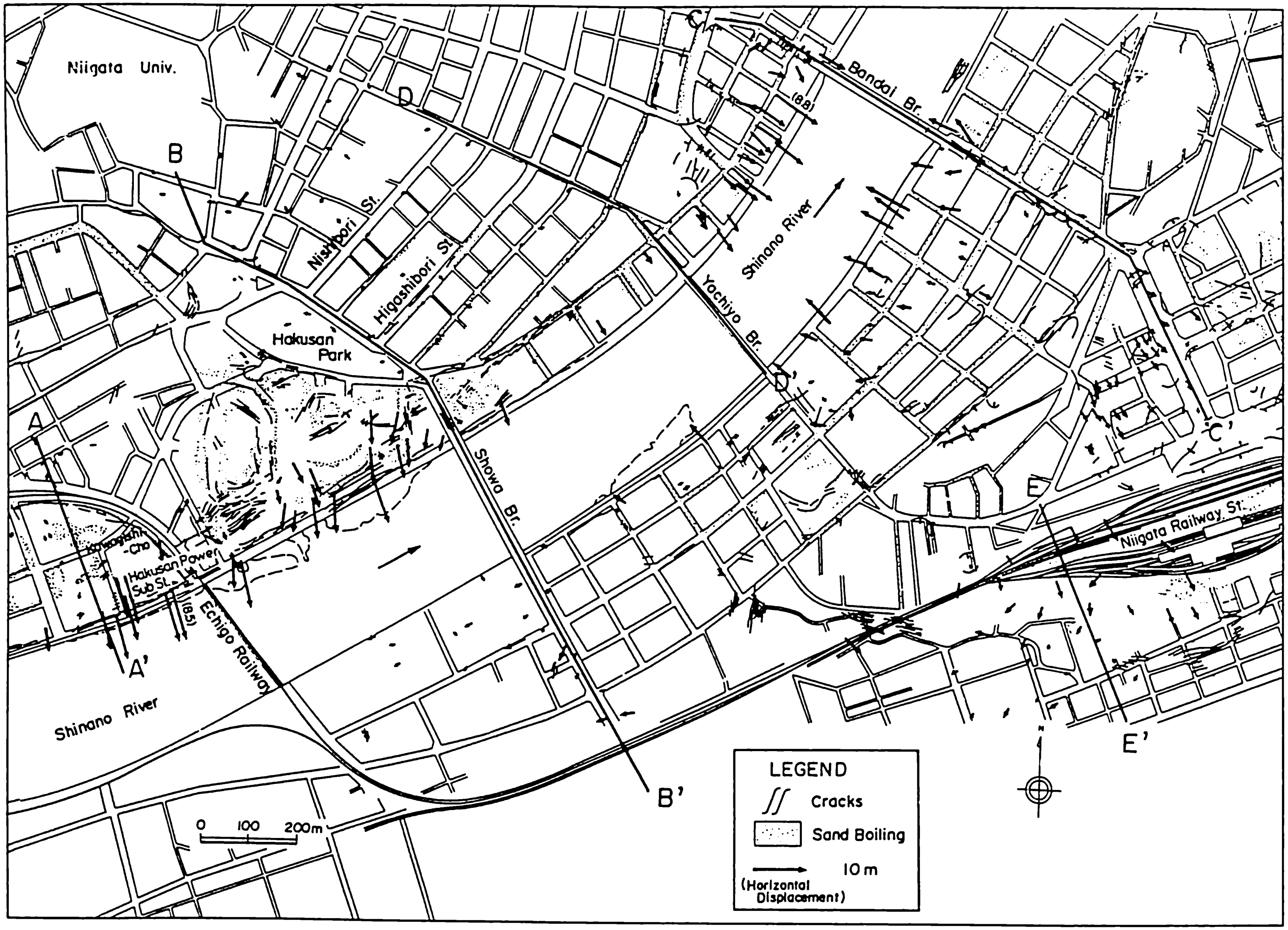

Figure 1: Permanent horizontal ground displacements in Niigata City (after Hamada et al., 1986)

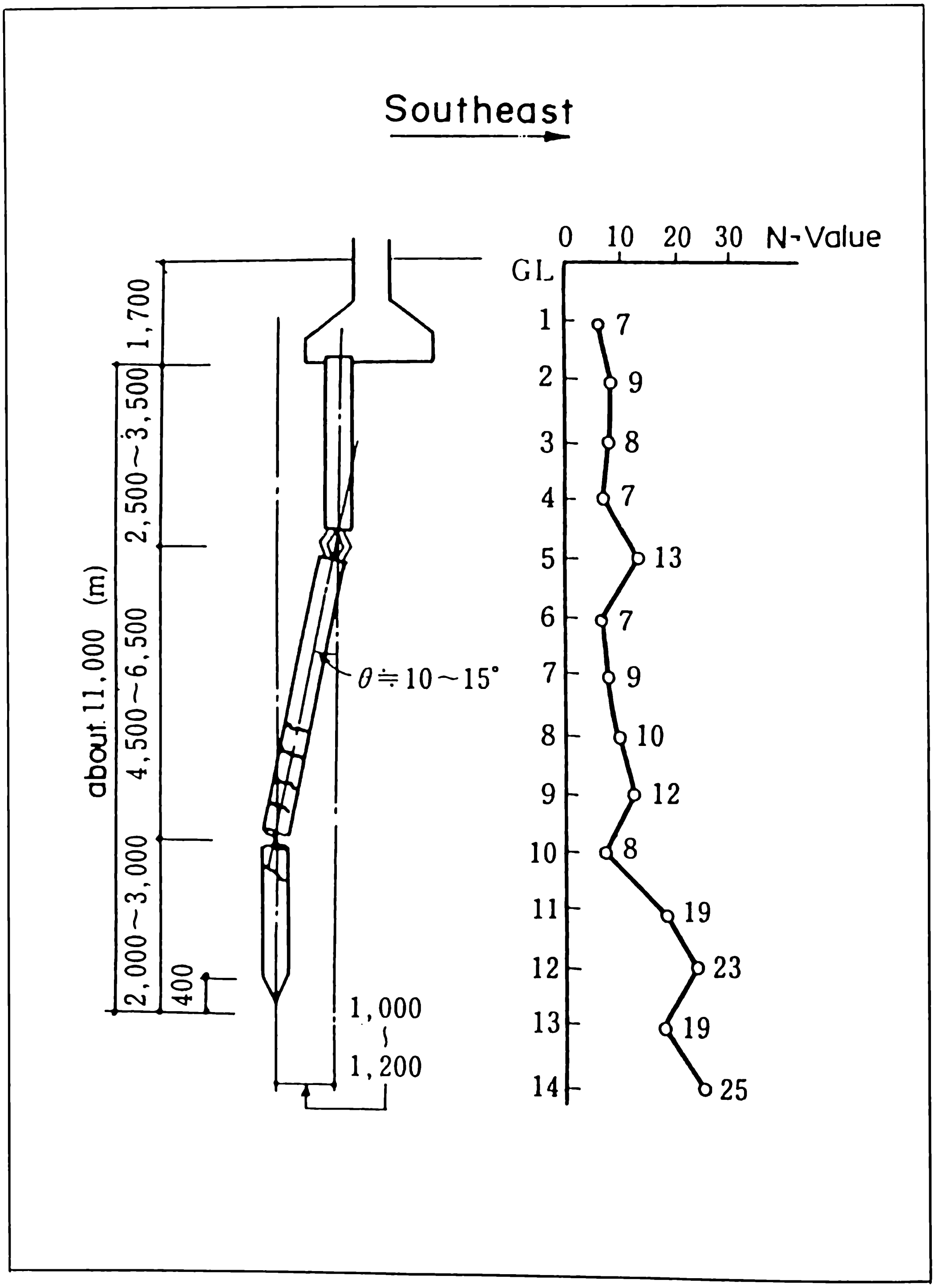

Figure 2: Broken pile in Niigata City (after Kawamura et al., 1985)

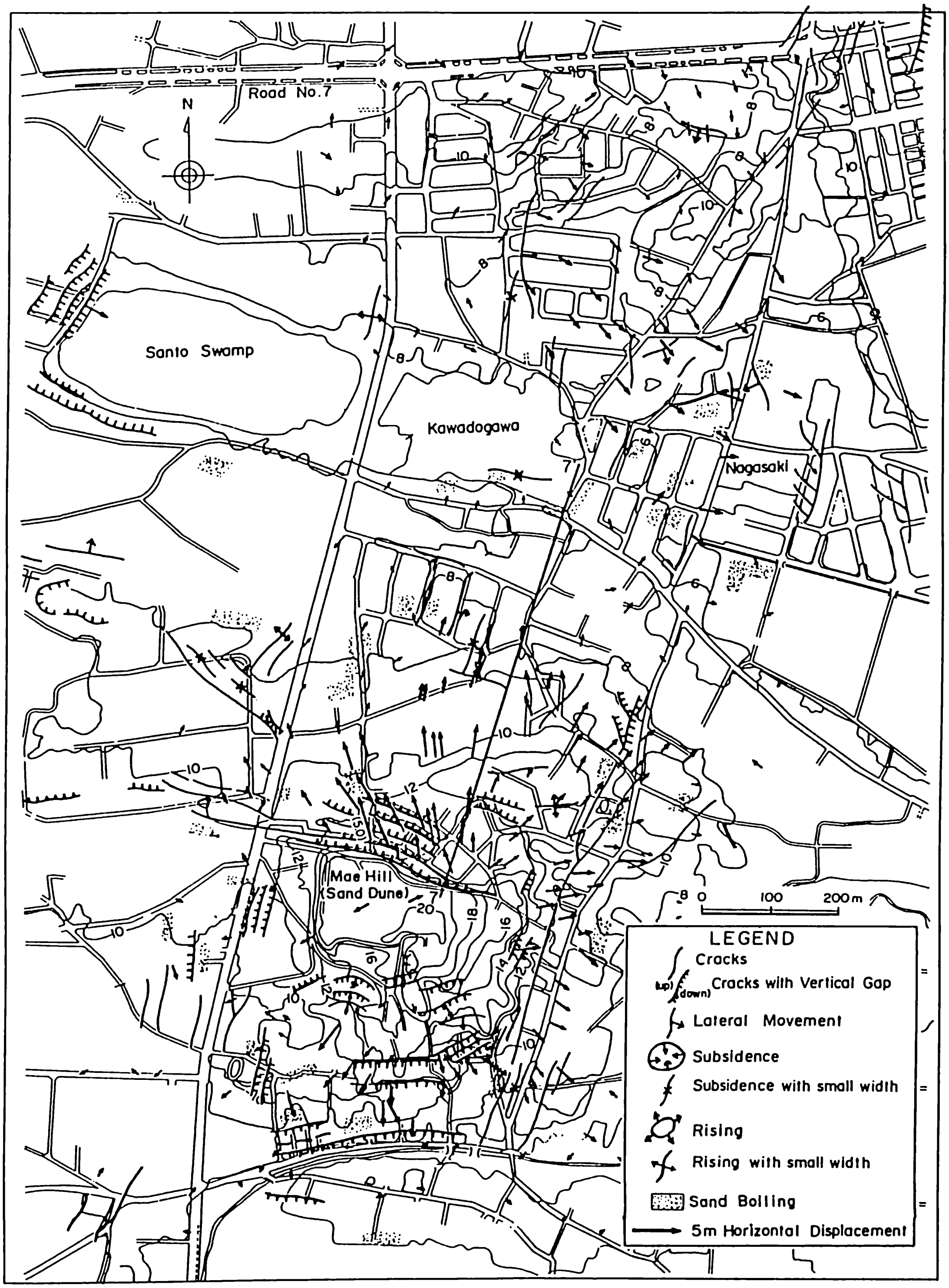

Figure 3: Permanent horizontal ground displacements in Noshiro City (after Hamada et al., 1986)

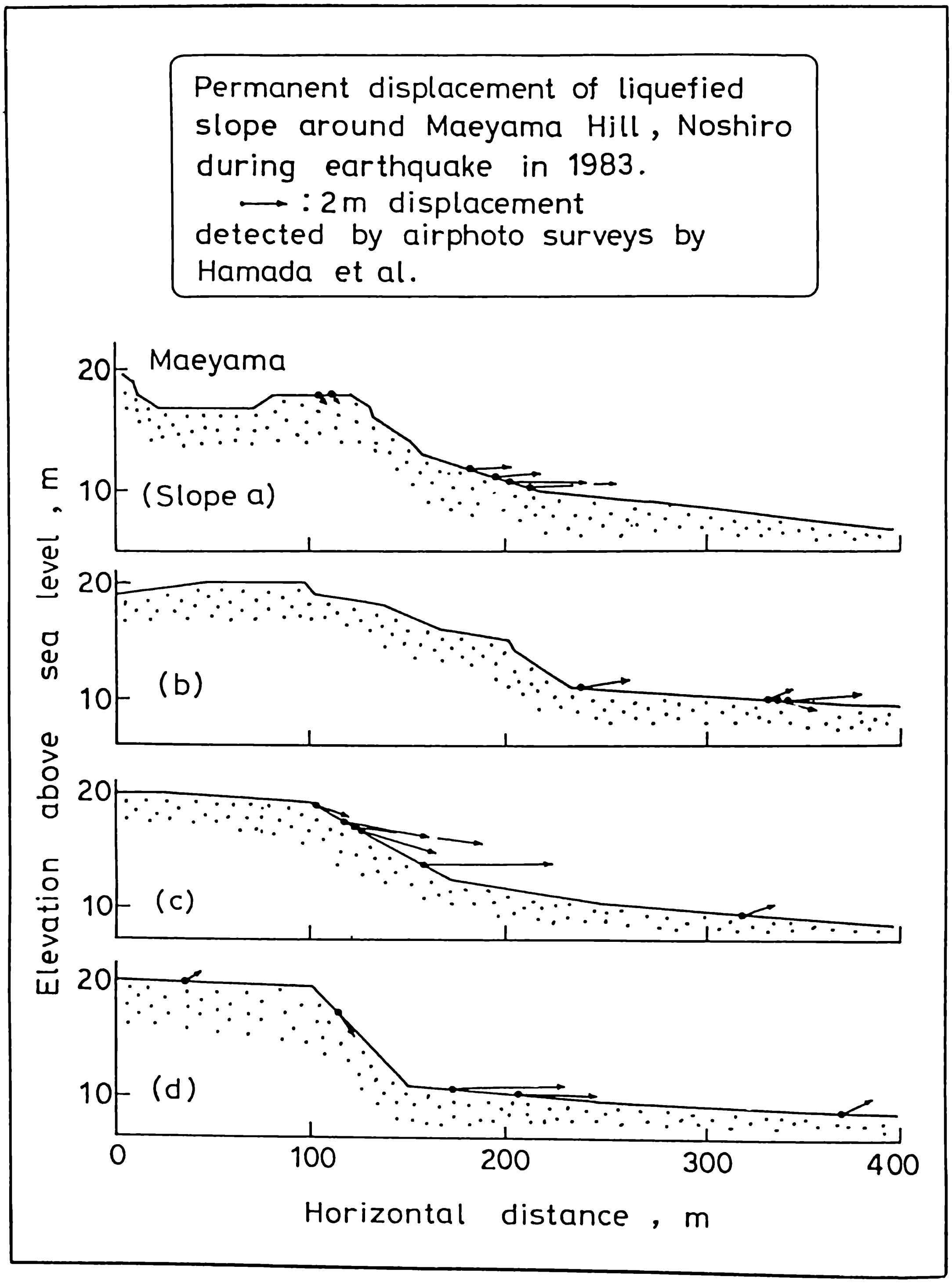

Figure 4: Ground movement in cross sections of Mae Hill slope

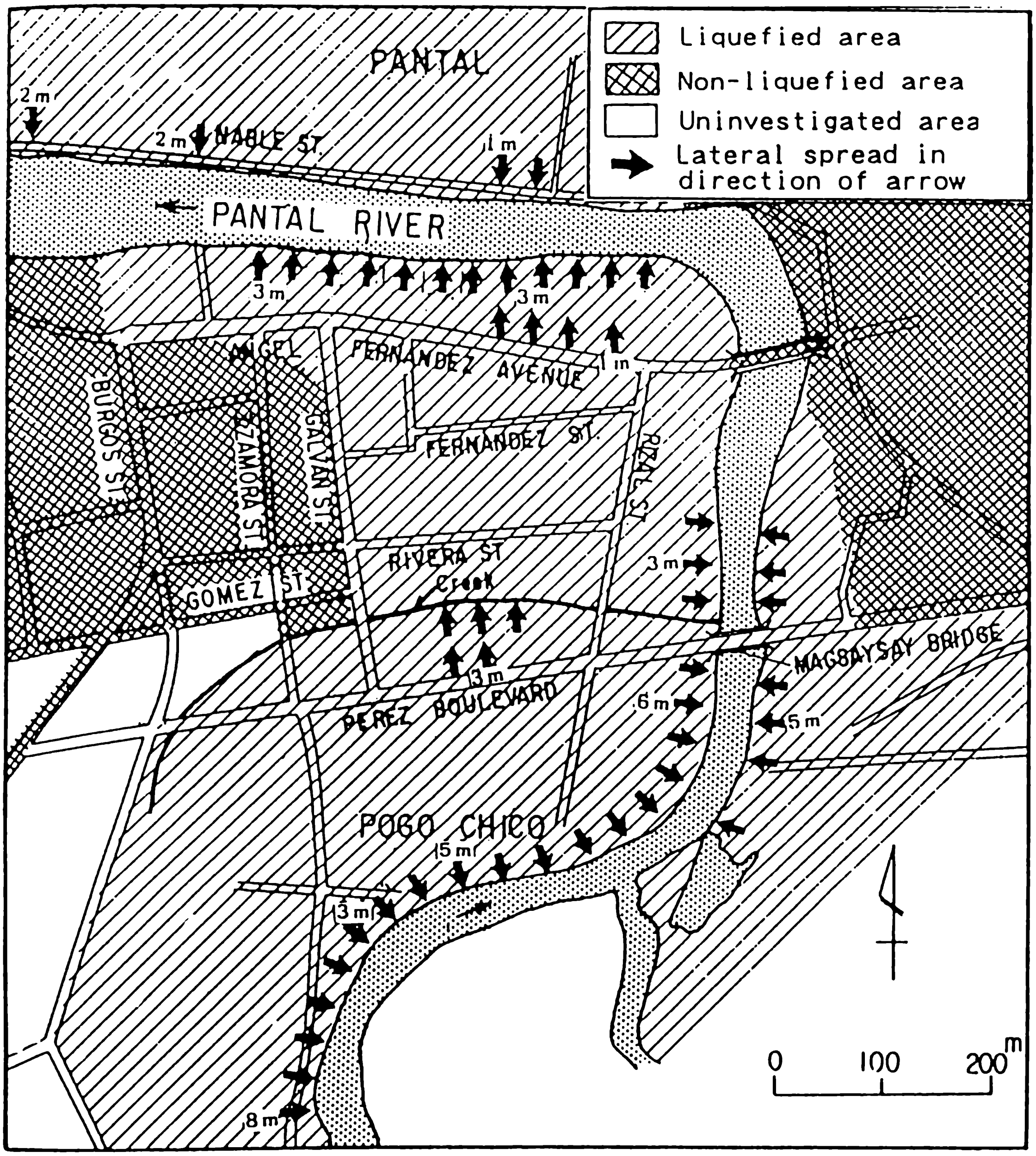

Figure 5: Approximate extent of liquefaction in Dagupan City (after Wakamatsu et al., 1991)

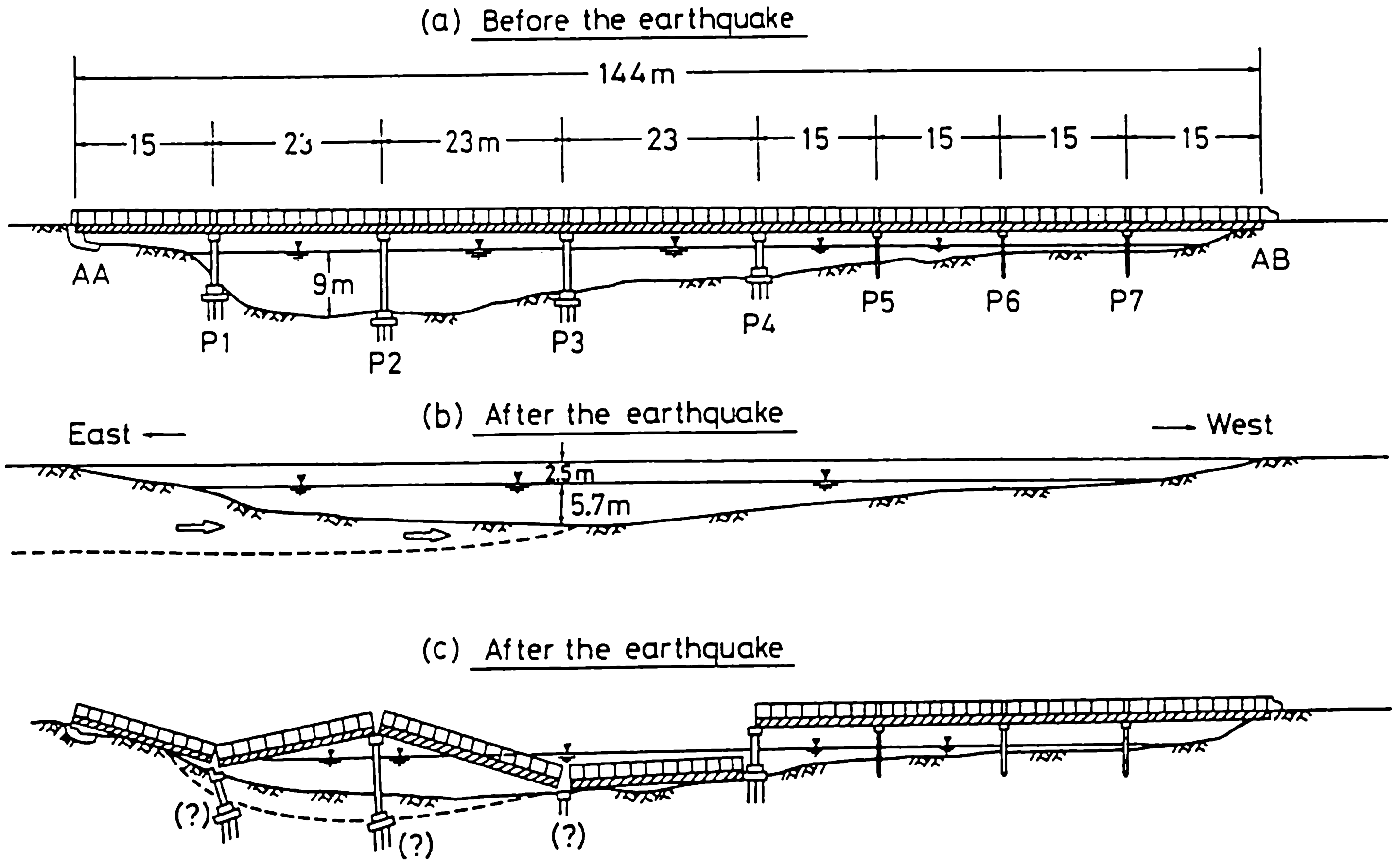

Figure 6: Sketch of Magsaysay Bridge before and after the earthquake (after Ishihara et al., 1993)

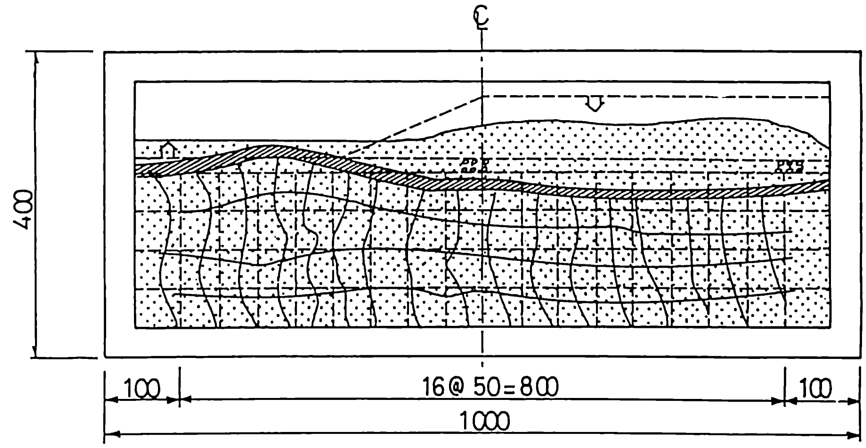

Figure 7: Deformation of model ground with small embankment on the surface

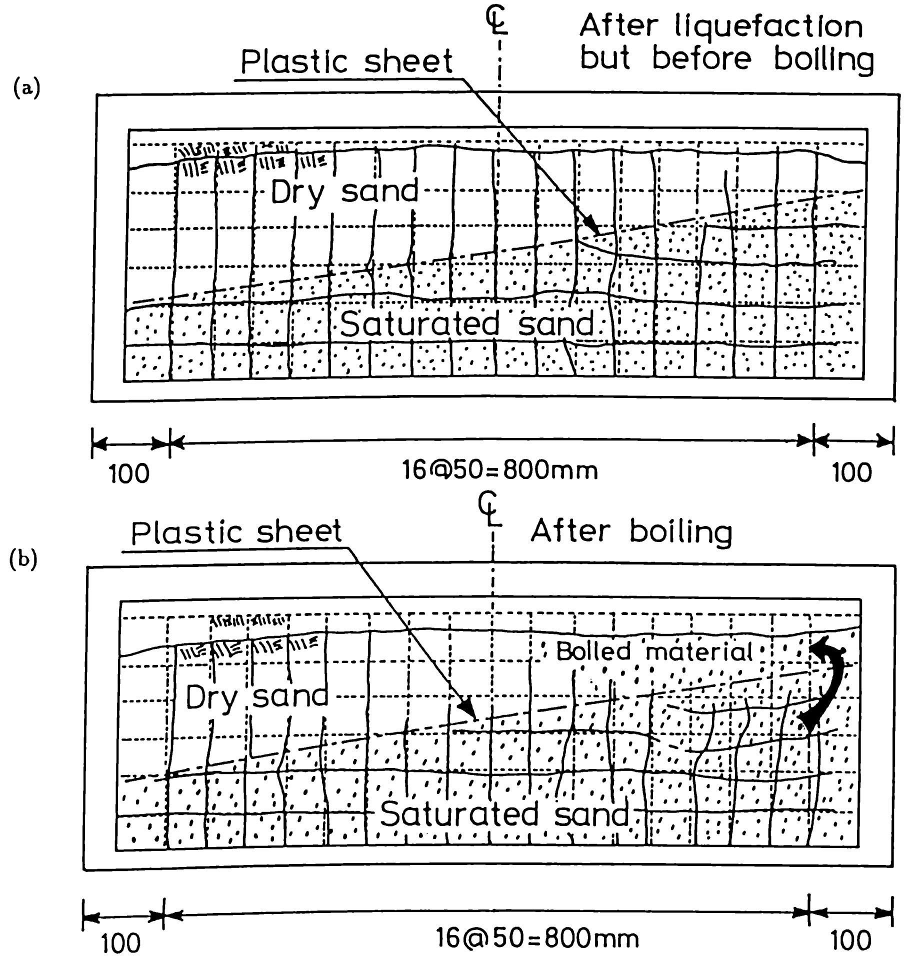

Figure 8: Configuration of model ground in which dry layer was underlain by water saturated one with inclined surface: (a) after liquefaction but before boiling; (b) after boiling

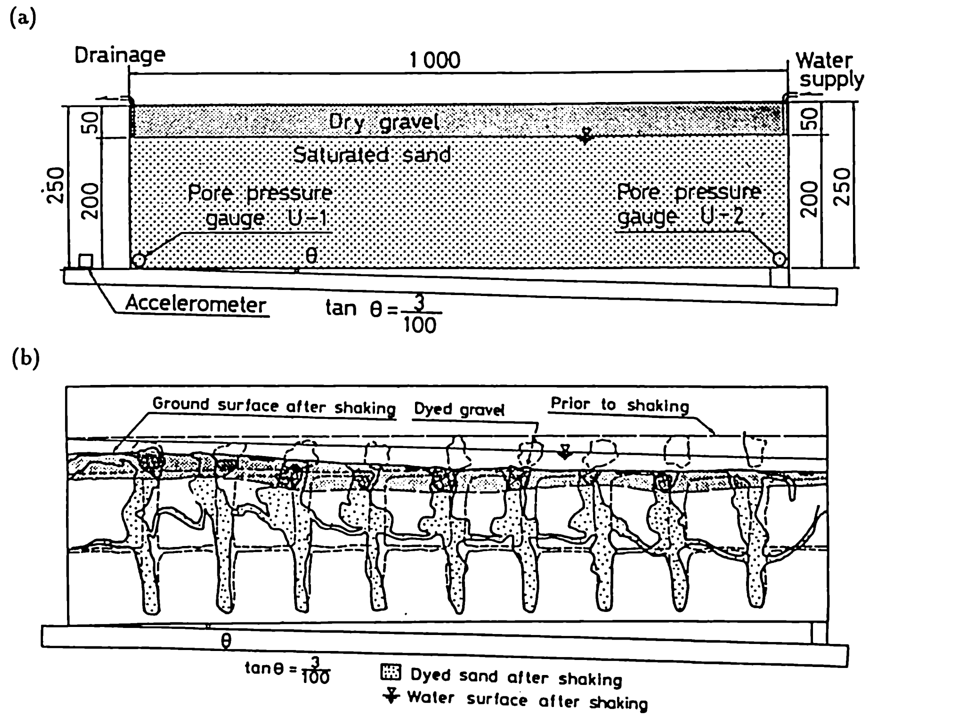

Figure 9: Model ground with inclined water table: (a) experimental set-up; (b) ground deformation

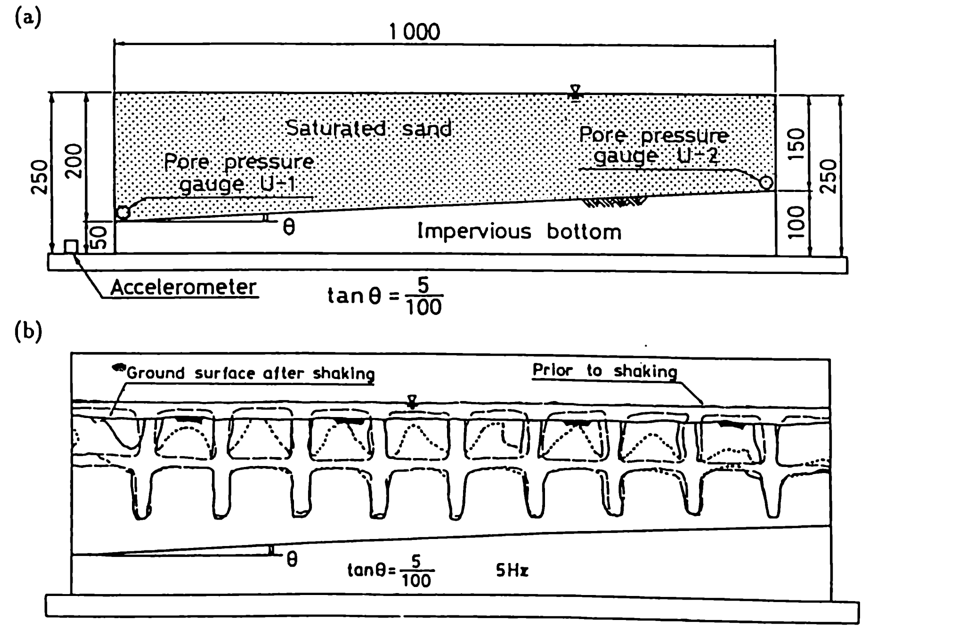

Figure 10: Model ground with flat surface and inclined (a) experimental set-up; (b) ground deformation

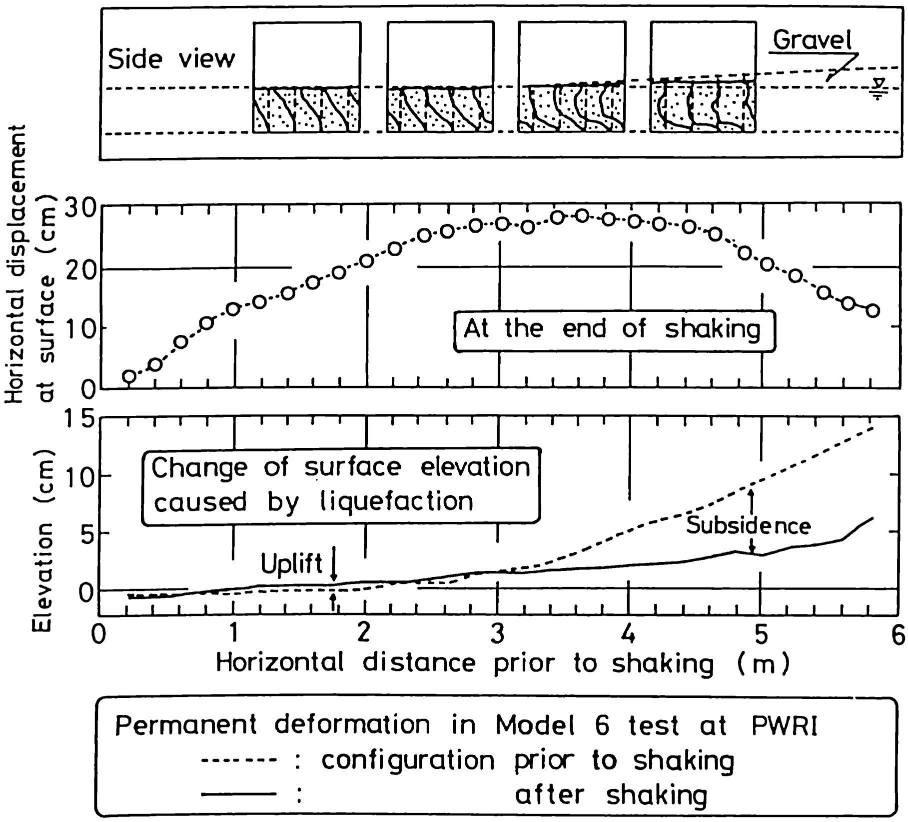

Figure 11: Permanent displacement of Model 6 test at PWRI (after Sasaki et al., 1991)

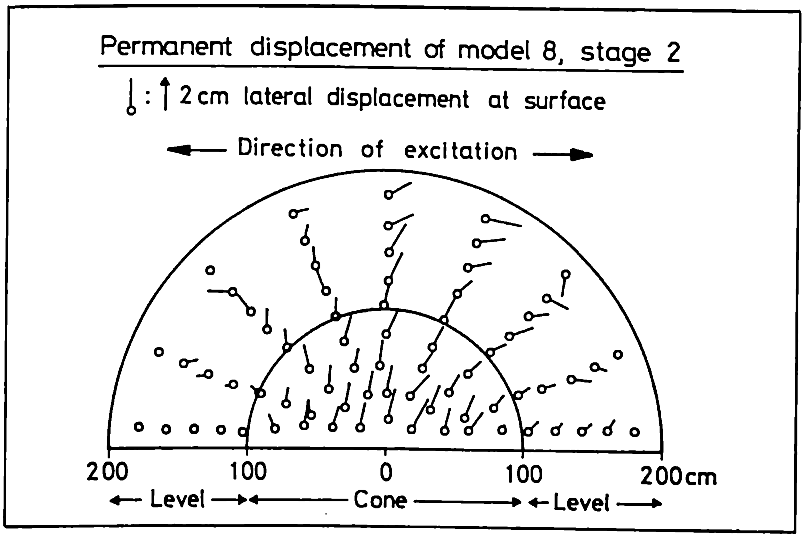

Figure 12: Permanent displacement of Model 8 test at PWRI (after Sasaki et al., 1991)

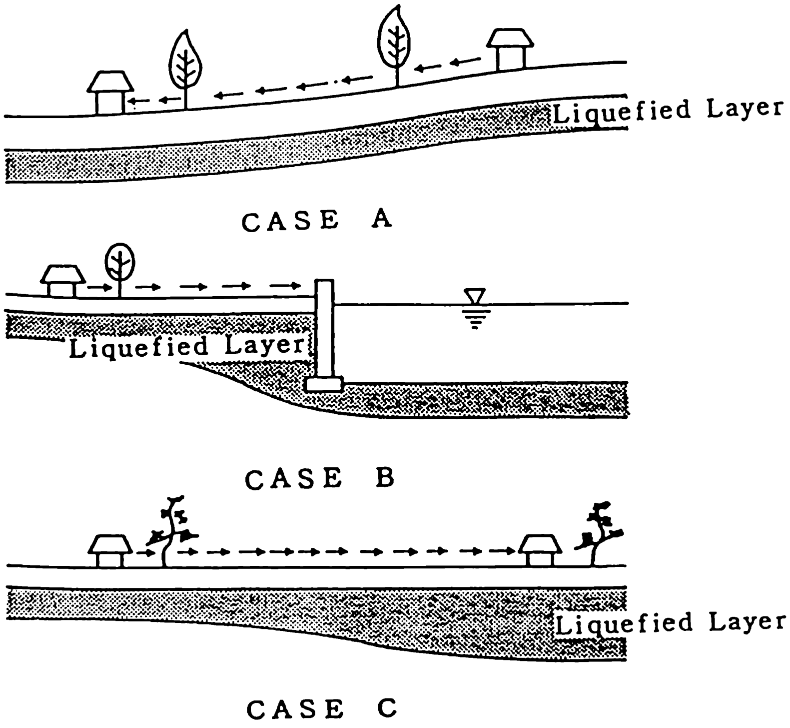

Figure 13: Types of permanent ground displacements (after Hamada et al., 1986)

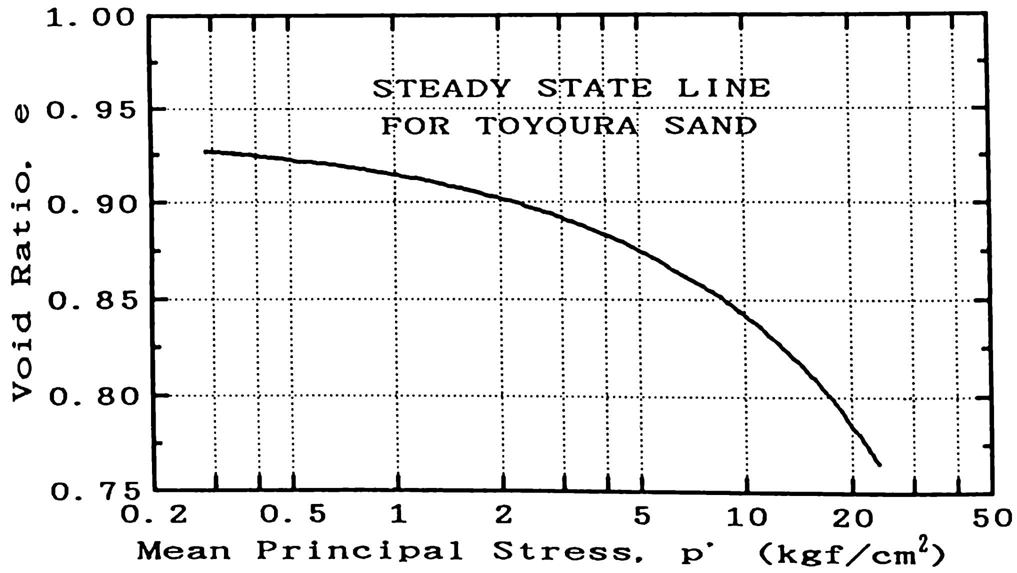

Figure 14: Relation between effective stress and void ratio at steady state for Toyoura sand (after Verdugo et al., 1991)^

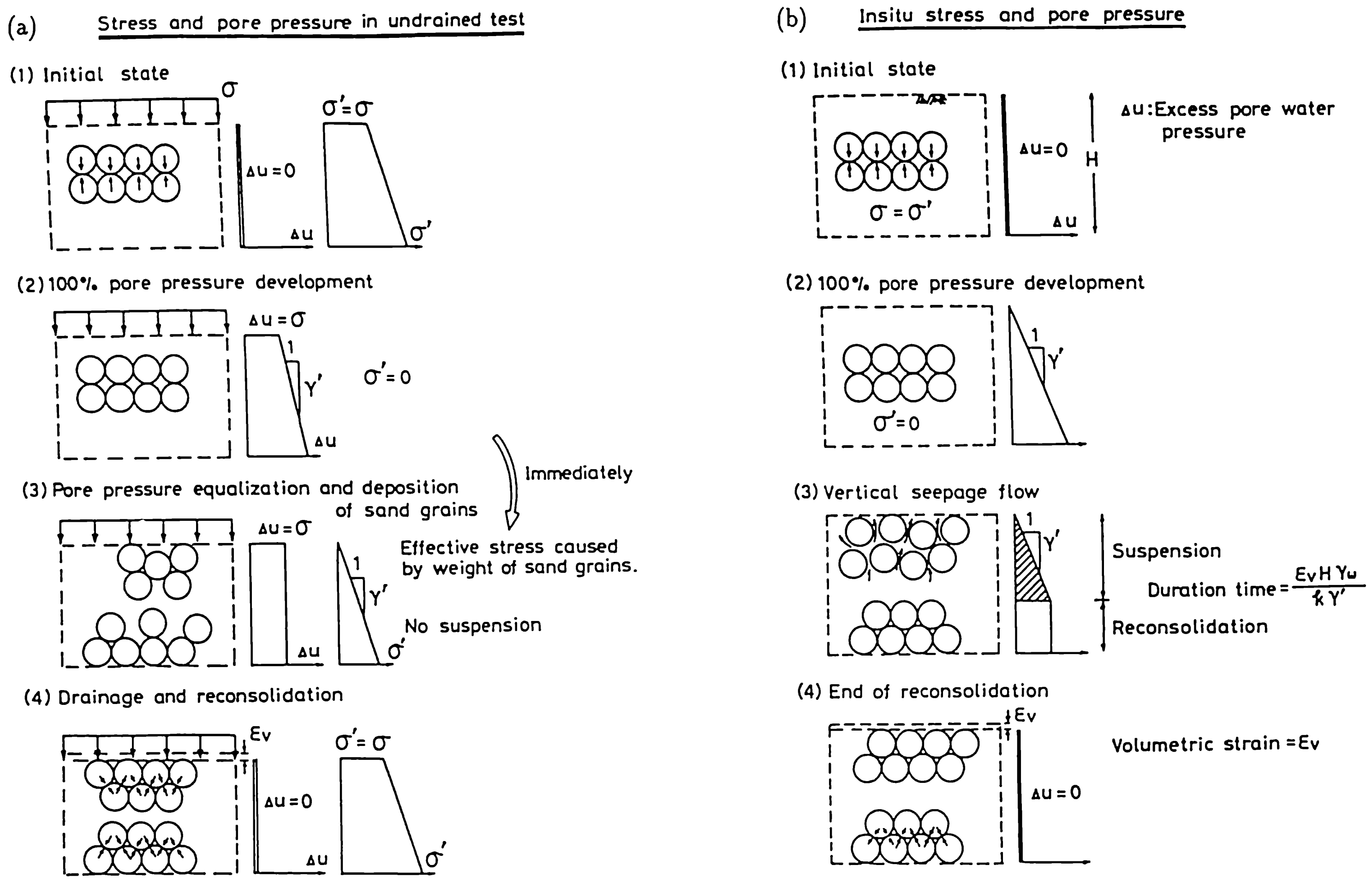

Figure 15: State of stress and pore water pressure: (a) in laboratory undrained test; (b) in-situ