CHAPTER 4: REQUIREMENTS AND PHENOMENA INFLUENCING PERFORMANCE

4.1 Overview

Design of improved ground zones to remediate liquefiable soils at existing highway bridges requires understanding both the performance requirements for the bridge and the phenomena associated with and affecting the treated ground behavior. Understanding the purpose and limitations of the bridge performance requirements is necessary so the implications of the improved ground zone and supported structure meeting or not meeting those requirements can be adequately assessed. Knowledge of the phenomena influencing improved ground behavior is needed so that they can be incorporated in the design process, as appropriate. If certain phenomena are not included during design, then the potential effects of leaving them out on the predicted performance must be understood.

4.2 Performance Requirements

Performance requirements for a bridge under seismic loading are generally established to provide a means of ensuring a certain degree of serviceability for the bridge following a seismic event of equal or lower magnitude than the design earthquake. Serviceability criteria used for bridges include: (1) minor damage that allows immediate use of the bridge, (2) repairable damage allowing service to be restored at a later date, and (3) nonrepairable damage that does not cause collapse but requires bridge replacement.

The performance of a bridge is typically dependent on the forces and displacements that its members are subjected to during and after a seismic event. These, in turn, are controlled in part by the displacements and motions that the foundation elements experience. Therefore, the objectives of designing improved ground zones to mitigate liquefaction effects are to limit the differential displacements of the foundations during and after the earthquake, as well as reduce the magnitude of ground motion transmitted to the structure through the foundation.

Stability and deformation analyses can both be used in the design of improved ground zones. However, it is the deformation analyses that provide actual estimates of the ground and structure movements ultimately governing design.

Pseudostatic stability analyses can be performed to evaluate the stability of foundations during earthquakes, where the seismic forces acting on the foundation and soil mass are represented as equivalent static loads. However, inadequate safety factors from these analyses do not necessarily indicate inadequate performance, because the condition of instability may only exist for short periods of time during shaking. The final displacements that accumulate during shaking due to transient instability may be acceptable. By the same token, a factor of safety greater than one from a pseudostatic analysis does not necessarily guarantee adequate performance, because accumulated displacements that occur during shaking could be excessive. Therefore, although the factor of safety for stability during earthquake loading may be used as a general indicator of potential performance, the ultimate indicator should be computed movements.

Unlike the earthquake-loading case, evaluation of stability for post-earthquake conditions will be a definitive indicator of inadequate performance if the factor of safety against failure based on the post-earthquake soil strength is less than one. In this case the post-earthquake condition would be unstable and associated movements would be expected to be excessive. On the other hand, if the factor of safety is adequate indicating a stable condition, displacements will usually need to be evaluated to ensure they are within acceptable limits.

Setting a limit on the magnitude of ground motion transmitted to a bridge is generally more difficult than establishing movement or safety factor limits due to the complexity of structure response to the motion. If possible, an improved ground zone should be designed to limit the motion transmitted to the bridge foundation, while at the same not exceeding the displacement limits.

4.2.1 Movement Criteria

Specific recommendations for movement criteria to limit damage to bridges can be found in the literature. Some of these criteria are summarized in Table 4.1. They are expressed in terms of either distortion, δ/l, or gross horizontal and vertical movements of the bridge foundations. Distortion, δ/l, is defined as the ratio of the differential settlement, δ, between bridge foundations to the distance between the supports on the foundations, l. These criteria, with the exception of that by Youd (1998), are for the bridge itself based on the foundation movement. For the case of a bridge on shallow foundations, the displacement of a bridge footing can be conservatively estimated to be the same as the supporting ground at footing level.

The limits presented in Table 4.1, with the exception of those by Youd (1998), were established for maintaining serviceability of the bridge under non-earthquake loading conditions. Therefore, when adopting these criteria for earthquake loading conditions, all post-construction movements must be incorporated in the displacement estimate, including those occurring before and after the earthquake. These criteria (excluding those by Youd, 1998) may be somewhat conservative for performance of a bridge subjected to earthquake loading and liquefaction conditions.

Youd (1998) provides movement criteria for maintaining bridge serviceability after an earthquake and liquefaction in terms of the ground movement at the bridge. He developed these criteria based on limited case history data of damage to bridges subjected to seismic loading and liquefaction of surrounding soils. The movement limits appear to apply to earthquake and post-earthquake induced movements only, and mark the boundary between displacement that can be tolerated without serious damage or loss of function of the bridge and those that can potentially cause significant damage. As indicated in Note 2 of Table 4.1, Youd et al. (1998) indicate that well-built, modern bridges, which tend to be more durable, can tolerate even larger displacements than those recommended by Youd (1998).

Larger movements than those given in Table 4.1 can be accepted if the design objective is either to (1) maintain the bridge in a repairable but not an immediately serviceable state following the earthquake, or (2) prevent collapse of the bridge but not maintain repairability. For these cases, the tolerable amount of ground and foundation displacements become increasingly a function of the particular structural details of the bridge.

4.2.2 Stability Criteria

Recommendations for appropriate factors of safety for stability of bridge piers, abutments, and approach embankments under earthquake and post-earthquake loading conditions are not given in codes such as the 1996 Standard Specifications for Highway Bridges by the American Association of State Highway and Transportation Officials (AASHTO, 1996) or Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, ATC 32 by the Applied Technology Council (1996). Typically, a factor of safety somewhere between

1 to 1.5 would be adopted depending on the conservativeness of the assumptions and parameters used in the analyses. As mentioned above, the factor of safety for stability is only a definitive predictor of failure for post-earthquake conditions. For post-earthquake conditions where failure is not predicted and earthquake loading conditions, the factor of safety for stability can only be used as an indicator of potential performance. In these cases it is the movements predicted from deformation analyses that provide definitive information regarding performance.

4.3 Phenomena Affecting Performance

As evident from the laboratory and numerical modeling results summarized in Chapter 3, the performance of a structure supported in liquefiable soil deposits by improved ground zones of limited size is highly dependent on the type, size, and location of the ground improvement. There are several phenomena that influence the performance of treated zones and will have an impact on the treatment configuration used. They include pore water pressure migration, ground motion amplification, inertial force phasing, dynamic fluid pressures, influence of a structure, and lateral spreading forces. A discussion of these phenomena and their implications for design is given below. Illustrations of the phenomena and observations in the literature regarding them are provided in Figure 4.1 and Table 4.2.

4.3.1 Pore Water Pressure Migration

Pore water pressure migration occurs during and after an earthquake, when pore water pressures will be higher in a liquefiable zone than in an adjacent improved zone due to the larger shear deformations that occur in the liquefiable soil. The difference in pressures between the two zones induces flow into the treated zone, as illustrated in Figure 4.1(a), causing an increase in pore water pressure and a potential decrease in strength in the improved area.

The observations given in Table 4.2 indicate that pore pressure migration into a densified zone occurs during and after shaking. The relative magnitude of the migration occurring during shaking, as opposed to after, is dependent on such things as the soil permeability, compressibility, and loading conditions.

In shaking table tests conducted on adjacent loose and dense zones with no surface loads, Iai et al. (1988) and Akiyoshi et al. (1994) noted that a portion of the densified zone along the perimeter, denoted as ABC in Figure 4.1(a), developed excess pore water pressure ratios (i.e. – ratio of the pore water pressure above the initial static level to the initial vertical effective stress), r_u, greater than 0.5. They stated that this zone, having a base angle of approximately 30 to 35 degrees (angle BAC in Figure 4.1(a)), could be treated as unstable and liquefied. This practice reduces the effective support area provided by a densified zone beneath a foundation, if it is adopted for this case. No such recommendation was made by Liu and Dobry (1997), who tested a circular footing on a densified zone in liquefiable sand, or Taguchi et al. (1992), who tested a dense zone with and without a surface surcharge and having liquefiable sand on both sides. In these two sets of tests the width of the densified zone beyond the edge of the surface load was smaller than the width of the unstable zone suggested by Iai et al. and Akiyoshi et al.

4.3.2 Ground Motion Amplification

Ground motion amplification occurs when the shear waves traveling upward through an improved ground zone are amplified, as shown in Figure 4.1(b). This amplification effect can result in more severe loading on the supported structure. As noted in Table 4.2, centrifuge tests by Liu and Dobry (1997) indicate that increasing the treated zone thickness within a liquefiable soil can reduce the settlement of a supported structure but increase the structures' acceleration. Therefore, it is desirable to strike a balance in design such that the size and stiffness of the improved ground zone results in acceptable movements and accelerations of the structure during an earthquake.

4.3.3 Inertial Force Phasing

As illustrated in Figure 4.1(c), there can potentially be differences in the times when the maximum inertial forces act on an improved ground zone and supported structure, referred to herein as the phasing between the inertial forces. The most severe phasing case for stability is when the inertial forces acting on the structure and soil mass are completely in-phase (peak force values occur at the same time and in the same direction) with each other during seismic loading.

At the present time there appears to be little experimental or analytical information in the literature about the phasing between inertial forces relative to improved ground design. In the related problem of seismic stability of footings on non-liquefiable soils, there is no clear consensus on whether the inertial forces acting on the soil during earthquake shaking are significant enough to include in design. However, as shown in Figure 4.1(c), the case of an improved ground zone of limited extent which supports a structure is similar to a seismic slope stability problem due to the loss of strength of the liquefiable soil. Therefore, inclusion of the inertial force acting on the soil when evaluating the seismic stability of a structure on an improved ground "island" appears appropriate. In the absence of information regarding the phasing between inertial forces, it seems prudent to assume they are completely in phase for simplified analyses.

4.3.4 Dynamic Fluid Pressure

Movement of a relatively stiff body, such as an improved ground zone, within a fluid medium, such as liquefied soil, may result in a dynamic component of fluid pressure acting on the body in addition to the static fluid pressure (refer to Figure 4.1(d)). At various times during an earthquake the sides of an improved ground block are assumed to be subjected to a total pressure which fluctuates between the maximum and minimum pressures (Figure 4.1(d)) obtained by taking the dynamic component of pressure and, respectively, adding and subtracting it to and from the static pressure. The dynamic component of pressure can be calculated using a formula such as the one given in Figure 4.1(d) proposed by Iai et al. (1988), which is based on a formulation developed by Westergaard (1931) for the dynamic component of water pressure acting on the vertical upstream face of a concrete dam during an earthquake. For stability and deformation of an improved ground block supporting a structure, the critical design case will likely occur when the minimum soil pressure acts on the sides, resulting in reduced lateral support of the block.

Although the presence of a dynamic component of pressure has been observed in liquefied soil acting on a full-scale retaining wall (refer to Table 4.2), no evidence has been presented of such a pressure acting on an improved ground zone that is not continuous, and generally not as rigid as a wall. The inclusion of this dynamic component of liquefied soil pressure could have a significant impact on the design of an improved ground zone by likely making it more conservative. Although there are some recommendations in the literature for including the dynamic component when designing an improved ground zone, it would be desirable to verify the development of this component before adopting this practice.

4.3.5 Influence of a Structure

The presence of a structure on improved ground alters the stress state within the ground and can thereby affect the dynamic response and stress-strain behavior of the treated soil. As noted in Table 4.2, the presence of a footing can result in the development of dilative behavior in densified sand. This dilative behavior can increase strength and limit deformations during shaking. However, post-earthquake stability and deformations can become more critical due to pore water pressure migration.

The performance of a densified ground zone and supported structure has been observed by Hatanaka et al. (1987) to be dependent on the width and height of the structure, as discussed earlier in Chapter 3, Section 3.2.2.1. For the same treatment width beyond the edge of the structure and same treatment depth below it, the wider the structure on a treated zone the smaller the settlement that was measured by Hatanaka et al. Likewise, the smaller the height to width ratio of the structure the smaller the observed settlement.

4.3.6 Lateral Spreading or Flow Forces

In areas of sloping ground, additional pressures/forces can be exerted on an improved ground zone as the surrounding unimproved soil undergoes lateral spreading and moves relative to the treated zone, as shown in Figure 4.1(e). Estimations of the force exerted by the liquefied soil on a stationary body typically involve treating the soil as a viscous fluid or a solid having a reduced modulus and using an appropriate analytical solution, such as a drag force calculation from fluid mechanics (refer to Table 4.1).

Although fully-liquefied soil moving relative to a stationary body will exert a force on that body, the forces exerted by partially- or non-liquefied layers will likely be higher than those from the fully-liquefied material. Field case history evidence presented by Berrill et al. (1997) indicates that an unliquefied surficial crust moving along with underlying liquefied material can exert a passive earth pressure type force which is much larger than the force exerted by liquefied soil.

4.4 Implications for Design and Research

Bridge performance requirements and the phenomena affecting improved ground behavior are important to rational design of treated ground zones for liquefaction mitigation at existing highway bridges. Both need to be carefully considered in the design process and in the evaluation of treated zone performance. Therefore these requirements and factors have been considered in this research work whenever appropriate and possible.

When evaluating the feasibility of using different ground improvement configurations at piers and stub abutments to reduce permanent movements to acceptable levels, as discussed later in Chapters 7 and 8, the limits given in Table 4.1 were used as a guide. In addition to evaluating the predicted displacements of the improved ground and supported structure system, the accelerations of the improved zone and structure were also reviewed, particularly in regards to the effect of treatment depth.

The phenomena of pore water pressure migration, ground motion amplification, inertial force phasing, and influence of a structure were considered in the analytical work performed. A dynamic component of liquefied soil pressure acting on an improved ground zone was not included in the analyses because its development still requires verification, most likely using shaking table or centrifuge tests. Likewise, the effect of lateral spreading involving mass movement of liquefied soil around a stationary improved zone, as illustrated in Figure 4.1(e), has not been directly investigated because it is a three-dimensional phenomena. The lateral spreading phenomena investigated in this study was limited to the two-dimensional case of the deformation of a liquefied sand layer beneath a stub abutment and approach embankment due to limitations of the analytical tools used.

The key to successfully assessing the performance of improved ground supporting a bridge in liquefiable soil is adequately predicting the deformations and accelerations of the treated ground system, taking into account the phenomena and performance requirements discussed above. In the next chapter some simplified, uncoupled analyses for doing this are evaluated.

TABLE 4.1: Distortion and Movement Criteria for Bridges

Authors	Basis of Assessment	Expected Bridge Performance	Findings: Distortion Criteria, δ/l	Findings: Gross Movement Criteria	Comments
Duncan and Tan (1991)	Literature review	Serviceability limit state	Simple span: 0.008 Continuous span: 0.004	Total settlement: 50 mm - not harmful 100 mm - somewhat harmful Horizontal movement: 38 mm	Distortion and horizontal movement criteria of Moulton et al. (1985) adopted except simple span value, which was found to be overconservative. State that distortion criteria preferred over gross movement criteria.
Lok and Mitchell (1994)	Literature review	Serviceability limit state	Simple span: 0.005 Continuous span: 0.004	Vertical: 50 to 100 mm Horizontal: 38 mm - without vertical movement 25 mm - with vertical movement	Distortion and movement criteria from Moulton (1985). State that displacements which bridge can tolerate during and after a seismic event is affected by bearing lengths of support spans and any end restraint mechanisms used.
AASHTO (1996)	Moulton et al. (1985)	Serviceability limit state	Simple span: 0.005 Continuous span: 0.004	Horizontal: 38 mm - with small vertical movement 25 mm - with vertical movement	Moulton et al.'s (1985) distortion and gross horizontal movement criteria have been adopted.
Youd (1998)[2]	Review of case histories	Upper limits of serviceability limit state	-	Ground settlement: 25 mm - shallow found.; 100 mm - deep foundation Lateral ground displacement: 100 mm	Settlement of deep foundation itself, as opposed to surrounding ground, should be 25 mm or less. Settlement limits apply when lateral displacement less than 100 mm.

Notes:
1. These criteria do not apply to rigid frame structures which AASHTO (1996) states " shall be designed for anticipated differential settlements based on the results of special analyses."
2. Youd et al. (1998) indicate that well-built, modern bridges on shallow foundations can tolerate up to 100 mm of ground settlement and 200 mm of lateral ground movement. Likewise, modern bridges on deep foundations can tolerate up to 200 mm of ground settlement and 200 mm of lateral ground movement.

TABLE 4.2: Past Studies of Phenomena Affecting Improved Ground Performance (after Mitchell et al. 1998)

Phenomena	Reference	Type of Study	Observations
Porewater Pressure Migration (Fig. 4.1(a))	Taguchi et al. (1992)	Shaking table tests of loose zone adjacent to dense zone with and without surcharge	• No collapse of dense zone into liquefied loose zone observed at interface of two zones. • Pore water pressure observed in dense zone basically dependent on distance of point from loose/dense interface and not significantly influenced by position relative to surface surcharge, except in early stages of shaking.
	Iai et al. (1988) and Akiyoshi et al. (1994)	Shaking table tests and numerical modeling of densified sand adjacent to loose sand.	• Migration occurs during and after shaking. • Area along perimeter of densified zones becomes unstable due to $r_u > 0.5$. [1] • When densified soil $k > 1\text{x}10^{-3}$ cm/sec seepage conditions in zone approach steady state at end of shaking. [2]
	Liu and Dobry (1997)	Centrifuge test of densified sand zone supporting circular footing in loose sand.	• Significant pore water pressure migration into densified zone during shaking when $k = 0.17$ cm/sec. • Dilative soil behavior observed beneath footing during shaking when sand had $k = 1.2 \text{ x } 10^{-3}$ and $1.3 \text{ x } 10^{-2}$ cm/sec. Most seepage into zone under footing occurs after shaking stops.
Ground motion amplification (Fig. 4.1(b))	Liu and Dobry (1997)	See above.	• Increasing thickness of densified zone beneath footing relative to overall liquefiable sand thickness resulted in increased horizontal acceleration of footing and reduced settlement.

Notes:
1. r_u denotes excess pore water pressure ratio defined as the ratio of pore water pressure above static levels to initial vertical effective stress.
2. k denotes hydraulic conductivity of soil.

TABLE 4.2(cont.): Past Studies of Phenomena Affecting Improved Ground Performance

Phenomena	Reference	Type of Study	Observations
Phasing of Inertial Forces[3] (Fig. 4.1(c))	Lam and Martin (1986)	-	• Seismic stability of footing can be evaluated with classical bearing capacity theory provided additional loads and moments from structure included. • Inertial force on soil mass not considered.
	Dormieux and Pecker (1995), Pecker and Salencon (1991)	Prandtl analysis of footing on sand or clay layer	• Horizontal inertial force on soil mass within failure zone does not significantly influence seismic bearing capacity and can be disregarded.
	Shi and Richards (1995)	Coulomb analysis of footing on frictional soil mass.	• Bearing capacity in frictional soils influenced by inertial forces acting on soil within failure zone and transfer of structures' inertial forces to soil.
Dynamic Fluid Pressure (Fig. 4.1(d))	Iai et al. (1988), Iai in JGS (1995) and Iai (1996)	-	• Dynamic component of fluid pressure acting on side of improved ground zone during an earthquake should be included in design using modified version of Westergaard's (1931) formula. • Validity of modified Westergaard's formula for estimating dynamic component of liquefied soil pressure verified experimentally by Tsuchida (1968) on full-scale retaining wall.
	Japanese Road Association (1990)	-	• A dynamic component of fluid pressure exerted by weak or liquefiable soil layers should not be used for design of column-type structures under earthquake loading.

Notes:
3. These studies evaluated the seismic bearing capacity of surface footings on level, uniform ground. They did not consider improved zones.

TABLE 4.2 (cont.): Past Studies of Phenomena Affecting Improved Ground Performance

Phenomena	Reference	Type of Study	Observations
Influence of Structure	Hatanaka et al. (1987)	Shaking table tests of rectangular block on dense zone in liquefiable sand (Dr = 50%).	• Settlement of rectangular block on densified zone extending fixed distance beyond edges decreases as width of structure increases. • Settlement of rectangular block on loose sand layer without densified zone increases as the height to width ratio of block increases.
	Liu and Dobry (1997)	Centrifuge tests of circular footing supported on medium dense sand (Dr = 55%)	• Dilative behavior observed beneath footing during shaking when sand had k =1.2 x 10^{-3} and 1.3 x 10^{-2} cm/sec. For these cases, most of excess pore water pressure migration into zone under footing occurred after shaking stopped.
Lateral spreading or flow forces (Fig. 4.1(e))	Ohtomo (1996)	Shaking table tests of lateral spreading around pile supported conduit	• Liquefied sand behaves like viscous fluid or muddy flow. • Force exerted on pile proportional to square of velocity and can be estimated using drag force approach.
	Kawakami (1996)	Shaking table tests of lateral spreading around model in-ground wall	• Pressures exerted by liquefied soil on wall were larger for more steeply sloping ground. Distribution of pressures dependent on 3-D characteristics of flow. • Non-liquefied crust exerted larger pressures than liquefied soils below.

TABLE 4.2 (cont.): Past Studies of Phenomena Affecting Improved Ground Performance

Phenomena	Reference	Type of Study	Observations
Lateral spreading or flow forces - continued (Fig. 4.1(e))	Tokida et al. (1992)	Model piles displaced through liquefied or partially liquefied ground in tank.	• Force required to move piles increased as r_u decreased below 1.0. • As velocity of piles relative to “liquefied” soil increased, force necessary to move piles increased. • As number of piles perpendicular to direction of movement increased, the force required to move the piles increased.
	Soydemir et al. (1997)	Literature review	• Liquefied soil can be treated as “viscous fluid or solid with significantly reduced modulus”. • Use of “subgrade reaction model” with appropriate p-y curves for laterally spreading soil (Jackura and Abghari, 1994) seems reasonable for design.
	Berrill et al. (1997)	Back analysis of field case where bridge piers subjected to lateral spreading.	• Unliquefied crust exerted passive forces on bridge piers which were much larger than drag forces exerted by underlying flowing (liquefied) sand.

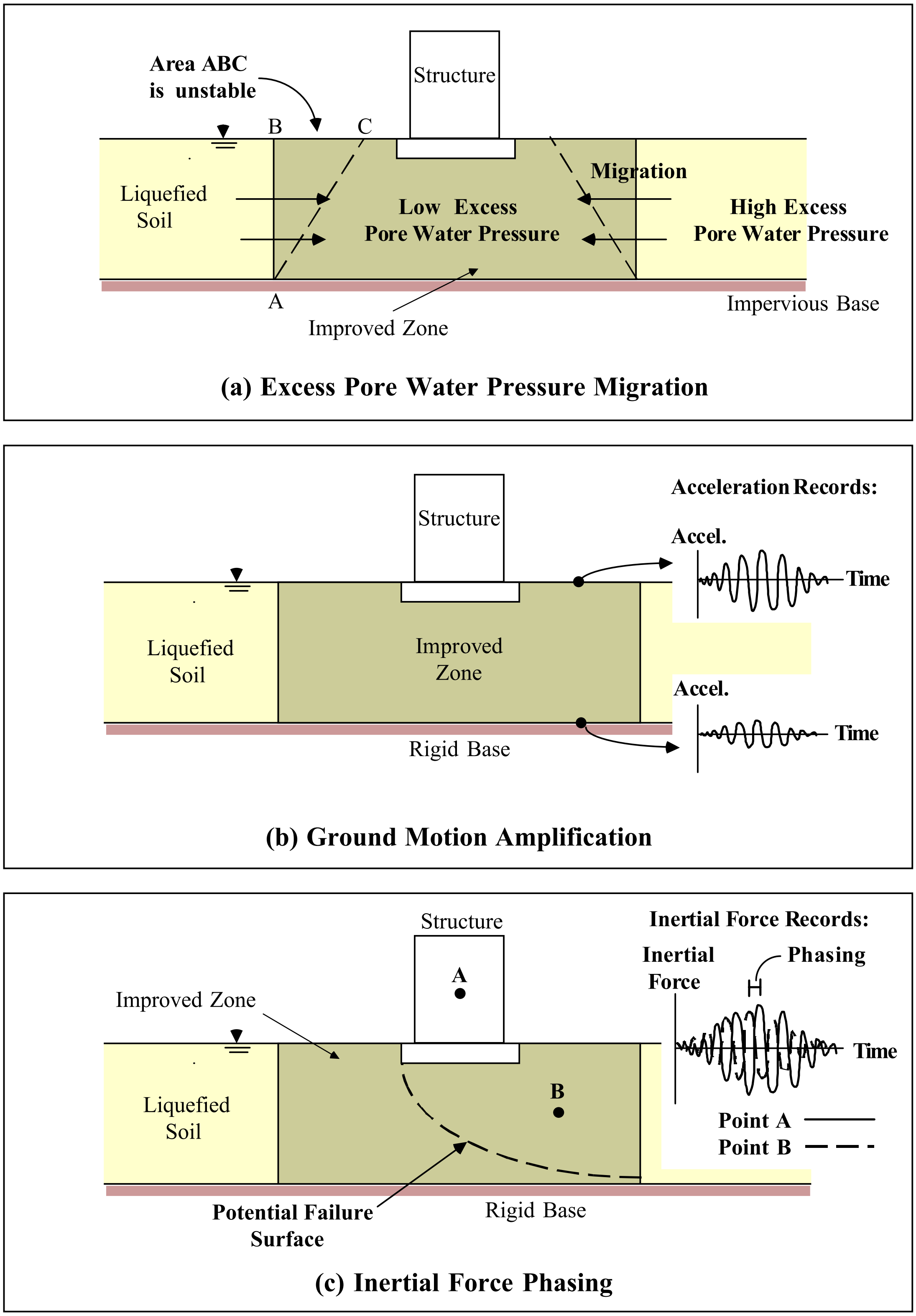
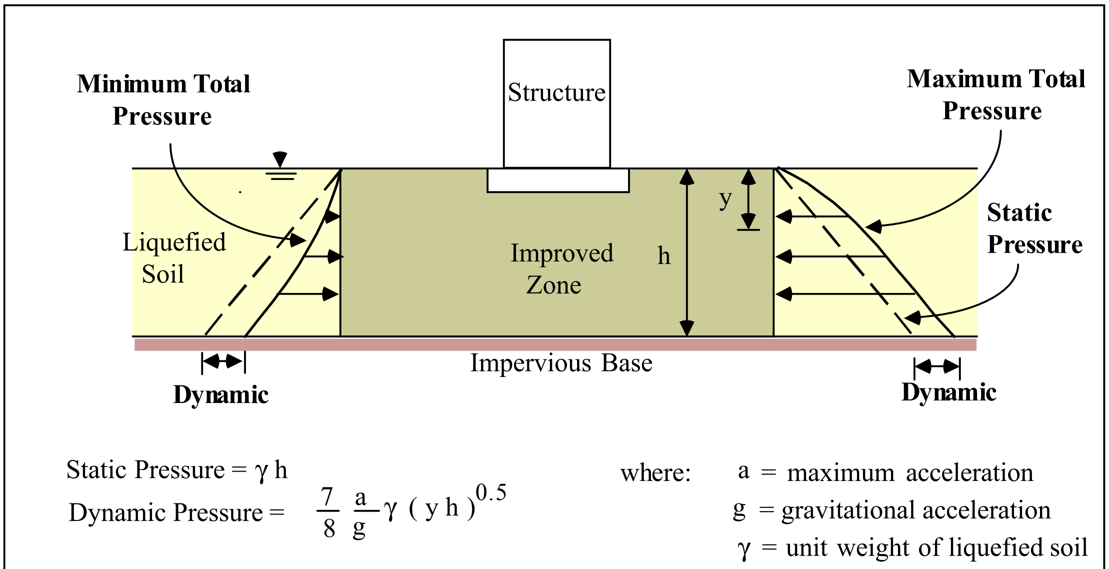
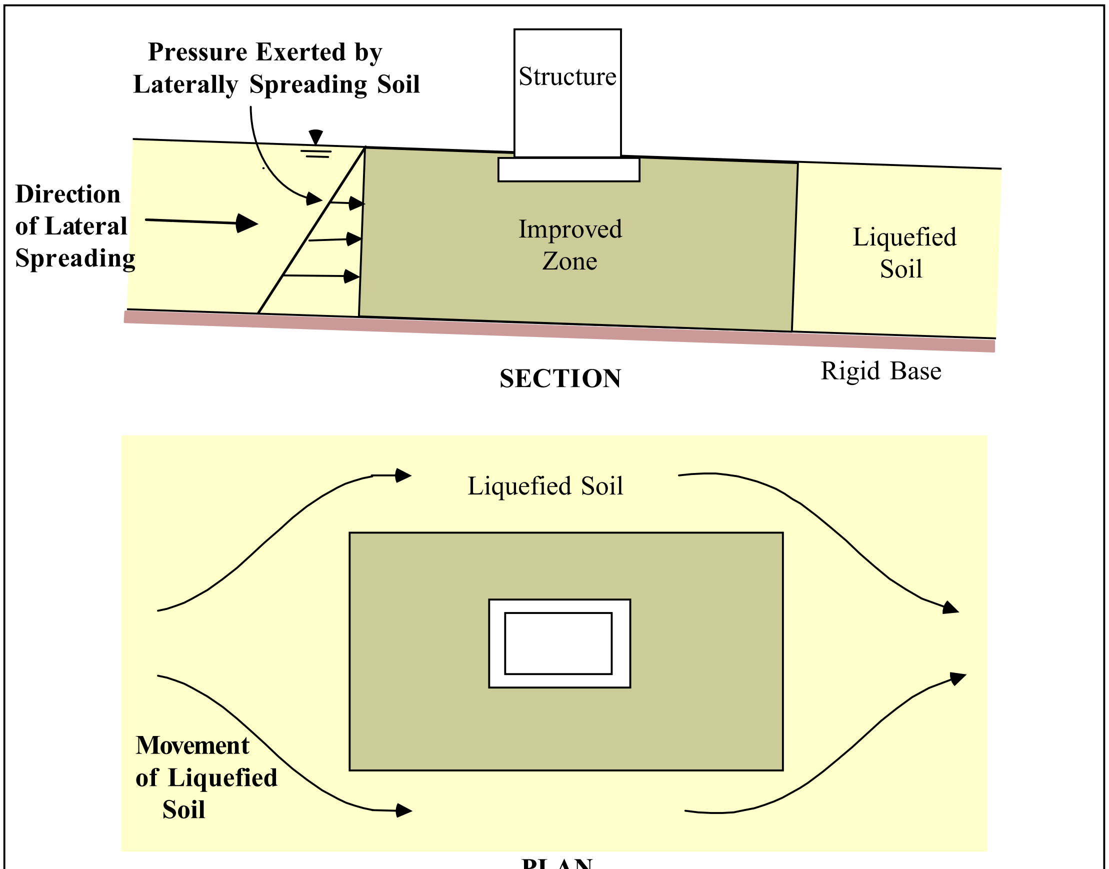


FIGURE 4.1: Phenomena Affecting Improved Ground Performance (after Mitchell et al., 1998)



(d) Dynamic Fluid Pressure



(e) Lateral Spreading Forces

FIGURE 4.1 (cont.): Phenomena Affecting Improved Ground Performance