CHAPTER 3: LITERATURE REVIEW ON OBSERVATION AND PREDICTION OF IMPROVED GROUND PERFORMANCE

3.1 Introduction

Over the last ten to fifteen years considerable information has been developed concerning the performance of improved ground within liquefiable soil deposits subjected to earthquakes. This information comes from physical modeling in the laboratory using centrifuge and shaking table apparatus, numerical modeling using a variety of computer codes, and field case histories. The results from physical and numerical modeling studies are particularly useful because in many cases a number of variables can be isolated for study of their impact on the improved ground behavior. In addition, several aspects of ground response are monitored in these studies, such as ground displacements, accelerations, and pore water pressure development. Information is summarized below from several key laboratory and numerical modeling studies having particular relevance to liquefaction mitigation at existing highway bridges. The focus of many of these studies is on the effect of treatment type, size, and/or location on performance.

As just mentioned, numerical modeling is one of the tools used for predicting improved ground performance under earthquake loading. Over the years, a variety of numerical modeling approaches have been used for these analyses with varying success. A summary is provided of those approaches along with a brief discussion of their performance, as cited in the literature.

3.2 Improved Ground Performance from Previous Studies

3.2.1 Abutment/Embankment Performance

3.2.1.1 Treatment Size and Location

Riemer et al. (1996) studied the effects of improved ground zone size and location on the performance of a section of the Highway I-57/Mississippi River Bridge Abutment, shown in Figure 3.1, by performing dynamic analyses of the zone using base acceleration records from the

1989 Loma Prieta (magnitude M = 7.1) and 1988 Saguenay (magnitude M = 6.0) earthquakes having predominant frequencies of about 1 and 10 Hertz, respectively. Both records had strong shaking durations of about 8 to 10 seconds, and were scaled to have maximum accelerations of 0.22g (where g is the acceleration of gravity). These analyses were performed using the two-dimensional finite difference computer program FLAC (Cundall, 1993). The soil properties used in the analyses are presented in Figure 3.1. Two sets of analyses were performed using simplified "stiff" and "soft" shear stress-strain models for liquefied sands, with the limiting shear stress being the undrained residual shear strength. The "soft" soil model assumed a large strain (5 percent) was required to reach the undrained residual shear strength of the soil, as opposed to a much smaller strain in the "stiff" soil model. Only the analytical results from the "stiff" model are presented here.

Figure 3.2 illustrates the effect of improved ground zone width on the predicted maximum lateral displacement of the embankment. As seen from the displacement values (given for earthquakes with predominant frequencies of 1 and 10 Hz) listed in the figure, increasing the distance that the treated zone extends beneath the embankment from the toe results in decreasing lateral displacements due to seismic loading. It can also be seen that although the predicted displacement progressively decreases as the width is progressively increased from 12 m to 36 m, the amount of change in the displacement reduction gets progressively smaller.

The effects of treatment zone locations on lateral displacements of an embankment based on the data are shown in Figure 3.3 (Riemer et al., 1996). According to Riemer et al., the values indicate that a 24-m-wide treatment zone is most effective when it is roughly centered beneath the sloping portion of the embankment. As the treated zone location moves outward from beneath the sloping portion, predicted lateral displacements become progressively larger, with the largest displacements occurring when the left side of the treatment zone is located at the toe and the remainder of the zone lies completely outside the embankment.

The results presented by Riemer et al. show quantitatively the effect of treatment zone size and location on the performance of an embankment underlain by a liquefiable soil layer. Although the soil model used in these analyses was not verified by evaluating laboratory or field case histories and comparing predicted to measured displacements, the results of the analyses provide an indication of the relative effectiveness of treated zone size and location. In particular, it appears that treated zones constructed entirely outside an embankment toe are not very effective in reducing deformations. One reason for the ineffectiveness is that the potential failure surfaces may pass between the toe and the treated zone, or only through a part of the treated zone.

In addition to the effects of the treatment zone size and location, the results in Figures 3.2 and 3.3 indicate the importance that the predominant frequency of the earthquake can have on the induced displacements. Riemer et al. note that a comparison of the displacements given in the figures for the 1 Hz and 10 Hz earthquakes, having scaled peak acceleration amplitudes of 0.22g, indicates that the higher frequency earthquake results in smaller predicted permanent displacements. However, this observation may not be true in cases where the higher frequency motion is closer to the natural frequency of the system than the lower frequency motion.

The results in Figures 3.2 and 3.3 also show that trends in the predicted post-earthquake factor of safety for equilibrium (factors of safety presented in figures) for the different ground improvement schemes are consistent with those observed for the displacements, in that the higher the factor of safety, the smaller the deformation. However, Riemer et al. (1996) state the factor of safety can not be used as a sole indicator of displacements because of its inability to capture the effects of the input motion frequency and ground motion amplification.

The conclusion by Riemer et al. that the location of a treated zone beneath an embankment on a liquefiable soil deposit is more effective when centered under the slope rather than located outside the toe is supported by the results of shaking table tests described by Yanagihara et al. (1991). In this study, three separate tests were conducted with a clay embankment constructed on a saturated sand deposit having a relative density of 40 percent. The three tests consisted of subjecting the embankment to a sinusoidal base motion having a frequency of 2 Hertz and acceleration amplitude of 200 centimeters per second squared (cm/sec^2) with no improvement in the liquefiable sand zone (Case 1), a densified sand zone having a relative density of 80 percent constructed in the liquefiable soil under the slope (Case 2), and a densified sand zone constructed just outside the embankment toe (Case 3), as shown in Figure 3.4. In case 1 (no treatment) high excess pore water pressures developed in the free field and extended under the embankment, resulting in large lateral deformations and settlements of the foundation soils and embankment. In case 3 (treatment outside toe) high excess pore water pressures developed in the free-field outside of the densified zone, as well as in the untreated liquefiable soil under the embankment. Although the improved zone did not deform significantly, the high pressures under the embankment caused the embankment to crack near the toe and liquefied material to be expelled out, resulting in the embankment settling 80 to 125 millimeters (mm). In contrast to this, Case 2 (treated zone under the slope) had high excess pore water pressures develop outside the embankment while excess pressures in the dense zone and under the embankment were lower, resulting in reduced lateral displacements and an embankment settlement of 22 millimeters. Based on the results from Cases 2 and 3, Yanagihara et al. conclude that a treated zone used at an embankment or dike will only be effective if there is sufficient embankment surcharge over the liquefiable soils behind the zone to resist the pore water pressures that develop in the untreated soil.

3.2.1.2 Treatment Type

The impact of different treatment types on the performance of an embankment is illustrated by centrifuge test results from Adalier (1996) and summarized in Adalier et al. (1998). In these tests a 4.5-m-high (all quantities presented in prototype scale which gives the equivalent representation of the centrifuge model in a 1g environment) clay embankment having side slopes of 1-horizontal to 1-vertical (1H:1V) was constructed on a saturated, 6-m-thick loose sand layer. The performance of the embankment was investigated with the following types of improvement constructed at the toes of the embankment as illustrated in Figure 3.5: (1) none, (2) 6-m-wide densified zones, (3) 6-m-wide cement treated blocks, (4) 3-m-wide gravel berms, and (5) 0.15-cm-thick steel sheet piles with tie rods. In all cases the base of each model was successively subjected to three 1.6-Hz sinusoidal acceleration records having magnitudes of 0.09g, 0.18g, and 0.3g. Each of the three records consisted of 10 uniform cycles of acceleration. During the tests, settlements, accelerations, and pore water pressures were monitored at various points in the foundation soils and on the embankment. Deformations were also measured at the end of the final shaking event.

The vertical and lateral deformations observed in each of the models after the third shaking event are illustrated in Figure 3.6. Table 3.1 presents approximate ratios of improved to unimproved (embankment) peak crest accelerations, crest settlements, and lateral toe displacements for the different remediation schemes for the third shaking event based on data presented in Adalier (1996) and Adalier et al. (1998). In addition, the ratios of the peak embankment crest accelerations to input base accelerations are presented.

Adalier et al. (1998) make several observations about the effectiveness of the various ground treatment methods on improving the performance of the embankment. They note that if the intent of the ground improvement is to minimize lateral displacement and vertical settlement of the embankment, then the methods in order of decreasing effectiveness are: (1) sheet piles with tie rods, (2) densification or gravel buttresses (the overall effectiveness of both were judged to be about the same), and (3) cement-treated blocks.

Adalier et al. (1998) note that the steel sheet pile enclosure with tie rods (Model 5) resulted in the smallest cracking and slumping of the embankment, with the embankment basically retaining its original shape. This good performance occurred despite the fact there was a sharp rise in the pore water pressure beneath the embankment during shaking. It is largely attributed to the reinforcement and confinement provided by the sheet piles and tie rods which prevented lateral deformation. In this model the tie rods were particularly important in restraining the outward movements of the sheet piles relative to each other, thereby playing a significant role in limiting the embankment movements. The peak acceleration measured at the embankment crest was the smallest (about 70 percent of the base input acceleration amplitude) with this improvement scheme. The low crest acceleration is attributed by Adalier et al. to liquefaction of the foundation soils directly beneath the embankment between the sheet piles resulting in dynamic isolation of the embankment.

Densification (Model 2) was the most effective improvement method for reducing settlement of the embankment and limiting the development of excess pore water pressures beneath the embankment. However, measured peak accelerations at the embankment crest were about 1.3 times the base input acceleration amplitude.

According to Adalier et al., the gravel berm (Model 4) improvement helped to improve embankment performance by providing lateral support for the embankment and increasing stress levels. They also attribute the improved performance to the longer distance that liquefied soil beneath the embankment had to move to reach the free ground surface. Part of the improved performance was also attributed to the experimental effect of the reduced free space between the berm and centrifuge box wall, as compared to the cases without the berm. Despite the improved lateral deformation performance, the berm treatment case had higher embankment settlements relative to other improvement methods due to the berm weight (Adalier, 1996). The measured peak accelerations at the embankment crest were about 1.1 times those of the base input amplitude.

The use of cement treated blocks (Model 3) was reported to have a negligible effect on embankment deformations in comparison to the case with no improvement. The large lateral deformations observed were attributed to the high excess pore water pressures and low soil strength that developed beneath the embankment during shaking. However, Adalier et al. postulate that the performance with the cemented blocks would be better if the bottom of the blocks were embedded in an underlying firm soil layer as opposed to resting on the base of the centrifuge model box. The measured "peak" accelerations of the embankment crest with the cement block zones were about 1.5 times those of the base input amplitude of 0.3g.

Based on the results presented above, the deformation performance of an abutment and approach embankment supported on liquefiable soils will generally improve in relation to the degree of confinement provided by a given ground improvement method. The degree of confinement increases as the stiffness of the confining system increases. Confining systems installed at the toe of the embankment appear to be effective provided the system is rigid and there is no opportunity for the liquefied soil to flow around the system. As noted in the previous section, ground improvement becomes less effective when it is performed outside the embankment. Accelerations at the top of an embankment can be reduced if high pore water pressures develop in the soil within the confining system, but this also results in larger densification settlements.

Although the improvement methods and configurations used in the centrifuge tests by Adalier (1996) did improve the model embankment performance, as illustrated by the ratios presented in Table 3.1, it is important to note that the measured deformations and accelerations were still relatively large (refer to values given in Figure 3.6). These data illustrate that reducing liquefaction-induced ground deformations to acceptable levels may require more than one improvement type. The data also indicate that: (1) there may be a limit to the reduction in deformations and accelerations that can be achieved and (2) a particular type of improvement may be effective for improving one type of ground response (i.e. – acceleration, deformation, or pore water pressure) but less effective in improving another.

The observation from the tests by Adalier (1996) that increased rigidity of a confinement system installed in liquefiable soil will result in reduced ground deformations is also supported by the results of shaking table tests reported by Yasuda et al. (1991). In these tests, a 20-centimeter-thick liquefiable sand layer having a relative density of 30 percent and slope of three percent was placed over a 45-centimeter-thick nonliquefiable layer of dense sand having the same slope. In each test a narrow band of improvement was constructed in the liquefiable soil layer, with the band extending perpendicularly across the slope. The different improvement types used to form the band included (1) a single row of equally-spaced sand compaction piles, (2) one or two rows of equally-spaced, rigid cylindrical piles, (3) a continuous densified sand zone, and (4) a continuous rigid wall. All of the improvements penetrated completely through the liquefiable layer and were shallowly embedded into the non-liquefiable soil layer, with the exception of the continuous rigid wall, which was deeply embedded. The smallest lateral displacement of the ground behind the improved zone was obtained with the continuous rigid wall. However, the displacements with the sand compaction piles and the continuous densified zone approached those of the continuous rigid wall as the number of compaction piles and the width (in the direction of the slope) of the densified zone were increased, respectively. Yasuda et al. note that numerical simulations of these tests supported the laboratory results. In addition, the numerical analyses indicated using a continuous densified zone penetrating approximately 50 percent of the liquefiable soil depth was not effective in mitigating lateral deformations.

3.2.1.3 Effects of Non-Liquefiable Surface Layer

Balakrishnan et al. (1998) performed a series of five centrifuge tests involving liquefaction and lateral spreading. The surficial soil in the model consisted of a non-liquefiable clay layer sloping up to 3 degrees toward a channel in the middle of each model. The clay layer was underlain by liquefiable, saturated fine sand (Nevada sand at 50 percent relative density) over dense sand (Nevada sand at 80 percent relative density), as shown schematically in Figure 3.7. In some of the models, abutments were constructed on top of the clay layer, on one or both sides of the channel, with the ground surface on the abutment side having little or no slope. Three of the models had no improvement within the liquefiable sand and two of the models had dense sand zones (Nevada sand at 80 percent relative density) within the liquefiable zone. The dense sand zone penetrated the full depth of the liquefiable layer and extended across the entire length of the model box, running under the abutments and slopes on both sides of the channel. The dense zone width was only slightly larger than the abutment width. In one of the models the dense zone was enclosed in a plastic membrane to reduce pore water pressure migration into the zone. The models were subjected to a series of simulated earthquakes, primarily by applying scaled records of the ground motion measured at 82 meter depth on Port Island during the 1995 Kobe earthquake in Japan. Peak acceleration amplitudes of the records ranged from approximately 0.15g to 0.7g.

According to Balakrishnan et al. (1998), the models with the dense zones within the liquefiable soil layer generally had reduced settlements, lateral deformations, and pore water pressures in comparison to the models with no improvement. They report that the improved zone settlements, which were generally in a range of 15 to 30 centimeters (all measurements presented in prototype scale), were lower by a factor of 2 to 4 compared to zones where there was no improvement. Lower settlements (20 to 40 percent) were observed in the model where the dense zone improvement was enclosed in a plastic membrane as compared to the case where no plastic membrane was used. The probable reason indicated for the lower settlements with the membrane was the reduction in pore pressure migration into the dense zone.

Ground improvement using a dense zone was less effective in reducing the lateral deformations (treated zone deformations were about 25 percent less than untreated zones) measured in the centrifuge tests. Surficial lateral displacements of the sloping ground, where no abutment was present, were on the order of 100 centimeters down slope for points located over the densified sand zones. The reduced effectiveness of the ground improvement in mitigating lateral deformations in these tests was attributed by Balakrishnan et al. (1998) to the improvement not extending through the overlying clay, thereby leaving a continuous clay layer over the treated and untreated zones. This configuration resulted in the clay over the treated zone being dragged along with the portion over the untreated zone. They conclude this indicates a need to tie an nonliquefiable surficial soil layer to the underlying improved zone to prevent it from moving with the surrounding laterally spreading soil. In addition they note that although a narrow width of treatment was effective in reducing settlements, it likely needed to be wider to reduce lateral deformations.

Balakrishnan and Kutter (1999) report on one additional centrifuge test that supplements the five tests presented in Balakrishnan et al. (1998). The test setup is the same as shown in Figure 3.7 with the exception that all of the sand beneath the surficial clay layer was placed at a relative density of 80 percent and no plastic membrane installed as a seepage barrier (herein referred to as the dense sand layer case). The measured surface settlements along the abutment alignment for this dense sand layer case were generally similar to those observed for the two cases where a densified zone of limited width (approximately the same width as the abutment itself) was used in the liquefiable soil. Balakrishan and Kutter point out that the use of the limited width of treatment was as effective as treating the entire liquefiable soil layer, and suggest that the effect of the width of treatment needs to be investigated further. For the same dense sand layer case (all sands at 80 percent relative density), the surficial lateral displacements of the sloping ground, where no abutment was present, were very similar to the lateral displacements measured in the unimproved (sand layer at 50 percent Dr) and improved zone (dense sand zone in sand layer at 50 percent Dr) cases. In the dense sand layer case most of the lateral displacement occurred at the interface of the clay and dense sand. Balakrishnan and Kutter attribute this to the likely buildup of water at the clay-sand interface due to upward seepage and suggest that this may present problems for bridges located at sites where the sands may not be liquefiable, but low permeability confining strata exist.

3.2.2 Shallow Foundation Performance

3.2.2.1 Treatment Width

Hatanaka et al. (1987) investigated the effect of the width of a densified sand zone within a liquefiable sand on the settlement of a supported structure by performing shaking table tests. The basic model configuration is illustrated in Figure 3.8. The model structures consisted of concrete blocks having widths of 10, 20, or 30 cm. These structures were supported on densified sand zones (relative density of 90 percent) having a thickness of 15 cm and varying width, which were constructed at the surface of a 40-cm-thick liquefiable sand layer. The models were then subjected to a 2-Hz sinusoidal acceleration record having a uniform peak amplitude of approximately 0.1g for a total duration of about 10 seconds.

Hatanaka et al. prepared a plot of the treatment width ratio (treatment zone width, SW, divided by the structure width, BW) versus the settlement ratio (measured settlement of the structure, S, divided by the liquefiable sand layer thickness, D), as shown in Figure 3.9. They observed that increasing the treatment width ratio results in a decrease in the settlement ratio of the structure. As noted by Hatanaka et al. and shown in Figure 3.9, for a given treatment width ratio the settlement ratio is smaller for a wider structure due to the increased confinement provided by a wider structure.

Figure 3.10 presents the settlement ratio of the structure, S/D, versus the ratio of the treatment distance beyond the edge of the structure, SL, to the treatment zone depth, SD. The observed trends in this figure are similar to those in Figure 3.9. As observed by Hatanaka et al., Figures 3.9 and 3.10 indicate there may be an optimum treatment width beyond which the decrease in structure settlement is minimal. Figure 3.10 indicates that this optimal treatment width may be on the order of 0.5 times the treatment depth or greater, and is dependent in part on the width of the supported structure.

Hatanaka et al. also investigated the effect of structure height (BH) to width (BW) ratio on settlement ratio for cases with no ground densification. Figure 3.11, which they developed based on experimental results, indicates that as the height to width ratio, BH/BW, increases the settlement ratio for the structure also increases. Such an effect may also be applicable to structures supported on improved ground, although no experimental evidence is provided in the paper to directly support this hypothesis.

3.2.2.2 Treatment Depth

The effect of treatment depth on the response of a footing supported on a densified sand zone within a liquefiable sand deposit was investigated by Liu and Dobry (1997) using centrifuge tests. In the tests a circular steel footing having a height of 0.8 m (all quantities presented in prototype scale), diameter of 4.56 m, and exerting a bearing pressure of 100 kilopascals (kPa), was supported on the surface of a circular densified sand zone (Dr of approximately 90 percent) having a diameter of 7.3 m (1.6 times the footing diameter) and varying thickness. The footing and densified zone were located within a 12.5-m-thick liquefiable sand layer having a relative density of about 50 percent and a permeability of approximately 0.168 centimeters per second (cm/sec). The models were subjected to a 10-cycle, 1.5 Hz uniform sinusoidal acceleration record having an amplitude of 0.2g applied at the base. A schematic of the model set up is shown in Figure 3.12.

Figure 3.13(a) by Liu and Dobry presents the measured footing settlement, S_f, versus the ratio of the compaction depth, Z_c, to footing diameter, B. The results in this figure indicate that the magnitude of footing settlement can be reduced by increasing the depth of treatment beneath the footing. This conclusion is also supported by Figure 3.13(b) where the footing settlement has been normalized by the settlement of the surrounding soil in the free-field, S_s, after liquefaction and dissipation of excess pore water pressures. Both Figures 3.13(a) and 3.13(b) suggest that there may be an optimum depth of treatment beneath a footing beyond which the reduction of the footing settlement is relatively minor. Liu and Dobry note that most of the observed footing settlement occurred during shaking and is primarily due to the inertial load acting on the footing.

Although increasing the depth of densification beneath the footing resulted in decreased settlement of the footing, it also had the adverse effect of increasing the peak footing acceleration. Figure 3.14 by Liu and Dobry, which presents the normalized footing acceleration (for both the peak footing acceleration and the footing acceleration at the sixth cycle) versus the normalized compaction depth, shows this trend.

The variation of footing settlement and acceleration with densification depth suggests it may be necessary to optimize the treatment depth to reduce the footing settlement to an acceptable level while at the same time ensuring that the footing acceleration is tolerable.

3.3 Numerical Modeling of Liquefaction and Ground Behavior Under Seismic Loading

3.3.1 Overview of Methods

Over the years a number of different approaches have been developed for evaluating the behavior of ground and supported structures under seismic loading, including cases where liquefaction is expected to occur. As discussed in the next chapter, one of the key factors affecting the performance of a structure subjected to an earthquake is the expected movement of the structure and the ground supporting it during and after the event. In addition, the accelerations that the structure experiences will likely have an impact on its performance. Therefore, the ability to predict the performance of a structure, such as a highway bridge, is dependent on predicting the ground and structure movements, as well as accelerations in some cases.

Predicting the response of a structure supported on liquefiable soil during an earthquake is highly dependent on adequately accounting for the effects of pore water pressure development, stress-strain softening, and strength reduction in the soil on the system behavior. Although simplified methods, such as evaluating the post-earthquake factor of safety for stability, have been found useful to provide a relative assessment of performance, such analyses generally can not be used as a predictor of deformation (Riemer et al., 1996). Predicting deformations usually requires performing some type of dynamic response and deformation analyses using numerical modeling techniques. In these analyses the phenomena affecting ground response are treated as either coupled or uncoupled to each other.

3.3.2 Uncoupled Analyses

Assessing ground and structure performance using uncoupled analyses involves evaluating different key aspects of behavior separately, and sharing results between analyses when appropriate. The success of such an approach depends on whether there is a strong interdependence between phenomena such that they significantly affect each other during dynamic loading. If there is not a strong interdependence between them, then an uncoupled analysis may provide reasonable predictions of ground behavior and performance.

One common, uncoupled analytical approach for calculating permanent ground displacements under seismic loading and liquefaction makes use of the Newmark (1965) method. In this approach, a dynamic ground response analysis is typically performed using the equivalent linear method with programs such as SHAKE (for one dimensional cases) or QUAD4M (for two dimensional cases). From this analysis the following information is obtained and used: (1) the strain that develops in different elements which indicates zones where significant softening or liquefaction of the soil will occur, and (2) average acceleration records for different potential sliding masses. Limit equilibrium analyses are then performed, using degraded soil properties in the soil profile, to calculate the yield acceleration producing a factor of safety of one for each potential sliding surface. The lateral displacement is then typically estimated for the surface with the lowest yield acceleration by double integrating the portion of the average acceleration record that exceeds the yield value.

In a recent study of the predicted movements of a highway bridge abutment described in Section 3.2.1.1, Riemer et al. (1996) found that predicted lateral displacements obtained using Newmark's method were in reasonably close agreement with those obtained using analyses performed with the computer program FLAC (Cundall, 1993) when a "stiff" soil model was used for the liquefied soil as opposed to a "soft" soil model (refer to Section 3.2.1.1 for an explanation of "soft" and "stiff"). According to Riemer et al., the Newmark analyses seemed to capture the important aspects of lateral deformation behavior and the results provided similar conclusions regarding the effectiveness of ground improvement as those obtained with the FLAC analyses.

They found that the Makdisi and Seed (1978) chart solutions for predicting lateral displacements of embankments, which were developed using Newmark's method, did not agree well with predicted displacements from the other methods. The poor agreement of the predicted embankment displacements from the Makdisi and Seed method with the other predicted values may have been due to the Makdisi and Seed method being developed for relatively high (i.e. – 23 m to 46 m) embankments/dams where no significant soil strength loss occurs. It is important to note that the predicted deformations from the FLAC analyses using the "stiff" soil model generally ranged from 0.25 to 0.50 of those obtained using the "soft" soil model, and verification was not performed of the ability of either model to predict deformations measured in laboratory tests or field case histories. Therefore, the conclusion that the Newmark method is able to predict lateral displacements under seismic loading and liquefaction is only circumstantial in this case.

There are several concerns and limitations of Newmark's method for predicting lateral ground movements where liquefaction is involved. Recent studies by Dobry and Abdoun (1998) indicate that even liquefied, loose soils may exhibit dilative behavior which could result in Newmark predictions being conservative if residual strengths (i.e. – Seed and Harder, 1990) are used. Finn (1998) indicates Newmark's method is only applicable when liquefied layers are relatively thin. This limitation exists because Newmark's method assumes the materials above and below the failure surface are rigid bodies and all of the deformation occurs along a sliding plane between the two blocks. Such an assumption does not fit the behavior of an improved ground zone within a thick, liquefiable soil deposit in the majority of cases.

3.3.3 Coupled Analyses

In a coupled analysis the key phenomena influencing the behavior of liquefiable ground under dynamic loading are incorporated in a single analysis. This type of analysis therefore typically includes some aspect of pore pressure development, stress-strain softening, and strength reduction in the soil under dynamic loading conditions. Groundwater flow may also be incorporated.

Although both total and effective stress coupled analyses exist, the more comprehensive approach is typically an effective stress analysis that includes pore water pressure development and migration. In both the total and effective stress analyses, varying degrees of coupling are possible. A fully-coupled analysis is one where the solution scheme directly accounts for the dynamic interaction between the solid and fluid media, and the mechanical equilibrium and flow equations are solved simultaneously. In a partially-coupled or loosely-coupled analysis, the solution generally does not directly account for all of the interactions between the solid and fluid media, but uses some simplifications. For instance, in some partially-coupled analytical solutions the mechanical equilibrium equations may be solved first and then plastic volumetric strain estimated on the basis of the shear strain that occurs. The plastic volumetric strain is then applied to the system to get the increment in pore water pressure that occurs at that time. This cycle of calculations is then repeated for subsequent times.

The advantage of fully-coupled analyses is the solution directly accounts for the solid-fluid interaction and, from that standpoint, is more complete. However, some parameters required in fully-coupled analyses to describe the interaction between the fluid and solid, as well as the overall soil mass behavior, may be difficult to determine. On the other hand, the partially-coupled analyses tend to use more readily obtainable soil properties and less complex soil models. However, these simplifications can result in inaccuracies in predicted behavior.

The ability of various fully- and partially-coupled solutions to predict the behavior of liquefiable soil under dynamic loading, with and without structures, has been investigated by a number of researchers. One of the largest studies of this type was the Verification of Liquefaction Analyses by Centrifuge Studies (VELACS) Project sponsored by the National Science Foundation from 1991 to 1993. This project focused on (1) the ability to obtain repeatable results from geotechnical centrifuge tests on models involving liquefiable soils and various structures subjected to dynamic loading and (2) the ability to predict the behavior observed in the centrifuge models using numerical modeling. In each of the centrifuge tests performed, pore water pressures, accelerations, and displacements were measured at key locations. The results of the centrifuge tests and numerical modeling are presented in the "Proceedings of the International Conference on the Verification of Numerical Procedures for the Analysis of Soil Liquefaction Problems" (Arulanandan and Scott, 1993/1994).

Results from the VELACS project indicated that the partially-coupled or uncoupled solution schemes used in that study did not perform particularly well. In a summary paper reviewing the numerical modeling procedures used, Smith (1994) expressed his opinion that any type of simplified procedure for modeling the liquefaction of soil could not compete with "… fully coupled codes operated by experts." Arulanandan (1996) states that one of the conclusions of the VELACS project was "only fully-coupled, effective stress based, elastoplastic numerical procedures are capable of predicting the deformation of earth structures due to earthquake-induced liquefaction." On the other hand, Scott (1994) performed a general overview of the numerical predictions done for the VELACS project and stated that "… overall, there is a lack of uniformity in the predictions." His statement indicates no one code or formulation predicted the results of all of the model tests consistently. In addition many codes, including those that were fully coupled, did not perform particularly well for model cases involving footings or walls supported on liquefiable soil.

In other published work, several researchers have reported the successful use of fully-coupled analyses for predicting the behavior of liquefiable soil measured in laboratory tests or field case histories. Arulanandan et al. (1997) reports on the successful use of DYSAC2 to predict the behavior of liquefiable soil deposits in the Marina District of San Francisco during the 1989 Loma Prieta earthquake. Both the codes DYSAC2 and DYNAFLOW are reported by Arulanandan (1996) as having adequately predicted the response of a submerged embankment subjected to dynamic loading in a centrifuge test. According to Madabhushi and Zeng (1998), the code SWANDYNE successfully predicted the seismic response of a gravity quay wall on a liquefiable sand modeled in the centrifuge. The program SUMDES (Li et al., 1992), which is a code utilizing a fully-coupled solution for evaluating liquefiable soil deposits under 1-D dynamic loading, has been reported by Li et al. (1998) as providing reasonable predictions of a site response for the 1986 Lotung earthquake in Taiwan.

Success is also reported in the literature with using solutions that are partially coupled in varying degrees for predicting the behavior and deformations of liquefiable soil. Finn (1988, 1991) reports the successful validation of TARA-3 using centrifuge studies. Puebla et al. (1997) report on the successful use of an elasto-plastic soil model (UBCSAND) implemented in the computer program FLAC (Cundall, 1995) to predict the pore pressure development and deformations in a liquefiable sand beneath an embankment built in the field. The embankment was constructed rapidly to induce static liquefaction. They also note that UBCSAND and FLAC were successful in predicting deformations and pore water pressures measured in a centrifuge model test used to design the field program. Beaty and Byrne (1998) used UBCSAND and FLAC to perform a 1-D simulation of the response of the instrumented Wildlife Site in Southern California where liquefaction and lateral spreading occurred during the 1987 Superstition Hills

Earthquake. In this case, they were able to predict the time that liquefaction occurred reasonably well, but the predicted post-liquefaction behavior in terms of surface velocity and relative displacement deviated from the measure response.

The successful use of a partially-coupled solution in FLAC for prediction of the behavior of caisson retaining walls in liquefiable soils is reported by Dickenson and Yang (1998). In their analyses, they used a nonlinear, effective stress analysis method based on the Mohr-Coulomb constitutive model, which required strength and elastic modulus parameters, and a pore water pressure increment scheme based on the work of Seed and his co-workers (e.g. – Martin and Seed, 1975; Seed and deAlba, 1986). They also incorporated adjustments for the effect of initial static shear stresses and overburden stresses on liquefaction, as well as magnitude scaling factors. According to the authors, predicted horizontal and vertical displacements were consistent with displacements measured at five caisson retaining walls along waterfronts in Japan during various earthquakes between 1983 and 1995. Likewise, McCullough and Dickenson (1998), who used the same analysis method and soil model in FLAC, report fairly good agreement between predicted and measured permanent horizontal displacements at the top of five anchored sheet pile bulkhead walls in liquefiable soils subjected to different earthquakes in Japan between 1978 and 1993.

Based on the information available in the literature, there has been reported success in predicting the measured behavior of liquefiable soils, with and without structures present, using both partially- and fully-coupled codes. The degree of success appears, in some cases, dependent on the type of problem analyzed. Therefore, a critical part of selecting and using a numerical code is verifying its ability to predict the behavior for the range of conditions of interest.

3.4 Issues Requiring Investigation

As discussed in this chapter, studies by various researchers indicate that ground treatment can improve the performance of structures, such as embankments and shallow foundations, constructed over or in liquefiable soil deposits. However, the predicted ground deformations for the improved ground cases evaluated in those studies were still sufficiently large that they would cause damage to many bridges. Therefore, the primary issue to be investigated in this research was whether permanent ground deformations at stub abutments and piers of existing bridges could be reduced to acceptable levels using available ground improvement technology. An integral part of assessing the feasibility of various treatment methods was evaluating how the predicted performance varies with the type, size, and location of the improved zone. The effects of these factors were studied through a parametric study performed using numerical analyses.

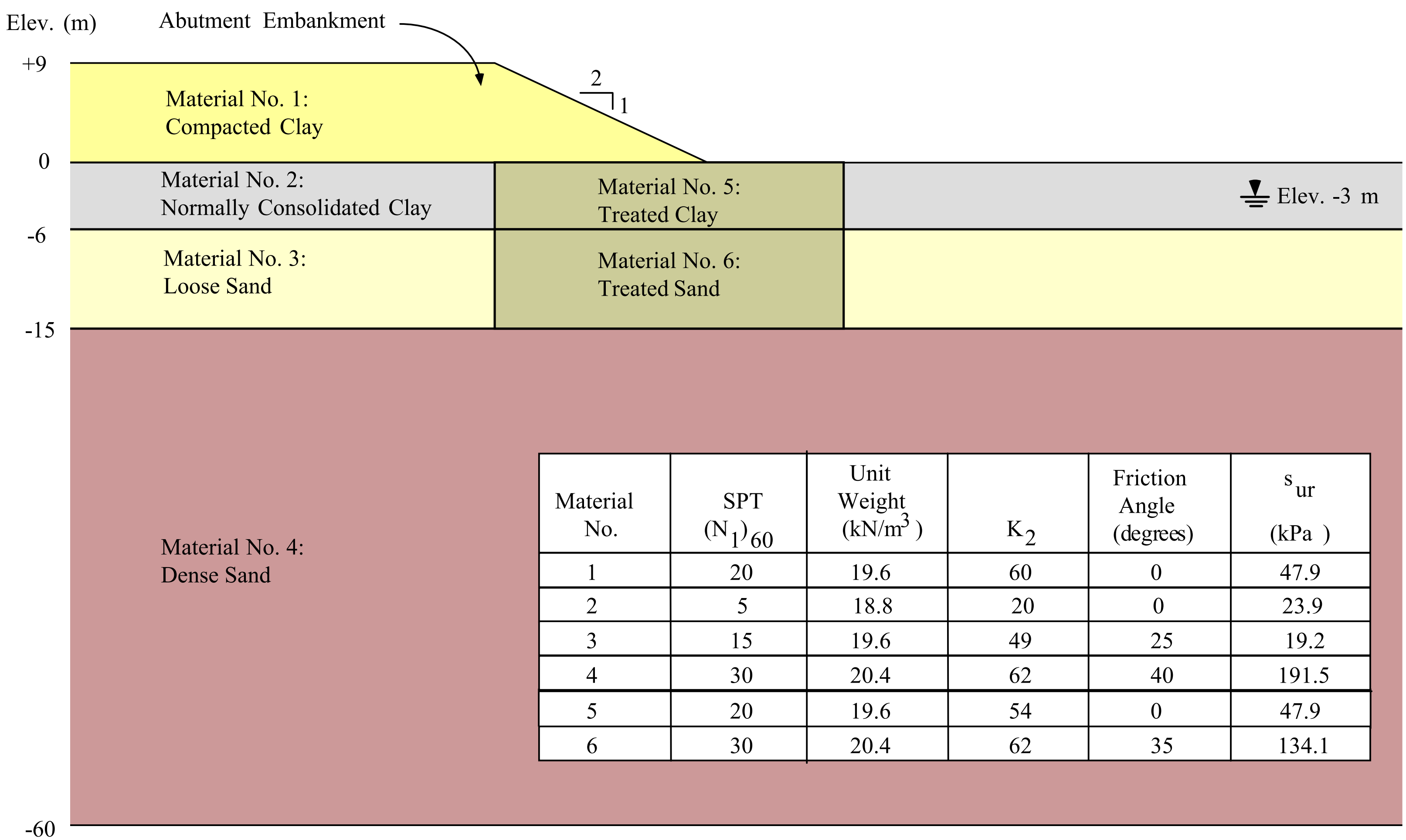
In order to conduct the proposed study, an analytical tool that could reliably predict the behavior of improved ground zones and supported structures within liquefiable deposits was needed. The wide variability in the reported success for various models and computer codes suggested that any analytical tool chosen had to be independently tested. Therefore, part of this study included selecting and checking the ability of a comprehensive, coupled analytical method to predict the ground behavior by using it to evaluate centrifuge tests and field case histories for which measured data was available. This work provided additional information regarding the capability of a coupled analysis to predict improved ground behavior, as well as verification of the method before it was used for a parametric study. In addition to evaluating a comprehensive method, selection and assessment of simplified analytical methods, such as those used in uncoupled analyses, were performed.

Prior to evaluating the use of improved ground for liquefaction mitigation at existing bridges, there was a need to clarify the associated phenomena that affect performance and design. These phenomena are identified and explained in the next chapter, along with performance requirements for highway bridges.

TABLE 3.1: Comparison of Improved and Unimproved Embankment Response for Adalier et al. (1998) Centrifuge Tests

Improvement Type	Ratio of Crest to Base[1] Peak Acceleration[2]	Ratio of Improved to Unimproved Embankment		
		Peak Crest Acceleration[2]	Average Crest Settlement[3]	Lateral Toe Displacement[3]
None	0.9	-	-	-
Densified Zones	1.3	1.4	0.5	0.5
Cemented Blocks	1.5	1.6	0.8	NA[4]
Gravel Berms	1.1	1.2	0.7	0.5
Sheet Piles	0.7	0.7	0.6	0.0

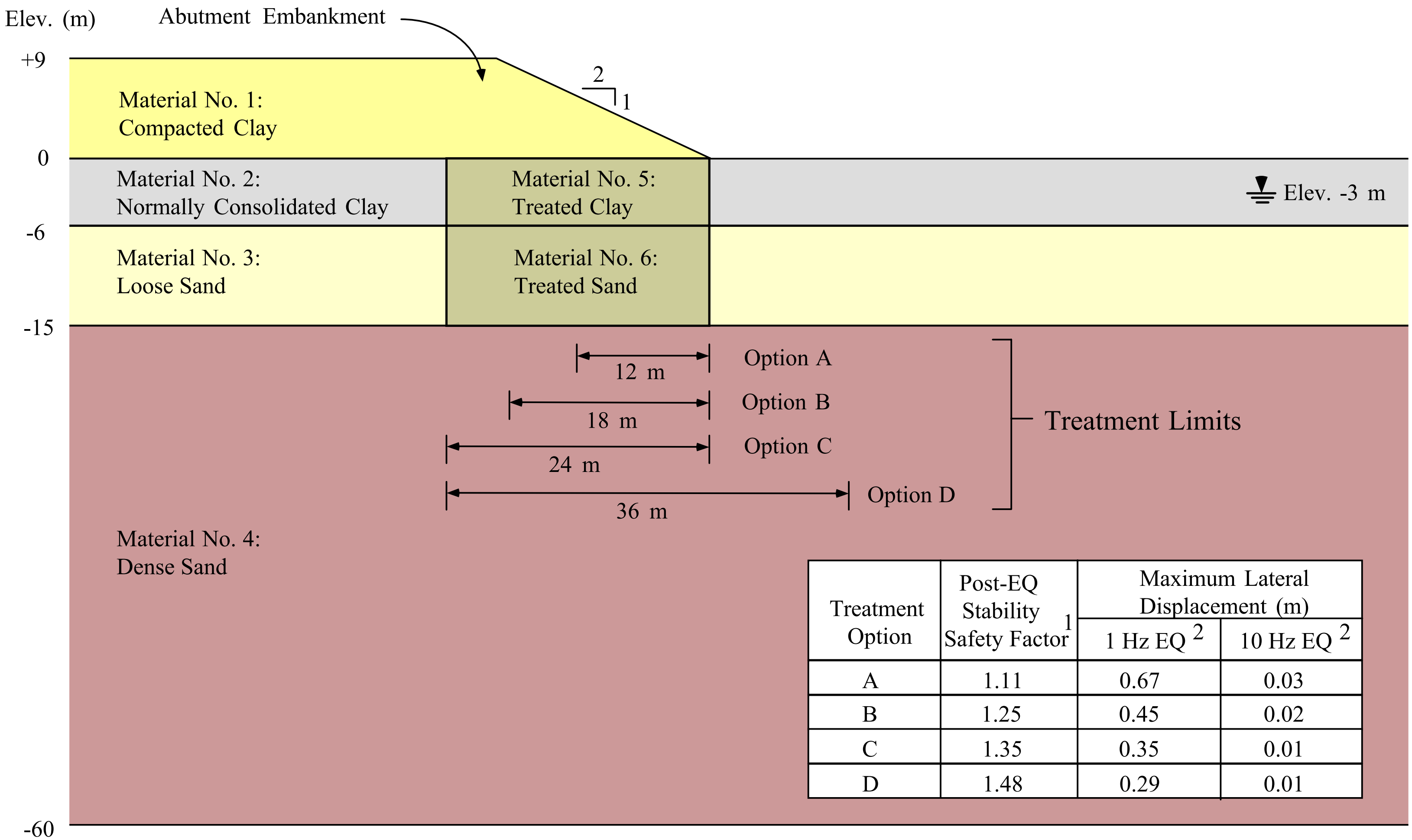
Notes:
1. Third shaking event used with centrifuge container base acceleration having amplitude of 0.3g.
2. Ratios of accelerations were computed using peak accelerations scaled from records for third shaking event presented in Adalier et al. (1998).
3. Ratios are for total deformations after all three shakes.
4. Large lateral deformations observed at toe but measured data not available (NA).



Material No.	SPT $(N_1)_{60}$	Unit Weight (kN/m^3)	K_2	Friction Angle (degrees)	s_{ur} (kPa)
1	20	19.6	60	0	47.9
2	5	18.8	20	0	23.9
3	15	19.6	49	25	19.2
4	30	20.4	62	40	191.5
5	20	19.6	54	0	47.9
6	30	20.4	62	35	134.1

Notes: 1. K_2 is factor used for calculating maximum shear modulus
2. s_{ur} is the post-seismic shear strength.

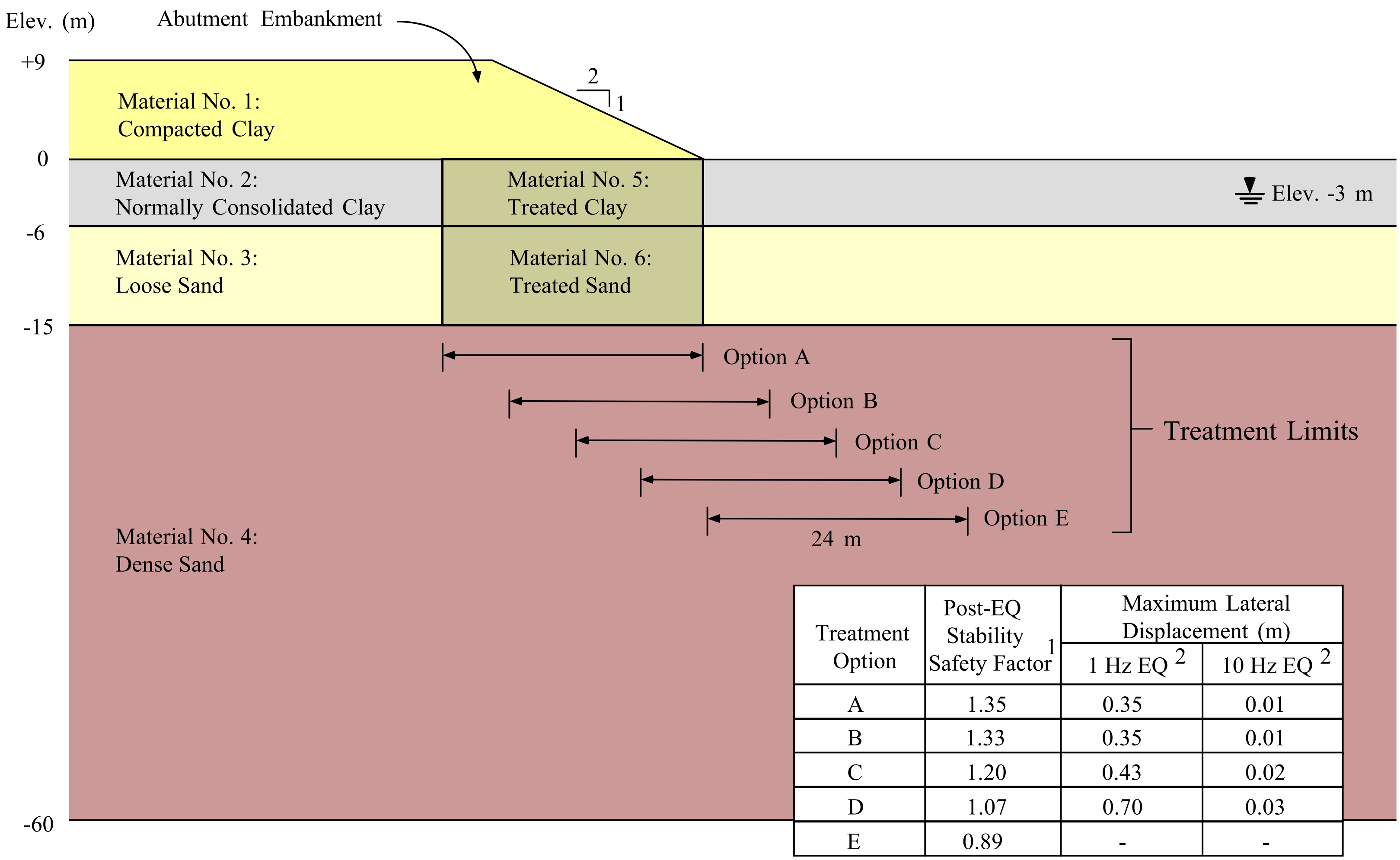
FIGURE 3.1: Soil Profile and Properties for Highway I-57 Bridge Used in Study by Riemer et al. (after Riemer et al., 1996)



Treatment Option	Post-EQ Stability Safety Factor [1]	Maximum Lateral Displacement (m)	
		1 Hz EQ [2]	10 Hz EQ [2]
A	1.11	0.67	0.03
B	1.25	0.45	0.02
C	1.35	0.35	0.01
D	1.48	0.29	0.01

Notes: 1. For no ground treatment the calculated post-earthquake factor of safety for stability was 0.71.
2. Input motions had durations of 8 to 10 seconds and peak amplitude scaled to 0.22g.

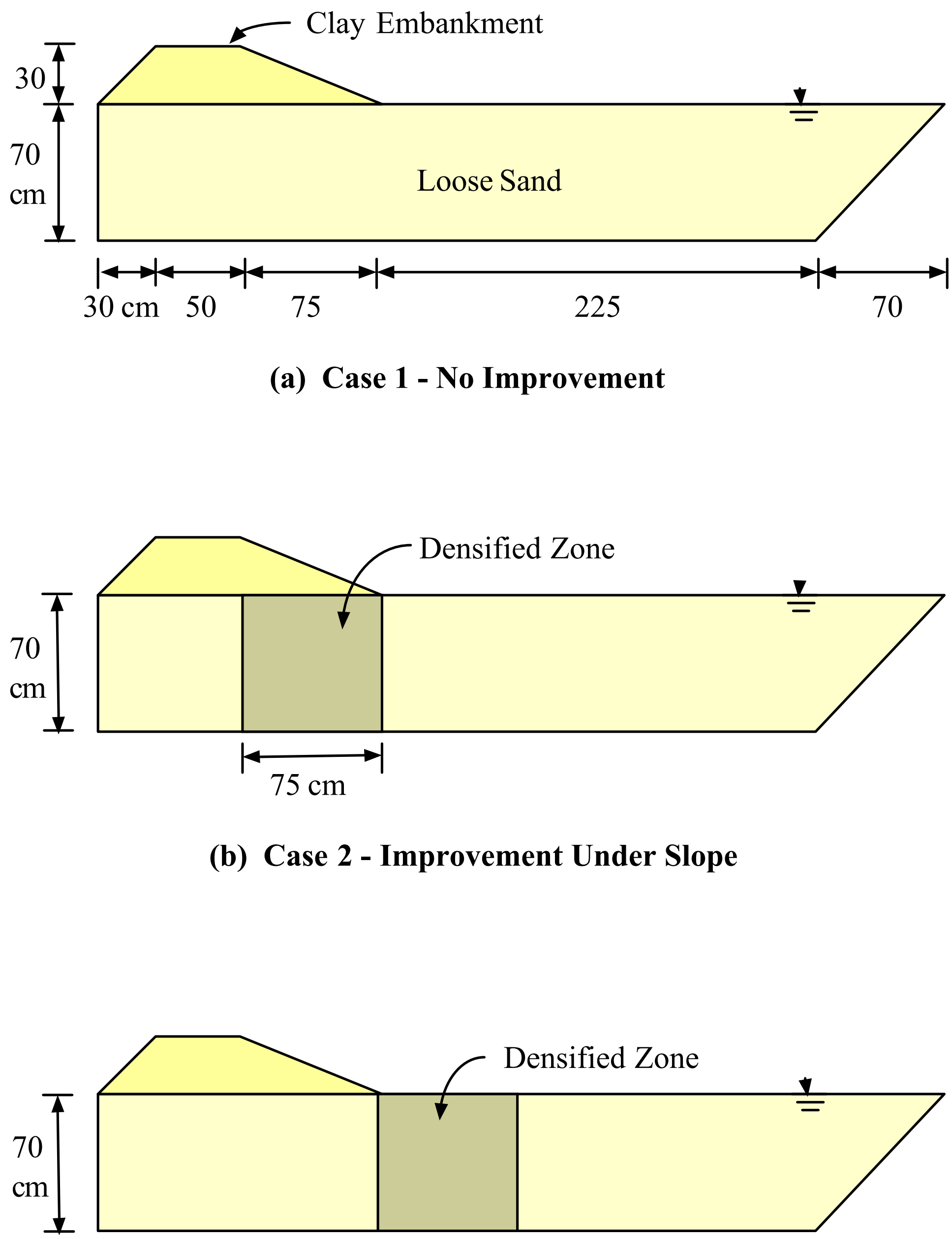
FIGURE 3.2: Impact of Treated Zone Size on Predicted Performance of Highway I-57 Bridge Abutment (after Riemer et al., 1996)



Treatment Option	Post-EQ Stability Safety Factor[1]	Maximum Lateral Displacement (m)	
		1 Hz EQ[2]	10 Hz EQ[2]
A	1.35	0.35	0.01
B	1.33	0.35	0.01
C	1.20	0.43	0.02
D	1.07	0.70	0.03
E	0.89	-	-

Notes: 1. For no ground treatment the calculated post-earthquake factor of safety for stability was 0.71.
2. Input motions had durations of 8 to 10 seconds and peak amplitude scaled to 0.22g.

FIGURE 3.3: Impact of Treated Zone Location on Predicted Performance of Highway I-57 Bridge Abutment (after Riemer et al., 1996)



(a) Case 1 - No Improvement

(b) Case 2 - Improvement Under Slope

(c) Case 3 - Improvement in Front of Toe

FIGURE 3.4: Shaking Table Test of Embankment With and Without Improvement by Yanagihara et al. (after Yanagihara et al., 1991)

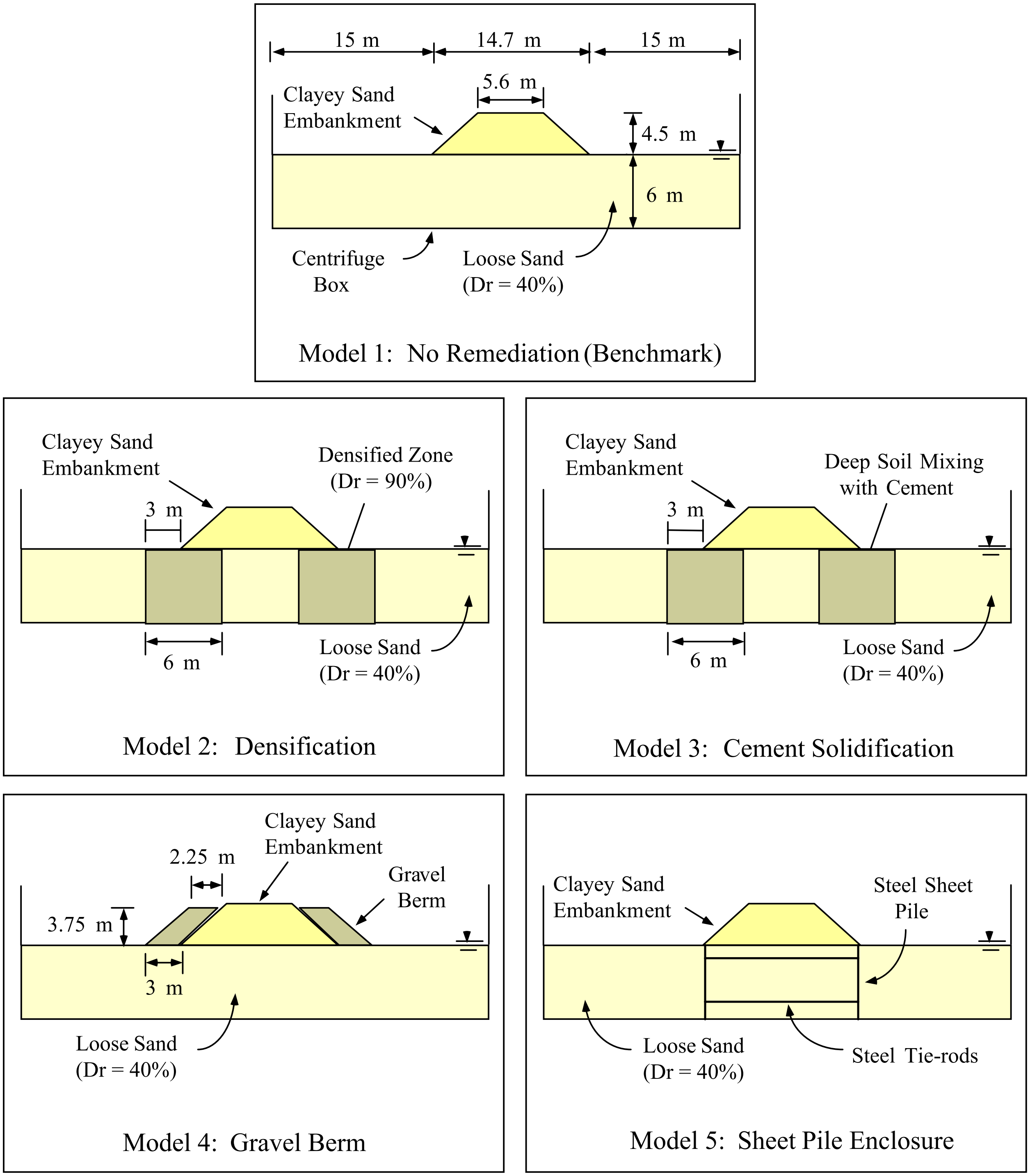


FIGURE 3.5: Improvement Schemes for Embankment Modeled in Centrifuge Tests by Adalier (after Adalier et al., 1998; dimensions in prototype scale)

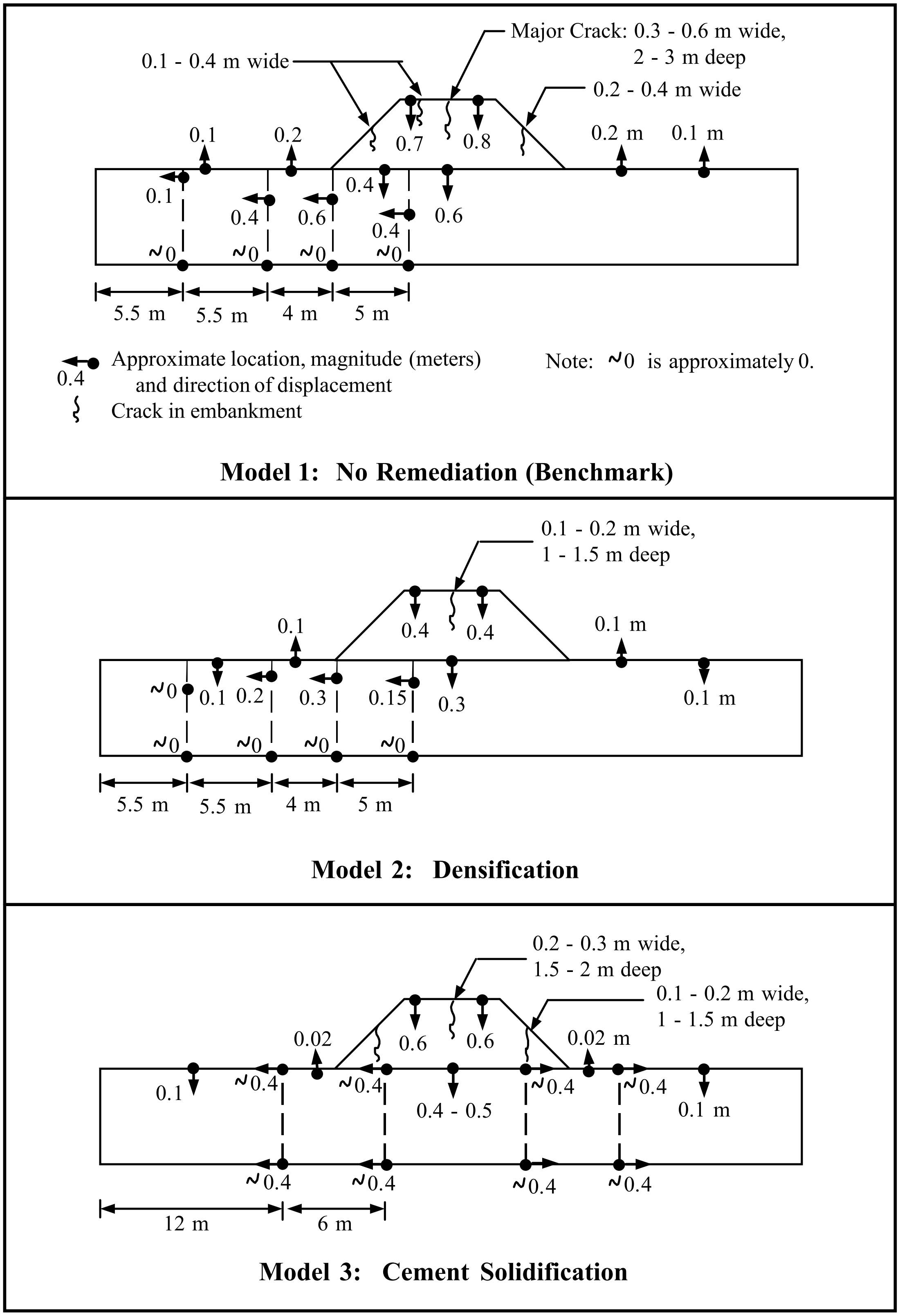


FIGURE 3.6: Ground Deformations (in Prototype Scale) Observed in Centrifuge Tests by Adalier (1996) After Three Shaking Events (after Adalier et al., 1998)

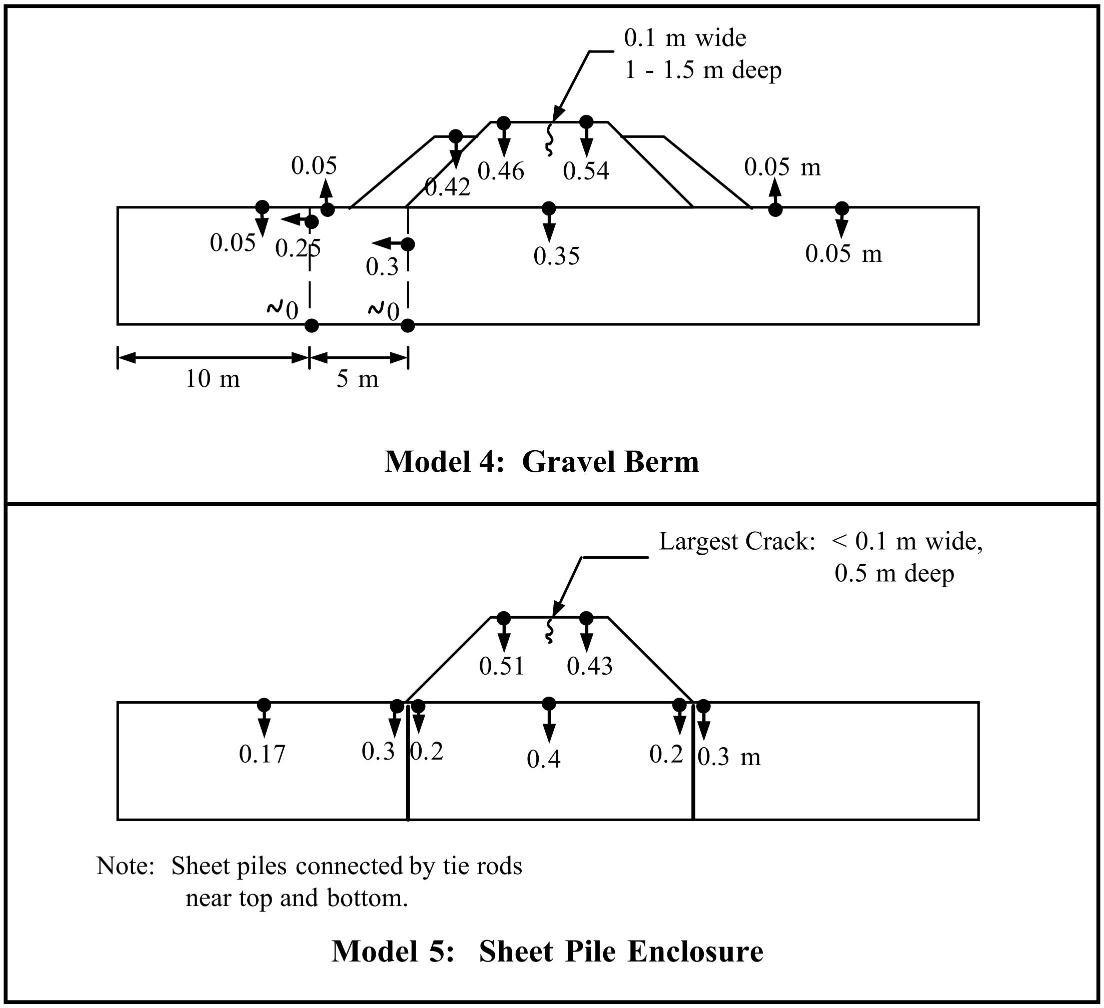


FIGURE 3.6 (cont.): Ground Deformations (in Prototype Scale) Observed in Centrifuge Tests by Adalier (1996) After Three Shaking Events (after Adalier et al., 1998)

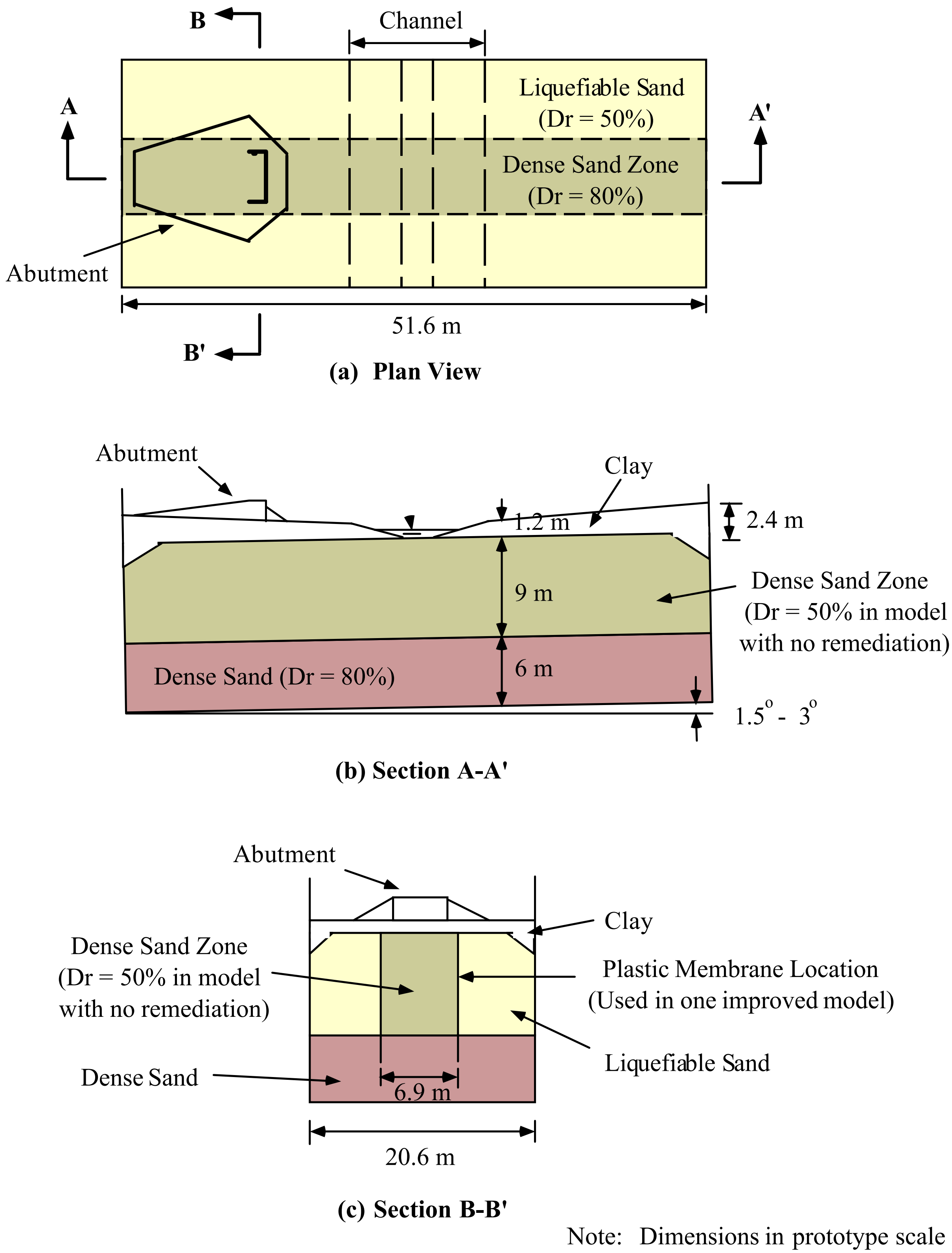


FIGURE 3.7: Schematic of Centrifuge Test with Improved (Dense) Zone by Balakrishnan et al. (after Balakrishnan et al., 1998)

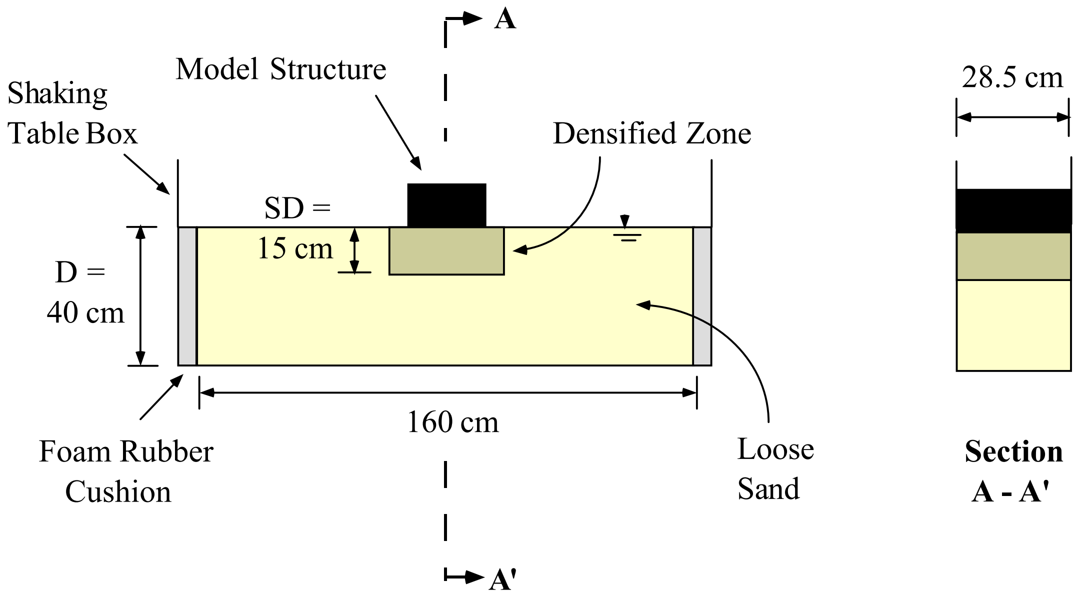


FIGURE 3.8: Sections Showing Structure on Densified Zone Modeled in Shaking Table Tests by Hatanaka et al. (after Hatanaka et al., 1987)

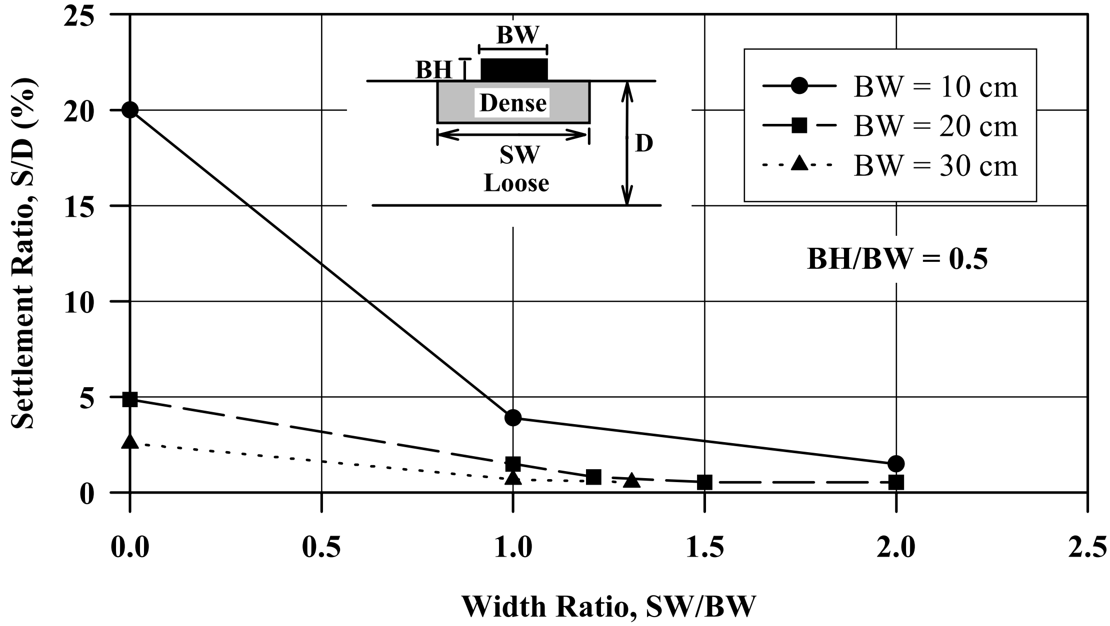


FIGURE 3.9: Relationship Between Settlement and Width Ratios for Improved Zone in Shaking Table Tests (after Hatanaka et al., 1987)

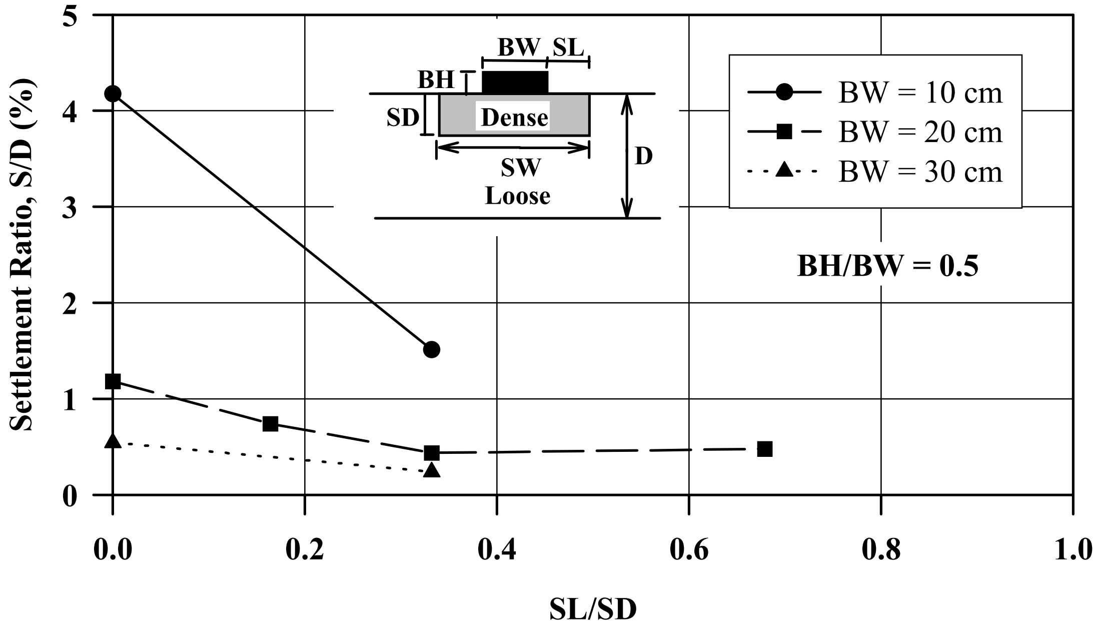


FIGURE 3.10: Relationship Between Settlement Ratio and Ratio of Treatment Width Beyond Structure to Treatment Depth for Improved Zone (after Hatanaka et al., 1987)

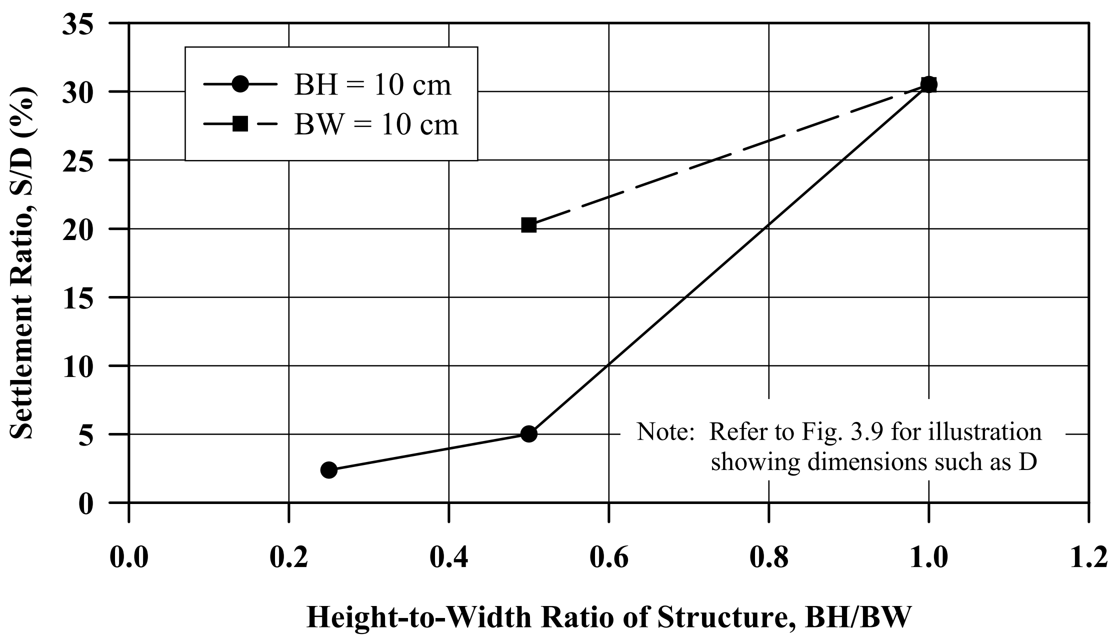
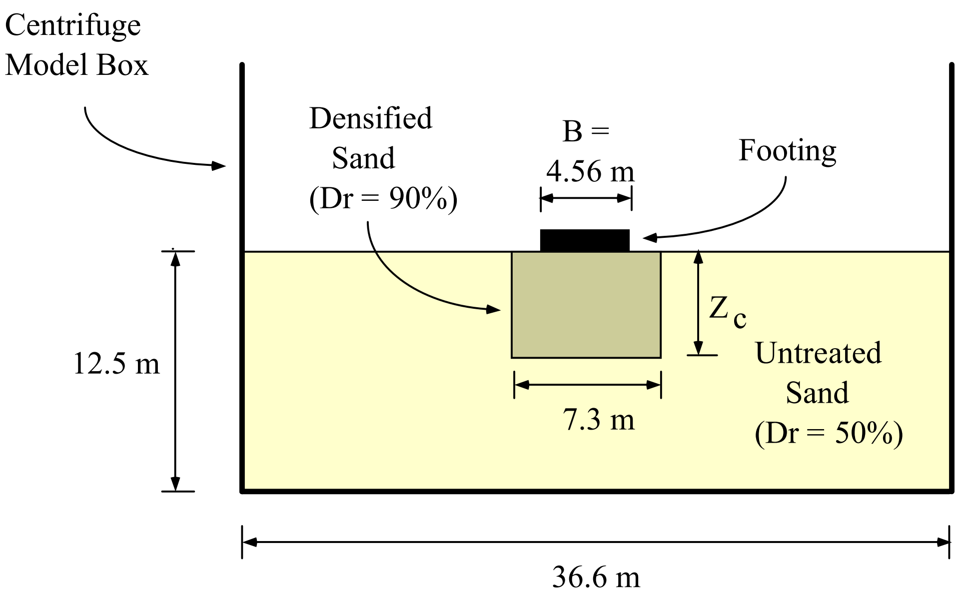
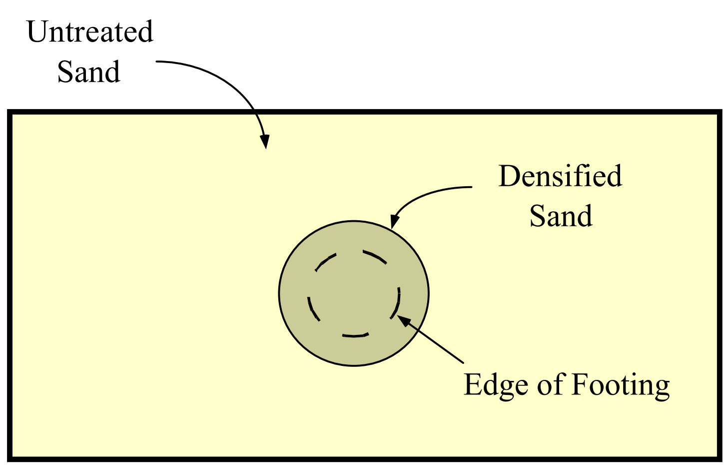


FIGURE 3.11: Relationship Between Settlement Ratio and Height-to-Width Ratio of Structure on Unimproved Soil (after Hatanaka et al., 1987)

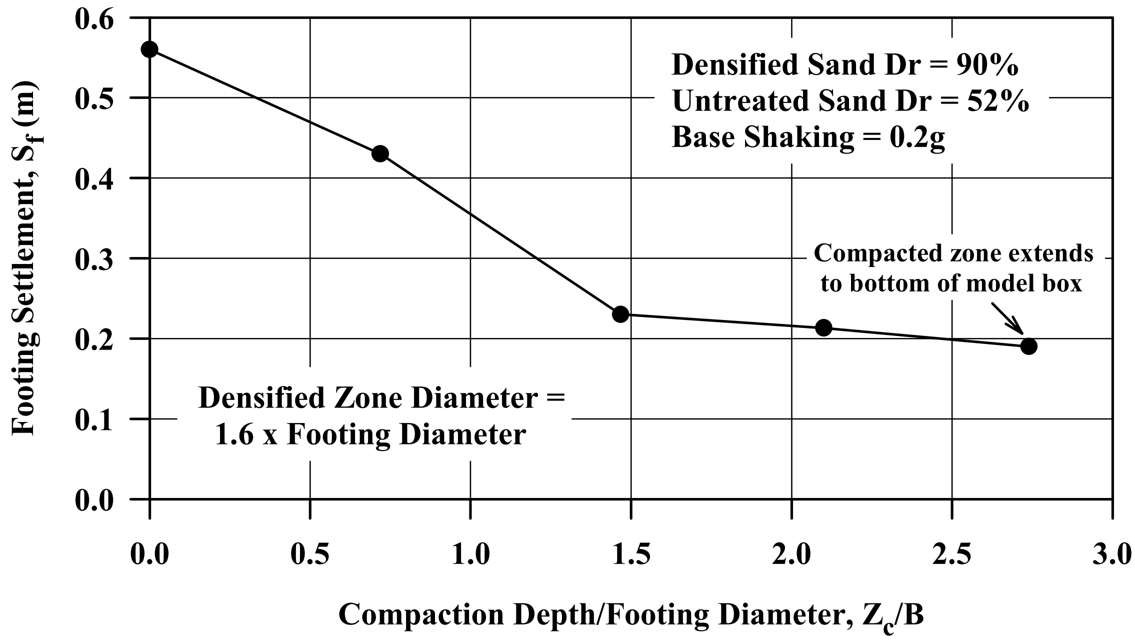


Profile

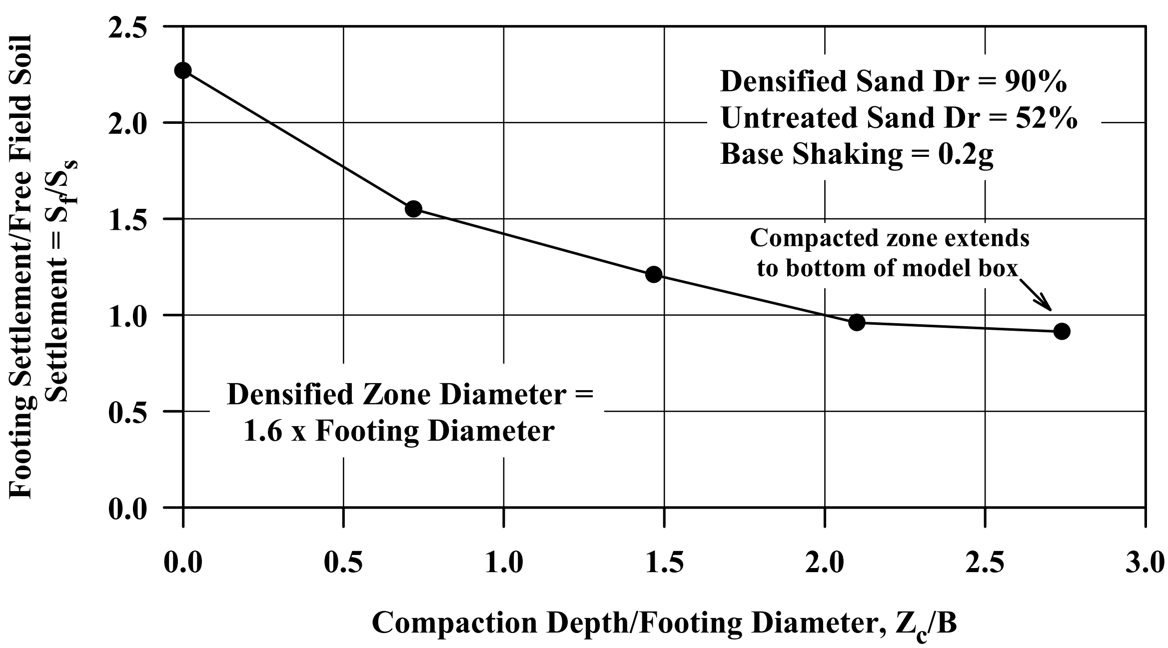


Plan View

FIGURE 3.12: Centrifuge Model of Footing on Densified Zone Tested by Liu and Dobry (after Liu and Dobry, 1997; dimensions in prototype scale)



a) Settlement Not Normalized



b) Settlement Normalized

FIGURE 3.13: Foundation Settlement versus Normalized Compaction Depth for Centrifuge Tests (after Liu and Dobry, 1997; settlement in prototype scale)

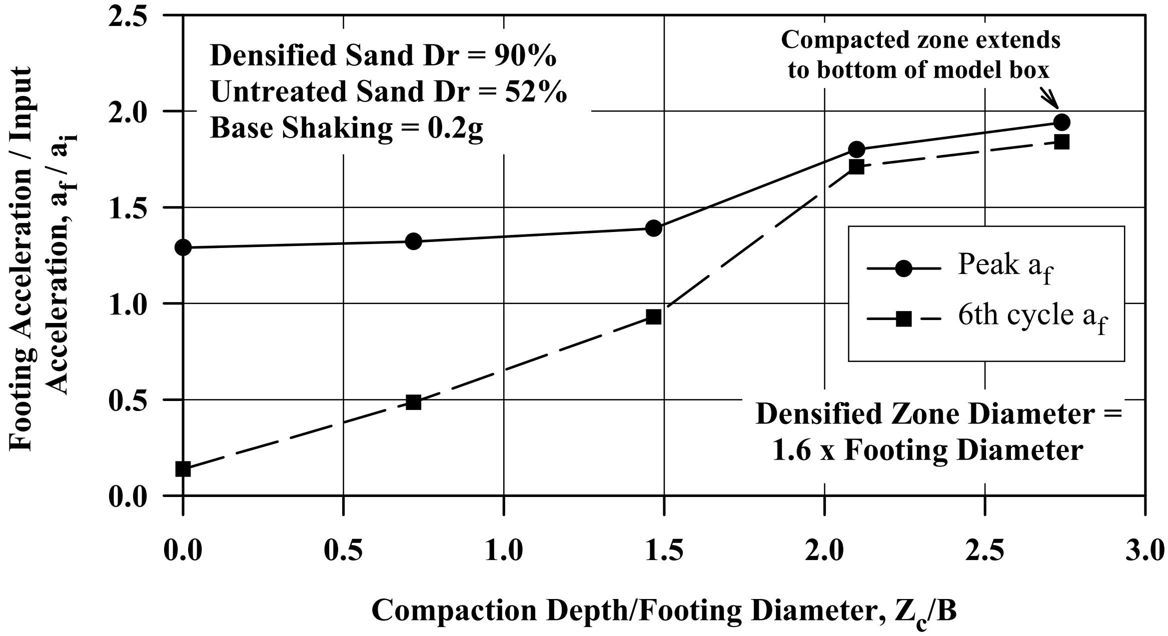


FIGURE 3.14: Normalized Footing Acceleration versus Normalized Compaction Depth for Centrifuge Tests (after Liu and Dobry, 1997)