6.0 SYNTHESIS OF ALL WORK AND REVISED HYPOTHESIS

6.1 Introduction

The objective of this chapter is to bring together what is currently known about aging of sands in the laboratory and in the field into a consistent framework. The results of the laboratory testing program presented in the previous chapter are interesting and yet they are contradictory with other laboratory studies in the literature by Dowding and Hryciw (1986) and Joshi et al. (1995). Another look is taken at the laboratory studies mentioned above to reconcile the differences in the results of these studies in the literature and the current research. In addition, a theoretical parametric study of the influence of stress level and stiffness on the penetration resistance of loose and dense sands is presented using cavity expansion theory. The objective is to see if reasonable changes in properties such as stiffness and horizontal effective stress can theoretically account for the time-dependent increases in penetration resistance observed in the field.

Finally, a revised hypothesis of what causes aging effects in sands that is consistent with the current study is presented.

6.2 Review of Significant Laboratory Studies on the Aging of Sands

The significant aspects of the current experimental study are summarized below:

- Uniform, saturated sand samples were aged in rigid wall cells for 30 to 118 days over a range of controlled densities, pore fluids, and temperatures.
- Some time-dependent property changes were observed, such as increases in the small strain shear modulus. In two tests, electrical conductivity measurements and chemical analyses suggested that precipitation of carbonate minerals and silica occurred. However, scanning electron micrographs were unable to find visible evidence of precipitation.
- These property changes did not translate into changes in mini-cone penetration resistance.

The findings of this research suggest that a controlled laboratory testing program may miss the true mechanism(s) responsible for aging effects in the field. This is generally consistent with the results of the laboratory testing program performed by Human (1992), but contradicts the published findings of laboratory testing programs by Dowding and Hryciw (1986) and Joshi et al. (1995). Therefore, a more detailed review of the latter two studies is warranted to reconcile the apparent differences in the results.

6.2.1 Laboratory Study of Blast Densification (Dowding and Hryciw 1986)

As described in section 2.6.2, Dowding and Hryciw (1986) presented the results of a laboratory study of blast densification. Evanston sand was used in a 1 m diameter, 1 m high liquefaction tank, as shown in Figure 6.1. Samples were prepared by flowing water up through the bottom of the tank until the soil liquefied, and then shutting off the water and allowing the sand to settle. A relative density of 50% was achieved by gently lifting one end of the tank and tapping it on the floor until the desired density was achieved. The study consisted of detonating charges with different delays in the center of the tank, and monitoring the change in penetration resistance both radially away from the blast and with time. The penetration tests were performed with a 1.25 cm^2 mini-cone. Figure 2.17 showed the results of the penetration tests, and is reprinted here as Figure 6.2. This figure shows the change in average penetration resistance (Δq_c) from pre-blast values as a function of time and radial distance from the point of detonation. The average penetration resistance was obtained by taking the integral of the penetration resistance-depth relationship and dividing by the height of the sample. The magnitude of the changes in penetration resistance is very low (0 to 0.25 MPa), which is consistent with the fact that the effective stress conditions in the liquefaction tank were very low (due only to the buoyant weight of the soil). Of the data shown in Figure 6.2, the authors presented two of the actual penetration resistance profiles. Figure 6.3 shows the penetration tests involving the detonation of 2 charges with a 17 ms delay between blasts. Figure 6.4 shows the penetration test results for freshly deposited sand in the tank with no blasting.

To compare this data more directly to changes in the field, it is more instructive to look at the time-dependent increases in terms of a percent change. The values of penetration resistance at a depth of 40 cm from Figures 6.3 and 6.4 were used. To assess the percent change in penetration resistance with time, these values are shown in Table 6.1. The initial penetration resistance at 40 cm ranged from 0.38 to 0.44 MPa, which corresponds to a mass of approximately 5.2 kg acting on the 1.25 cm^2 cone. The percent change of the penetration resistance with time from the initial values is shown in Figure 6.5 for the test involving blasting and Figure 6.6 for the freshly deposited sample. Figure 6.5 shows that within 20 cm of the blast hole, there were significant time-dependent increases in the penetration resistance with time. Fifteen days following blasting there was a 75 to 90% increase in the penetration resistance within 20 cm of the blast. These increases are comparable to those reported in the field, such as at the Jebba Dam project.

Table 6.1 Penetration resistance at 40 cm depth in liquefaction tank.

BLASTING (2 charges, 17 ms delay)

Radial Distance from Center	Initial q_c at 40 cm	1 day (MPa)	5 days (MPa)	15 days (MPa)
10 cm	0.42	0.39	0.57	0.80
20 cm	0.41	0.46	0.64	0.72
30 cm	0.42	0.45	0.49	0.53
40 cm	0.38	0.42	0.42	0.45
FRESHLY DEPOSITED				
10 cm	0.41	0.46	0.46	0.51
20 cm	0.39	0.42	0.46	0.51
30 cm	0.44	0.46	0.48	0.50
40 cm	0.42	0.45	0.50	0.53

After 15 days, the sample was re-liquefied and large amounts of gas were seen exiting the sample for a radius of 20 cm around the blast hole. Since the areas that had gases showed the largest aging effects, the authors attributed the large changes in penetration resistance to an arching effect by the blast gases. As seen in Figure 6.6, time-dependent increases in penetration resistance were also observed in the freshly deposited samples with no blast-

ing. These magnitudes were smaller than for the blasting, but there was still a 25 to 30% increase in penetration resistance after 15 days.

There are several aspects of the blasting study, which were different from the current laboratory testing program, and may explain the differences in the results between the two. These factors include boundary conditions, the stress level at testing, the preparation of the samples, and the energy imparted to the samples.

One important factor that is not different between the two studies is the sand that was used. Evanston sand was used for both studies, and, in fact, it was chosen specifically for the current study because Dowding and Hryciw (1986) had measured aging effects using Evanston sand.

The boundary conditions of the two testing programs were very different. The penetration tests in the liquefaction tank were much closer to simulating free field conditions than those performed in the rigid wall cells. The latter tests were definitely influenced by the presence of the walls, however it is unclear how the walls may have affected time-dependent changes in penetration resistance.

The stress level at which each set of tests was performed was also very different. All of the samples in the rigid wall tests were loaded to a vertical stress of 100 kPa, whereas the tests in the liquefaction tank were performed at very low effective stress conditions. However, the current study did include four bucket tests, which were performed at the same stress levels and did not observe any significant increase in penetration resistance.

There were also differences between the two testing programs in the way the samples were formed and the energy imparted to the soil by blasting. Dowding and Hryciw (1986) liquefied their samples and then tapped the tank to reach a relative density of 50%. In the current study, sand was pluviated through the pore fluid and vibrated on a vibrating

table to reach a relative density of either 40% or 80%. These two methods may have resulted in samples with slightly different fabrics.

The blasting clearly affected the time-dependent increases in penetration resistance. The large, time-dependent increases occurred within a radius of 20 cm, which corresponded, to the radius of trapped gas that was released upon re-liquefaction at the end of the test. It is possible that, as discussed in section 3.5.2, the gas bubbles decreased the degree of saturation around the blast hole and created an arching effect. However, this mechanism can not explain all aging effects that are observed in the field, such as those in connection with vibrocompaction, dynamic compaction, or hydraulically placed fills.

The blasting did impart energy to the soil, which may be significant to the aging process. The samples in the rigid wall tests of the current study were vibrated to 40 or 80% relative density. However, this may not have imparted enough energy to influence time-dependent changes.

Thus, the significant differences between the current study and the study performed by Dowding and Hryciw (1986) are different sample preparation, different amount of energy imparted, and the generation of gases by blasting.

6.2.2 Laboratory Test on the Penetration Resistance of Sand (Joshi et al. 1995)

There are both similarities and differences between the laboratory aging study performed by Joshi et al. (1995) and the current research. Joshi et al. (1995) performed small penetration tests in rigid wall cells and studied the effect of pore fluid (air, distilled water, and seawater) on the time-dependent penetration resistance for two sands. Details of the testing program were presented in section 2.6.14.

Figure 6.7 shows a section and plan view of the equipment used for their study. The cells were 360 mm in diameter with a wall thickness of 20 mm, and were made of PVC pipe.

The height of specimens inside the cells was 290 mm. The penetration tests were actually small plate load tests performed by four, 10 mm diameter probes, which were embedded in the samples throughout the entire test. At different times, a load was applied to one of the penetrometers until ~2 mm of settlement occurred and the load-deformation relationship was recorded. A series of these penetration tests is shown in Figure 6.8. As discussed in section 2.6.14, the data presented by Joshi et al. (1995) suggests that for sand saturated in seawater, distilled, and dry, in one year of aging one could expect a 90%, 80%, and 30% increase in penetration resistance, respectively. Scanning electron micrographs of the saturated samples indicated the presence of precipitate on and around the sand grains, as shown in Figure 3.12. Joshi et al. (1995) concluded that the observed increases in penetration resistance were caused by a combination of rearrangement of particles and precipitation of the soluble fractions of the sand.

A comparison between the Joshi et al. (1995) study and the current research is shown in Table 6.2. The fact that each study came up with very different results makes a comparison very useful.

Table 6.2 Comparison between the current study and that of Joshi et al. (1995).

	Joshi et al. (1995)	Current Study
Diameter of rigid wall cell	360 mm	145 mm
Proximity of wall to penetrometer	8 diameters	5 diameters
Composition of cell wall	20 mm thick PVC	6 mm PVC; 6 mm steel composite
Vertical effective stress	100 kPa	100 kPa
Relative density	87%, 95%, 100%	40%, 80%
Pore fluids	Dry, Distilled water, Seawater	Dry, Distilled water, Ethylene glycol, CO_2-saturated water

The diameter of the cell in Joshi et al. (1995) was more than twice the diameter of the cells in the current study. However, the proximity of the cell wall to the penetrometers in both studies was similar. The saturated samples for both studies were prepared in the

same way by pluviating through the pore fluid and then vibrating the samples to the desired density. The stress level was also the same for both studies.

With so many similarities, why did the penetration resistance almost double in two years in the Joshi et al. (1995) study, while there was no change in the penetration resistance for the current research? The fact that large increases occurred using PVC cells by Joshi et al. (1995) suggests that possible lateral expansion of the PVC-steel cells used in this study was not a factor. For the current study, PVC cells were abandoned and replaced with the stiffer composite cells because it was found that the PVC crept with time and adversely affected the shear wave velocity measurements. For both studies, the presence of the rigid walls most likely influenced the penetration resistance of the probes in a similar manner, considering that the walls were only 5 to 8 diameters away from the penetration tests.

Although the tests in the Joshi et al. (1995) study are described as penetration tests, they are quite different from cone penetration tests. As shown in Figure 6.7, the tests are akin to small plate load tests, and the penetrometers are like miniature drilled shafts with casing to remove the effects of shaft friction. The penetrometers remained in the samples throughout the aging process. Unlike the current study where the mini-cone was pushed continuously for approximately 12 cm, the displacements for the penetration tests performed by Joshi et al. (1995) were very small. At each time, the penetrometer was pushed approximately 2 mm and the load-deformation curve was recorded. After two years of aging, the total displacement for each of the four penetrometers in the sample was about 10-12 mm. The small scale of the tests is better illustrated by plotting the cumulative displacements for each test, which is shown in Figure 6.9. From this figure, it is also apparent that the testing program consisted of a series of loading-unloading cycles. Again, this is quite different from what occurs during cone penetration tests.

The penetration test results shown in Figures 6.8 and 6.9 were plotted in terms of total load, in newtons, N. To better compare the results to other studies, it is instructive to convert the results into a penetration resistance in units of stress. For all the tests, the penetration resistance ranged from 2 MPa to 3.7 MPa. In comparison, the penetration resistances for the current study ranged from approximately 10 MPa for loose sand to an average resistance of 30 MPa for dense sand in water. It appears that the measured penetration resistances in the Joshi et al. (1995) study are quite low.

In an effort to evaluate whether the magnitude of the penetration resistance was comparable to field measurements, estimates of the penetration resistance for Joshi et al. (1995) test results were made using analytical methods. The results of the penetration tests were compared to theoretical estimates using bearing capacity and cavity expansion theory. Using bearing capacity theory, the penetrometers were considered to be a deep foundation system and the ultimate resistance was estimated as follows (Kulhawy et al. 1983):

$$q_{ult} = \sigma_v' \, N_q' \qquad (6.1)$$

where

q_{ult} = ultimate bearing capacity

σ_v' = vertical effective stress

N_q' = modified bearing capacity factor

The modified bearing capacity factor, N_q', is a function of σ_v', which was 100 kPa, and the effective friction angle, ϕ'. The friction angle was assumed to be 45°, which is reasonable for a dense sand. For $\phi' = 45^o$, $N_q' = 150$ and the estimated penetration resistance for the tests performed by Joshi et al. (1995) was 15 MPa.

The penetration resistance was also estimated using cavity expansion theory, as proposed by Salgado (1993). This approach is discussed in more detail in section 6.3. Using cavity expansion theory, an estimated penetration resistance of 28 MPa was obtained. Thus, it

appears that the magnitude of the penetration resistance is lower than expected, and may not be comparable to penetration tests in the field.

There is also a direct contrast between the scanning electron micrographs of the two studies. Joshi et al. (1995) presented micrographs of sand showing a clear presence of precipitate on and in between the grains. However, the micrographs from this study showed no visible evidence of precipitation, despite other chemical evidence that suggested some precipitation of silica and carbonate material occurred. For the study by Joshi et al. (1995), the micrographs showing precipitation came from small (1.25 cm diameter, 1.9 cm high) complementary samples that were submerged, loaded, and allowed to age for two years. After aging, the samples were drained (Achari 1999) and allowed to air-dry. It is possible, however, that if the samples were not fully drained, solute would precipitate on the grains during air-drying, thus suggesting the presence of precipitation.

To investigate if this was possible, a simple test with two small samples (approximately 200 g) of dry Evanston sand was performed. One sample was mixed with a salt solution (35 g/L NaCl) to simulate seawater, and the other sample was mixed with the pore fluid from the aged tests involving carbon dioxide-saturated water. The latter was chosen because it contained approximately 150 ppm calcium in solution, as indicated by the ICP results shown in Figure 5.19. The samples were air-dried without draining and the resulting scanning electron micrographs, shown in Figure 6.10, indicate clear evidence of precipitation. In Figure 6.10 (a), the precipitate was found to be sodium chloride using energy dispersive spectrum analysis, and the white dots on the sand grains in Figure 6.10 (b) were found to be calcium.

Thus, there are some questions regarding the results of the Joshi et al. (1995) study. Although the percent increases in penetration resistance with time were comparable to increases observed in the field, the magnitude of the penetration resistances were quite low. In addition, the penetration tests performed are significantly different from cone penetra-

tion tests. The penetration tests were mini-plate load tests with loading-unloading cycles, and involved very small displacements. Lastly, the visible evidence of precipitation could have been caused by inadequate drainage of the samples.

6.3 Can Aging Effects be Accounted for Using Analytical Techniques?

Time-dependent increases in penetration resistance should be caused by some change in engineering properties. These properties could include the effective friction angle, the stiffness of the sand, and the horizontal effective stress. A way of investigating the effect that changes in engineering properties have on the penetration resistance is to use an analytical technique for predicting cone penetration resistance. For this study, cavity expansion theory, as developed by Salgado (1993) was used. A brief description of the analysis for cone penetration resistance using cavity expansion theory is included followed by a parametric study on the effect of relative density, small strain shear modulus, and horizontal effective stress on the predicted cone penetration resistance.

6.3.1 Estimating Cone Penetration Resistance Using Cavity Expansion Theory

When a cone penetration test is performed, a cylindrical cavity is created in the soil. During the penetration, the cavity expands from a radius of zero (no penetration) to a cavity with the radius equal to the radius of the cone. An analytical technique was developed by Vesic (1972) to determine the pressure required to create such a cavity in a soil medium. This cavity pressure can be related to the cone penetration resistance. Salgado (1993) improved the analysis for cohesionless soils by treating the soil as a non-linear elastic-plastic material. Details of the analysis can be found in Salgado et al. (1997) and applications of the approach can be found in Salgado and Mitchell (1994) and Ghionna et al. (1994).

In determining the cavity pressure, and thus the penetration resistance, cavity expansion theory characterizes the surrounding soil into three zones of behavior: a plastic zone, a non-linear elastic zone, and a linear elastic zone. These zones are shown in Figure 6.11.

Immediately surrounding the cavity is the plastic zone where the soil is at a state of failure. Outside the plastic zone is an area in which the material has yielded but not yet failed. This zone is modeled as a non-linear elastic material. Farther away from the cone, the strains in the soil from the cavity are so small that the soil can be treated as a linear elastic material.

Because of the non-linearity of the material, the determination of the cavity pressure can not be determined with a closed-form solution. The program CONPOINT (Salgado 1993) can be used to determine the penetration resistance iteratively. CONPOINT accounts for the effects of density, stress, dilatancy, stiffness, and boundary conditions (for calibration chamber testing) in determining the cone penetration resistance, and has been shown to predict the results of calibration chamber cone penetration tests to within approximately 20%.

The cone penetration resistance in sands is primarily a function of relative density, mean effective stress, effective friction angle, and shear stiffness. However, there is evidence that some of these variables do not change significantly during the aging process. Time-dependent increases in penetration resistance with time in the field following ground modification occur with no observed settlements of the ground surface. Thus, it appears that no significant changes in the average density of a sand deposit occur that can account for the measured increases in penetration resistance. In addition, results of triaxial tests performed by Daramola (1980) and Human (1992) suggest that, despite changes in shear stiffness with time, the effective friction angle does not change with time. Therefore, a parametric study was performed using CONPOINT to determine if realistic changes in stress conditions and stiffness could account for the observed time-dependent increases in cone penetration resistance.

For this analysis, the properties of Ticino sand were used. This is a quartz sand used in calibration chamber tests in Italy (Salgado et al. 1997), the properties of which are known

and listed in Table 6.3. The small strain shear modulus, G_o, was determined from an empirical equation originally proposed by Hardin and Richart (1963) based on laboratory test data. G_o is a function of the mean effective stress and the void ratio, and is shown in Table 6.3.

Table 6.3 Properties of Ticino Sand.

$\gamma_{d\ min}$	13.6 kN/m^3
$\gamma_{d\ max}$	16.7 kN/m^3
G_s	2.68
e_{min}	0.574
e_{max}	0.931
D_{50}	0.58 mm
D_{10}	0.36 mm
ϕ_c	34.8^o
$G_o = 647 \frac{(2.27-e)^2}{(1+e)} \sigma'^{0.43}_m$	

$\gamma_{d\ min}$ = minimum dry density
$\gamma_{d\ min}$ = maximum dry density
G_s = specific gravity
e_{min} = minimum void ratio
e_{max} = maximum void ratio
ϕ_c = critical state friction angle

The approach to the parametric study was as follows:

- Ticino sand properties were used.
- For a given relative density (30% or 80%), the corresponding void ratio and unit weight were determined.
- Based on the unit weight and assuming a homogenous sand deposit with the water table at the ground surface, the vertical effective stresses were calculated at depths of 5, 10, and 20 m.
- For a given value of K_o, the small strain shear modulus and the horizontal effective stress were estimated at each depth. The small strain shear modulus, G_o, was calculated from the empirical equation in Table 6.3. For this equation, the void ratio was determined from the

relative density and the known maximum and minimum void ratios listed in Table 6.3. The mean effective stress was calculated from the buoyant unit weight, the depth, and the given value of K_o.

- Using CONPOINT, the penetration resistance, q_c, was estimated for the given density, small strain shear modulus, and stress conditions.
- G_o or K_o was then increased, and then a new penetration resistance was calculated.

A 20% increase in G_o is realistic based on experimental data for sands. For the sands tested in this study, 30 to 40 days of aging resulted in at most a 23% increase. A 33% increase in K_o under constant vertical stress is larger than would be normally expected for sands. However, for the parametric study, these levels were chosen to investigate if relatively large changes in G_o and K_o translate to large changes in q_c in order to bound the problem.

The results of the parametric study are shown in Table 6.4 for a relative density of 30%. The results corresponding to a depth of 5 m will be discussed in detail to aid in understanding the results of Table 6.4. The first row contains the baseline values to which the other rows are compared. Row 2 assumes a 20% increase in G_o (from 43,104 to 51,699 kPa), which results in a 6.0% increase in q_c. Row 3 assumes a 33% increase in K_o (from 0.45 to 0.60). Note that G_o is dependent on the mean effective stress, and as K_o increases, this causes K_o to increase as well by 6% (from 43,104 to 45,896 kPa). These changes in properties result in a 16.2% increase in q_c over the baseline value. Finally, row 4 shows that a 33% increase in K_o (0.45 to 0.60) and a 28% time-dependent increase in G_o (from 43,104 to 55,046) results in a 23.3% increase in the penetration resistance over the baseline value. For depths of 10 m and 20 m, the largest increases were 23.8% and 24.8%, respectively, for increases in both K_o and G_o.

The results of the parametric study for a relative density of 80% are shown in Table 6.5. The increases in penetration resistance are less than the increases for the loose sand, with the largest increase being a 14.5% increase q_c at a depth of 20 m.

Table 6.4 Results of the parametric study using CONPOINT for Dr_o = 30%.

	Depth (m)	K_o	G_o (kPa)	q_c (kPa)	Percent Increase
1	5	0.45	43,104	4,402	Baseline
2	5	0.45	51,699	4,667	+ 6.0%
3	5	0.60	45,896	5,115	+ 16.2%
4	5	0.60	55,046	5,427	+ 23.3%
5	10	0.45	58,072	6,650	Baseline
6	10	0.45	69,650	7,043	+ 5.9%
7	10	0.60	61,859	7,786	+ 17.1%
8	10	0.60	74,193	8,230	+ 23.8%
9	20	0.45	78,276	10,178	Baseline
10	20	0.45	93,883	10,751	+ 5.6%
11	20	0.60	83,411	11,999	+ 17.9%
12	20	0.60	100,042	12,701	+ 24.8%

Table 6.5 Results of the parametric study using CONPOINT for Dr_o = 80%.

	Depth (m)	K_o	G_o (kPa)	q_c (kPa)	Percent Increase
1	5	0.45	62,536	22,451	Baseline
2	5	0.45	75,004	23,678	+ 5.5%
3	5	0.60	66,638	24,195	+ 7.8%
4	5	0.60	79,925	25,502	+ 13.6%
5	10	0.45	84,308	28,095	Baseline
6	10	0.45	101,118	29,626	+ 5.4%
7	10	0.60	89,795	30,418	+ 8.3%
8	10	0.60	107,698	32,014	+ 14.0%
9	20	0.45	113,635	35,546	Baseline
10	20	0.45	136,291	37,399	+ 5.2%
11	20	0.60	120,975	38,633	+ 8.7%
12	20	0.60	145,095	40,687	+ 14.5%

From Tables 6.4 and 6.5, the results of the parametric study using cavity expansion theory can be summarized as follows:

1. Increasing G_o by 20%:
    - Increases q_c by 5.5 - 6% for D_r = 30%
    - Increases q_c by 5 - 5.5% for D_r = 80%
2. Increasing K_o by 33%:
    - Increases G_o by 6%
    - Increases q_c by 16 – 18% for D_r = 30%
    - Increases q_c by 8 – 9% for D_r = 80%
3. Increasing K_o by 33% and G_o by 28%:
    - Increases q_c by 23 – 25% for D_r = 30%
    - Increases q_c by 13.5 – 14.5% for D_r = 80%

From the above results, it is clear that, using cavity expansion theory, realistic changes in soil properties, such as horizontal effective stress and small strain shear modulus, can not account for the time-dependent increases in penetration resistance in the field, which can exceed 100%. This strongly suggests that modeling the soil as a homogeneous deposit with homogeneous soil properties does not capture the aging phenomena in sands.

6.4 Revised Hypothesis

It may be that laboratory studies and analytical techniques can not capture the aging phenomena because both are idealized or simplified views of conditions found in nature. In the laboratory, care is taken to form reproducible uniform samples, and in the analytical approaches, soil is treated as a homogeneous continuum. Perhaps the inherent variability of natural soil deposits contributes to the aging phenomena. Cone penetration tests in sands often show considerable variation, or heterogeneity, in penetration resistance with depth. The cone penetration resistance is primarily a function of relative density and horizontal effective stress, and some of the observed heterogeneity observed in cone

penetration test results may be caused by localized variations in stress as well as variations in density.

It is known that stresses are not uniformly distributed throughout granular soils. The concepts of load chains and internal stress arching, discussed in section 3.2.4, suggest that there are stiffer zones within soils which carry greater stresses than adjacent weaker zones. Schmertmann (1991) postulated that such an arching effect may be responsible for some of the measured increases in penetration resistance with time. By imparting energy through impact, vibration, or blasting, loose sands are densified and the equilibrium stress distributions are disturbed. Perhaps aging effects in sands are due to the gradual redistribution of stresses, creating new stronger load chains, which result in an increase in the measured penetration resistance. This redistribution of stresses may also cause changes in the heterogeneity of the penetration resistance with time

Figure 6.12 shows an example of this possible effect from the Jebba Dam project. This shows a series of penetration test results at one location before and at four times after blast densification. The initial penetration resistance shows significant heterogeneity with depth. In fact, blast densification was performed in this area because of the low penetration resistances below a depth of 30 m. Four days after the first coverage of blasting, both the magnitude of q_c and the degree of heterogeneity were less than before blasting. Twenty one days after the second pass of blasting, however, there had been a measurable increase in the penetration resistance and its heterogeneity with depth.

A qualitative assessment of time-dependent changes in the heterogeneity of cone penetration test results was made for a number of examples from the literature. The results are divided in four tables, which separate the examples into categories of blast densification, vibrocompaction, dynamic compaction, and hydraulically placed fill. In all of the examples, the penetration resistance increased with time. The plots of penetration resistance vs. depth for all of the examples are assembled in Appendix C.

Table 6.6 shows the time-dependent changes in heterogeneity associated with the examples involving blast densification. Eight examples are listed. Based on the results of the cone penetration tests, the initial heterogeneity of the deposit before blasting was assessed as being slight, moderate, or extreme in column 2. The penetration resistances shown in Figure 6.12 can be used to illustrate these gradations of increasing variability. The penetration test performed 4 days after the first pass of blasting is an example of a penetration resistance with slight heterogeneity. The penetration test performed 124 days after blasting was classified as having moderate heterogeneity, and the penetration test performed before blasting was classified as having extreme heterogeneity.

The next two columns (3 and 4) in Table 6.6 indicate whether, immediately after blasting, the heterogeneity and penetration resistance, q_c, increased, decreased, or stayed the same. Columns 5 and 6 indicate the change in heterogeneity and q_c with time compared to the values assessed immediately after blasting. In six of the eight cases, there was an initial decrease in heterogeneity of the deposit after blasting. In four of these cases, there was also a decrease in q_c. With time, six of the eight cases showed an increase in heterogeneity. In all eight examples, there was an increase in q_c with time.

Tables 6.7, 6.8, and 6.9 detail the changes in heterogeneity in the cone penetration resistance associated with examples of dynamic compaction, vibrocompaction, and hydraulically placed fill, respectively. A total of six examples are cited, and an increase in the heterogeneity of the deposit with time occurred in four of the six cases. As stated previously, there was an increase in q_c with time in all six cases.

From this qualitative assessment, there appears to be a trend which suggests that, following disturbance or ground modification, there is a time-dependent increase in the heterogeneity of the deposit, as indicated by the penetration resistance. It is possible that this change does not reflect changes in density; rather it could indicate that stresses within

the soil are being redistributed to a more stable arrangement. This redistribution of stresses may translate into an increase in the overall cone penetration resistance.

Table 6.6 Changes in heterogeneity as indicated by cone penetration resistance associated with blast densification.

Reference (1)	Initial Hetero-geneity (2)	Immediately After Blasting (Change from Initial)		Final (Change from Immediately After)		Notes (7)
		Hetero-geneity (3)	q_c (4)	Hetero-geneity (5)	q_c (6)	
AGRA (1995)	Extreme	Same	Same	Same	Increased	200-300% increase in q_c after 12 days; no further increase at 30 days.
Charlie et al. (1992)	Moderate	Decreased	Decreased	Increased	Increased	18% increase in q_c after 18 weeks; q_c did not reach pre-blast values.
Gohl et al. (1994)	Moderate	Increased	Increased	Increased	Increased	Increase in q_c up to 440 days; increase not consistent throughout site.
Jefferies and Rogers (1993)	Moderate	Decreased	Increased	Increased	Increased	>200% increase in q_c in 16 days.
La Fosse and von Rosenvinge (1992)	Extreme	Decreased	Increased	Same	Increased	Only slight changes in heterogeneity.
Mitchell and Solymar (1984)	Extreme	Decreased	Decreased	Increased	Increased	Blast Zone 2, CPT 101. Large decrease in heterogeneity im-mediately after; mod-erate increase w/ time.
Mitchell and Solymar (1984)	Extreme	Decreased	Decreased	Increased	Increased	Test blast 17. Very large decrease in het-erogeneity 8 days after blasting.
Solymar (1984)	Extreme	Decreased	Decreased	Increased	Increased	Test blast 3.

Table 6.7 Changes in heterogeneity as indicated by cone penetration resistance associated with dynamic compaction.

Reference (1)	Initial Hetero-geneity (2)	Immediately After Blasting (Change from Initial)		Final (Change from Immediately After)		Notes (7)
		Hetero-geneity (3)	q_c (4)	Hetero-geneity (5)	q_c (6)	
Dumas and Beaton (1988)	Slight	Increased	Increased	Increased	Increased	20% increase in q_c from 1 day to 8 days
Mitchell and Solymar (1984)	Slight	Same	Increased	Increased	Increased	30% increase in q_c from 17 days to 31 days.

Table 6.8 Changes in heterogeneity as indicated by cone penetration resistance associated with vibrocompaction.

Reference (1)	Initial Hetero-geneity (2)	Immediately After Blasting (Change from Initial)		Final (Change from Immediately After)		Notes (7)
		Hetero-geneity (3)	q_c (4)	Hetero-geneity (5)	q_c (6)	
Mitchell and Solymar (1984)	Extreme	Decreased	Increased	Same	Increased	
Ng et al. (1996)	Slight	Same	Increased	Increased	Increased	

Table 6.9 Changes in heterogeneity as indicated by cone penetration resistance associated with hydraulically placed fill.

Reference	Initial Hetero-geneity	Final (Change from Initial)		Notes
		Hetero-geneity	q_c	
Jefferies et al. (1988)	Extreme	Same	Same	No change in q_c in 11 months.
Mitchell and Solymar (1984)	Slight	Increased	Increased	

Clearly more research is needed to determine conclusively what causes time-dependent increases in the penetration resistance in sands in the field. This study has shown that performing laboratory tests may not be a good way of studying aging effects that have been observed in the field. Therefore, future research in this area should focus more on field studies. Studies specifically designed to investigate aging effects are needed. For most of the case studies cited in this report, aging effects were not the primary focus of study. As such, there is often incomplete data regarding sand mineralogy, groundwater conditions, location of tests, and spatial variation of soil properties. Any study of aging effects in the field should assess the spatial variation and changes in the heterogeneity of the deposit. In this way, true time-dependent increases can be separated from the natural variability of the site.

6.5 Summary and Conclusions

The purpose of this chapter was to bring together the findings of the current experimental study and what is presented in the literature into a consistent framework. From the results of the current study, it appears that a controlled laboratory testing program may miss the true mechanism(s) responsible for time-dependent increases in penetration resistance in the field.

This contradicts the findings of two laboratory studies from the literature. A comparison was made between the two studies and the current research to reconcile the differences in the results. A summary of this comparison is as follows:

- Dowding and Hryciw (1986) observed time-dependent increases in penetration resistance in a liquefaction tank following blast densification. The stress level at which the tests were performed were very low, and thus the increases in q_c with time were also correspondingly low. However, the percent increases in q_c were comparable to field observations (75 to 90% increase in q_c in 15 days). Differences between the Dowding and Hryciw (1986) study and

the current research include sample preparation, boundary conditions, the different amount of energy imparted to the samples, and the generation of gases by blasting.

- Joshi et al. (1995) observed increases in penetration resistance with time in rigid wall cells. There were a number of similarities between this study and the current research, including sample preparation, stress level, and size of samples. However, the penetration tests performed by Joshi et al. (1995) were significantly different than the penetration tests performed in the current research and may not be representative of cone penetration tests. The tests involved a series of load-unload cycles of mini-plate load tests and the displacements for each were very small. The magnitude of the penetration resistances were also significantly lower than would be expected of cone penetration tests in the field. Lastly, scanning electron micrographs from the current research suggest that the precipitation observed in the specimens from the Joshi et al. (1995) study may have been caused by incomplete drainage of the specimens.

A parametric study on the effect of relative density, small strain shear modulus, and stress conditions on the cone penetration resistance was performed using cavity expansion theory. The results suggest that large increases in small strain shear modulus and horizontal effective stress cause the cone penetration resistance to increase up to approximately 25%. This can not account for increases in the q_c of 100% or more that are observed in many field cases of aging. Modeling the soil as a homogeneous deposit and/or modeling the penetration test with cavity expansion theory does not capture the aging effect in sands.

From the results of time-dependent increases in cone penetration resistance in the field, a qualitative assessment of changes in heterogeneity with time was made. There is a gen-

eral trend of an increase in heterogeneity with time that accompanies increases in q_c. This trend suggests that time-dependent redistribution of stresses, or internal stress arching, may be the mechanism responsible for aging effects in sands. Presently, however, there is no direct evidence in support of this hypothesis and more detailed field studies are needed.

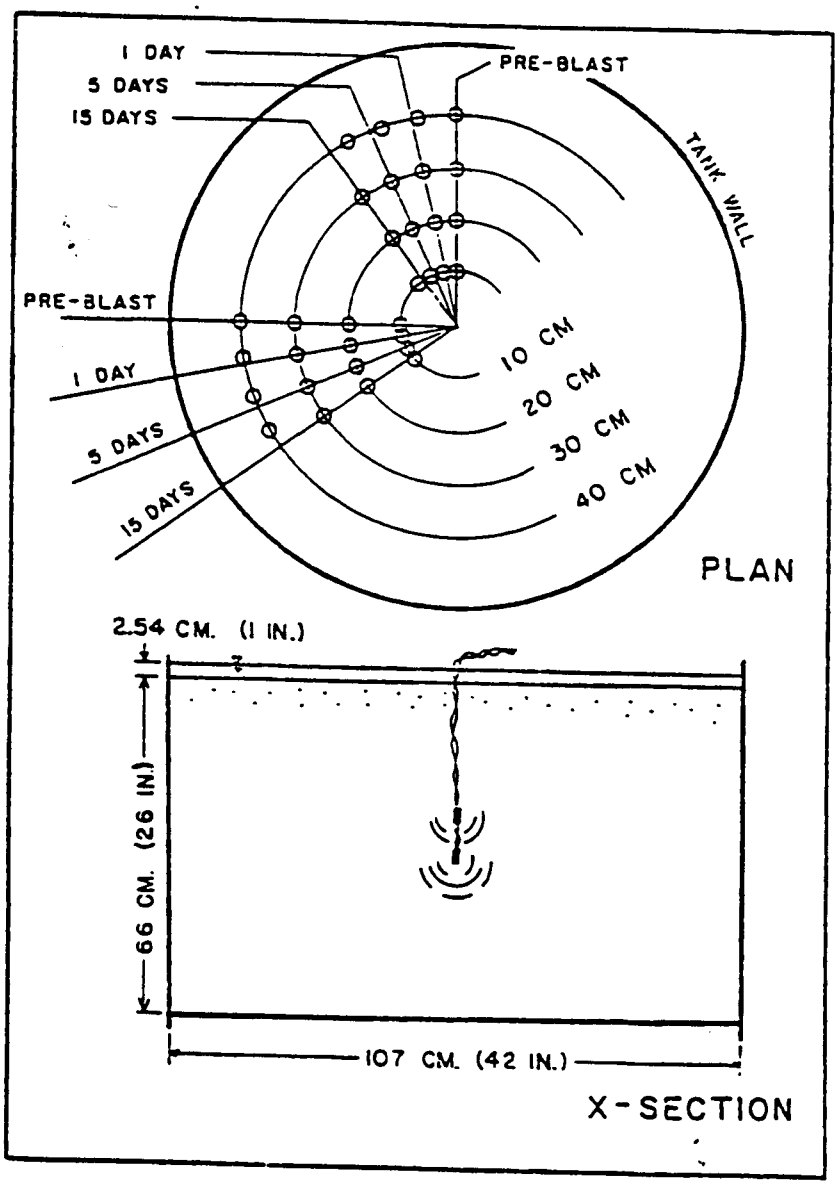


Figure 6.1 Diagram of liquefaction tank used by Dowding and Hyrciw (1986).

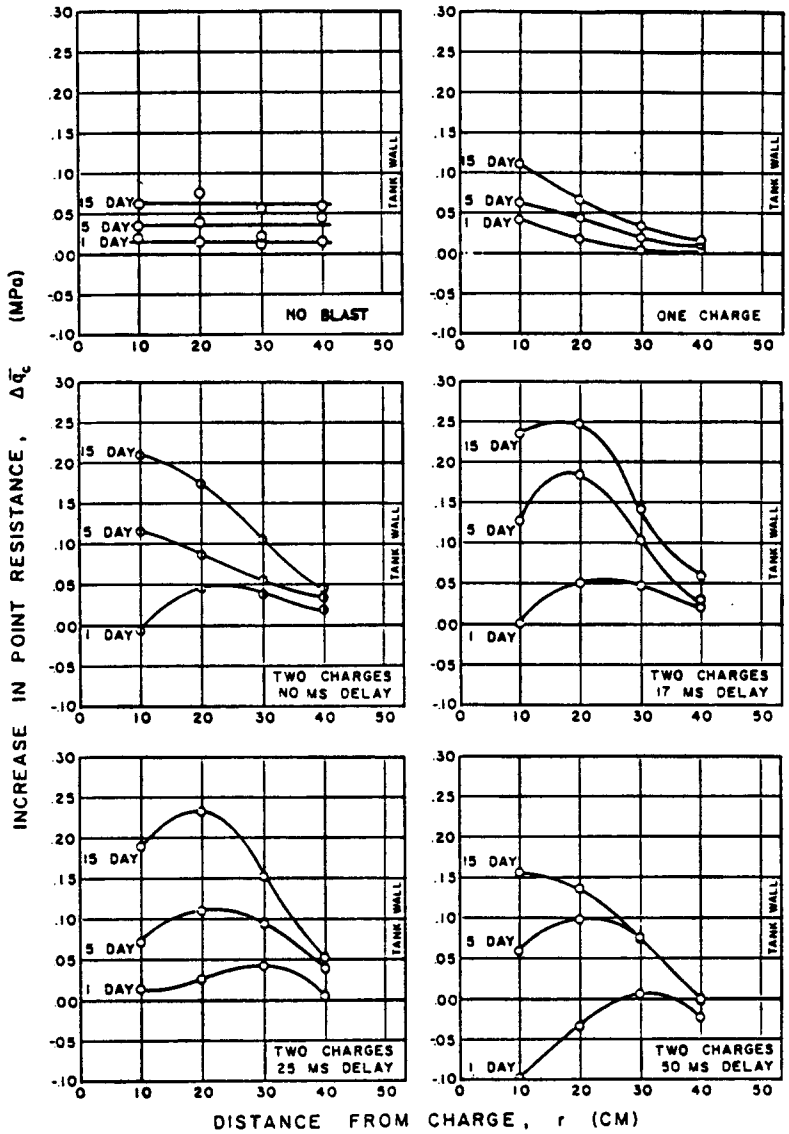


Figure 6.2 Increase in penetration resistance with time in a laboratory blasting experiment (Dowding and Hryciw 1986).

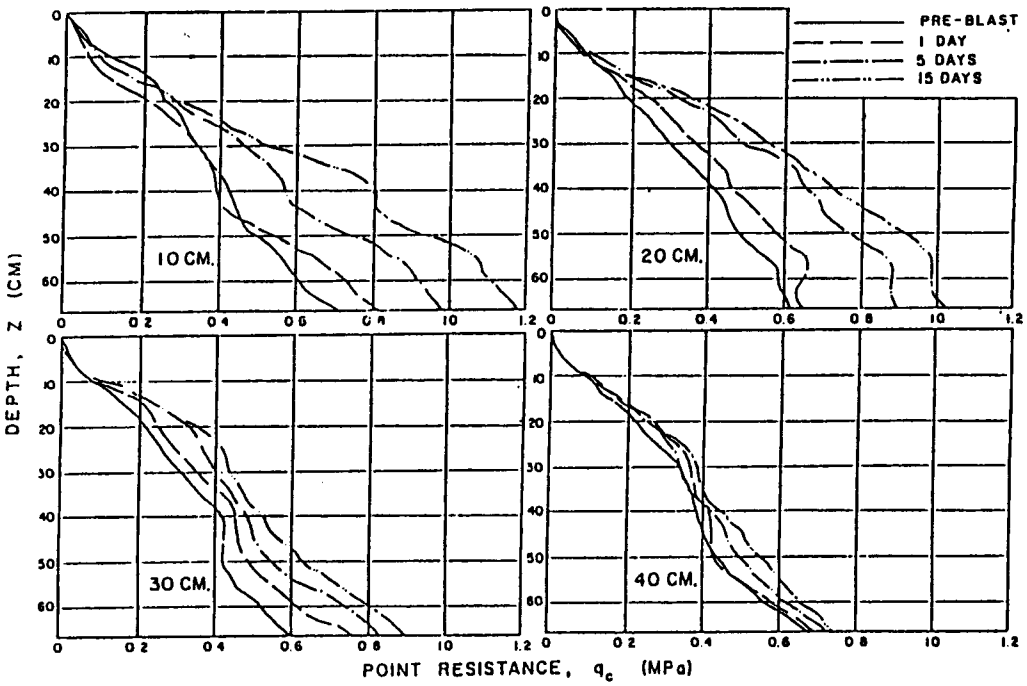


Figure 6.3 Penetration test results after detonation of 2 charges (Dowding and Hryciw 1986).

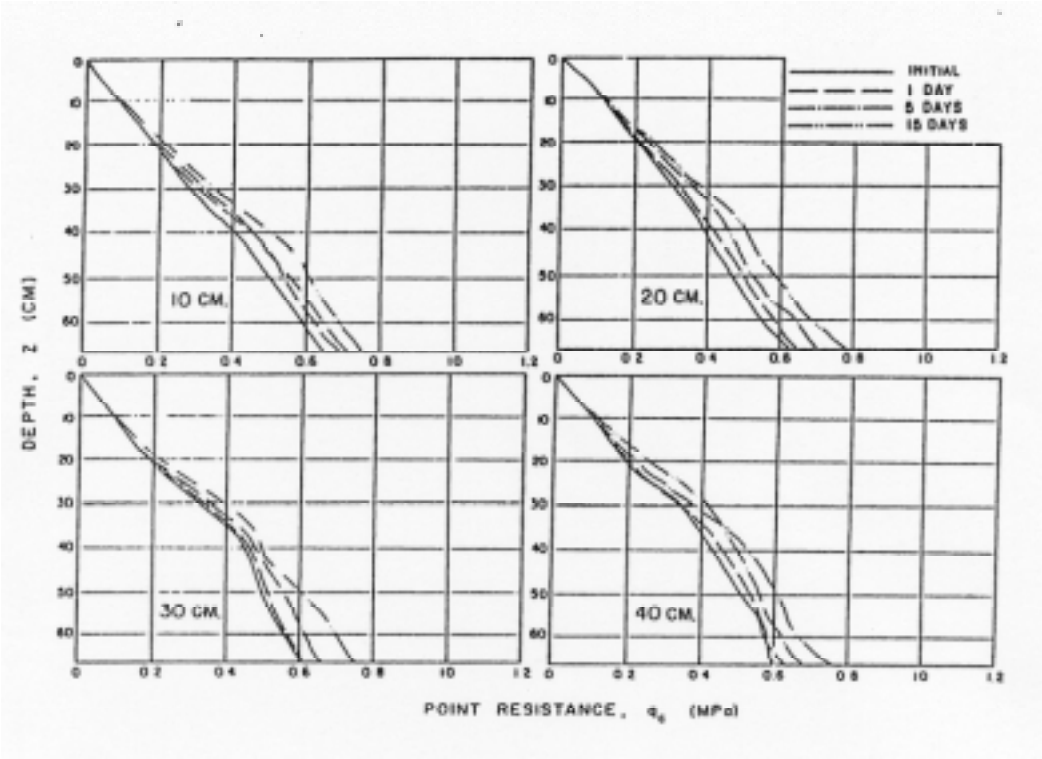


Figure 6.4 Penetration test results for freshly deposited sand (Dowding and Hryciw 1986).

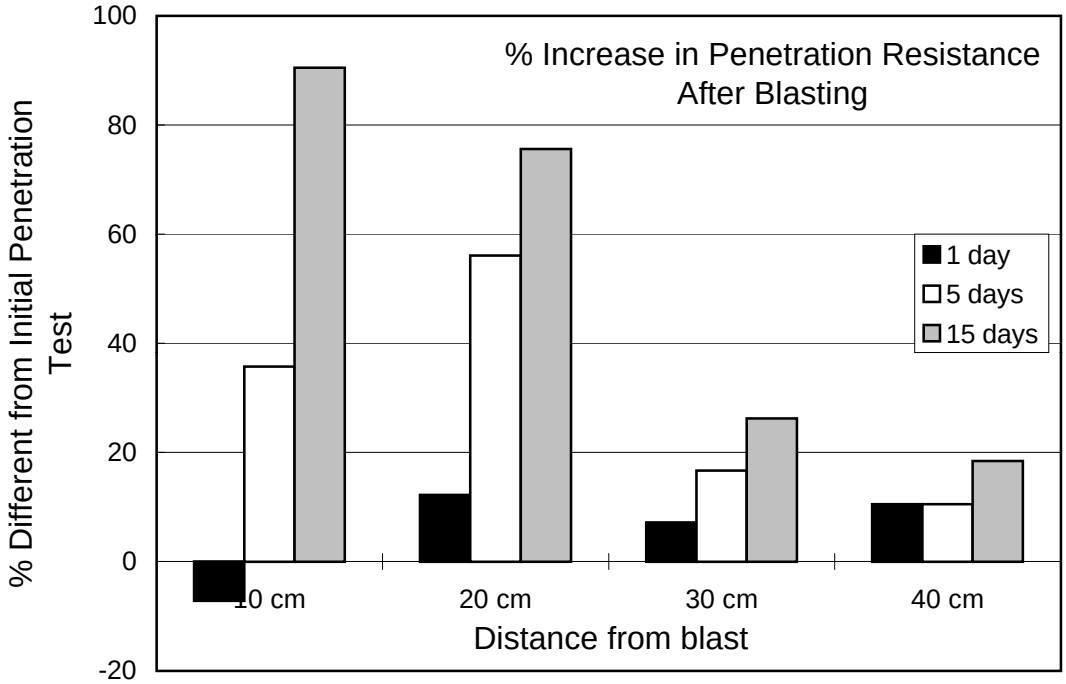


Figure 6.5 Percent increase in q_c after blasting (Dowding and Hryciw 1986).

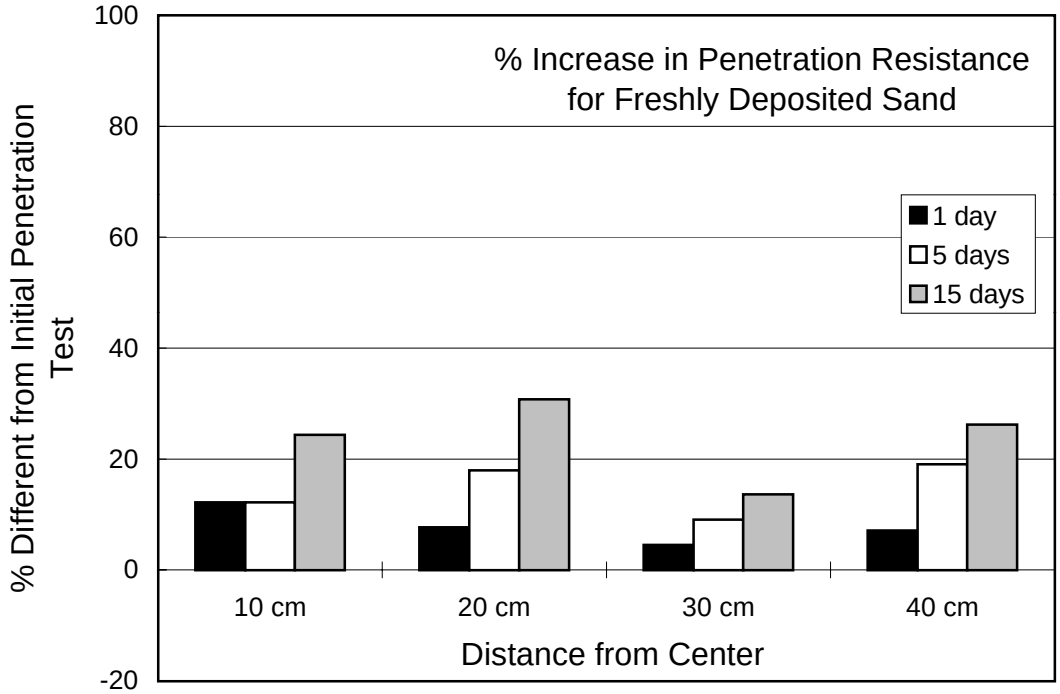


Figure 6.6 Percent increase in q_c in freshly deposited fill (Dowding and Hryciw 1986).

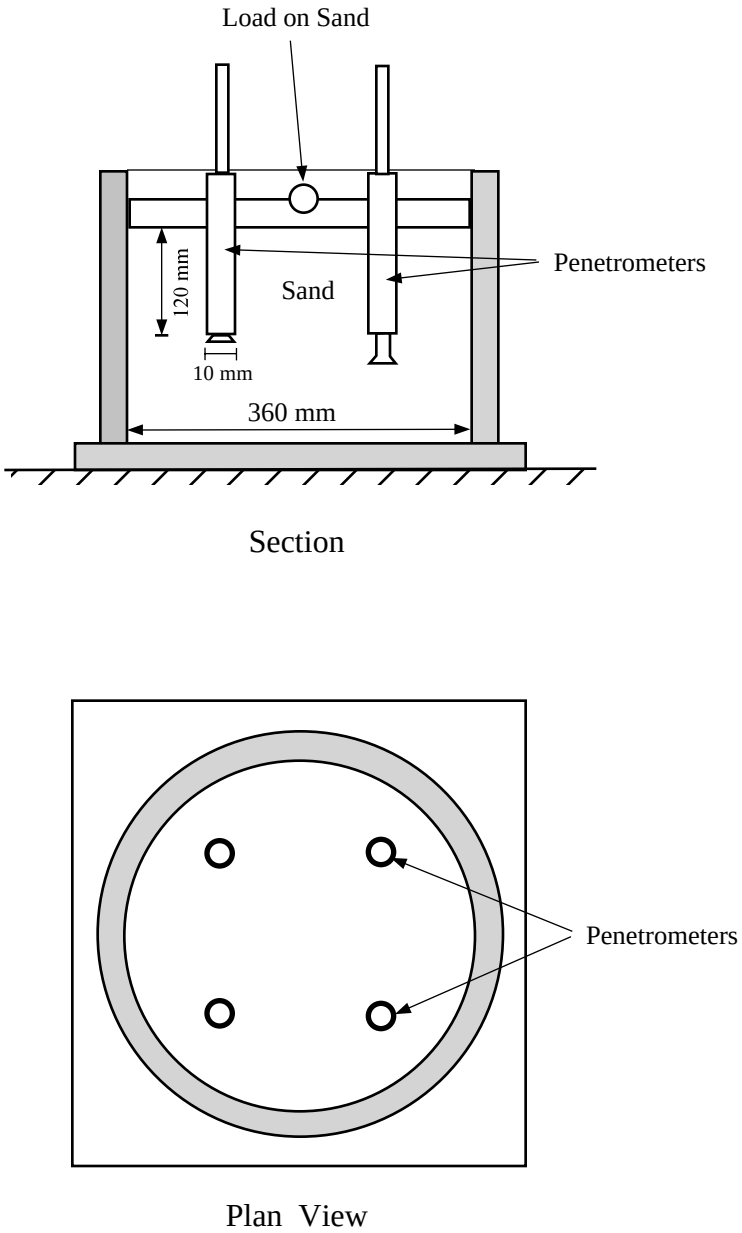


Figure 6.7 Section and plan view of equipment used by Joshi et al. (1995).

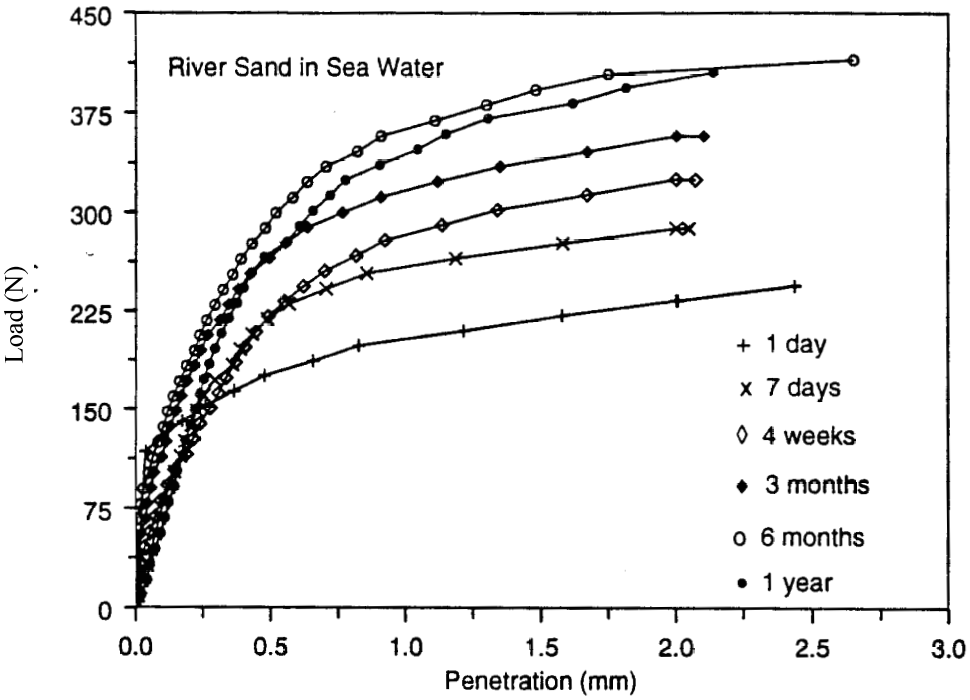


Figure 6.8 Penetration test data for Joshi et al. (1995).

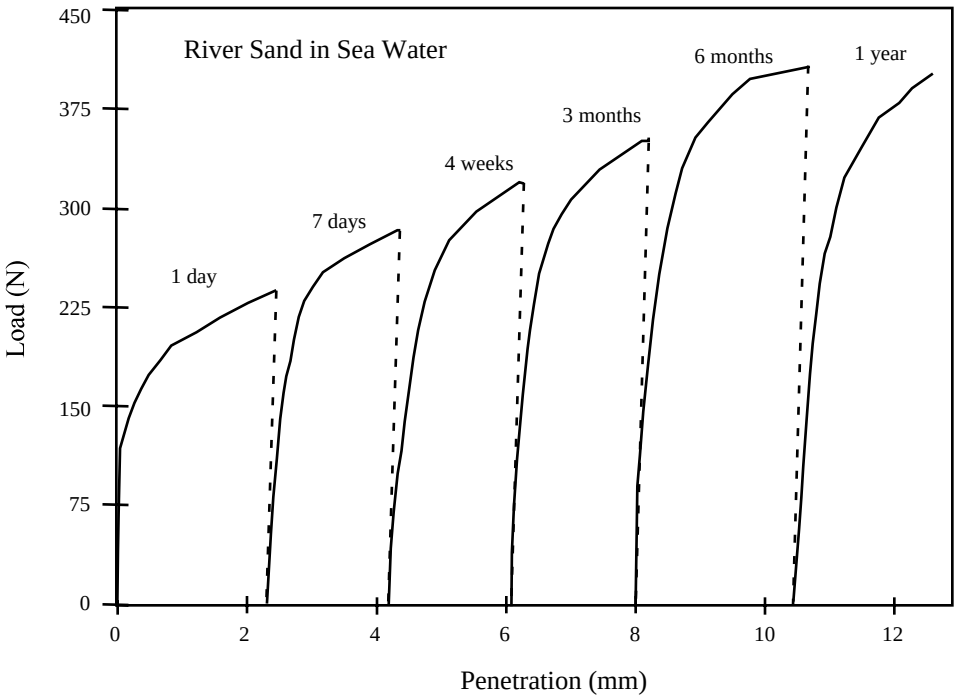
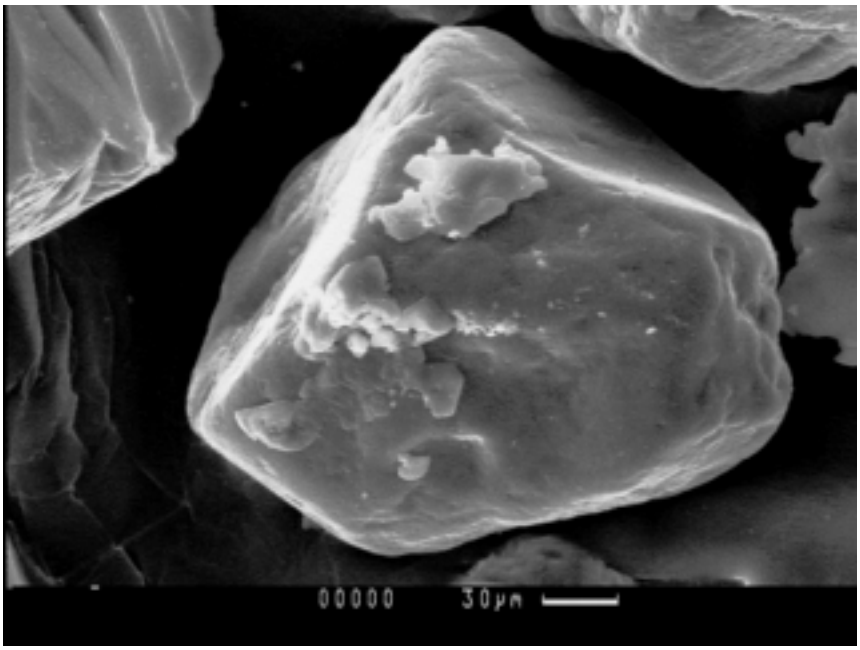
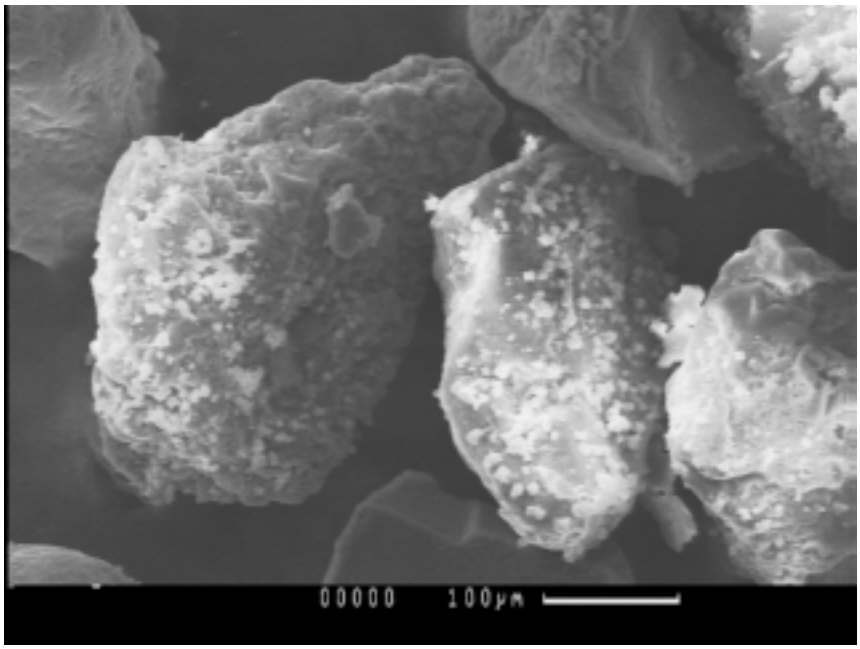


Figure 6.9 Penetration test data from Joshi et al. (1995) showing cumulative displacements and loading-unloading cycles.



(a)



(b)

Figure 6.10 Evanston sand saturated with (a) seawater, and (b) calcium rich water, and then air-dried without drainage.

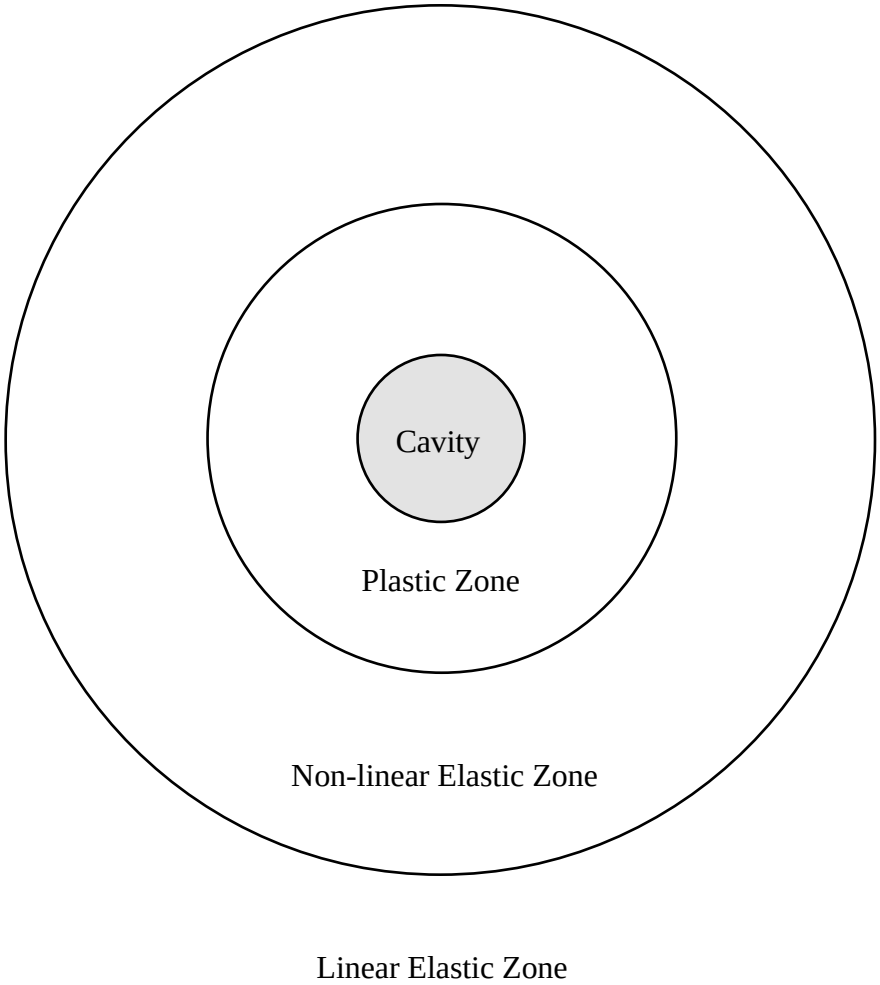


Figure 6.11 Zones of different stress-strain behavior used in cavity expansion theory to model cone penetration resistance (after Salgado 1993).

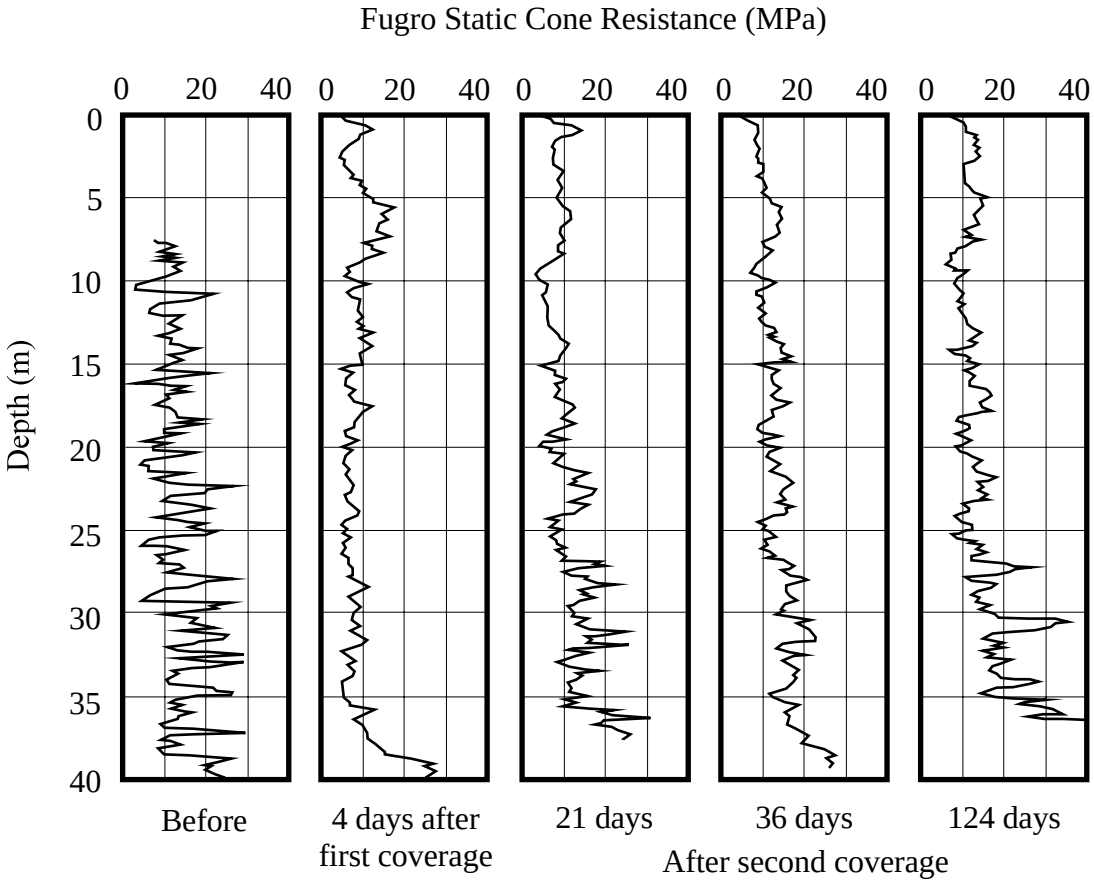


Figure 6.12 Example of increasing heterogeneity with time following blast densification, Blast zone 2, CPT Area 101 (after Mitchell and Solymar 1984).