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Lateral In-Situ Stress Measurements to Diagnose Liquefaction

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1. Introduction

Death by earthquake usually involves toppling and crumbling of masonry walls, falling roofs, and pancaking floors in multistory buildings. Residents of underdeveloped countries are particularly vulnerable because structures only rarely are designed to resist earthquakes. The safety of masonry construction could be improved with buttressing or cellular design, but those procedures are little known and seldom used, suggesting that effective charity should include education and innovation.

Even earthquake-resistant structures remain vulnerable to a hidden danger, that posed by liquefaction. Most susceptible are saturated loose sands and silts where shaking movements can cause individual soil grains to become dislodged and temporarily suspended in water, turning the soil into a dense liquid analogous to quicksand. The process occurs at depth due to increased horizontal shearing stresses, so gentle slopes can turn into landslides, buried tanks can float up out of the ground, and buildings can sink or tip over. Fortunately the destruction is relatively slow compared with walls crumbling and roofs falling, and can allow enough time for people to escape with their lives. A famous example in Niigata, Japan, where as shown in Fig. 1 multistory apartment buildings remained intact and in one



Fig. 1. Apartment buildings at Kawagishi-Cho, Niigata, Japan, tipped after the foundation soil liquefied during the 1964 earthquake.

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instance tipped far enough that residents could crawl out of windows and walk to safety down the exterior wall.

Perhaps the most serious situation is where a potentially liquefiable soil supports a dam, where a failure would be sudden and could take many human lives.

Liquefaction is soil specific, so building sites in earthquake areas can be tested to determine their susceptibility. However, the tests are largely empirical and based on case histories. This chapter examines the role of a more fundamental soil property, lateral in-situ stress, on soil collapse and liquefaction, and emphasizes the use of a relatively new tool to measure that stress.

2. The nature of liquefaction

Since both lateral and vertical stresses in soils tend to increase with depth, they are conveniently expressed in terms of a ratio of effective stresses, K_0 . During liquefaction, soil grains are temporarily suspended in water: Effective or intergranular stresses are zero, and there is no K_0 . Then the grains settle and displace water so it seeps upward, often taking sand particles with it. The reconstituted soil density therefore is higher than it was prior to liquefaction, so liquefaction can be a one-time event, mainly limited to geologically youthful sandy deposits such as deltas, alluvial fans and floodplains, and to loose, man-made sandy fill soil that is dumped without being compacted.

Liquefaction probably occurs with all major earthquakes, but in severely damaged areas the evidence lays buried underneath the devastation. Prehistoric earthquakes can be dated from sedimentary sequences that contain sand carried upward by the displaced water and deposited as shallow cones at an earlier ground surface.

Stress ratios existing in soils prior to and after liquefaction have not yet been fully explored, the major restraint being the difficulty of obtaining reliable in-situ stress data.

3. Current methods for evaluating the liquefaction potential of a soil

The pioneering work of Seed and Idriss (1971) developed criteria for identifying potentially liquefiable soils from Standard Penetration Test (SPT) data. The method has since been improved through the use of empirical corrections to the data, and is supplemented by cone penetration tests (CPT) and measurements of shear wave velocity (Youd et al. 2001).

Schmertmann (1971) suggested that the SPT is essentially a side friction test, implying a dependence of blow counts on horizontal stress existing and developed in the soil. Kelly and Lutenecker (2004) show close relationships between blow counts and side friction measured by torquing the SPT sampler, with the exact relationship depending on the type of soil. The lateral stress ratio K_0 appears in several formulas for cone resistance based on cavity expansion theory (Yu and Mitchell 1998). Thus even though the lateral in-situ stress changes in the neighborhood of a test instrument, it still may play an important role in a liquefaction process.

4. Measuring K_0

Vertical effective stress in soil can be obtained from depth, soil density, and elevation of the groundwater table, but lateral stress must be measured. Empirical estimates of the lateral stress ratio K_0 based on depth, density, friction angle, and overconsolidation ratio, do not address effects or likelihood of liquefaction.

One approach for measuring lateral in-situ stress in soil is to bore a hole and insert a measuring device without disturbing the soil. This is the concept of the self-boring pressuremeters. However, tests are time-consuming, and some plucking or shoving aside of individual soil particles during drilling is unavoidable.

Other devices such as the Cone Penetration Test (CPT) and Dilatometer laterally displace the soil to the extent of creating a lateral bearing capacity failure. Empirical relationships can be developed based on pressure chamber testing, but the changes in lateral stress caused by disturbance involve an unknown combination of soil responses including elastic compression, consolidation and shearing. These occur simultaneously and create changes in lateral stress that are larger and more variable than the stresses that are intended to be measured. A third approach is to leave a flat pressure cell in place long enough for stresses to equilibrate, but that ignores the permanence of soil arching action.

A different approach is used with the K_0 Stepped Blade.

4.1 The K_0 stepped blade

The methodology used with the K_0 Stepped Blade was adapted from analytical chemistry, to intentionally introduce different levels of disturbance and extrapolate the results to a condition of zero disturbance. This is accomplished with four blade steps ranging in thickness from 3.0 to 7.5 mm, Fig. 2.

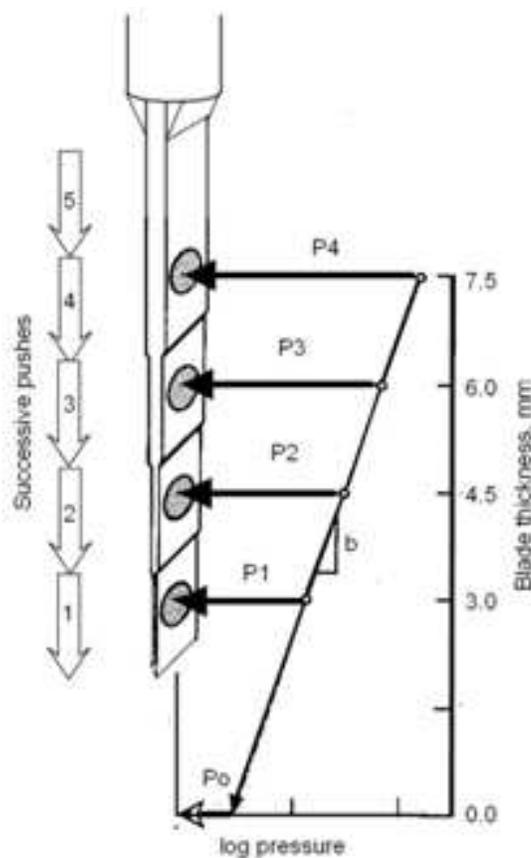


Fig. 2. Schematic showing the method used to interpret Stepped Blade data. A linear e - $\log p$ relationship indicates consolidating behavior. Reprinted with permission from Handy and Spangler (2007). The McGraw-Hill Companies, New York.

The blade is advanced incrementally into the bottom of a boring so each measurement is made on a progressively increasing step thickness at the same depth. While the test is rapid, enabling as many as 10 determinations in an hour, the blade cannot be pushed continuously, but must be withdrawn and the boring extended deeper for each new test series. A blank space at the top of the Blade allows four multi-stress measurements to be made at 127 mm (5 in.) depth increments in each series. Measurements are directional owing to the flat shape of the Blade.

4.2 Interpretation of K_0 stepped blade data

Extensive measurements in the field and in laboratory simulations show a consistent relationship between step thickness and the logarithm of pressure, illustrated by the graph in Fig. 2 (Handy et al. 1982; White and Lutenegger 2008). Analogy to the familiar e - $\log p$ linear relationship suggests that soil penetrated by the Blade must be laterally consolidating. Data that do not meet this consolidation criterion are not included in the interpretations.

Stepped Blade steps are thin in order to minimize the tendency to create a lateral bearing capacity failure. The two most common causes for data rejection are (1) elastic response that increases the pressure on the thinnest step prior to a breakdown of the soil structure, and (2) a lateral bearing capacity failure that relieves stress on the thickest step(s). The three behaviors, elastic response, consolidation, and lateral bearing capacity failure, are illustrated in Fig. 3. Elastic and bearing capacity responses must not be used in the extrapolations, and are readily recognized from pressure decreases instead of increases on successive blade steps.

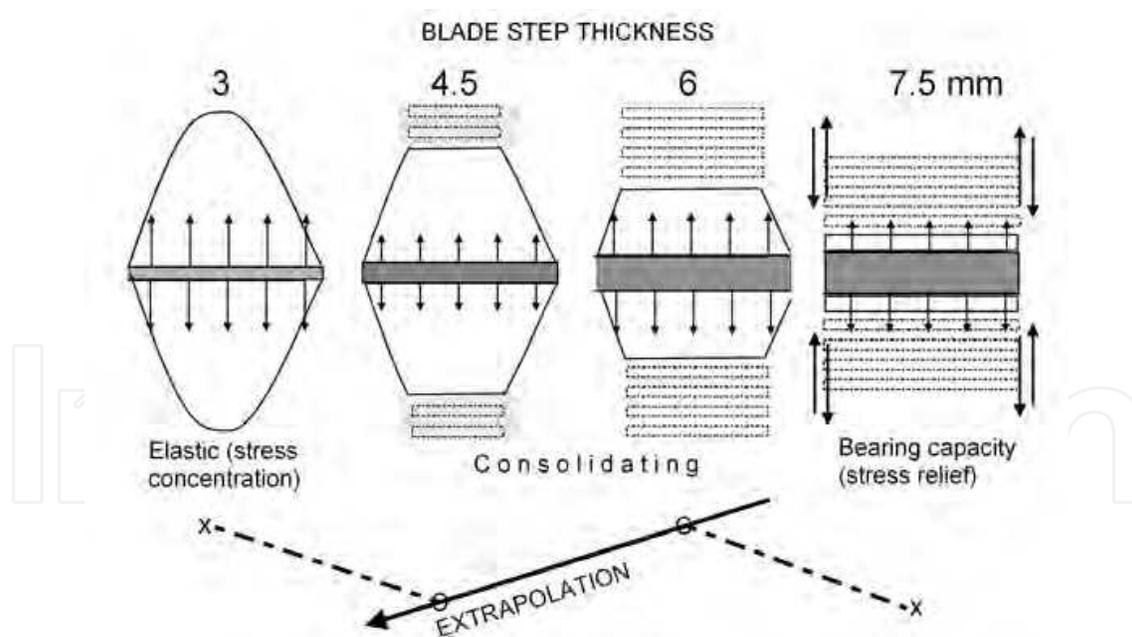


Fig. 3. Stress diagrams inferred from pressure measurements on different thickness steps, showing the critical role of consolidation and reasons for rejecting specific data points.

An elastic concentration of stress most commonly occurs with the thin step in stiff soils, and a relief of pressure from a bearing capacity failure most commonly occurs with the thick step in soft soils. Usable data sequences therefore may have two, three or four data points. A discriminating evaluation of each data point is required before using a regression analysis to

extrapolate the lateral in-situ stress, or the analysis will yield incorrect and erratic interpretations including a low value for R^2 .

A linear regression analysis of the logarithm of measured pressures versus blade step thickness yields a slope and intercept. The intercept is the lateral stress that existed prior to insertion of the blade, and the intercept can be examined for consistency. A test performed close to a hole or hard inclusion in the soil will change the slope and can be another reason to question some data. However, since such a condition cannot be confirmed, questionable data points are shown on graphs but are labeled with question marks.

The distribution of soil pressure across the face of the Blade was investigated by pulling narrow friction strips (Handy et al. 1982) and emphasized the importance of focusing measurements in the central area of each step.

4.3 The K_0 stepped blade in relation to other in-situ tests

In Fig. 4, soil response mechanisms interpreted from Stepped Blade data are applied to other in-situ displacement soil tests. For example, it appears likely that the width/thickness ratio of the Dilatometer causes a lateral bearing capacity failure, so empirical correlations are used to estimate both stress and modulus from expansion of a metal diaphragm. A more direct measure of modulus might be obtained from elastic response of soil to penetration by the thinnest step of the Blade, a possibility that has been examined but not yet fully investigated.

Elastic.....consolidating.....bearing failure

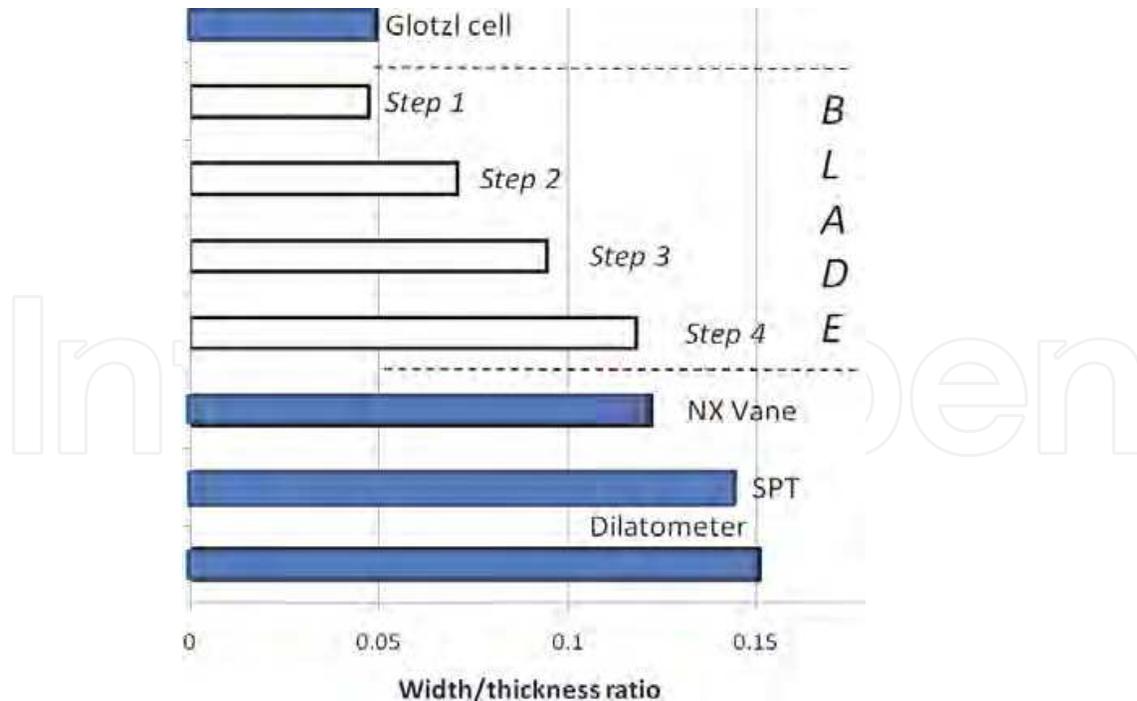


Fig. 4. Width: thickness ratios for various in-situ displacement test devices and (at top) relationships to soil behavior determined from Stepped Blade tests. Consolidating behavior is required to interpret lateral in-situ stress from Blade tests. The w/t ratio for the cone is 1.0. Modified from Handy et al. (1990)

4.4 Pneumatic pressure cells

The several requirements for pressure cells used in the Blade are (1) that they be as thin as the thinnest Blade step, (2) that they measure pressure on a discrete centralized area, (3) that they be durable and reliable, and (4) that when damaged they can be easily replaced or repaired in the field. A back-pressured pneumatic cell was developed to meet these criteria and is shown schematically in Fig. 5. An important advantage of the pneumatic cells is that measurements involve gas pressures and do not require calibrations.

The flat surface of each cell is covered with Teflon over an elastomeric membrane to minimize friction with the soil, and to create an internal seal between the two cell chambers. The chambers are slowly pressurized with gas pressure while maintaining a small differential between the two chambers of the cell. At the instant when the gas pressure equals the soil pressure anywhere around the threshold, the membrane is lifted sufficiently to cause an internal leak so pressures in the two chambers equalize. At that time pressure is read and recorded, and pressures to both chambers are simultaneously vented to prevent bulging of the membranes and ready the cell for the next pressure reading. Selector valves are turned to direct the gas pressures to the next Blade step, and the test procedure is repeated by simply closing the two relief valves. If desired, a reading can be repeated for additional confirmation.

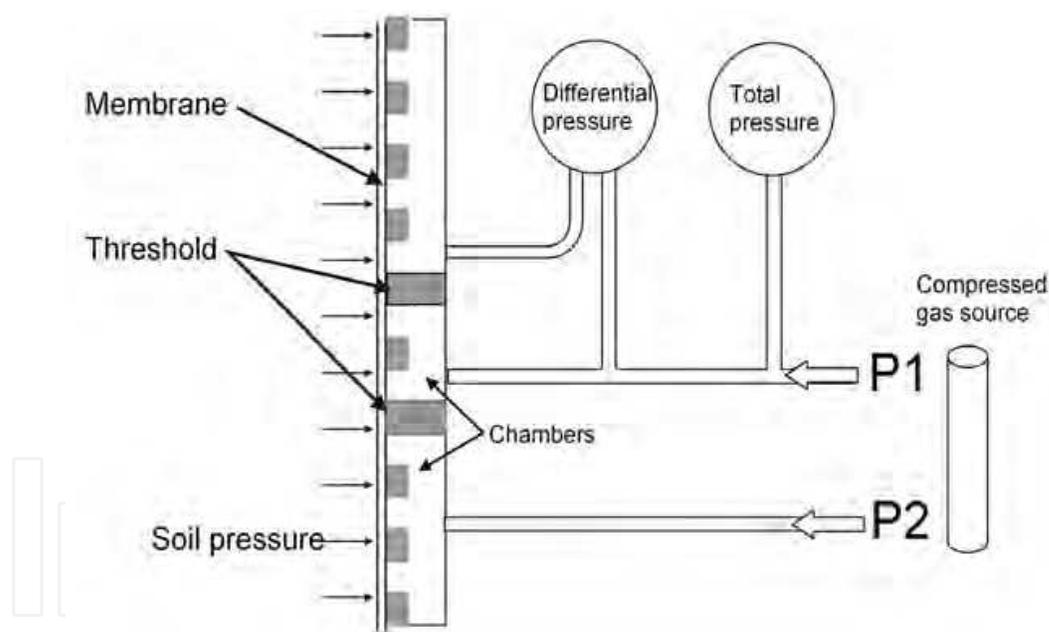


Fig. 5. Part of the secret of the K_0 Stepped Blade is the back-pressured pneumatic cells. At the instant when gas pressure equals soil pressure, the differential pressure goes to zero so the total pressure is read from the appropriate pressure gauge. After Handy and Eichner, U.S. Pat. No. 4,662,213, 1987.

Experiments have shown that the Teflon cover is much more durable than thin sheet metal but nevertheless can be scraped or punctured. If a leak occurs, positive gas pressure prevents water and soil from entering the cell, and since cells plug in they can be quickly replaced in the field. Experiments with electrical strain-gauged metal membranes showed that they were not reliable under the severe test conditions imposed in the field.

A hollow rib attached to the back of the Blade carries the multiple gas lines and reinforces against bending.

5. Test sites

Several test sites were selected in potentially liquefiable soils:

1. The Jackson Lake Dam was built to increase the water level in a natural glacial lake located on the Snake River in northwestern Wyoming, U.S.A. The log-crib dam was constructed in 1907 and failed three years later. It was replaced between 1911 and 1916 by a concrete and earth dam.

In 1964 the destructive nature of liquefaction was emphasized both by the Niigata Earthquake in Japan and the Alaska Earthquake, and in 1976 the Jackson Lake Dam foundation soil was determined to be a potentially liquefiable on the basis of Standard Because a failure could be catastrophic, in 1986-89 the dam was removed and the gravelly sand foundation soil was densified by Deep Dynamic Compaction, shown in Fig. 6. Stepped Blade Tests were conducted to measure lateral pressures in the sand after the dam was removed, both before and after the site was compacted.



Fig. 6. Deep Dynamic Compaction to prepare a gravelly sand foundation soil for the reconstructed Jackson Lake Dam, Wyoming. Reprinted with permission from Handy and Spangler (2007), The McGraw-Hill Companies, New York.

2. Considerable areas of poorly compacted sand fill exist in many bay areas including San Francisco, California, where there is a history of liquefaction dating from the 1906 earthquake. A potentially liquefiable sand was identified at the San Francisco Naval Shipyard, so experiments were conducted by the U.S. DOT-FHWA to determine the effectiveness of driving displacement piles to densify the sand. K_0 Stepped Blade tests

were conducted to measure lateral pressures in the sand both before and after pile driving.

3. A relatively recent development called Rammed Aggregate ® Piers, or the Geopier ® method for soil reinforcement, is becoming increasingly popular for improving foundation soils. As shown in Fig. 7, the rammer head is shaped to push aggregate outward into the soil and thereby increase lateral stress. K_0 Stepped Blade tests were conducted to measure those stresses at various depths and distances from the piers.
4. K_0 Stepped Blade tests conducted in collapsible loess soil to determine relationships between collapsibility and lateral in-situ stress, and suggested a history of liquefaction in a basal, saturated zone. This should affect slope stability, and massive landslides in the thick loess in China have taken many human lives. The trigger usually is excessive rains, but also can involve earthquakes.



Fig. 7. The Geopier hydraulically actuated rammer is shaped to direct part of the ramming energy outward and increase lateral stress in the surrounding soil.

5.1 Test results from the Jackson Lake Dam site, Wyoming

The Deep Dynamic Compaction shown in Fig. 6 involved dropping an approximate 100 tonne weight about eight times at each location on a grid pattern. This left a pattern of surface depressions that were 2 to 3 m deep. The results from K_0 Stepped Blade tests conducted before dynamic compaction are shown in the left diagram in Fig. 8, and afterwards in the right diagram in the same figure. The site was difficult because of angular gravel-size rock fragments in the soil.

At the left in Fig. 8, below about 5.5 m (18 ft) depth Stepped Blade tests gave K_0 in the neighborhood of 0.5 without giving consideration to the weight imposed by the dam. The Jaky relationship indicates that this corresponds to an angle of internal friction of 30° , which is reasonable for normally consolidated sand.

In the same figure, the weight of the dam compressed the soil and increased lateral stress down to a depth of about 4m (13 ft), indicating that normal consolidation penetrated to that depth. Below that depth there is a transition to a K_0 of about 0.3, indicating

underconsolidation and a potential for collapse that can be attributed to the weight of the dam.

At the right in Fig. 8, Deep Dynamic Compaction is shown to cause a substantial increase in lateral stress such that the average K_0 is 1.0, perhaps from seepage forces created when the weight of the overlying soil was supported by pore water pressure (Handy 2012). The uneven distribution of lateral stress after compaction may be symptomatic of localized shearing that concentrated stress in one area while reducing it in another.

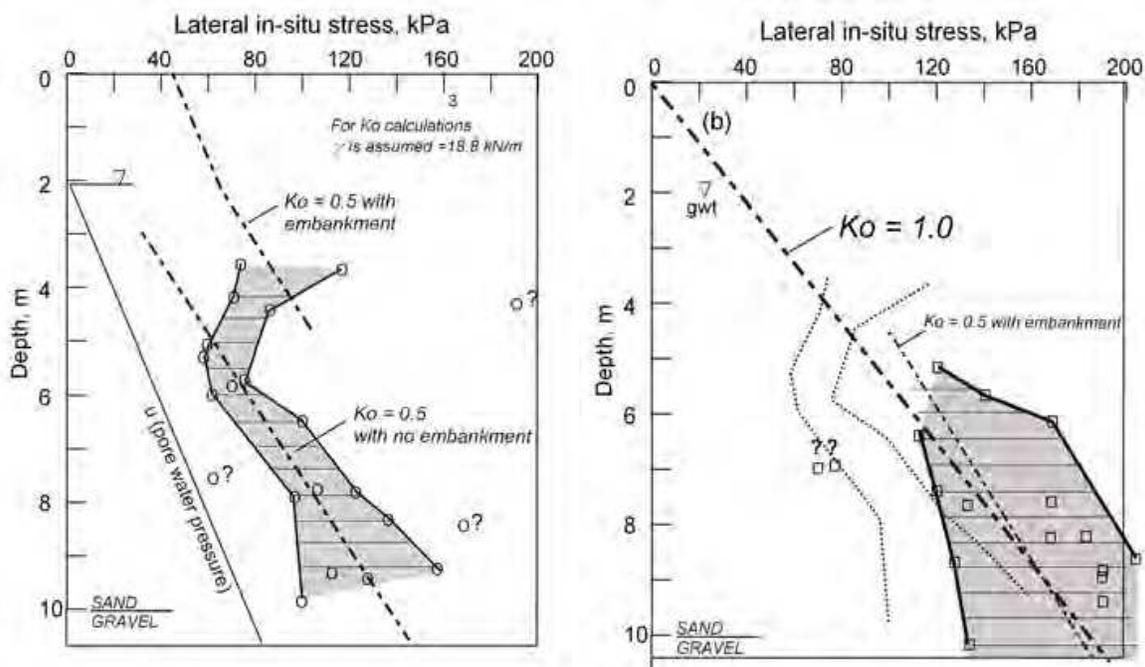


Fig. 8. Lateral stresses measured in foundation sand at the Jackson Lake dam site, (left) before and (right) after compaction. At the left, the soil was normally consolidated until after the embankment was built, when the weight of the dam compacted the upper part but left the lower soil underconsolidated and potentially liquefiable. At the right, compaction increased the average K_0 to 1.0, indicating a history of induced liquefaction, and $K_0 = 0.5$ after the dam was replaced. From Handy (2012), modified from Handy et al. (1990).

In this connection the narrow range of lateral stresses measured before compaction, in a range as low as ± 8 kPa or $\pm 10\%$, compared with $\pm 50\%$ after compaction, indicates that the variability is real and should not be assigned to random experimental error. Large variations in lateral stress also have been measured within a compacted earth embankment (C. Anderson, personal communication).

After compaction and restoration of the weight of the dam, K_0 is about 0.5, indicating that compaction was successful in creating a quasi-normally consolidated condition, and the dam should be safe from future collapse and liquefaction of the foundation soil.

5.2 Test results from the San Francisco Naval Shipyard, California

The San Francisco Naval Shipyard was first established in 1870 at Hunters Point in the southeast part of the city. Starting in the early 1900's the area was expanded by dumping fill

sand, making this an appropriate site to measure the effectiveness of driving piles to compact the sand. The discovery of contamination with nuclear and other wastes led to the shipyard being closed in 1994.

Lateral stresses measured prior to pile driving were quite consistent, shown by data points and the shaded area to the left in Fig. 9. A trend line drawn through the data and extended upward to the y axis indicates a prior surcharge equal to the weight of about 5 m (16 ft) of the same soil.

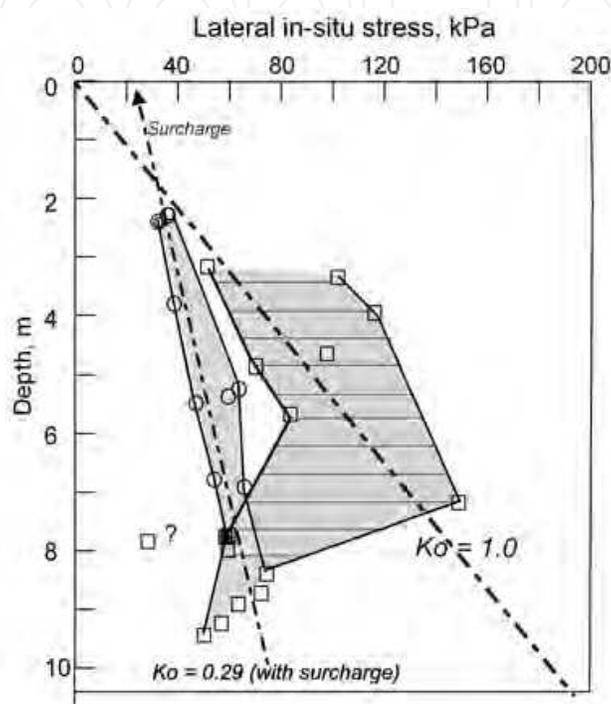


Fig. 9. Tests in loose sand at the San Francisco Naval Shipyard the slope of the lateral stress line gives $K_o = 0.29$ prior to pile driving, indicating a potential for liquefaction. As in the case of DDC, pile driving increased the average K_o to 1.0, indicating temporary liquefaction down to the bottoms of the piles. Data are from Handy et al. (1990)

The K_o value obtained from the slope of the trend line is 0.29, compared with a value of 0.5 that can be expected if the soil is normally consolidated. (A K_o calculated without taking into account the surcharge is meaningless as it becomes infinite at the ground surface.) The measurements of lateral stress confirm that the soil is underconsolidated and potentially liquefiable, and a suitable candidate for remediation.

Lateral stresses measured after pile driving were more erratic, but nevertheless cluster around $K_o = 1.0$ down to the bottom level of the piles. Below that depth, lateral stress remained unaffected.

The K_o value of 1.0 may be from seepage forces during temporary liquefaction that occurred during pile driving. Liquefaction would be beneficial for enabling a wider distribution of lateral stress, but also would tend to relieve passive pressures that otherwise would act to increase skin friction. A liquid condition cannot sustain a passive ratio any more than it can sustain an active ratio. The lack of passive pressure is consistent with a group effect of 1.0.

5.3 Test results after Geopier® soil reinforcement

The Geopier method for soil reinforcement involves boring holes that typically are 760 mm (30 in.) in diameter, and incrementally filling the holes with layers of rammed coarse aggregate. As previously noted, the hydraulic rammer head, Fig. 7, is shaped to direct a substantial portion of the ramming effort laterally outward into the soil.

5.3.1 K_0 no longer constant

Fig. 10 shows lateral stress measurements at different depths and distances from rammed piers at the site of a stadium in Memphis, Tennessee. The soil was saturated loessial silt. Because of the variability of the pier diameter caused by incremental ramming, data are plotted versus distance from pier centers. K_0 is not constant with depth, which is quite a different distribution of lateral stress from those in the preceding examples. Instead of K_0 it is the lateral stress at particular radial distances that is constant with depth. This probably relates to the uniform distribution of ramming energy throughout the length of a pier, and may be influenced by compaction that proceeds from the bottom up instead of from the top down.

5.3.2 Another evidence for temporary liquefaction

In Fig. 10 the pier contact stress is obtained by extrapolation of data using plastic theory. The open arrow pointing to the right in the figure indicates an undiminished transfer of radial compaction stress outward to a radial distance of almost 2 m ((6 ft). This radial distribution of stress is possible only if the soil temporarily behaved as a liquid.

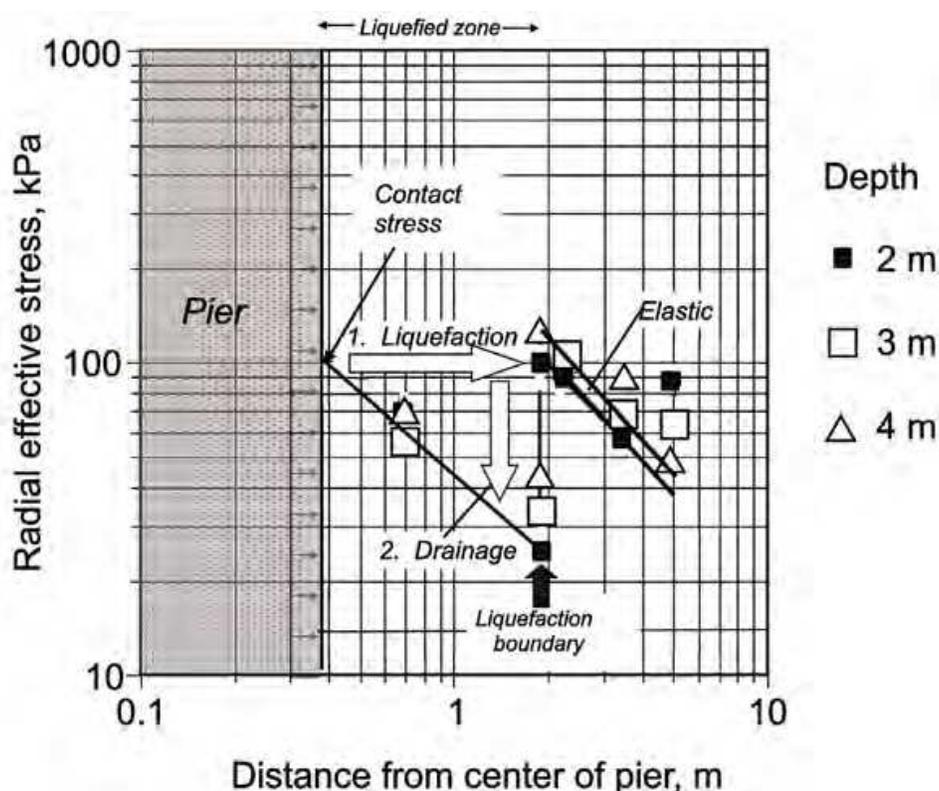


Fig. 10. Radial stresses measured in saturated loess soil near Rammed Aggregate Piers in Memphis, Tennessee. The data suggest a succession of behavioral modes that includes temporary liquefaction. Modified from Handy and White (2006).

5.3.3 Where does the water go during Geopier ramming?

Rammed Aggregate Piers are constructed incrementally by compacting successive layers of aggregate such that each layer has a final compacted thickness of about 0.3 m (1 ft). During ramming, the data in Fig. 10 indicates that within the liquefied zone the pore pressure exceeds 100 kPa (14.5 psi, 1 Tsf). Beyond a radial distance of 1.9 m, which is outside of the liquefied zone, the decrease in radial stress is linear with distance. Elastic theory indicates that this can occur only if there is a lack of restraint from circumferential tension, which can be explained by radial tension cracking. Radial cracks have been observed emanating outward from piers, and can extend down to a depth where the fluid pressure exceeds the existing lateral stress. In a normally consolidated soil below the groundwater table that can be as deep as 20 m (60 ft).

Open cracks are an open invitation for injection by a soil-water mixture, and soil particles carried in with the water can prop the cracks open and create a drainage gallery. That is the concept used in the “Hydrofrak” process to enhance oil and gas production from wells. After the fluid pressure dissipates, the injected cracks should create a ring of compression that preserves stress in the elastic zone.

Cavity expansion theory allows an estimate of the post-drainage friction angle from the slope of the line in the drained zone (Baguelin et al. (1978). This gives a friction angle of about 40°, which represents a substantial increase over friction angles commonly measured in loess (Handy and White 2006).

It will be noted in Fig. 10 that identical behaviors were observed at all three test depths.

5.3.4 How lateral stress affects preconsolidation pressure

Fig. 11 (a) represents a normally consolidated soil where intergranular friction acts to support the grains and reduce the lateral pressure. If a high lateral stress is applied, it can reverse the directions of the friction arrows, as shown in Fig. 11(b). Then when a vertical load is applied, before the grains can slip and the soil can consolidate, the load must be sufficient to again reverse the friction arrows, Fig. 11(c). This concept is further illustrated by the succession of Mohr circles in the lower part of the figure. It means that imposing a high lateral stress can cause a substantial increase in the preconsolidation pressure so within specific limits the soil will tend to compress elastically instead of by consolidating. This can reduce total settlement of a structure by an order of magnitude. This has been confirmed by load tests and is an important feature in Geopier design.

It should be noted that in a conventional laboratory consolidation test the lateral stress is not controlled, and in the conventional triaxial it is difficult to apply a lateral stress that exceeds the vertical stress, so in both cases the stress ratio remains in a normally consolidating mode.

What if lateral stress later is released, for example by trenching? During compaction the soil cohesion is greatly reduced by remolding, and then is recovered, a source of “setup factor” for driven piles. Schmertmann (1991) suggests that even the strength of sand can be doubled by aging, and a time-related recovery of strength is characteristic of liquefaction (Youd et al. 1996).

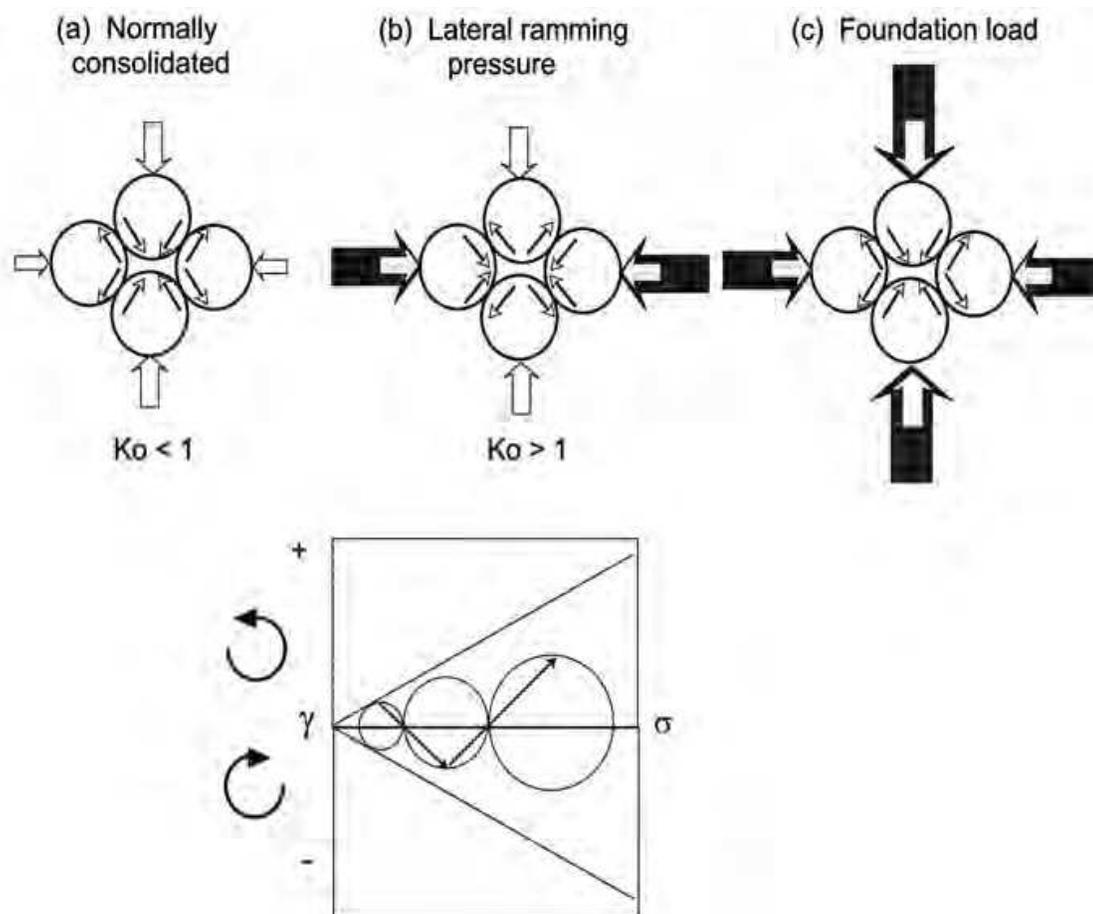


Fig. 11. Model illustrating how a high imposed lateral stress can reverse friction directions at grain contacts and increase the preconsolidation pressure without actually consolidating the soil. Modified from Handy (2001).

5.4 Test results in collapsible loess

Loess is wind-deposited silt, and comes from “löss,” which is German for loose. Thick loess deposits that are close to a river floodplain source are so porous that it does not retain a groundwater table. Only lower parts of deposits are saturated, where the groundwater table is perched on an underlying less permeable stratum.

The strength and ability to stand in high steep banks such as shown in Fig. 12 is attributed to negative pore water pressure that is enhanced by a small content of active clay minerals. The strength therefore is sensitive to the moisture content, and sudden collapse can occur upon wetting. A concentration of surface water therefore can contribute to rapid and intense gullyng and can crate vertical “chimneys.” Severe conditions can result in an almost karst-like terrain (Derbyshire et al. 1994).

5.4.1 Loess K_0 before collapse

Some of the early tests of the Blade were in collapsible loess, and gave K_0 values that were so low, 0.1 to 0.2, that they were met with considerable skepticism. However, the data were repeated and found to be consistent.

Low lateral stress probably results from dust settling into a nest of grasses and then being washed down by rains until grains come into contact, where they are held in place by negative pore water pressure or suction. As the loess accumulated, vegetation migrated upward and left root channels that although often mineralized still may enhance drainage. Densities as low as 100 kg/m^3 (64 lb/ft^3) have been measured in loess actively being deposited in Alaska.

The loess deposit investigated in this study is thinner and farther from the source than that depicted in Fig. 12, in a relatively flat area in the western part of Omaha, Nebraska. It has a weathered soil profile and probably has been subjected to modification of the lateral stress through compaction. K_0 values in Fig. 13 indicate that except for near-surface modifications the loess is collapsible down to a depth of about 5 m (16 ft).

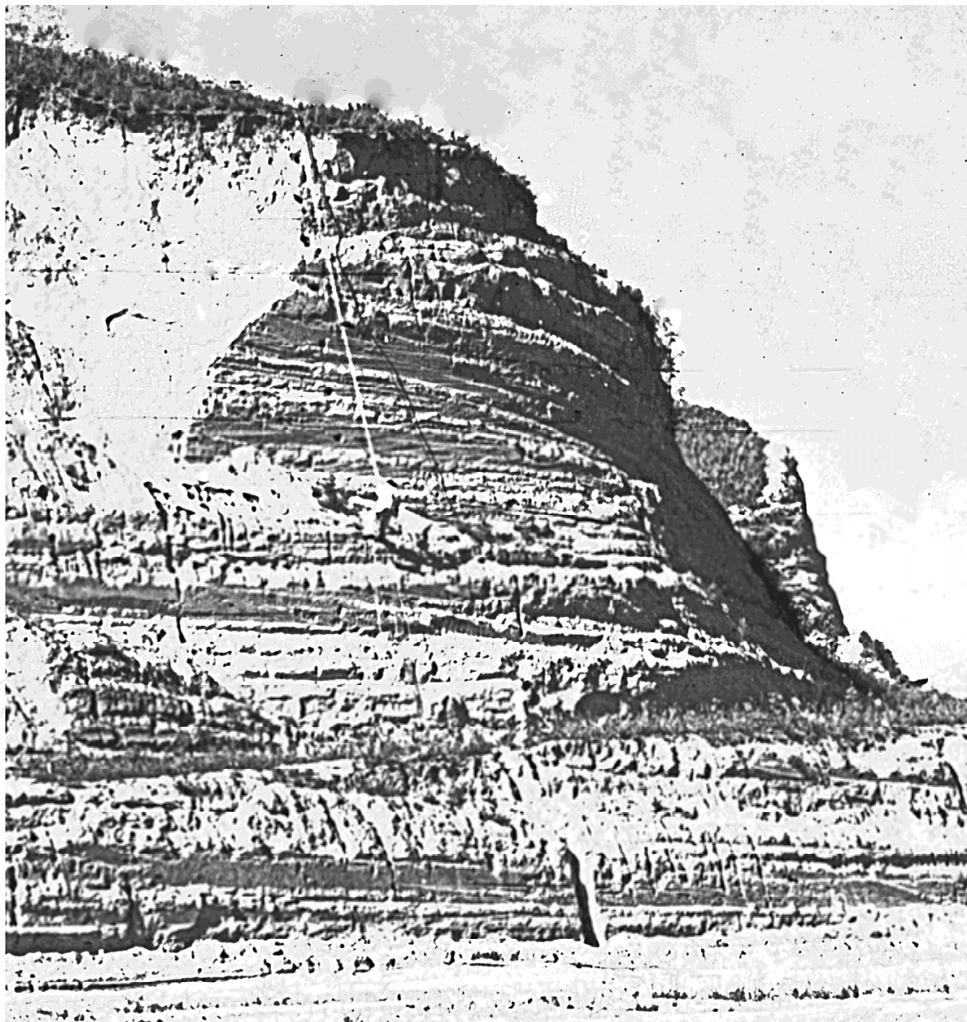


Fig. 12. Western Iowa loess deposits are thick close to the Missouri River floodplain, where glacial outwash was the major source of wind-blown silt. The author in his younger days is sampling about half-way down the cut slope.

5.4.2 Loess K_0 after collapse

In Fig. 13 in the lowest 2 m (6 ft) of the deposit, four independent data points give $K_0 = 1.0$. The reproducibility confirms the precision of the measurements, and the value may be an indication of accuracy since it characterizes a liquid condition.

The highest K_0 in Fig. 13, at a depth of 1 m (3 ft) translates into a vertical pressure of about 100 kPa (14 psi), which can be explained by ordinary tire pressures.

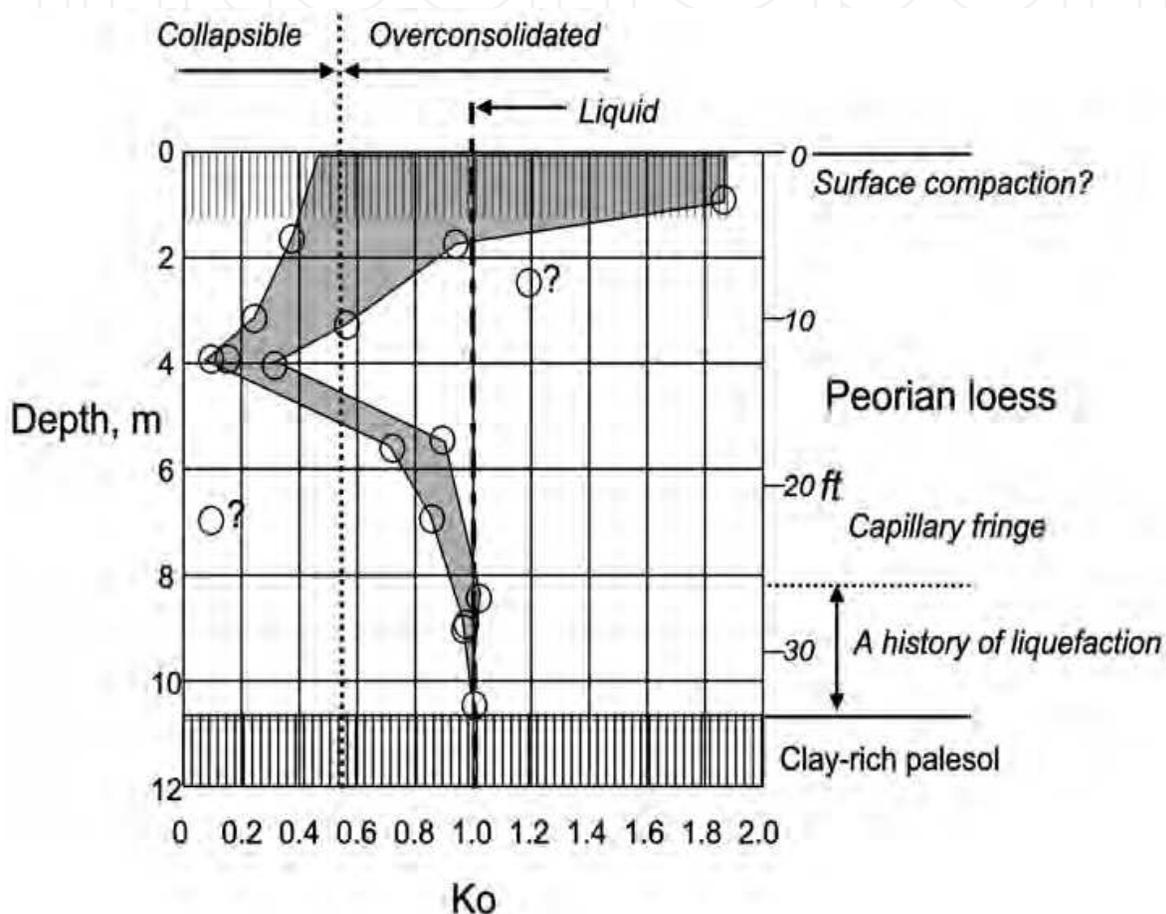


Fig. 13. K_0 Stepped Blade data from Nebraska loess about 17 km (11 miles) from the Missouri River floodplain source. From Handy (1994).

5.4.3 K_0 and partial collapse

Between depths of about 4 and 8 m (13 and 26 ft) in Fig. 13 there is a transition zone from the lowest value of K_0 to $K_0 = 1.0$. Ordinarily values that are higher than those resulting from normal consolidation are attributed to overconsolidation, but that appears to be unlikely at this site since it would require that loess deposition would be interrupted by a major erosion cycle.

The explanation may relate to a capillary fringe that includes saturation in the lower part, where pore water pressures are negative and diminish to zero at the groundwater table. The implication is that the soil may be only *partly* collapsed, increasing lateral stress to a value that is higher than normally would occur but not high enough to create a $K_o = 1.0$ liquid condition. To the author's knowledge this may be the first indication of a partial collapse.

5.4.4 Geography of collapsible loess

Loess deposits become thinner and finer grained with increasing distance from a source. Larger near-source thicknesses such as shown in Fig. 12 can be attributed to randomly variable wind directions that pile up more soil when winds are blowing nearly parallel to a source (Handy 1976).

A commonly used predictor for loess collapsibility is the saturation moisture content, which can be calculated from the soil unit weight and mineral specific gravity, in relation to the liquid limit. Loess containing less than 16 percent 2 μm clay is very likely to be collapsible, and one containing more than about 32 percent clay is not (Handy 1973).

5.4.5 Loess collapse and slope stability

A relationship between liquefaction of loess and slope stability is instantly appreciated if one walks along the toe of an active landslide involving saturated basal loess. The soil squeezing out of the basal zone only appears to be firm enough to walk on. The most likely condition for sliding is after extensive rains that raise the groundwater elevation to a new level in underconsolidated loess.

6. Conflicting observations

A K_o that is less than about 0.5 for sand and 0.55 for silt indicates an underconsolidated condition that is susceptible to collapse and liquefaction. This can be triggered by an earthquake or other ground vibrations such as those induced by pile driving or compaction, or in the case of loess soils, by saturation.

However, a low K_o is not required for lateral compaction used in the Geopier process.

Whereas compaction from the top down can result in a constant K_o with depth, Geopier compaction results in a variable K_o and constant lateral stress with depth, with the developed lateral stress depending on radial distance from the piers.

What may explain this difference? One possibility is that the pier ramming process involves compaction energy that is uniformly distributed with depth, resulting in a different stress distribution than when compaction is from the ground surface down.

A Rammed Aggregate Pier is surrounded by a zone of highly compacted soil that should help to contain the ramming energy for the next lift. That may not be the case for pile driving, although this possibility is yet to be investigated. Pile driving from the top down may generate a peripheral zone of sheared and dilated soil that is less able to confine the pressure created by driving.

With normal consolidation, K_o normally is in a range 0.5-0.6, whereas the K_o acquired after drainage of a liquefied soil is consistently 1.0. Why the difference? In both cases soil grains are settling out of water that is under pressure.

One difference is that normal consolidation that occurs as vertical stress gradually is increased activated intergranular friction as shown in Fig. 1(a), where it acts to partly support the individual soil grains and reduce lateral stress. During liquefaction that friction does not exist, and during drainage there are large seepage forces and a transfer from fluid pressure to effective stress. The process would be difficult to simulate but might be examined with laboratory tests.

7. Conclusions with regard to K_o , collapse and liquefaction

A low K_o is an indicator of a potential for collapse and liquefaction, provided that there is a sudden change in conditions that is sufficient to break down the soil structure.

The defining value for collapse and liquefaction is a K_o less than about 0.5-0.55. The angle of sliding friction of quartz is about 25° , which according to the Jaky relationship should result in $K_o = 0.58$ without a contribution from positive or negative dilatancy.

During liquefaction as water is ejected from soil its density increases, tending to make liquefaction a one-time event that is mainly limited to geologically young or poorly compacted man-made sand or silt deposits.

Lateral in-situ stress is a manuscript for stress history, as certain values are indicative of normal consolidation, overconsolidation, and underconsolidation. For example, the amount of removal of overburden often can be determined from a linear relationship between lateral stress and depth. In a preciously collapsed soil K_o is consistently found to equal 1.0.

Artificial compaction results in a much higher variability of lateral stress than does consolidation involving natural processes.

8. Conclusions with regard to the instrumentation

Lateral stress has been called the "Holy Grail" of soil mechanics (M. Jamiolkowski, personal communication). It plays a major role in shallow and deep foundation bearing capacity, pile skin friction, slope stability, and pressures on retaining walls.

The stated goal of the research sponsor, the U.S. DOT-FHWA, was to support development of an instrument that would rapidly determine lateral *in situ* soil stress within an arbitrary margin of ± 1 psi (7 kPa). In Fig. 11 the several data points for $K_o = 1.0$ occupy a much narrower range, within about $\pm 0.4\%$. The narrow range of data in Fig. 8 fall in a range ± 6 kPa, which still is within the target range even though it includes a soil variable.

The reproducibility and speed of the test make it useful not only to determine average values, but also to define standard deviations that may be equally meaningful, for example by diagnosing man-made compaction as opposed to natural consolidation.

The possible use of lateral in-situ stress to indicate a potential for collapse or liquefaction was not anticipated during development of the test. However, it is not unusual for a new approach to open new vistas and paths that can unexpectedly lead in diverse directions.

The easy path to lateral stress is through computer modeling, and before that there was mathematical modeling. However, every model should be tested against the real world because otherwise it may be accepted but nevertheless flawed. Measurements inspired Terzaghi to develop consolidation theory; it was not the other way around. Measurements made in the 20th Century of pressures on retaining walls revealed inadequacies of 19th and 18th Century Rankine and Coulomb theories (Terzaghi 1943; Handy 1985). Measurements of soil loads on buried pipes made over 100 years ago led to a theoretical model that requires empirical corrections and only recently was shown to incorporate a simple and obvious error (Handy 2004).

Lateral stress existing in soil at the toe of an embankment or earth dams should be an index to their stability against sliding, by allowing a determination of an existing factor of safety. Lateral stress existing in soil next to a shallow foundation can test and evaluate the safety against a foundation failure, and can be helpful to determine if an additional load may be allowable. Measurements of lateral pressures in soils behind a retaining wall target the existing factor of safety and how it may change under varying conditions. With so many potential uses the K_0 Stepped Blade may be underutilized.

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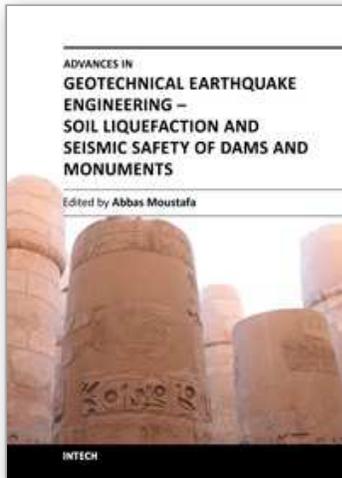
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This book sheds lights on recent advances in Geotechnical Earthquake Engineering with special emphasis on soil liquefaction, soil-structure interaction, seismic safety of dams and underground monuments, mitigation strategies against landslide and fire whirlwind resulting from earthquakes and vibration of a layered rotating plant and Bryan's effect. The book contains sixteen chapters covering several interesting research topics written by researchers and experts from several countries. The research reported in this book is useful to graduate students and researchers working in the fields of structural and earthquake engineering. The book will also be of considerable help to civil engineers working on construction and repair of engineering structures, such as buildings, roads, dams and monuments.

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