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Evaluating the Potential for Earthquake-Induced Liquefaction in Practice

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ABSTRACT

Evaluation of the potential for earthquake-induced liquefaction of sandy soils has become a routine part of geotechnical engineering practice worldwide, but there are practical considerations that are frequently overlooked in these evaluations and these omissions, in combination with over-reliance on overly simplified methods of analysis, frequently lead to excessively conservative evaluations. As a result, liquefaction is often predicted even when there is no evidence of past liquefaction in the type of soils and seismic environment in question. These practical considerations have mostly to do with the impact on the potential for liquefaction of what are sometimes thought of as secondary factors including ageing, prestraining, overconsolidation and the effect of fines, in addition to external factors such as a lack of continuity of sand layers, lack of complete saturation and partial drainage. Taken together these effects can be anything but secondary, but they tend to be overlooked, particularly when use is made of simplified methods that are based on correlations that largely rely on case histories of liquefaction of relatively clean, relatively recent sandy deposits. These points are illustrated by examples from practice in Australia, New Zealand and California.

Introduction

There is no doubt that the liquefaction of sands and silts with non-plastic fines in earthquakes is real and has been the cause of significant damage to port structures and to other facilities constructed on recent alluvium or man-made fills with a high water table. However, even though this phenomenon has been the subject of detailed study for about fifty years, there are two recurring problems in practice. One that is particularly germane to the host city for this conference is that the basic hazard is still ignored in some planning and political contexts. It should not have been necessary to wait for the occurrence of the recent earthquakes to either recognize the hazard to both urban and suburban development, and in particular utilities, due to the likelihood of liquefaction and settlement in recent alluvial deposits along the Avon River, just as it should have been more obvious that the active tectonics that are very visible in the North Canterbury Fold and Thrust Belt are concealed by the sediments that underlie the Canterbury Plain. But the other side of the coin is that as geotechnical engineering has been commoditized, too many engineers are cranking out simplified analyses of the potential for liquefaction and settlement that greatly exaggerate the actual hazard and are predicting liquefaction where there is no precedent for liquefaction in similar soils and tectonic environments. This paper examines some of the causes of this second problem and provides three examples of projects where the initial evaluations were unnecessarily conservative. This paper can be viewed as a companion to Semple (2013).

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Background

There are actually two broad problems with the use of simplified methods for the evaluation of the potential for liquefaction and settlement in the real world. The first is that that this is just one example of the increasing use of canned spreadsheets both in teaching and in practice that make it difficult for the user to exercise his or her own judgment, rather than encouraging the user to exercise judgment that takes into account the both local geology and local experience with the phenomenon in question. The second is that penetration resistance is an imperfect measure of liquefaction resistance. Because penetration resistance does not fully account for the effects of overconsolidation, previous cyclic shearing, age and the presence of fines, especially clayey fines, on the potential for liquefaction, these simplified methods based on penetration resistance should not be used for site conditions which are outside the ranges of site conditions in the databases of case histories on which they are based. Those case histories consist almost entirely of observations of liquefaction in recent alluvium and hydraulically-placed sand fills. In the case of the case histories which rely on CPT data, this means that the parameter I_c is likely to have been less than 2.05, indicative of a relatively clean sand.

The initial laboratory studies of the phenomenon of liquefaction under cyclic loading were conducted by Harry Bolton Seed and his co-workers Kenneth Lee and Clarence Chan in the mid-nineteen-sixties. Those tests were conducted on reconstituted samples of clean, washed sands. In the earliest studies the sands both in the laboratory and in the field were simply characterized by their relative density. However, it quickly became apparent that relative density alone was an inadequate measure of the resistance to liquefaction of a sandy soil. In the early nineteen seventies various students of Professor Seed including the author, John Mulilis and Kenji Mori, showed the importance of soil fabric, and hence the method of specimen preparation in the laboratory, the time under sustained pressure, or age, the previous straining history, overconsolidation and the coefficient of lateral earth pressure on the resistance to liquefaction. These effects were first brought together by Professor Seed in a paper presented at the BOSS ’76 conference, Seed (1976). However, the recognition that it was difficult both to prevent the loss of these effects as a result of sampling disturbance and to fully recreate them in the laboratory, along with pressure from Professor Ralph Peck and others to emphasize a more empirical procedure, led Professor Seed in an important paper in 1979 to emphasize the use of empirical methods, at that time focusing on use of the SPT, over the use of laboratory tests because the same factors that tended to increase the resistance to liquefaction also tended to increase the penetration resistance, as measured by the SPT blowcount. However, it was never stated or implied that penetration resistance was a perfect analog for liquefaction resistance, because it is not. For instance, overconsolidation is known to increase the coefficient of lateral earth pressure, but the increase in the resistance to liquefaction caused by overconsolidation is greater than that implied just by the increase in the coefficient of lateral earth pressure.

Unfortunately, after 1979 the emphasis in most academic studies has been directed to refining the empirical evaluation methods based on penetration resistance, rather than quantifying the divergence between penetration resistance and the actual resistance to liquefaction of real soils. This is not a big deal in the case of very recent alluvial deposits and hydraulically placed sand fills which constitute the bulk of the case histories of liquefaction in earthquakes, but it is very important for all other soils. Had Professor Seed lived longer he surely would have paid more attention to the divergence between the penetration resistance and the resistance to liquefaction of these other soils. It was particularly disappointing that this was not addressed in the NCEER and MCEER workshops which led to the paper by Youd, Idriss et al. (2001). That is why the author contributed a discussion on that paper, Pyke (2003). That discussion has been widely quoted in practice but has had less impact on academic research and publications.

A strong argument can in fact be made that the shear wave velocity is a better indicator of the liquefaction resistance of soils than penetration resistance, but critics of that belief argue that under earthquake loading the most sensitive aspects of soil fabric that control the low strain shear modulus, and hence the shear wave velocity, might be lost. However, Professor Ricardo Dobry and others have shown rather conclusively that the rate of excess pore pressure development is very dependent on small strain behavior, e.g. Dobry et al., (2015) and there is no evidence that cyclic loading significantly disrupts soil fabric short of complete liquefaction.

Example No. 1

This industrial site is located on the coast of Western Australia, an area of relatively low seismicity, and is underlain by clayey sands to a depth of about 20m. It is notable that the sands below the water table were apparently laid down in the last interglacial period and are 100,000 years or more old. It is notable that the sand particles are coated with clay and that most samples have measurable plasticity indices. Because the ground surface slopes towards the coast, the depth to the water table is not constant but varies from as little as 5m to more than 20m, and there is a wedge of the sands that is saturated. The sands at the site are somewhat unusual, but not unique. Some years ago the author worked with the late Professor Seed on a US Army Corps of Engineers project, Arcadia Dam in Kansas, that had similar soils in its foundation. I recall that we were very pleased with ourselves for inventing a new correction factor to account for the fact that the penetration resistance in clayey sands is less than in a similarly compact clean sand, although that particular correction factor never found its way into print.

The configuration of the sands at the site might be a problem if the saturated wedge of the sands is loose and thus contractive in behavior, but that does not appear to me to be the case. There are several lines of evidence on this. The samples of the sands tested in the laboratory showed strong dilation in monotonic simple shear and, although some test specimens finally “failed” after a large number of cycles, they generally showed “clayey” behavior in cyclic simple shear. That is, they showed some development of excess pore pressures and cyclic strains under cyclic loading but did not exhibit runaway failure within the number of cycles felt in an earthquake. Examples of this kind of behavior can be seen, for instance, in Idriss and Boulanger (2008). Unfortunately, because the low strain moduli or shear wave velocities of the samples that were tested were not measured in the laboratory, it is not possible to evaluate the effects of inevitable sample disturbance, and thus it is not possible to use the laboratory test results in a quantitative manner.

Some of the limitations of using penetration resistance as an analog for liquefaction resistance have already been noted but the CPT in particular is an excellent tool for assessing stratigraphy and, if used with caution, can be quite revealing of soil properties. Careful examination of the CPT results in the saturated wedge suggests that the sands in this zone are not so much looser but have lower penetration resistance both because they are below the water table and are more clayey. It should not be surprising that the sands might exhibit softer behavior where the clay coating the particles is saturated rather than dry.

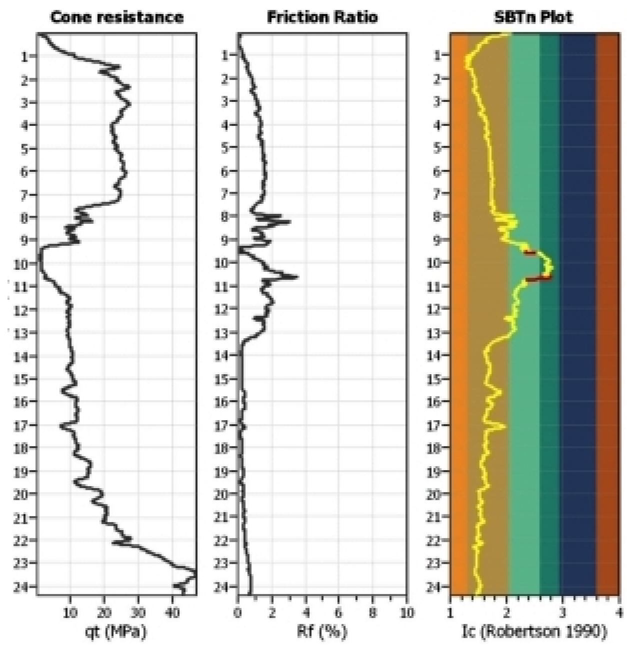

Figure 1. Typical CPT, Western Australia Site

The effect on the penetration resistance of varying amounts of fines, which in this case are largely clayey fines, may be seen in Figure 1, generated using the computer program CLiq v.1.7.1.6 developed by GeoLogismiki, which shows a representative CPT that penetrates the saturated wedge. It may be seen that when I_c increases, penetration resistance decreases. Conversely, in this and the other CPTs in this zone, when the I_c falls below 2.05, the penetration resistance tends to be higher. Thus, if one were to attempt to do a quantitative analysis using CPT data, the data should probably be screened at an I_c value of 2.05 and an ageing correction should be applied. The phenomenon of ageing of sands is documented in the Terzaghi lectures by Mitchell (1986) and Schmertmann (1991) and the specific impact of ageing on the resistance to liquefaction under earthquake loadings is addressed by Leon et al. (2006), Lewis et al. (2008), and Hayati et al. (2008), although these authors all provide a fairly wide range for possible correction factors.

But, the measured shear wave velocities provide the clinching argument at this site. Seismic CPTs indicate typical shear wave velocities in the order of 500 m/sec, with low values in the order of 400 m/sec and many higher values. The upper limit of shear wave velocity for sands reported to have liquefied is, according to Andrus and Stokoe (2001), and as updated by Dobry et al. (2015), is only in the order of 210 m/sec. Thus, in spite of other opinions to the contrary, the author concluded that the probability that the sands at this site might liquefy in any conceivable earthquake is so low that a more detailed quantitative analysis was not required.

Example No. 2

This site is located in the Hamilton Basin in the North Island of New Zealand in an area of moderate seismicity. The materials in question are mid to late Pleistocene terrace deposits that have no history of liquefaction and lateral spreading of engineering significance, even adjacent to incised streambeds. Kleyburg et al. (2015) have recently found two examples of paleo liquefaction in similar deposits but, more likely than not, these incidents of liquefaction occurred when the deposits were more youthful, and they are not in any case of engineering significance.

The geotechnical engineers on the project used as the basis for their evaluation the EERI Monograph by Idriss and Boulanger (2008), which can be viewed as an update of Youd, Idriss et al. (2001). However, the first step in any evaluation of liquefaction should be to ask the question "is there any historic evidence of liquefaction of these soils in this seismic environment" and the second question should be "are the standard analytical procedures applicable". The most striking thing about the soils at this site is their variability. As an example, the descriptions of the soils at a depth of 3m in two adjacent borings were quite different. In one there was "tightly packed 'dense' grey coarse gravel and cobbles" and in the other there was "loose grey silt, some fine sand, minor organics". At a depth of 6m in second boring there was "medium dense orange brown fine to coarse sandy fine gravel" and in a third boring at that depth there was "medium dense dark grey fine to coarse sand". These are just examples, but the overall impression is that this is a very heterogeneous deposit that likely varies widely over quite short distances. The importance of this heterogeneity in which looser sands and silts likely occur as lenses or tortuous channels rather than continuous layers is that, as explained in Pyke (2003), the cyclic shear strains within soft inclusions are controlled by the stiffer surrounding matrix, so that even in materials that might develop high excess pore pressures in a uniform deposit, cyclic shear strains and excess pore pressure development are limited.

Such a variable deposit can also make it difficult to use CPT data for the analysis of liquefaction potential because the tip resistance is affected by the materials at least several diameters in front of the tip, whereas the soil type is interpreted from the friction ratio that is measured behind the tip. CPT data should be screened to eliminate data obtained in "transitions" from one material type to another, as indicated in Figures 1 and 2, and it was not clear whether that was done in this case. But, more importantly, the effects of ageing were not accounted for and the use of the shear wave velocity measurements was rejected because of the "conflicting results" that were obtained. Even though the measured shear wave velocities are likely low relative to the stiffness of these materials because of the presence of pumice, the normalized shear wave velocities are generally in the vicinity of, or slightly higher than the value of 210 m/sec, the upper limit for the shear wave velocity of soil layers that have been observed to have liquefied. There were some lower values but these are in materials that have a value of I_c greater than 2.6, suggesting that the materials in question are clayey and not susceptible to liquefaction. The fact that the shear wave velocities are not well above 210 m/sec might be of some concern if the site were located in an area that is expected to have both longer and more intense ground shaking, but should not have been at this site. There was in any case a very simple design measure that can eliminate any residual risk and that was to use pervious backfill so that any excess pore pressures that might be generated in the vicinity of the planned structures would quickly dissipate. Such drainage was successfully incorporated into the design and construction of the combined sewer box that runs along the edge of the Marina Green in San Francisco. In the 1989 Loma Prieta earthquake, there was clear evidence of liquefaction on either side of this box but no visible displacement of the box itself. Thus, although the engineers on this project had presented a comprehensive report on this site, their recommendation that significant and expensive ground improvement be undertaken was not necessary to keep any risks to acceptable levels.

Unfortunately the formulae and charts for computing post-liquefaction settlements provided by Idriss and Boulanger (2008) and used by the engineers to compute post-liquefaction settlements of up to 700 mm, are not only out of date but are not applicable to this site. Following complete liquefaction there may indeed be large settlements on reconsolidation but since liquefaction is not expected in this instance and the site has presumably been shaken many times since these materials were deposited, any settlements resulting from seismic shaking should be negligible.

Likewise the slope stability calculations performed by the engineers were not applicable to this site. The assumption of a widespread drop in shear strength to residual values would be very conservative even if there were pockets of liquefaction, and the empirical relationships of Youd et al. (2002) are at best a screening tool that is only applicable in recent deposits of the same kind that are in their database.

Example No. 3

This site is located on the margin of San Francisco Bay and lies between the San Andreas and Hayward faults. The site is underlain by 1½ to 9 feet of undocumented fill, young alluvial clays that vary from zero to about 7½ feet in thickness, and older alluvial deposits to the maximum depth explored of 100 feet. The exact age of these older deposits is not known, but suffice to say that they are not recent Holocene deposits as are the young alluvial clays. Shear wave velocities were measured in a single seismic cone penetration test and range from a low value of 628 ft/sec to a high value of 957 ft/sec.

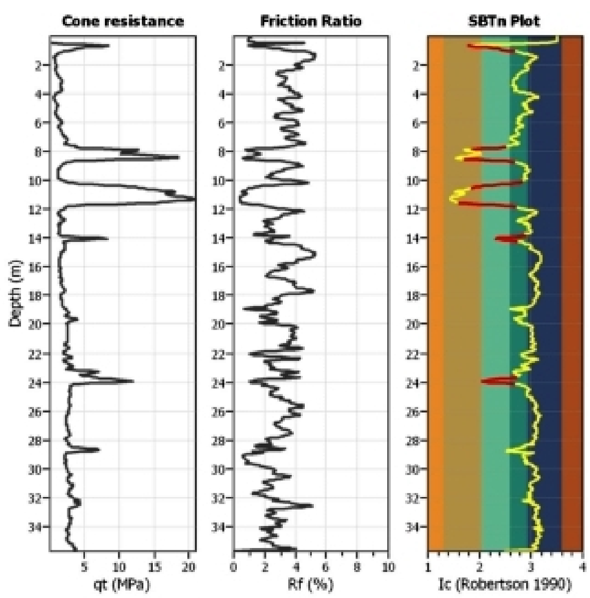

Figure 2. Typical CPT, California Site.

The older alluvial deposits contain lenses of sand that might be susceptible to liquefaction and the geotechnical engineer, using the procedures recommended by Idriss and Boulanger (2008), calculated that these sand lenses would liquefy under the expected MCE ground motions at the site. Because a very high profile building was planned at the site, and in accordance with the building code, the site was classified as Soil Type F, requiring a site specific response analysis to determine the ground surface motions for design of the building. The author was approached to conduct these analyses but felt that although a site specific response analysis was appropriate given the importance of the building, the evaluation of liquefaction potential was quite conservative because of the lack of continuity of the sand deposits and the effect of ageing.

A typical CPT result, again generated using CLiq, is shown in Figure 2. It may be seen that there are indeed several occurrences of clean sand indicated by values of I_c of less than 2.05, but these occurrences had varying thicknesses and occurred at different depths in each of eight cone soundings. Thus, again, the clean sands appear to be lenses, rather than continuous layers, and some correction to the standard procedures for evaluating liquefaction should also be made for the effect of ageing.

In order to address these issues the author conducted one-dimensional nonlinear site response analyses to test the effect of some generation of excess pore pressures on the expected ground surface motions at this site using his own computer program TESS (TAGAsoft, 2015). A representative profile was used that included two sand layers. Because in a one-dimensional analysis it is not possible to represent anything other than continuous horizontal layers, the assumed properties for these layers generated excess pore pressures at a rate that might be expected in lenses rather than continuous layers. Although the program allows modelling of redistribution and dissipation of excess pore pressures, that option was not used in this instance. Acceleration histories corresponding to Site Class C were input at depth and the computed ground surface response spectra were generally similar to a Site Class D spectrum. Thus, while it was recommended that the foundations be well tied together, it was concluded that for the purposes of structural design the site should simply be treated as Site Class D and that the effects of potential excess pore pressure development on ground motions were not significant.

Conclusions

In summary, the simplified methods such as those proposed by Youd, Idriss et al. (2001), might be fine as screening tools but in general they should not be used for anything beyond that. In particular, they should not be relied on for projects that have significant financial considerations both in terms of the cost of construction and the consequences of failure. Rather than emphasizing use of these simplified methods, academics should be pursuing more studies of the kind recommended by Dobry et al. (2015) including "further laboratory and field research toward clarification of the specific factors causing the observed increased liquefaction resistance of natural sands discussed in the paper, such as geologic age and preshaking by previous earthquakes."

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