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# **New Insight in Liquefaction After Recent Earthquakes: Chile, New Zealand and Japan**

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Additional information is available at the end of the chapter

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## **Abstract**

Liquefaction has proved to be one of the major geotechnical issues caused by earthquakes. It is one of the most costly phenomena and has affected several cities around the world. Although the topic has been studied since the 1960s, new questions are emerging. The earthquakes of Chile in 2010, New Zealand in 2010 and 2011, and Japan in 2011 had in common not only being some of the largest earthquakes of this decade but also having a problem of extensive liquefaction. Although most seismic codes have provisions against liquefaction, there are still some misconceptions regarding the characteristics of soil susceptibility and the effect of repeated liquefaction. This chapter introduces a detailed report of the damage caused by liquefaction in the cities affected by those earthquakes and also highlights observations in liquefied areas that were unexpected. Advanced geotechnical testing was conducted and compiled to compare them with previous assessment criteria and observations. A more comprehensive framework for the evaluation of liquefaction susceptibility and countermeasures will be presented and a roadmap of future work in the area will be described.

**Keywords:** iquefaction, fines content, repeated liquefaction, 2010 Chile Earthquake, New Zealand Earthquake, 2011 Japan Earthquake

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## **1. Introduction**

Liquefaction is a hazard that has caused a large number of casualties and economic losses. Although the phenomenon has been observed for a long time, only relatively recently it has been acknowledged and more discoveries are being done during the latest seismic events. Dutch engineers recognized the phenomenon of strength loss and pore-pressure increment after the severe flow slides caused by vibration near a railway bridge at Weesp in 1918 [1].

Mogami and Kubo [2] reported small heavings of sand at some places as Amagasaki during an earthquake in 1951. They performed experiments on Kumiho sand and other materials, using a metal box able to move vertically and sinusoidally. They found that as acceleration increased, shearing strength decreased to almost zero which made the soil behave as a liquid and they decided to call this phenomenon liquefaction. Nevertheless, it was until the earthquakes of 1964, in Anchorage, Alaska and Niigata, Japan, that liquefaction was acknowledged as an important engineering problem. From that point, a vast research has been conducted on liquefaction. However, recent earthquakes in Chile, New Zealand and Japan have proved that there is still a gap of knowledge in this topic regarding susceptibility of silty soils, repeated liquefaction and ageing effects.

### 1.1. Chile Earthquake

A  $M_w = 8.8$  earthquake hit the west coast of Maule, Chile, on February 27, 2010, at 3:34 local time. The epicentre, 335 km away from Santiago, was located offshore at 35.909°S, 72.733°W with a depth of 35 km and had a plate rupture area of about 550 by 150 km. This earthquake was the second largest in the Chilean History and was accompanied by a large number of aftershocks that continued over several months and reached magnitudes higher than six. The area affected included the cities of Santiago, Vina del Mar, Angol and Concepcion in three different regions as shown in **Figure 1**. One of the most distinctive characteristics of this earthquake was the long duration of large accelerations. In some areas, values greater than 0.05 g lasted longer than 60 s or even 120 s.

A group of Japanese experts (Architectural Institute of Japan, Japan Association of Earthquake Engineering, Japanese Geotechnical Society and the Japanese Society of Civil Engineering) along with Chilean specialists prepared a report regarding liquefaction in the affected areas of Chile [3]. Their reconnaissance showed that given the magnitude of the earthquake, several places experienced liquefaction, although given the season of the year, the groundwater table was low and only sites located near a saturation source liquefied. The soil was found to be composed of quaternary deposits in the coastal area, the general stratigraphy included alluvial deposits, strata of loose sandy silt and a sand backfill, where liquefaction usually started [4].

Several buildings, including modern structures, underwent differential settlement in the Concepcion area and some buried tanks emerged due to liquefaction. Ports, a fundamental infrastructure for the Chilean economy, were affected by liquefaction and lateral spreading. Bridges such as the Llacolen bridge, the Juan Pablo II bridge and the Bio-Bio bridge showed column shear failures and pier settlement [5]. There was also damage in dams, slopes and embankments, where cracks were observed. In tailing dams, the remaining of ore account for particles as fine as silt which increased the liquefaction potential. Mines as Curico, Veta del Agua and La Florida exhibited signs of liquefaction [4]. Liquefaction also destroyed several water distribution pipelines and large-diameter steel transmission pipelines, but high-density polyethylene (HDPE) pipes remained undamaged as reported by Duhalde [6].

On the other hand, in residential developments, it was observed that mitigation measures such as dynamic compaction were very effective against liquefaction, for instance, at the Ribera Norte Bío Bío housing project [3].

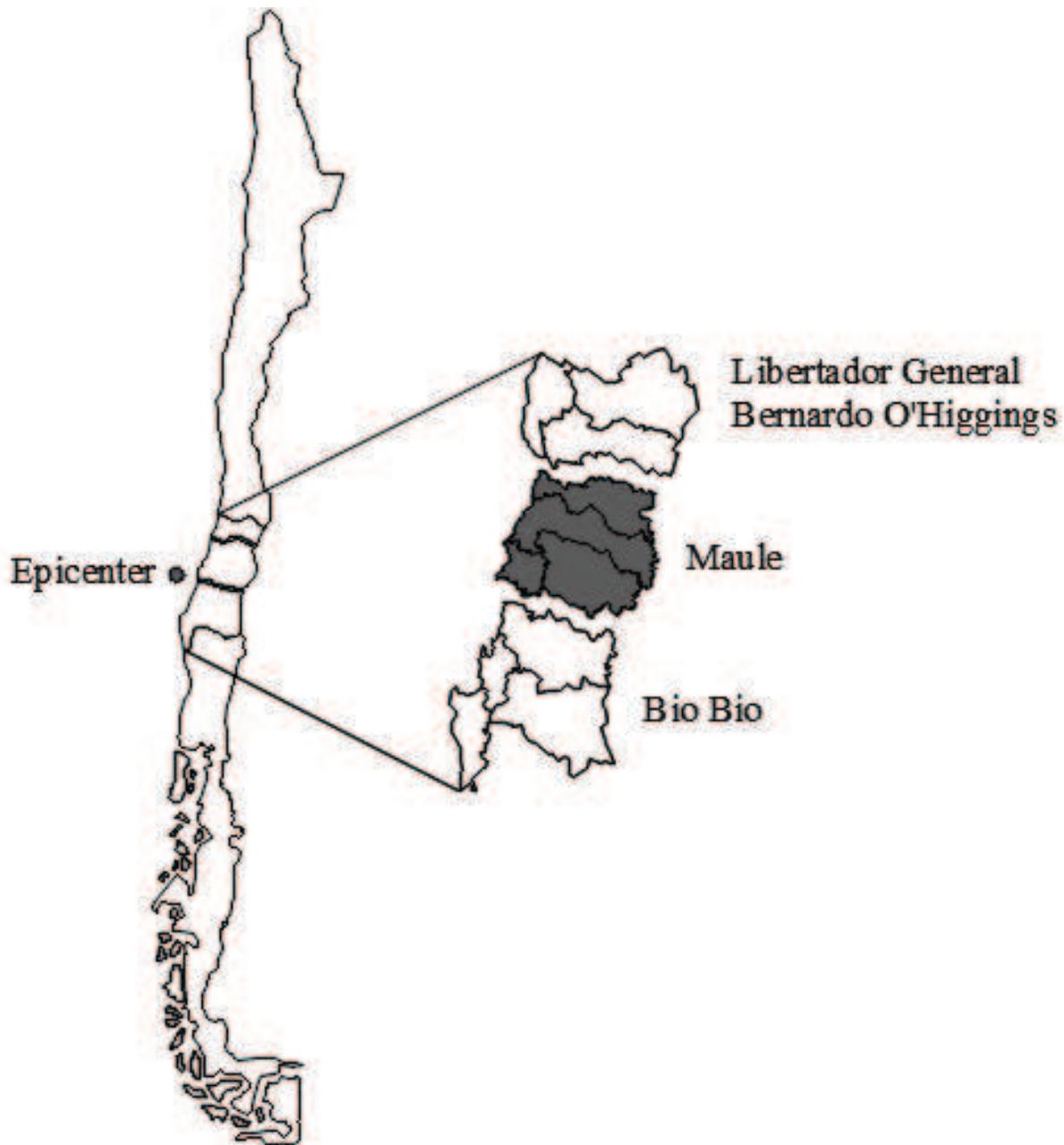


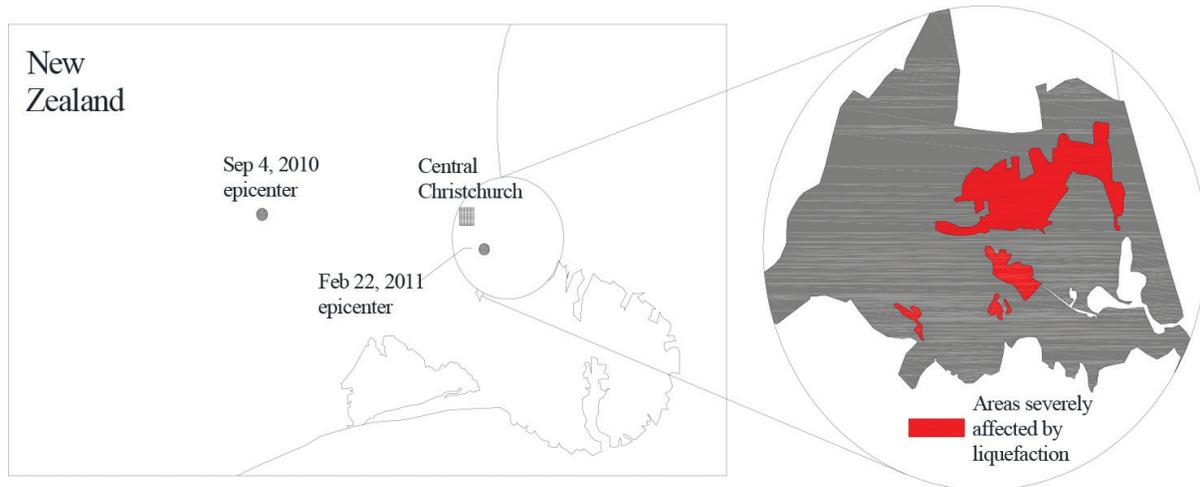
Figure 1. Areas affected and epicentre location, Chile Earthquake 2010.

## 1.2. New Zealand Earthquake

Two earthquakes struck the New Zealand Island on September 4, 2010, and February 22, 2011. The first one,  $M_w = 7.1$ , is located in the Darfield area and the second,  $M_w = 6.3$ , in the Christchurch area. Extensive liquefaction, affecting primarily the residential areas and pipelines, was observed (Figure 2). The lifelines were severely compromised and recovery was a complex task given the continuous aftershocks and their large magnitudes.

During the Darfield earthquake, there were clear signs of liquefaction in the eastern part of Christchurch near the Avon River in Avonside, Dallington, New Brighton and Bexley. At that

time, the Central Business District (CBD) was not severely affected but later, during the February earthquake, the liquefaction extended to the southern part and caused even more damage leading to a 'flood' due to the large amounts of liquefied soil [7].



**Figure 2.** Areas affected and epicentre locations of the New Zealand Earthquakes.

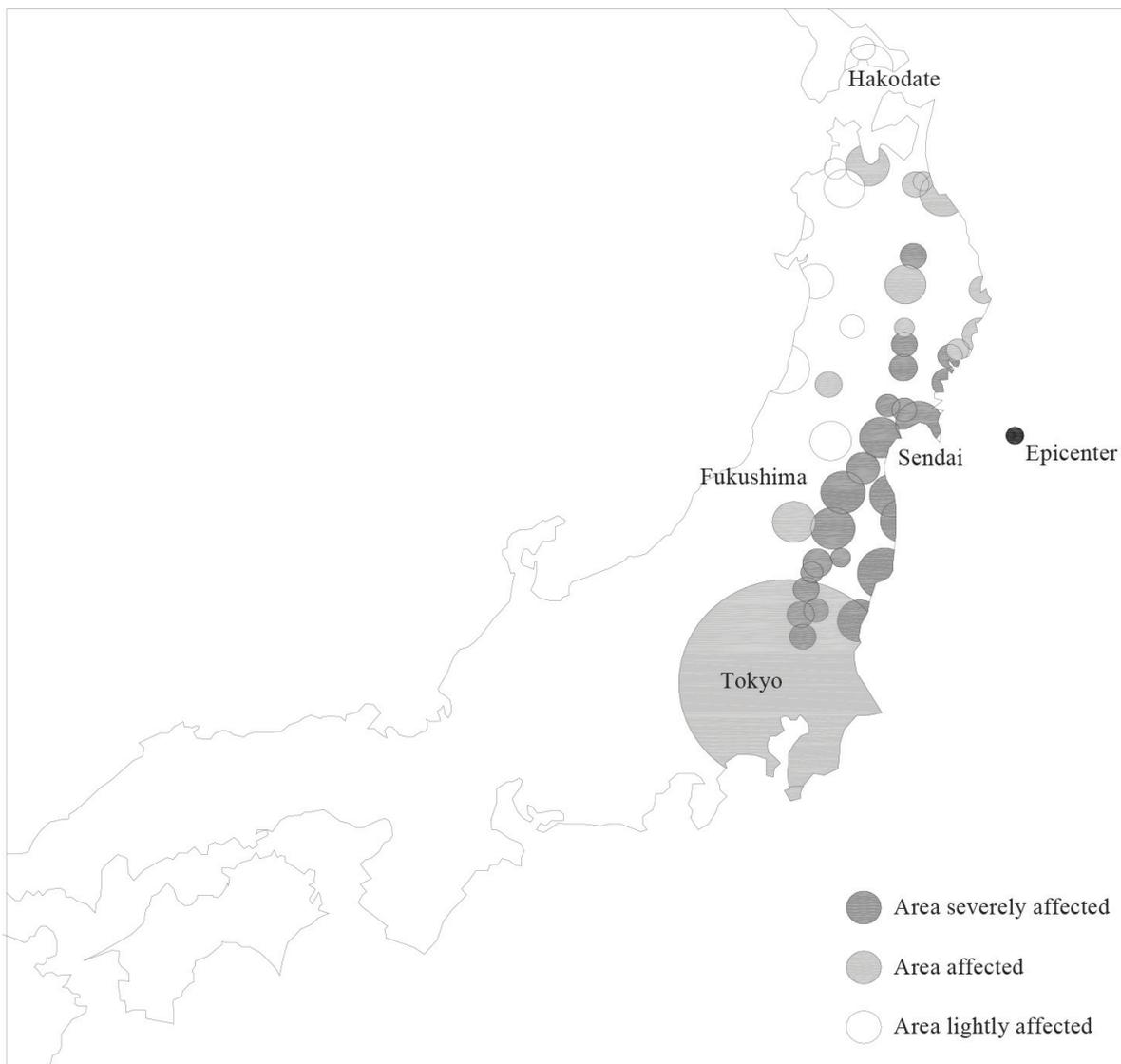
The City of Christchurch is located on the east coast of New Zealand, and it was mainly built on reclaimed swamp. Soils in the area are clean sands around the Avon River, along with loose silts and peat in the southeast part of the CBD. Boiled sand along the Avon River contained 5–20% of silty fines and the fines were low to non-plastic [8].

Damage in the residential area was extensive in the east and northeast of the CBD, where soils are mostly clean fine to medium sands with non-plastic silt. More than 15,000 residential properties and buildings were affected particularly due to lateral spreading and differential settlement. The evaluation of the liquefaction potential of soils containing non-plastic fines is of major interest in the prevention of future damage in the city. Some studies have been conducted on the effect of fines content (FC) in the sandy soils of the surroundings in this area, finding a more contractive behaviour with the addition of fines when density measures as void ratio or relative density are used (e.g., [8, 9]).

Some other remarkable characteristics in these events are the cumulative effects of these strong earthquakes and repeated liquefaction. In different areas, it was observed that sites liquefied in previous earthquakes re-liquefied and sometimes with more intensity than the previous times.

### 1.3. Japan Earthquake

The 2011 off the Pacific Coast of Tohoku Earthquake,  $M_w = 9.0$ , hit the east coast of Japan on March 3 and triggered a tsunami (**Figure 3**). The disaster caused a tremendous number of casualties and economic loss and became a watershed in earthquake and risk engineering due to the combined events which also included a nuclear accident in the Fukushima plant. This event, one of the five most powerful earthquakes in the world since 1900, was followed by two aftershocks inducing additional damage of  $M_w = 7.4$  and  $M_w = 7.7$ , 15 and 30 min after the first event.



**Figure 3.** Areas affected and epicentre location of the 2011 Great East of Japan Earthquake.

Several reports (e.g., Refs. [10–15]) showed that liquefaction occurred along the coast of Japan, although most of the traces were erased by the tsunami in the northern part. In spite of being more than 200 km away from the earthquake fault, the Kanto area was significantly affected, especially the Tokyo Bay, where there is a large extension of reclaimed land. Severe liquefaction-induced damage was observed in Tokyo Bay, where reclamation of the coastline started around 1600, from Sumida to Yokohama and expanded to Kanagawa and Chiba Prefectures around the 1950s. These areas were affected before by other seismic events. On December 17, 1987, a  $M_w = 6.7$  Earthquake hit eastern Chiba Prefecture (Chibaken Toho-oki Earthquake) causing liquefaction in many areas of the city. Therefore, some zones that experienced liquefaction in 1987 liquefied again in 2011.

Urayasu City, a zone of reclaimed land, was built in three different stages; the first reclamation was done before 1945 and later had extensions in 1948, 1968, 1971, 1975, 1980 and 2009. The reclamation work was done by the hydraulic method, consisting in dredging the shallow seabed and transporting the soil hydraulically by pipes and having the soil sedimented under

water. There was more damage in Ichikawa City at the east of Urayasu, in Funabashi City and Makuhari City.

On March 2011, boiled sand was observed in the reclaimed cities of Urayasu, Ichikawa, Narashino, Odaiba, Shinonome, Tatsumi, Toyosu, Seishin, Yokohama, Kawasaki, Kizarasu and Chiba [15]. However, the most devastating effects were found on lands reclaimed after the 1970s where differential settlement and lateral displacement affected Residential areas, roads, sea walls, pipelines and other structures. Amid those areas, Urayasu City was the most affected, where more than 9000 private properties had detriment.

As stated in the previous sections, there were distinctive features in the liquefaction events; however, one of the coincidences is the presence of non-plastic fines in the sand (also considering the tailings) and the increased susceptibility to liquefaction. This topic has been debated since it is believed in the current practice that the addition of fines will decrease the liquefaction potential. Other observed phenomena were repeated liquefaction and ageing effects and will be discussed in the following sections.

## 2. Liquefaction of sand containing non-plastic fines

### 2.1. Previous cases of liquefaction of sand containing non-plastic fines

On October 17, 1989, the San Andreas fault in California ruptured over a length of approximately 45 km and generated a  $M_w = 6.9$  earthquake. In the San Francisco Bay Area, there were hydraulic fills that had 3–9 m of loose, silty sand. Liquefaction-induced damage in building, infrastructure and pipelines was found mostly in the South Market area, Mission Creek and Mission District. One of the most representatives proves of liquefaction during the 1989 Loma Prieta Earthquake was the extreme damage of the Valencia Street Hotel. Most of these fills were placed during the 1900s and were composed of sand that was deposited in hydraulic suspension and allowed to settle freely [16]. Sand dredged from San Francisco Bay contained fines compared to clean Dune sand that was also present as fill material. It was reported that sand with fines had lower values of density and had more tendency to liquefy than clean sand [17].

In a study conducted by Rollings and McHood [18], the liquefaction-induced settlement in Marina District was computed and compared to the measured values. They used a correction for fines adjusting the volumetric-strain curves as pointed out by Seed et al. [19] instead of modifying the SPT  $N$ -value. Their results had a difference to the measured value of about a factor of 2, which led them to conclude that more studies should be done on defining the effect of fines on the correlations.

Another example of liquefaction on silty ground was observed during the Taiwan Earthquake. On September 21, 1999, the mountainous village of Chi-Chi was the epicentre of a  $M_w = 7.6$  earthquake causing extensive liquefaction damage in foundations, embankments, riversides, retaining walls, and so on. The counties affected, Yunlin, Zhonghua, Nantou and Taichung, are in Central Taiwan where soils were mostly compressible sands with large amounts of low to medium plastic fines. These soils originated from the process of weathering and abrading of shales, slates and sandstones from the central mountains; at some spots,

there are layers of very loose sand susceptible to liquefaction, their fines content ranges from 10 to 50% and some of these layers are capped by thick layers of clay material [20].

Back analyses performed on the liquefaction potential of the soil showed discrepancies between the results using simplified methods and the actual observed response. For instance, [21] evaluated the methods by Refs. [22] and [23] to observe the adequacy of these procedures to be applied on the soil conditions in Central Taiwan. They found that one of the major differences is the correction factor used for fines content, while the factor of safety computed for the liquefied area was similar in both methods for fines content less than 35%. Tokimatsu and Yoshimi's correction for fines caused an overestimation and Seed's, an underestimation for the real correction in their study. Ni and Fan suggested correction of fines for the simplified methods they discussed.

Similarly, Juang et al. [20] proposed a model based on artificial neural network of limit-state data that resulted in more accuracy for considering more fines than 35%, than the method by Youd and Idriss [24]. The extensive economic loss during this earthquake also enforced the development of new methods for sampling, testing and evaluating the liquefaction potential of sands containing large amounts of fines.

## 2.2. Current treatment of soils containing fines

In the late 1970s, researchers as Seed and Tokimatsu developed different procedures for evaluating the liquefaction potential. Observations of liquefied sites, where it was observed that liquefaction also occurred in deposits formed by different materials as gravel and silt, were added to various databases and used for guidelines. Currently, some of the simplified procedures used worldwide are those proposed by Seed et al. [22], the Japan Road Association (1990 and 1996), [23], the Chinese Building Code (1989) or the Arias intensity method [25].

Case studies [19, 26] have shown that the existence of fines in sands increases the liquefaction resistance at the same level of standard penetration test,  $N$ -value.

The first approach to liquefaction of sands containing fines was taken on by Wang [27] who compiled a series of liquefaction events in different soils to estimate the liquefaction potential of silty soils according to its fines content, FC, plasticity index, PI, water content,  $w_c$ , and liquid limit, LL. Later, Seed et al. [28] summarized Wang's findings into the three following conditions for soils vulnerable to liquefaction:

$FC < 15\%$  (per cent finer than 0.005 mm)

$LL < 35\%$

$w_c > 0.9 LL$

Seed et al. [28] compiled data of silty sand from liquefied sites and added them to a chart of cyclic stress ratio and modified penetration resistance. They concluded that the boundary between liquefiable and non-liquefiable soils is significantly higher for silty sands than that for clean sands. After the liquefaction events in Adapazarı during the Kocaeli Earthquake of 1999, Bray and Sancio [29] studied the limits proposed by the so-called Chinese criteria. They concluded that the use of FC for separating liquefiable from non-liquefiable soils should be avoided, redefined the relation between water content and liquid limit as  $w_c > 0.85 LL$  and stated that plasticity index, PI, was a good index of liquefaction susceptibility since soils with  $PI < 12$  can liquefy.

The presence of fines during liquefaction has caused divergent conclusions regarding its effects. While field test data of sites with fines have been added in charts for design (e.g., [19, 30]), there is no clear differentiation between plastic and non-plastic fines.

Robertson and Campanella [30] in their studies on cone penetration tests found that silty sands and silts cause a decrease in penetration resistance. According to this, soils with fines at the same penetration resistance have greater liquefaction resistance than clean sand.

Given the advantages of testing soil in controlled environments, laboratory testing has been a very recurrent choice when dealing with the influence of fines on the undrained behaviour of sands. While testing, it becomes necessary to keep one parameter constant to observe the effect of the variation of others. Some of the most common parameters to keep constant during comparison are overall void ratio or simply void ratio,  $e$ , and relative density,  $D_r$ , which are good measures of particle contact. When testing clean sand, it is easy to compare results while keeping constant both of them, which has made these parameters quite useful and widespread. For that reason, in experiments with silty sand, most researchers have employed them. However, there are different issues when testing sand containing fines, which have encouraged researchers to not only understand the limitations of void ratio and relative density but also develop different parameters for comparison, as explained in the following section.

Although gradation and mineralogy of sand as well as the amount of fines tested are key factors, the difference in the results obtained by several researchers might be explained by considering the concept of void ratio. While sand has no fines, voids are only occupied by water (in a saturated soil) and void ratio is an index of particle contact and force transmission. As a small amount of fines is added to the sand matrix, voids are occupied by water and fines, reducing global void ratio although there is no contribution of the fines to the intergranular force transmission. If fine content increases, it reaches a threshold point B (**Figure 4**) when fines fill all the voids. From such a point, fines start gradually influencing the mechanical behaviour, until sand grains are fully surrounded by them and do not make contact with each other anymore; then the force is totally supported by fines. It can be deduced that the concept of void ratio as an index of particle contact is not valid after the threshold point. In this regard, variations in void ratio have been used to be representative of the behaviour of silty sand, such as the intergranular contact index void ratio [31] and the equivalent void ratio [32] both shown in **Figure 5**. These parameters seem to solve the disjunctives concerning real particle contact. However, there are still uncertainties regarding the values that must be used when fine content is very high or regarding the parameter that reflects the fraction of fines participating in the force structure of the solid skeleton (b). Some researchers as Rahman and Lo [33] have shown formulas for estimating (b), but they require different assumptions and an iterative process.

Nevertheless, since it is important to be able to compare soils with different fines content at their natural state in ground, in this paper another standpoint is taken.

The use of density measures for comparison has made the laboratory research on liquefaction of silty sand ambiguous, given the restrictions of each parameter and the impossibility to keep them all constant at the same time.

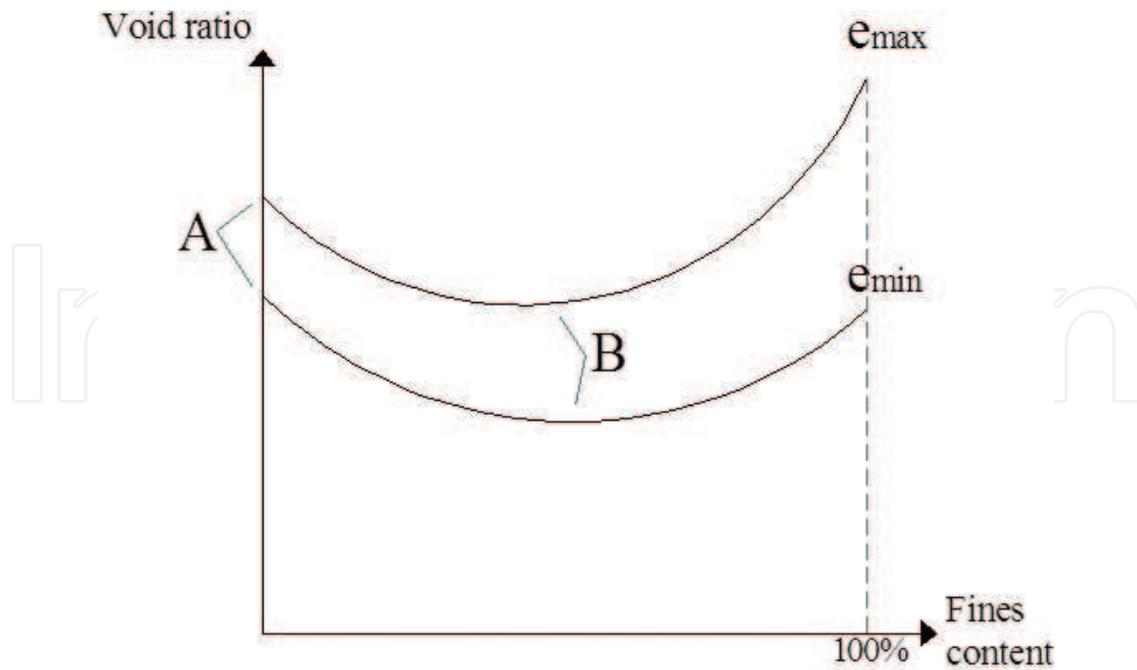


Figure 4. Theoretical variation of void ratio in binary packings with fines. After Ref. [34].

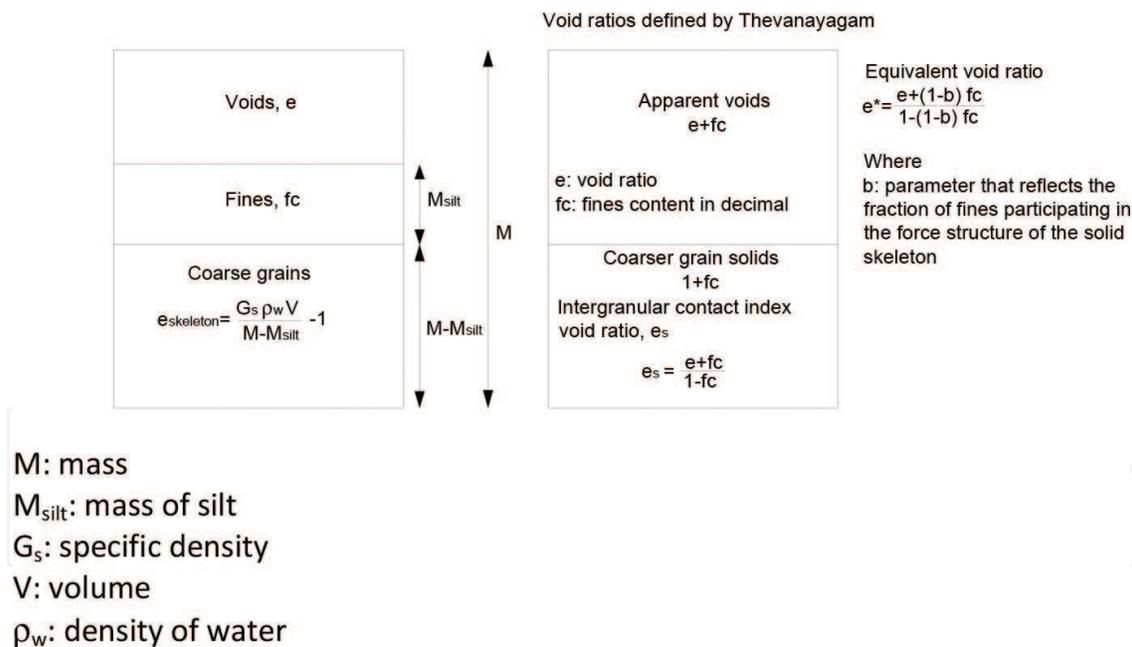


Figure 5. Definition of sand skeleton void ratio, intergranular contact index void ratio and equivalent void ratio.  $M$ : mass;  $M_{silt}$ : mass of silt;  $G_s$ : specific density;  $V$ : volume;  $\rho_w$ : density of water.

Lee and Fitton [34, 35] performed tests on alluvial sand and gravel deposits at El Monte, Los Angeles, CA. Grain particles were composed of quartz, feldspar and dark minerals; fines varied from 0 to 90%, the fines being a mixture of silt and clay. Samples were isotropically consolidated to 15 psi (100 kPa) and pulsating-loading triaxial tests were conducted at relative

densities of 50 and 75%. They found that very fine sands and silty sands showed the weakest response.

Iwasaki and Tatsuoka [36] performed tests with a resonant column apparatus with a hollow cylindrical specimen of sands with different gradations and fine content from 0 to 33%. While keeping constant void ratio, it was seen that sands other than clean sands had smaller shear moduli.

Shen et al. [37] conducted one of the first researches carried out on the effect of fines in the liquefaction potential. In their tests, they used a triaxial machine that allows for cone penetration tests on specimens with the same stress conditions as those of the static and cyclic triaxial tests. They used Ottawa sand and clayey silt with  $PI = 11$  and observed that, at the same sand skeleton void ratio, fines increase the liquefaction resistance.

These primal experiments on sand with fines provided some insight on the expected influence of fines, according to the parameter used for comparison for further research. For instance, when keeping constant void ratio it has been found that liquefaction resistance decreases as fines rise (e.g., [38–42]). If relative density is held constant used for comparison, liquefaction resistance grows with the addition of fines (e.g., [40, 42–44]). Some researchers as Kuerbis [43] found the sand skeleton void ratio, which assumes that the volume occupied by fines is part of the volume of voids, to be a more appropriate parameter because it seemed to be independent of fines content; yet, Polito and Martin [45] identified a growth in liquefaction resistance with fine content for Yatesville sand when maintaining constant sand skeleton void ratio.

Liquefaction resistance in silty sand has demanded the attention of many researchers throughout the years. When researchers compared the same parameter, they found similar conclusions. It is important to note that most researchers have focused only on fines content below 30%, which is usually the limit for using parameters as void ratio, relative density, sand skeleton void ratio or even equivalent void ratio.

### 2.3. Hollow torsional shear tests conducted on sand containing non-plastic silt

Torsional shear tests were conducted on boiled silty sand collected from Urayasu City after the 2011 Tohoku Earthquake in Japan, hereinafter called Urayasu sand. A typical grain size distribution is shown in **Figure 6**.

The variation in minimum and maximum void ratios with fines content is shown in **Figure 7**, and it can be seen that there is a “V-shape” in these curves, as pointed out by Lade et al. [34]. The lower minimum and maximum void ratios indicate the threshold value where the voids in the sand matrix are completely filled with fines, having the lower resistance condition at the bottom part of the curve (fine content between 30 and 40% for the maximum void ratio).

As stated before, as more fines are added, minimum and maximum void ratios vary, making it difficult to select void ratio or relative density as constant parameters for the evaluation of soil behaviour, since all parameters cannot be kept constant at the same time. To avoid this concern, constant energy for sample preparation is used in this study as the comparison

parameter for several fines contents to offer a different perspective in this matter. As pointed out by Lade and Yamamuro [44], any depositional process will produce different densities depending on the gradation of soil in nature, hence the use of the same compaction energy for sample preparation is by some means the reproduction of the same natural environment, although in a very naive way.

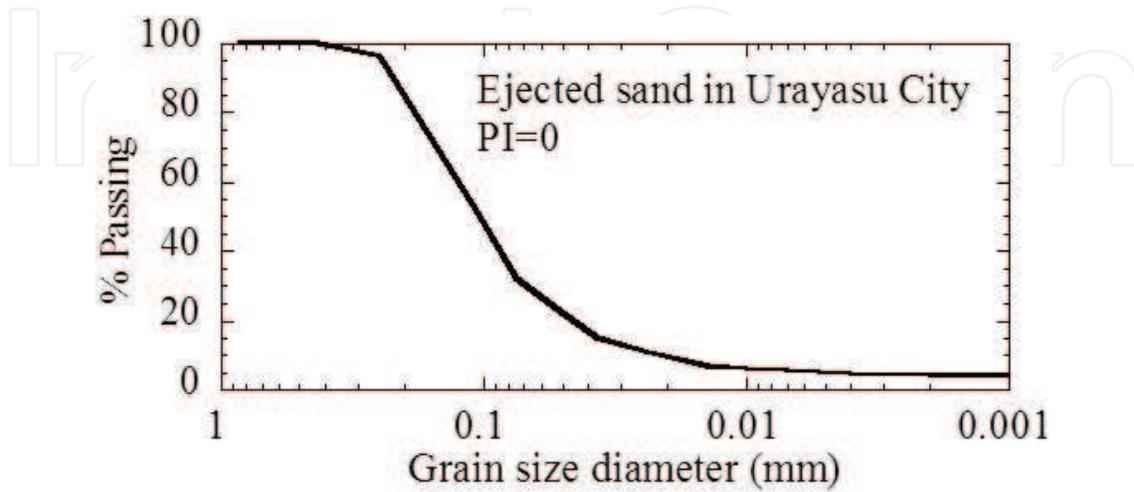


Figure 6. Grain size diameter of Urayasu sand.

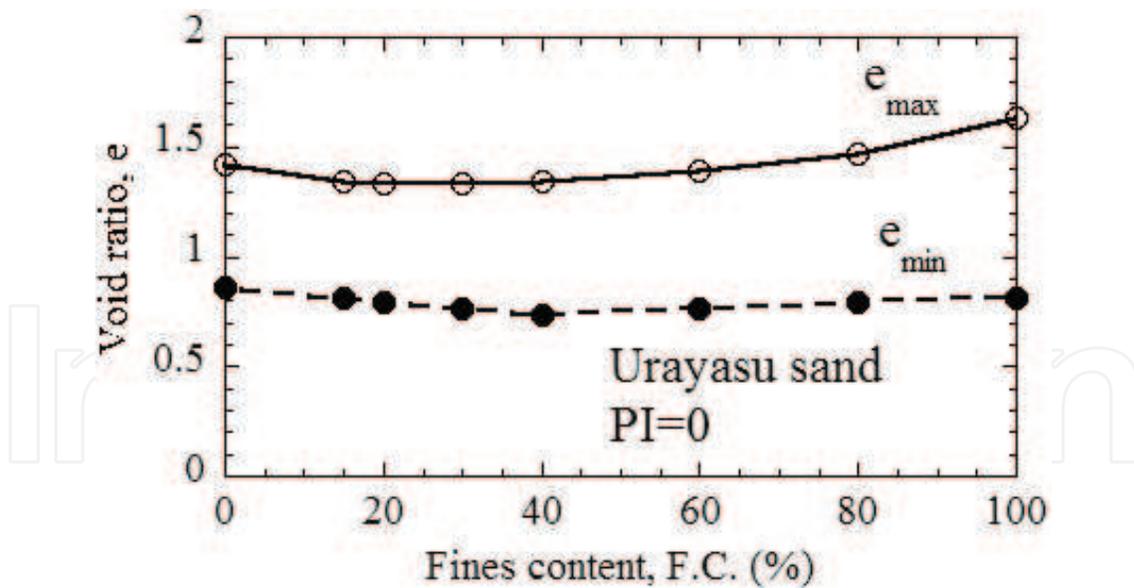


Figure 7. Minimum and maximum void ratio of Urayasu sand.

The device selected to carry out these tests was a hollow cylindrical torsional shear device that can subject a 190-mm height specimen (internal diameter of 60 mm and external diameter of 100

mm) to a combination of axial and torsional stresses, in addition to the fluid stresses inside and outside the cylindrical surfaces. Sand was sieved to separate fines from sand grains. After washing and drying, both sand and fines were thoroughly mixed together, varying the amount of fines from 0 to 80%. Dry pluviation was chosen for practical purposes since it does not overestimate the cyclic resistance [46], and some tests by Huang et al. [39] proved that dry deposition samples with fines content up to 30% shows uniformity. This procedure consists of pouring dry sand with a funnel with a fixed inner diameter and a constant height of fall, in this case 5 cm (AP-5 cm). The funnel should be turned slowly around the sample, first clockwise and then counter-clockwise to allow for even distribution of the grains; this practice was repeated keeping the same height of fall at all times, to use this energy of compaction as the parameter of comparison.

Saturation of Urayasu sand was completed using the double vacuum method described by Ampadu and Tatsuoka [47]. While keeping the specimen at a constant effective stress of 20 kPa, vacuum was incrementally applied to the inner and outer cells, as well as the interior of the specimen reaching -70 and -90 kPa, respectively. After 1 or 2 h of vacuuming, depending on the amount of fines and density, deaired water was flushed through the specimen allowing for full saturation. According to the fines content, this process could take up to 4 h. After this, backpressure saturation was used to dissolve any air remaining in the voids. The degree of saturation was measured with Skempton's  $B$ -value and all samples reached values of  $B \geq 0.96$ . Once satisfactory  $B$ -values were achieved, sand was isotropically consolidated to an effective confining stress  $\sigma'_{c0}$  of 100 kPa. During this stage, volumetric and axial strains were measured. After consolidation, cyclic shear tests were conducted with a strain rate of 0.12%/min. Several cyclic stress ratios ( $\tau/\sigma'_{c0}$ ) were chosen for samples with fines content varying from 0 to 80%, in order to define liquefaction curves.

### 3. Volumetric strain during consolidation

One important factor to understand the effect of different fines contents is the amount of volumetric strain during consolidation. Since the energy for preparation is the same, it is possible to compare the volumetric strain during consolidation for all samples (**Figure 8**). The graphs show that there are three groups of behaviour regarding the fines content; for AP-5 cm, the 0–20% samples seem to increase their volumetric strain as the fine content increases, while the 30–40% seems to decrease the volumetric strain as the fine content increases, as well as the 60–80% group. However, in this group, due to the amount of silt, the volumetric strain is larger.

Another key factor is the coefficient of volume compressibility,  $m_v$ , which is most likely inversely proportional to the strength of soil if Young modulus is also proportional to strength. In such case,  $m_v$  can be a laboratory parameter used for evaluating strength in the field. Considering the values in **Figure 8**, the coefficient of volume compressibility was computed, and it was found that from 0 to 20%, the value becomes larger, from 30 to 40%,  $m_v$  decreases, and it also decreases from 60 to 80%, although with larger values than the previous group.

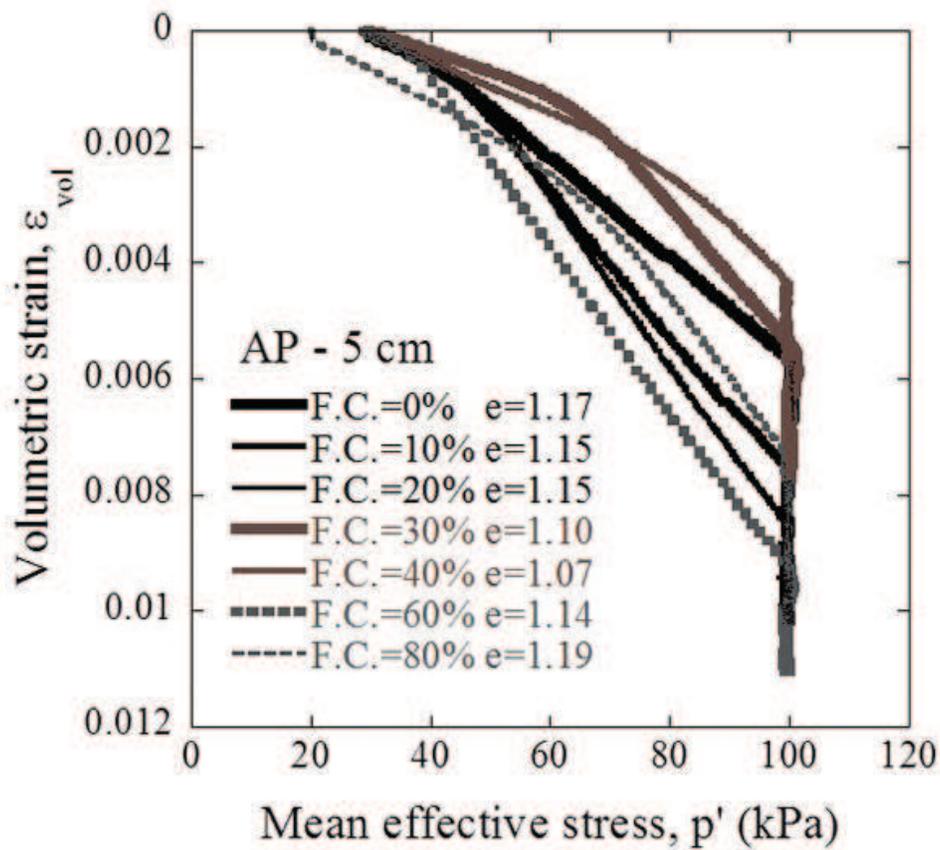


Figure 8. Volumetric strain versus mean effective stress.

#### 4. Stress-strain curves and stress paths

Once the test programme was completed, stress-strain curves and stress paths were plotted. Results showed that there are three different behaviours, according to their relation to the threshold fines content. Below the limiting fines content, from 0 to 20% there is response dominated by the sand grains, from 30 to 50% there is a transition stage between sand and fines behaviour, then, above the threshold value, from 60 to 80%, the behaviour seems to be dominated by the contacts along the fines. Outcomes are described considering this perspective. **Figure 9** shows a comparison of the final cycle of liquefaction in which the strain amplitude is in the range of -9 to 10% for three samples with 0, 30 and 80% fines content. It can be seen that the reduction in the tangent shear modulus seems to be smaller as the fines content increases.

The corresponding stress paths are depicted in **Figure 10**, and it is noted that the 80% curve does not reach the zero-effective stress point as the other curves. It is also seen that there are very small differences between the 30 and 80% samples formed at different densities.

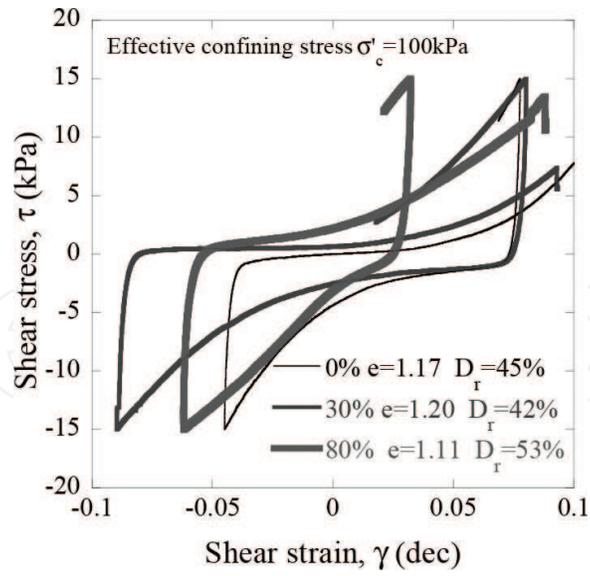


Figure 9. Stress-strain comparison of samples containing 0, 30 and 80% of fines.

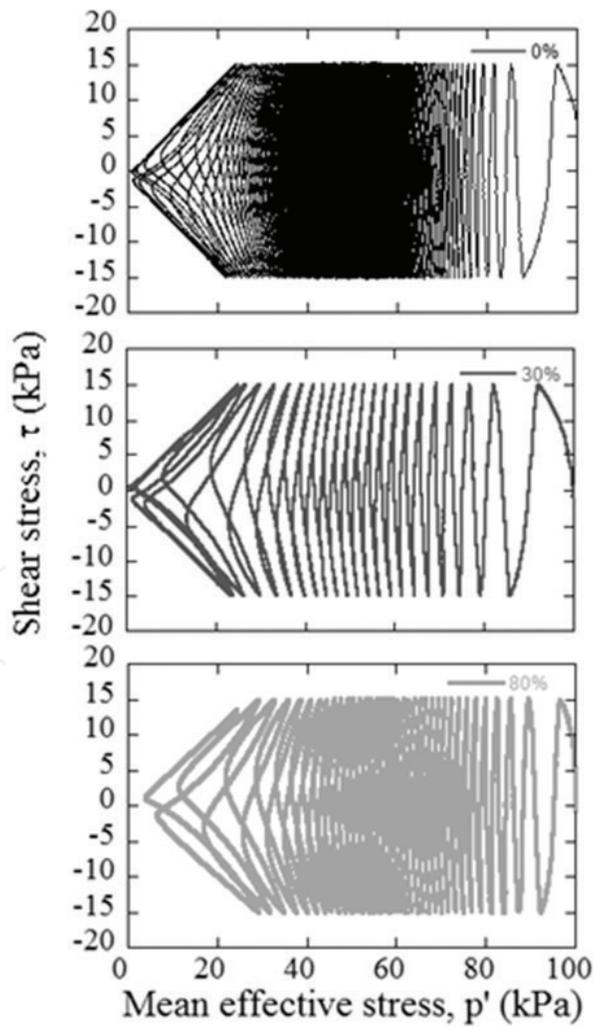
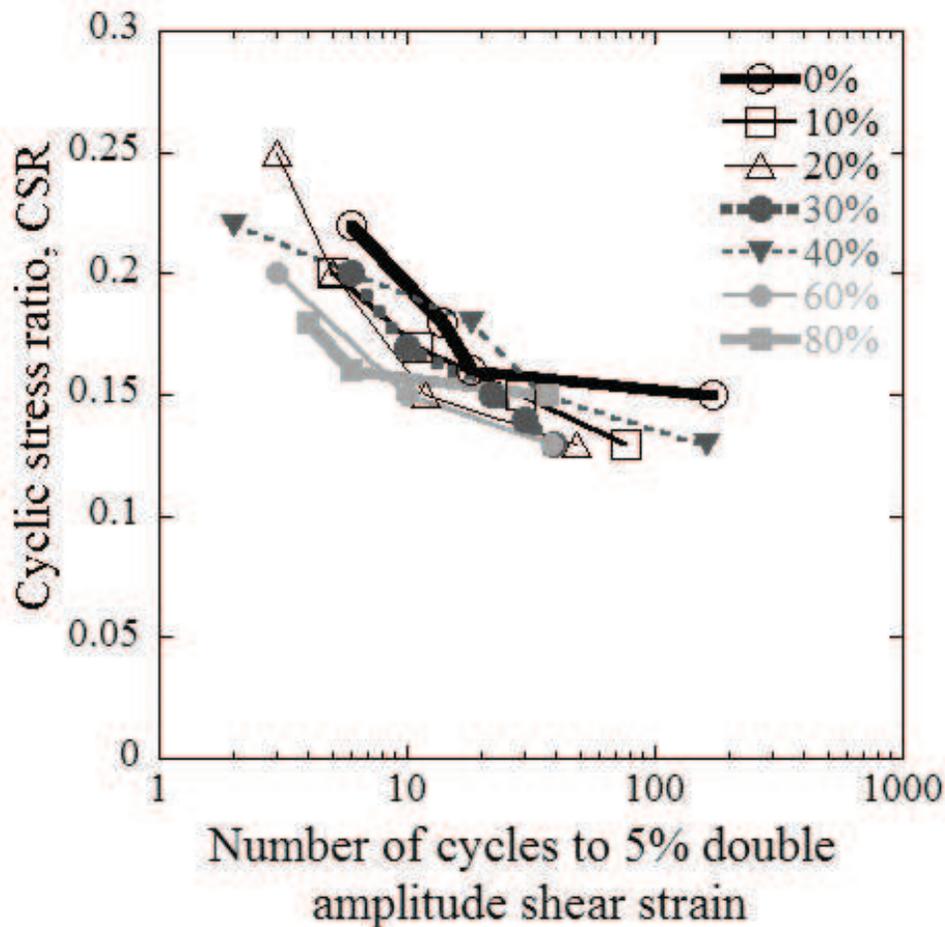


Figure 10. Stress-path comparison of samples containing 0, 30 and 80% of fines.

## 5. Results of cyclic shear tests

Two criteria were used for defining liquefaction, the generation of total excess pore-pressure ratio,  $r_u = 1$  and the 5% double amplitude of shear strain. Given the nature of loose samples, they yielded similar results for AP-5 cm. The curves shown in **Figure 11** were constructed using the criterion of 5% double amplitude of shear strain for liquefaction.



**Figure 11.** Liquefaction curves for samples containing 0–80% fines content.

From the series of tests, it can be seen that there are three noteworthy groups within the fines content, which are sand-like behaviour, intermediate behaviour and clay-like behaviour. It is important to remark that the clay-behaviour in this paper does not refer to clayey material but to clay-size material. In the same way, sand-like behaviour refers to the typical response of granular materials. It can be observed that the clean sand specimen has larger resistance than the samples that have fines.

There is a distinction from 0 to 20% of fines, where the resistance drops as the fines content increases; from 30 to 40%, where there is the threshold of the maximum amount of fines that can be fit in the sand matrix voids, there is a rise in resistance as fines content augment. Finally, there are the high contents of silt, from 60 to 80%, where the sand loses contact

between grains and each grain is surrounded by silt which controls the response of soil. In this group, there is an overall reduction in the resistance, compared to the first two groups, but the larger the fines content, the larger the resistance. The more resistant samples within their respective ranges of fines content are the 0, 40 and 80% specimens, which might indicate that arrange of fines results in the best resistance, independently of the amount of fines content.

## 6. Repeated liquefaction

There were several examples of repeated liquefaction in New Zealand and Japan. The large amount of sand ejected in the residential area of Kaiapoi in New Zealand showed that repeated liquefaction represented a significant issue. Similarly, liquefaction spots identified during the Tohoku Earthquake had liquefied more than three times during previous seismic events. It could be expected that liquefaction will actually increase the relative density of soil; however, that increment could be about 10%, not large enough to reduce the liquefaction potential [14]. The major problem with repeated liquefaction is the disruption caused for reconstruction after earthquakes, especially underground lifelines and foundations, given the cumulative damage that increases the impact.

Wakamatsu [48] presented a series of borehole data in Chiba Prefecture after the 1987 earthquake and compared it to the damage observed in 2011. They found that similar damage patterns and the sites where repeated liquefaction was observed consisted of alluvial deposits or artificially filled areas.

## 7. Ageing effect

The ageing mechanism has brought up several questions given that observations seem to disagree with existing procedures to estimate liquefaction resistance. Aged sand having similar SPT  $N$ -values to those of younger sand has exhibited greater liquefaction resistance. During the 2011 Great East Japan Earthquake, this phenomenon was observed again in the coast of Tokyo, where only soils reclaimed after 1960 liquefied. Kokusho et al. [49] conducted a series of experiments on sands containing different fines contents and found a correlation between cone resistance and liquefaction strength independent from relative density and fines contents, which is contrary to the current practice. Their specimens of Futtsu sand with cement to simulate geological age showed higher liquefaction strength compared to specimens without cement at the same cone resistance. Towhata et al. [50] compiled data from liquefied and unliquefied sites to compare the factor of safety for liquefaction,  $F_L$ , and the time of land reclamation. They observed that some soils with values of  $F_L$  lower than one but older than 1960 did not liquefy.

## 8. Future trends and conclusions

This chapter introduced the liquefaction-induced damage reported during recent earthquakes in Chile, New Zealand and Japan. One of the most significant issues was the liq-

uefaction potential of sands containing non-plastic fines. The current practice indicates that sand containing fines is generally more resistant to liquefaction than clean sand; however, the observations in tailing mines, reclaimed areas and alluvial deposits showed that the amount of silt had a significant influence.

The authors conducted a review of previous experimental research on liquefaction potential of silty sands. Torsional shear tests were conducted on silty sand from Urayasu City, varying the fines content from 0 to 80% to consider all ranges of behaviour. Three different responses were found and their characteristics were evaluated in terms of excess pore-pressure build-up, shear strain and cyclic resistance ratio. Keeping the same energy for sample preparation gives some useful insight into the behaviour of silty sand. It was found, by using this criterion, that the resistance of clean sand is always greater than that of sand mixed with fines. However, the behaviour of the silty sand depends more on their relation to the limiting fines content. When it is below this value, liquefaction resistance decreases with increasing fines content, around this value liquefaction resistance increases, and for high values of fines content, soil behaviour is dominated by the fines and liquefaction resistance increases as less sand grains are immersed into the sample.

Regarding repeated liquefaction, during the events in New Zealand and the earthquakes in Japan of 1987 and 2011, it was observed that the risk of liquefaction does not decrease after the first event given that soil does not gain enough resistance in the densification process.

As for the ageing effect, the mechanism was observed during these earthquakes, as well. Researchers conducted experiments to prove the benefit of geological ageing and they found that even at similar penetration resistances, older soils exhibited larger liquefaction resistance; this is of vital importance for areas of young deposits and mapping of liquefaction risk that might underestimate the resistance of aged deposits.

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