Earthquake-induced settlements in port facilities in Chile

Les tassements provoqués par les tremblements de terre dans les installation d'un port au Chili

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SYNOPSIS: A large earthquake took place on March 3, 1985 in the central region of Chile. The earthquake had a surface wave magnitude of M=7.8 on the Richter scale and the epicenter was located in the Pacific Ocean approximately 40 km from the port of San Antonio. The earthquake's intensity in the port was VIII on the modified Mercalli scale and the damage to port facilities was very extensive. Most spectacular were the liquefaction-related failures of portions of a seawall. Also, heavy settlements occurred in many backfill and yard areas. Simplified available procedures have been applied to evaluate liquefaction potential as well as to estimate settlements of sandy deposits using the SPT N-values and the earthquake parameters, and, in several locations, the results show good correlation to actual observations. The authors have also studied the influence of local soil conditions on seismic response and earthquake-induced settlement mechanisms in different sites of the port.

INTRODUCTION

The Chilean earthquake of March 3, 1985, became one of the most significant, well recorded, and major earthquakes to date. The main earthquake occurred at 19:47 hours local time. Figure 1 (Connor, 1985) shows the general area affected by the earthquake, which included San Antonio, Valparaiso, the metropolitan region and Santiago, and many surrounding cities. Significant ground motions were recorded for a relatively long duration. Seismological records indicate that the earthquake actually consisted of two main shocks occurring within 11 seconds of each other as shown in Figure 1, with surface wave magnitude of 5.2 and 7.8, respectively.

FIGURE 1. THE GENERAL AREA OF THE EARTHQUAKE (AFTER CONNOR, 1985)

Peak ground acceleration (PGA) values recorded by strong motion accelerometers of the Chilean instrumentation network in the area most affected by the earthquake were reported by Saragoni (1986). The maximum recorded PGA value was 0.67 g for the N10E component at Llolleo.

An earthquake intensity of VII on the modified Marcelli scale was generally applicable in the coastal area between the cities of Valparaiso and San Antonio (near the central zone of the energy release). However, in the city and port of San Antonio intensity was rated as VIII.

The geotechnical effects of the earthquake were of great interest to the profession, and are still subject to ongoing research. A large area was affected by strong ground shaking for a relatively long duration and geotechnical failures were observed in many areas as described by Wyllie et al. (1986), Ortigosa (1986), and others. The port of San Antonio, which suffered significant damage during the earthquake, is of particular interest. The liquefaction-related failure of portions of a seawall, substantial seaward movements of anchored sheetpile seawall, and large settlements in the backfill areas of these walls are the focus of this paper and discussed in detail as follows.

THE PORT OF SAN ANTONIO

The port of San Antonio is one of the two major ports that function as a gateway of the Santiago metropolitan area, and therefore the activities of this port significantly affect the Chilean economy. The extensive damage to the port from this earthquake has caused significant problems in cargo handling capacity and port operations. The general port layout is shown in Figure 2, and typical cross sections of sites 1 and 3 are shown in Figures 3 and 4, respectively.

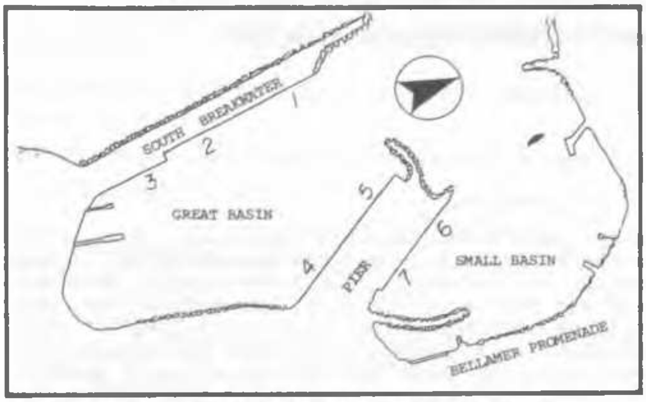

FIGURE 2. PORT OF SAN ANTONIO AND SITE LOCATIONS

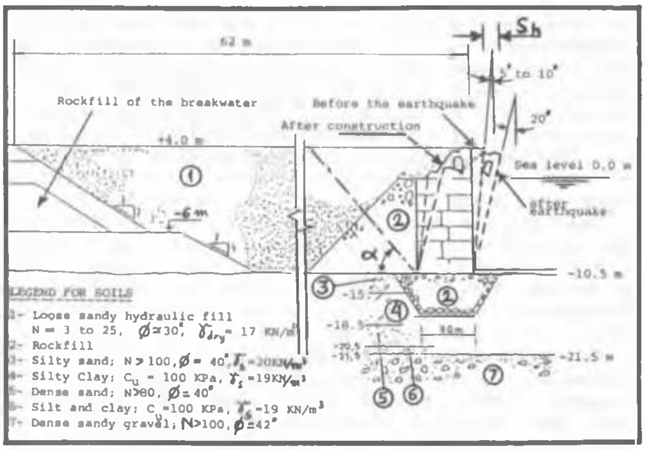

FIGURE 3. TYPICAL CROSS SECTION OF SITE 1.

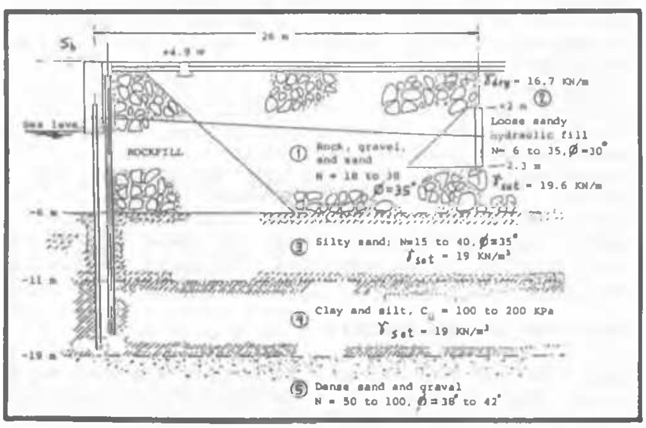

FIGURE 4. TYPICAL CROSS SECTION OF SITE 3.

Soil data included in this paper are based on a large number of soil reports summarized in Development Program of Ports of the Vth Region and Feasibility, 1st Stage (1988), Ortigosa (1986), and Ortigosa et al. (1986).

Unfortunately no instruments were installed at the port and therefore no ground acceleration records are available. However the ground station located at Llolleo, less than 4 km south of the port, recorded PGA values of 0.426 g and 0.669 g for the S80E and N10E components, respectively. It is estimated that the base rock depth for both the Llolleo station and the south breakwater at the port is approximately 100 m. Therefore, it was assumed (consistent with the literature) that the ground motion in the port is similar to the motion recorded at Llolleo.

The main damage from the earthquake occurred at sites 1 and 2. A typical cross section of site 2 is similar to Site 1, as shown in Figure 3, except that the seawall in site 2 collapsed into the sea. The sheetpile wall in site 3 suffered severe seaward tilt and copeline deformations of up to 1 m, and severe settlements were reported at the apron and backyard areas of sites 1, 2, and 3 as discussed in more detail.

LIQUEFACTION ANALYSIS

The hydraulic fill between the gravity seawall and the rockfill of the breakwater in sites 1 and 2 consists of loose, sandy soils. Similar hydraulic fill is found beyond the anchors of the sheetpile bulkhead in site 3 (Figure 4). In some locations these sands included occasional gravel. The D_{50} values range between 0.26 mm for the clean sands to 0.12 mm for the silty sands, which generally contain no more than 20% fines of low plasticity silts.

Low SPT N-values were recorded in many borings performed in this sandy fill area before and after the earthquake, indicating low relative densities and high liquefaction potential. The failure of the seawall in sites 1 and 2 indicates increased lateral pressure against the wall which occurred, most probably, caused by the liquefaction of the sandy backfill. The loss of strength of the backfill material is the result of high water pressure development due to shear strains induced by the earthquake. A large sand volcano was observed in the backfill debris behind the collapsed wall (Wyllie et al., 1986). This volcano is typical when a sand layer liquefies below denser surface layers as is the case near the seawall where the top 4 m (above the sea level) consists of relatively well compacted sandy soils with little fines and some content of gravel and rock. The backfill immediately adjacent to the seawall apparently consists of rockfill as indicated in Figure 3. In spite of the fact that only one sand volcano was observed, it is believed that the liquefaction of the sandy hydraulic fill was extensive.

In site 3 the soil between the rockfill immediately adjacent to the sheetpile wall and the anchors consists of sandy gravel and rocks from the surface to approximately 6 m below sea level (Figure 4). It is unlikely that liquefaction occurred in this layer, which is generally well compacted. However, partial liquefaction may have occurred in the underlying silty sand layer, which extends between 6 and 11 m below sea level. Liquefaction may have been more extensive in the hydraulic fill between the anchors and the breakwater. This fill consists of loose, sandy soils similar to sites 1 and 2. Liquefaction of the fill near the anchors may have caused anchor displacements, which contributed to the large seaward deformations of the sheetpile wall.

Liquefaction analysis was conducted based on the procedure outlined by Seed et al. (1983, 1984) and the results are presented in Figure 5. The critical envelope shown in Figure 5 is a convenient method for evaluating liquefaction potential based SPT N-value for a large number of borings in leveled ground in which the water table and soil conditions are similar, such as the conditions at sites 1, 2, and 3. The critical envelope was determined by the cyclic stress ratio induced by the earthquake. The overburden pressures were calculated based on a dry unit weight of 16.7 KN/m³ and a saturated unit weight of 19.6 KN/m³, which is typical for these loose, silty sands and which corresponds to a relative density of approximately 50%. Based on boring data, an average water table was assumed at 4 m below ground level.

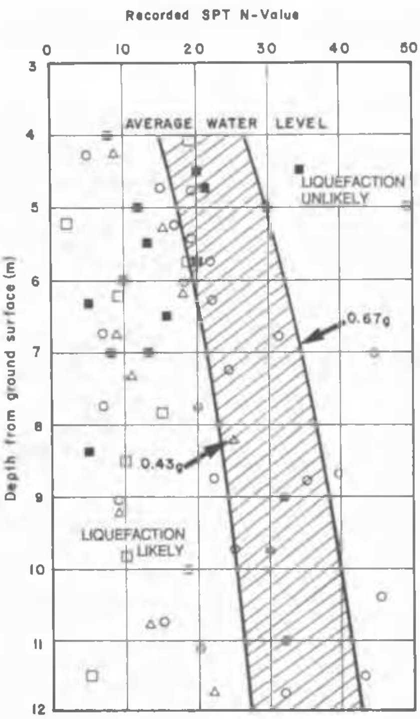

FIGURE 5 - CRITICAL LIQUEFACTION RANGE FOR SANDY SOILS IN TERMS OF SPT N-VALUE vs. DEPTH

LEGEND: CRITICAL RANGE BEFORE THE EARTHQUAKE OF MARCH 3, 1985

■ SITE I

AFTER THE EARTHQUAKE OF MARCH 3, 1985

□ SITE I △ SITE 2 ○ SITE 3

The limits of the critical range were calculated based on induced stress ratios for ground accelerations of 0.67 g and 0.43 g for the upper and lower limit curves, respectively, based on earthquake magnitude M=7.5. These stress ratios were then used to obtain the critical values of $(N_1)_{60}$, which is the normalized SPT N-value corrected for a 60% energy ratio. The upper limit curve (for a_{max} = 0.67 g) corresponds to $(N_1)_{60}$ values obtained from the correlation presented by Seed et al. (1983) for sands with $D_{50} > 0.25$ mm, and therefore represents the most severe condition for these sands. The lower limit curve (for a_{max} = 0.43 g) corresponds to the correlation presented by Seed et al. (1984) for silty sands with 15% fines. Generally, SPT in Chile is considered to have an energy ratio of 60% and therefore, the final step to determine the critical range boundaries in terms of recorded SPT N-value was based on C_n, which is a correction coefficient proposed by Seed et al. (1983) to normalize N-values for an overburden pressure of 1 tsf (106 KPa).

As shown in Figure 5, boring data from sites 1 and 2 indicate that liquefaction of the sandy hydraulic fill was probably very extensive during this earthquake. The concrete slabs in the apron and backyard areas at these sites suffered significant settlements as a result of liquefaction. Many of the data points from site 3 fall within the critical range in Figure 5. It is possible that these sands, which have higher fines content, did not liquefy, especially in the zone between the sheetpile bulkhead and the anchors. Some of these data points, from depths greater than 10 m below ground surface, are probably natural soil deposits that are less likely to liquefy. On the other hand, N values from borings in the hydraulic fill between the anchors and the breakwater (which consists of loose, clean sands) indicate that these sands liquefied during the earthquake.

EARTHQUAKE-INDUCED SETTLEMENTS

Settlements in apron and yard areas of sites 1, 2, and 3 were severe. Several mechanisms were involved in the settlements: vertical settlements, which were the result of the horizontal soil deformations in the failure zone of the fill adjacent to the failed seawall; earthquake-induced vertical settlement of the dry, sandy deposits in the top 4 m of the fill areas; liquefaction-induced settlements of the saturated sandy deposits; and certain combinations of these mechanisms.

Settlement Due to Horizontal Deformations: As shown in Figure 3, the seawall at site 1 failed by severe tilting. Based on the literature, it is reasonable to assume that the wall had an average initial tilt of 7.5 degrees, and as a result of this earthquake the average wall tilt increased to 20 degrees when measured at the copeline. In order to estimate vertical settlement the following assumptions are made: the extent of the soil mass affected by the wall tilt is confined by the wedge shown in Figure 3 where α = 45 degrees (this assumption is consistent with Ortigosa et al. [1986]); and the soil mass in the wedge had no volume change. Based on these assumptions, the maximum vertical settlement equals the horizontal seaward deformation at the copeline S_h, therefore the calculated vertical settlement is 2.95 m. This result correlates well with the measured settlement of 2.98 m in the fill immediately adjacent to the failed seawall.

Vertical settlement near the sheetpile wall in site 3, shown in Figure 4, was calculated based on similar assumptions. The maximum seaward deformations of the copeline were between 70 to 100 cm where the sheetpile wall had approximately 5 degrees seaward bend. Maximum vertical settlement of 65 cm was measured in the adjacent apron area, which is slightly less than what is calculated based on the previously mentioned assumptions.

The design of site 4 is essentially identical to site 3. However, site 4 suffered very slight damage from the earthquake compared to the severe damages to site 3. Boring data from site 4 reveal that the backfill in the apron and yard area consists primarily of rock with some gravel and sand. This rock fill is superior to the fill found in site 3, and therefore the seismic performance of the sheetpile wall in site 4 was much better than in site 3.

Settlement of Dry Sand Deposits: Generally, at the top of the fill areas the dry soils from underneath the yard pavements to water level consist of relatively well-compacted sand and gravel. However, very low SPT N-values were recorded in several locations at which large settlements were observed. An analysis of the possible settlement of these loose, sandy deposits was conducted using the procedure proposed by Tokimatsu et al. (1987). The analysis is based on average properties for a 4 m layer of dry sand with recorded SPT N-value of 3 and dry unit weight of 16.7 KN/m^3, and on the cyclic strains induced by a M = 7.5 earthquake with a a_{max} = 0.67 g. It should be noted that the calculated values for the dynamic shear modulus at low strain level correlate well with values obtained from PS prospecting performed in these sites. The results indicate that settlement of up to 21 cm may have occurred at the location with loose dry sandy fill.

Liquefaction Settlements: Liquefaction settlements were calculated according to the procedure proposed by Tokimatsu et al. (1987). The calculations for sites 1, 2 and 3 were based on average SPT N-values for the saturated sandy soils at each location and on the induced shear strains evaluated for a M = 7.5 earthquake with a_{max} = 0.67g. Liquefaction settlements of 12, 18, and 15 cm were calculated for sites 1, 2, and 3 respectively.

Total Settlements: The earthquake-induced settlement at any location is obviously, some combination of the three mechanisms previously discussed. Good correlation is obtained between calculated and observed total settlements at several locations as shown in Table 1.

Table 1. Comparison of Calculated and Observed Settlements

Site	Vertical Settlements (cm)				
	Calculated				Observed Total
	Due to seawall tilt	Dry sand	Lique-faction	Total	
1	295	0	0	295	298
2	0	21	18	39	50
3	70-100	0	0	70-100	65
	0	21	12	33	30

SUMMARY AND CONCLUSIONS

This paper examines liquefaction and settlements induced by the Chilan earthquake of March 3, 1985 in the port of San Antonio. A simplified available method based on SPT N-values and the cyclic stress ratio induced by the earthquake was used to evaluate liquefaction potential of several sandy deposits, and generally the results show good correlation to the observations.

Available simplified methods are also used to estimate settlements of sandy deposits; the cyclic stress ratio and maximum shear strain induced by the earthquake motion in saturated sands and the cyclic strains induced in dry sands, together with the SPT-N values and the earthquake magnitude are the parameters used for these estimates; and, in several locations settlements are also correlated to horizontal deformations of seawalls. Generally, the estimated results show good correlation with settlements observed during the earthquake.

The results demonstrate that these simplified methods may be useful to estimate liquefaction potential and earthquake-induced settlements, provided that local site conditions are taken into consideration.

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