

SPSI ANALYSIS OF CEMENT SILOS EXPOSED TO LIQUEFACTION SPSI ANALIZA CEMENTNOG SILOSA IZLOŽENOG LIKVIFAKCIJI

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Keywords

- vertical pile interaction
- liquefaction
- lateral spreading
- pile failure/fracture mechanism
- pore water pressure (EPWP) ratio

Abstract

The paper presents some simplified SPSI methods, as well as static analysis and the effect of liquefaction. During the Hokkaido Nansei-Oki earthquake with a magnitude of 7.8, in Japan in 1993, liquefaction occurred on the filled soil in Hakodate Port. Of the 2 silos, one larger with a mass capacity of 5,000 t, and the other in the immediate vicinity, with a mass capacity of 2,500 t, the smaller experienced significant settlement and rotation. Due to local lateral spreading, 1/20 tilt and 90 cm differential settlement occurred. Cement silo with a mass capacity of 2,500 t was supported on 64 pre-stressed concrete piles. All piles experienced fracture. The damaged silo superstructure was urgently demolished, so as not to damage neighbouring buildings. Mori and Numata performed detailed analyses of piles of this silo, after demolition, including displacement, acceleration, shear stress, shear strain and excess pore water pressure ratio. The same authors also provide models of interaction with the spring system, such as Gazetas, Mylonakis, Tazoh, Novak, Nogami, Penzien, Wilson.

INTRODUCTION

Liquefaction occurs during strong earthquakes (at least 7 degrees on Richter scale) when insufficiently compacted sandy soil saturated with water, behaves like a heavy liquid and loses its load-bearing properties. The phenomenon of liquefaction and some of its effects has been known for centuries [7], but this phenomenon was only explained in the late 60s of XX century, more specifically after the earthquake in Alaska and Japan, Niigata 1964. The phenomenon of liquefaction itself consists of a whole set of effects [2, 3, 6-8, 10, 18] that do not all have to be manifested in a specific case. Some of these effects will be explained below.

Ripples in the ground

The movement of the particles of the soil skeleton, before and during liquefaction, is performed first at the level of granular skeleton, as the passage of seismic waves. During the passage of seismic waves, dilatations and changes in the stress field occur in the granular skeleton. Those waves through the soil skeleton are both longitudinal and transversal

Ključne reči

- interakcija vertikalnog šipa
- likvifikacija
- bočno širenje
- mehanizam otkaza/loma šipa
- odnos porenog pritiska vode (EPWP)

Izvod

U radu su predstavljene neke pojednostavljene TŠKI metode, kao i statička analiza i efekat likvifikacije. Tokom zemljotresa Hokaido Nansei-Oki, magnitude 7,8, u Japanu 1993. godine, došlo je do likvifikacije na nasutom tlu u luci Hakodate. Od 2 silosa, jednog većeg sa masenim kapacitetom od 5.000 t, i drugog u neposrednoj blizini, sa masenim kapacitetom od 2.500 t, manji je doživeo značajno sleganje i rotaciju. Zbog lokalnog bočnog širenja, došlo je do nagiba od 1/20 i diferencijalnog sleganja od 90 cm. Cementni silos sa masenim kapacitetom od 2.500 t bio je oslonjen na 64 prethodno napregnuta betonska šipa. Svi šipovi su doživeli lom. Oštećena gornja konstrukcija silosa je hitno srušena, kako se ne bi oštetile susedne zgrade. Mori i Numata su izvršili detaljne analize šipova ovog silosa, nakon rušenja, uključujući pomeranje, ubrzanje, smičući napon, smičuću deformaciju i odnos prekoračenja porenog pritiska vode. Isti autori takođe pružaju modele interakcije sa sistemom opruga, kao što su Gazetas, Milonakis, Tazoh, Novak, Nogami, Penzijen, Vilson.

and can be spatial and planar or polarised. Although they can be specifically understood as linear, in the case of a needle-shaped granule, which is longitudinally supported, in points.

When the oscillations of the soil granules collect in such a way that the characteristic oscillations of almost every granule overcome the contacts with the neighbouring granules, the granular skeleton collapses, and liquefaction occurs. The entire incoherent soil together with the water then behaves as a heavy liquid. This phenomenon resembles quicksand, but unlike that, here we have internal kinematics and dynamics, but the duration of liquefaction is time-limited. The movement of particles is mostly rotational and turbulent. Laminar movement of soil layers is also possible, but it is usually short-lived, and of course it is not a Newtonian fluid.

Through the soil, the load is transferred via the granular skeleton (solid phase) and pore water (liquid phase), we can ignore the gas phase during liquefaction. Total stress (TSA) consists of effective stress (ESA) and pore pressure (PWP). The granular skeleton transmits the effective stress, and in

water the stress acts like pore pressure. The pore pressure does not transfer the load directly but acts as a confinement, like hydrostatic pressure. It should be noted that transverse waves do not propagate through water.

Liquefaction testing

During the testing of soil where liquefaction occurred, previous soil data was initially scarce. That is to say, there were rarely any previous extensive soil tests, at locations where liquefaction occurred, but the properties were tested afterwards, when liquefaction had already occurred, i.e., on already disturbed soil. Detailed soil tests were rare in the 1960s, among other things, because they were expensive and not widely available. So, it was rarely possible to compare soil conditions before and after liquefaction. The dominant large-scale complex geo-mechanical tests were then in North

America and Japan. This is not so unusual because modern soil mechanics was formed in the 20th century.

Vibro-platforms and centrifuges

In the beginning, experimental laboratory tests on seismic effects were performed on vibro-platforms, often striving to make the scale of the model as close as possible to the real thing. Of course, when examining a building that has tens of floors, a scale of at least 1:10 must be applied, and the appropriate proportions of dynamic modulus of elasticity and stress must be used. But after the end of cold war, a lot of centrifuges remained, which until then were used for pilot training under multiple gravitational accelerations, and new application were found in the early 90s in seismic models. Therefore, the liquefaction testing on vibro-platforms is less often used, but later reduced models on centrifuges were widely applied.

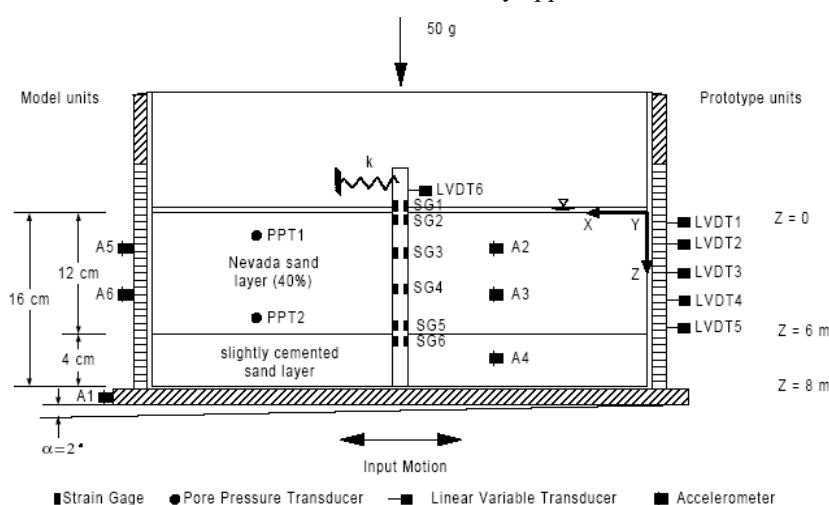


Figure 1. Centrifugal test /26/. Instrumentation: strain gage, pore pressure transducer, accelerometer, piezoceramic bender elements to measure shear wave velocities.

Quantity	Full Scale (Prototype)	Model at n g's
Linear Dimension, Displacement	1	$1/n$
Area	1	$1/n^2$
Volume	1	$1/n^3$
Stress	1	1
Strain	1	1
Force	1	$1/n^2$
Acceleration	1	n
Velocity	1	1
Time - In Dynamic Terms	1	$1/n$
- In Diffusion Cases	1	$1/n^2$
Frequency in Dynamic Problems	1	n

Figure 2. Scaling relation for centrifugal model, /12/.

Through experimental testing on centrifugal platforms, the instruments register the change of the entire set of necessary parameters, which enable the application of backward calculation methods to determine the forms of load intensity of the piles due to the occurrence and effect of liquefaction. Namely, the phenomenon of lateral spreading of the soil due to the effect of liquefaction is known. There are simplified expressions for calculating the amount of ground movement due to horizontal expansion. This displacement can amount to several cm, dm or metres. The same procedures can also be used to determine the degree of rotational constraint of the pile into the ground or elastic rotational spring.

Mix	Components	Percent-by-Weight	Sample Age
1	Sodium-Silicate	14%	3-days
	Sodium-Fluo-Silicate	2%	
	Bentonite	15%	
	Tap-water	69%	
2	Colloidal-Silica-Sol-(830)	82%	7-days
	Colloidal-Magnesium		
	Aluminum-Silicate	3%	
	NH ₄ OH	3%	
	Bentonite	12%	
3	Fumed-Silica	17%	
	Bentonite	13%	
	Tap-Water	70%	3-days

Figure 3. Soil mix models, for dynamic SPSI test, /11/.

Let's briefly analyse the centripetal force $F_c = mv^2/r$, we see that it is inversely proportional to the radius of the curve, in this case it is the length of the centrifuge arm added to the distance to the model of construction and soil. If the arm is approximately equal to the height of the basket in which the model of the soil and the construction is, we have curvature on the surface of the soil, that is, different accelerations on the surface of the soil, in the plane of rotation of the centrifuge. Often the soil surface is cemented. The basket usually hangs down, and then, with gradual acceleration,

rises into the plane of rotation. When we analyse the effects of forces on a particle, we have a vector triad: centrifugal, centripetal and gravitational forces. There is little difference between the mixed vector product of these orthos and the unit 'volume'. That's why the accelerations are on the order of 40g and more, underground water is not just water, the ground surface is often cemented (Fig. 3), and it is desirable that the arm is significantly longer than the height of the model, due to the 'curvature' effect.

Analysis models and the physics of the problem

There are different forms of load distribution and transfer along the pile casing. Also, in the classification of piles, the ratio of bearing capacity to the pile base and casing is used. A standing pile mainly transfers the load to the base, while the floating pile does so by friction on the casing. Due to overloading or changes in the condition of the soil during an earthquake, the bearing capacity is redistributed between pile and soil. The loss of shaft friction of pile, redistribution and reduction in base capacity of the pile are also possible.

Vesić is among the first to analyse the reduction of bearing capacity in the base of the pile /16/, where the calculation of the expansion of spherical and cylindrical cavities is presented. Vesić experimentally investigated dynamic bearing capacity of shallow footing on sand, /17/.

Inertial and kinematic interaction /9/

The calculation of the response of piles under seismic effects is done in two parts. The first is the kinematic part where the free movement is modified over the pile before it is transferred to the structure, and the second is inertial part in which the analysis of coupled vibrations /6/ is required for the group of piles. Several parameters are needed to analyse these calculation elements, /29-31/.

Inertial interaction originates mostly from inertial forces of the upper structure. Kinematic interaction is considered on a massless model. The depth to which certain influences extend is divided into a surface, intermediate and deep zone, expressed over the diameter of the pile. Near the surface zone, about 8 diameters of piles, dominate inertial effects. Intermediate zone influences both inertial and kinematic interaction. At the deep zone below 12 to 15 diameters from the surface, dominate kinematic effect.

There is also a division into the near field and far field, and it refers to the proximity of the ground in relation to piles, when seismic waves occur. It is also used at dynamic and kinematic interactions of piles.

Critical pile buckling force

Here, there are practically two borderline cases of embedding piles (bound) in the raft. One is almost total clamping, and the other is a free support-joint. The movable support is considered only in the case of moving the entire raft. Depending on the shape of the elastic line, the free length of buckling can be less than the actual length of the pile or even significantly greater than the double value. Therefore, for the correct determination of Euler's critical buckling force /4, 15/, it is essential to determine the boundary conditions of support or clamping/pinching. Table 1 shows the summary of critical force of all piles for the same

support conditions and buckling length, for both small and large capacity silos.

Table 1. Piles properties and critical axial force.

<i>n</i> pile	type	D_u/D_{inner} (cm)	<i>L</i> (m)	E_c (GPa)	E_I (kPa)	ΣN_{kr} (kN)
64	PC	40/25	13.8	30	31946.7	423845.25
122	PHC	35/20	13.8	50	32903.9	832163.40

$$N_{kr} = \pi^2 E_I / (kL)^2, k = 0.5, \text{ both ends rotationally restrained}$$

The total critical force of the larger silo is twice the total force of the smaller silo. It corresponds to the live load ratio, but it is necessary to take into account the total load. Here, however, the useful load should also consider the variant of an empty, half-full, and full large silo, because unlike a small silo, the amount of cement stored at the time of the earthquake is not known.

In Table 1, individual parameters, clamping/fixed boundary conditions, modulus of elasticity, active pile length, etc. can be varied. Thus, the influence of individual input values on the overall stability of the foundation-soil-structure system can be evaluated.

Conditions in the soil

To assess the state of the soil, it is necessary to analyse Table 2 and the cross-section through soil layers in Fig. 1. According to EC8 /30/, soil profiles of type D, (E), S1 and S2 require a special check for seismic effects and the occurrence of liquefaction. These layers have a speed of shear waves V_s , less than 180 or 100 m/s. Also, the number of blows per feet NSPT is less than 15. That is practically all soil layers, except for Ds1. Ds1 is of volcanic origin and resembles Roman cement.

Table 2. Cross section soil layer /22, 23/.

Soil layer	Depth (m)	SPT (N)	V_s (m/s)
embankment	0.8	-	-
Bk	6	2-4	94-98
As1	7	3-7	151
As2	2.8	5-13	338
Ac1	3	8-13	159
Ds1	>30	20-24	222

According to EC8 /26/, in the case of the large change of the stiffness of adjacent soil layers /28/, due to the effect of the earthquake, a plastic joint occurs near the border of these layers. This is already happening at the modulus ratio of layers 4. In layer As2, silty sand involving thin stiff sand sub-layer, there is a big change in stiffness that is visible by the number of blows *N*, SPT test, in Fig. 4, and that would be mark D for a new group of pile failure shapes. Type A shear failure, type B bending failure, type C compression failure and lateral shift.

Cyclic stress ratio (CSR) and cyclic resistance ratio (CRR)

For the evaluation of the resistance to liquefaction, the following ratio is important $FS_L = SRR/CSR$, where FS_L is the factor of safety against liquefaction. Cyclic stress ratio is the seismic demand on a soil layer, expressed in terms of CSR; and the capacity of the soil to resist liquefaction, expressed in terms of CRR, /24/.

$$CSR = (\tau_{av} / \sigma'_{v0}) = 0.65 \frac{a_{\max}}{g} \frac{\sigma_{v0}}{\sigma'_{v0}} r_d, \quad (1)$$

where: $a_{\max}$ is peak horizontal acceleration at the ground surface generated by the earthquake (discussed later); g is gravitational acceleration; σ_{v0} and σ'_{v0} are total and effective vertical overburden stresses, respectively; and r_d is the stress reduction coefficient,

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m}, \quad (2)$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} \leq z \leq 23 \text{ m}, \quad (3)$$

where: z is depth below ground surface in metres.

Field tests commonly used to assess liquefaction resistance, including the standard penetration test SPT, are the cone penetration test CPT, shear wave velocity V_s measurements, and BPT. Since there are different SPT apparatuses, the N_{60} is introduced to standardise different instruments,

$$(N_1)_{60CS} = \alpha + \beta(N_1)_{60} = 5 + 1.2(N_1)_{60}, \quad (4)$$

where: α and β are determined from relationships, Eq.(5), for fine contents $FC \geq 35\%$, $\alpha = 5.0$ and $\beta = 1.2$,

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S, \quad (5)$$

where: N_m is measured standard penetration resistance; C_N is factor to normalise N_m to a common reference effective overburden stress; C_E is correction for hammer energy ratio ER; C_B is correction factor for borehole diameter; C_R is correction factor for rod length; and C_S is correction for samplers with or without liners. Similar to [1],

$$C_N = 2.2 / (1.2 + \sigma'_{v0} / P_a), \quad (6)$$

where: C_N normalises N_m to an effective overburden pressure P_a of approximately 100 kPa (1 atm). C_N should not exceed a value of 1.7.

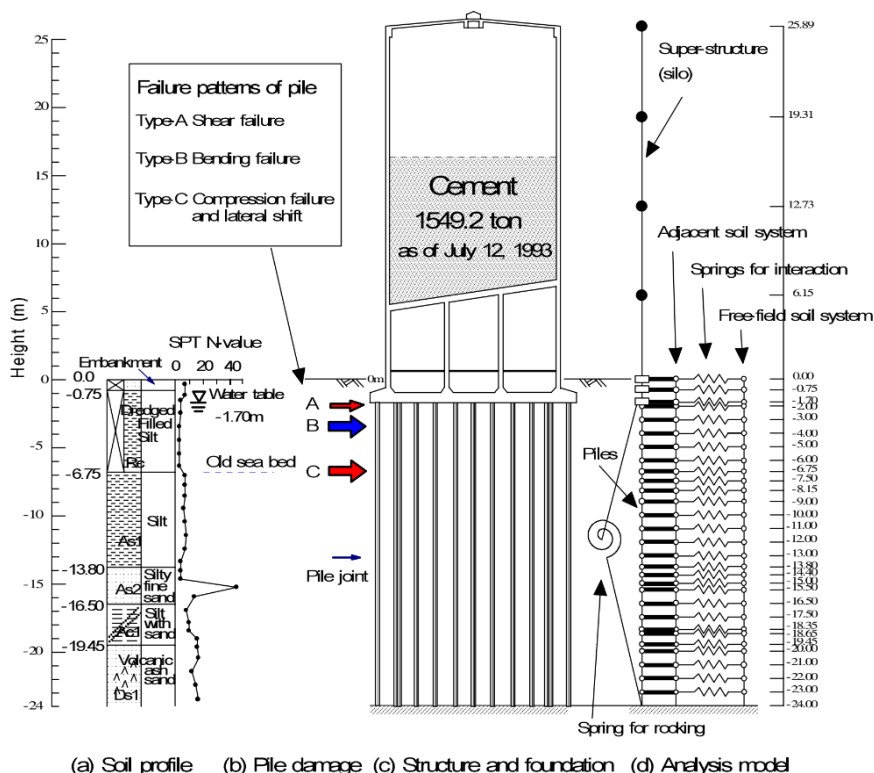


Figure 4. Specific model of analysis in association with ground profile of the site, the three depths of damage to piles, [22, 23].

Table 3. Cross section of soil layers.

Soil layer	Type soil
embankment	
Bk	Dredged-and filled silt
As1	Holocene silt marine deposit
As2	Silty sand involving thin stiff sand sub-layer
Ac1	Sandy silt
Ds1	Pleistocene volcanic ash sand

Table 4. Cyclic stress ratio.

Depth (m)	σ_{v0} total	Pwp (kPa)	σ'_{v0} eff	r_d	CSR
6.75	81	60	21	0.9484	0.427982
13.8	183.23	130.5	52.73	0.8055	0.327522
19.45	282.10	187	95.10	0.6547	0.227217

Water level 0.75 m



Figure 5. Tilted silo, [22, 23].

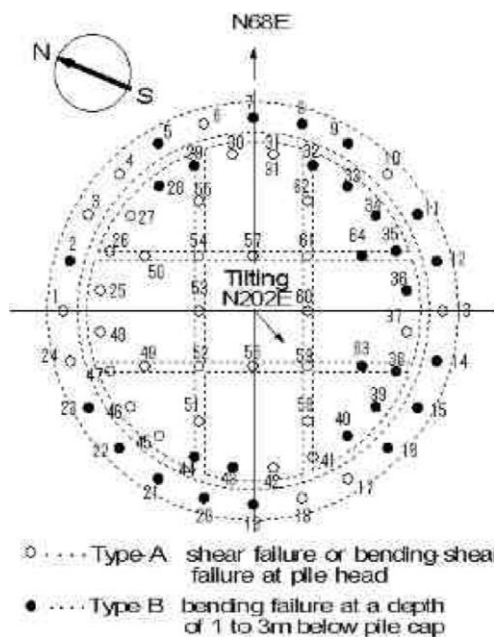


Figure 6. Shear or bending shear, Type A and Type B bending failure piles, in plane (Mori and Numata, 2004) /22, 23/.

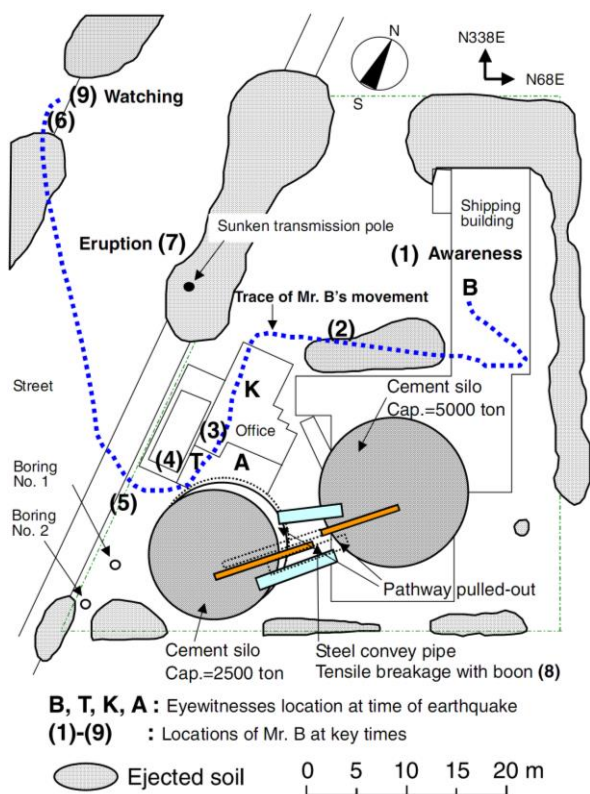


Figure 7. Plan view of damaged site and location, /22, 23/.

Tables 5 and 6 are continuations for the same depths of soil layers in the first column of Table 4.

Table 5. $(N_1)_{60cs}$ corrected SPT N for clear sand.

C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	$(N_1)_{60cs}$
1.56	0.9	1	0.95	1	5.3	11.4
1.27	0.9	1	1	1	5.7	11.9
1.02	0.9	1	1	1	7.4	13.8

FC > 35 %, assumed after soil layer section, borehole diameters (65-125 mm)

Cyclic resistance ratio and factor of safety

$$FS = (CRR_{7.5} / CSR) MSF, \quad (7)$$

$$MSF = 10^{2.24 / M_w^{2.56}}, \quad (8)$$

MSF - Magnitude Scaling Factors.

Table 6. Factor of safety.

$CRR_{7.5}$	CSR	$CRR_{7.5} / CSR$	MSF	FS
0.126	0.43	0.29	0.90	0.27
0.130	0.33	0.40	0.90	0.36
0.149	0.23	0.65	0.90	0.59

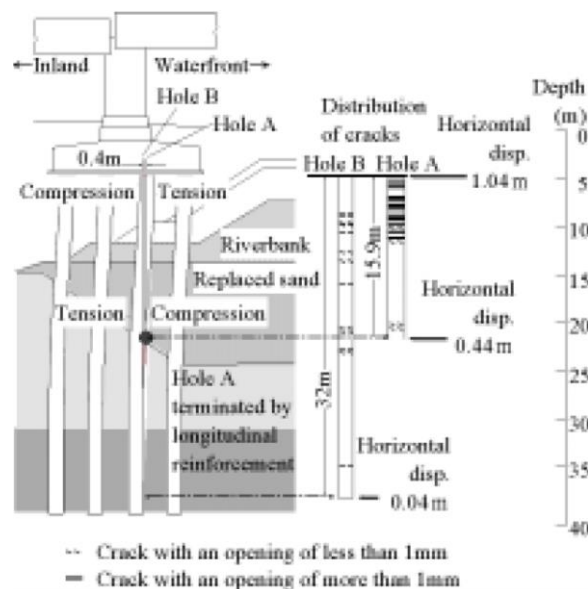


Figure 8. Example of pile damage investigation, /25/.

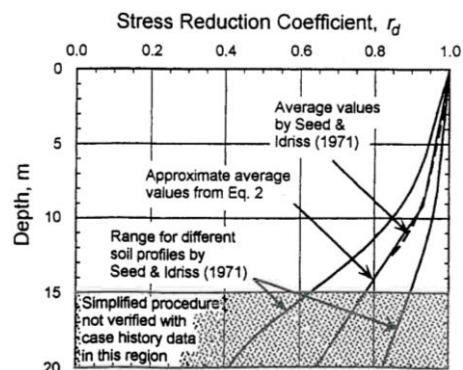


Figure 9. r_d in relation to depth, curves developed by Seed and Idriss (1971) with added mean-value lines plotted from Eqs.(2) and (3), /24/.

There are also correction coefficients K_σ and K_α , which depend on the effective overburden pressure, and the age of the deposit including relative density, stress history and over-consolidation ratio. These coefficients will not correct such a low factor of safety of the layers As1, As2 and Ac1. However, layer Ds1 has an SPT greater than 20, and liquefaction will not occur there. This is also confirmed by the speed of shear waves in that soil layer, which is $V_s = 222$ m/s. The assumed depth of liquefaction is about 18 to 20 m. It is a miracle that liquefaction did not occur as early as 1968, a year before the construction of the silo during the M8.2 magnitude earthquake, if it was of a similar distance to the 1993 Hokkaido Nansei-Oki earthquake. Although it may have occurred, but there was no loading or tilting of the soil sur-

face to show it, and a few months after the earthquake, the soil was over-consolidated.

Let us assume that half of the piles are clamped on both sides, and the other half is freely supported at the ends, and that the buckling length is 18 m, then the total critical force decreases 3 times compared to that in Table 1 and amounts to 155704.26 kN. However, the actual critical force is significantly lower, due to the way piles break /21-23/ and Figs. 4, 6 and 7 in this paper.

It is interesting to note that the sand breaking out (surface eruption of sand in Fig. 7) on the surface in Fig. 7 occurs almost in the vicinity of both silos, and that only the smaller one experiences fracture, rotation, tilt and translation.

The impact of stored material

The cement in the silo is located on an inclined plate, due to the effect of strong earthquakes, movements of the stored mass are possible that are not coordinated with the movement of the silo structure. Namely, it is possible to have the centre of mass of cement risen up, towards the upper part of the inclined plate, or down. These movements can sometimes be synchronous, and sometimes asynchronous with the movement of the silo. When the cement moves up, the moment of action on the raft is greater. This movement is only reminiscent of a hydrodynamic impact from a fluid, but here we are dealing with powdery material, so special analyses are required that we will not deal with in detail. The direction of movement of the stored material does not have to move symmetrically in relation to the co-structure but is initially carried out in the direction of the effect of the earthquake.

Calculation of the critical buckling force

In this model, the support conditions given for bound piles in raft are adopted. The conditions from the upper end are taken over for the lower end of the supports. In order to achieve buckling, the lateral spreading must increase 19 times, Fig. 10. The introduced horizontal force distributed over the entire length of the pile is 1.60 kN/m. The maximal kinematic force is about 20 and rarely reaches 40 kN/m². The calculated value in Tower Radimpex is 30.51 kN/m (76.28 kN/m²), which suggests that there may be some other form of fracture, or that a mechanism may have been formed. Based on the actual tests, we see that there was a formation of a mechanism, and a rotation of the silo.

Mechanism of pile failure, in coherent soil, for: short, pile according to Broms, /14/, is shown in Fig. 11. Diagram: soil reaction, shear force and bending moment. This system can be used in limit equilibrium models.

P-y curve model

Models of p-y curves /19, 5/ are developed at the end of the 50s of the XX century, based on the analogy with the triaxial shear test. Horizontal p-y and vertical p-z curves are especially considered. Figure 12 shows how the shape of the p-y curve changes depending on the degree of critical pore water pressure r_u . P-y curves can be entered as link elements in any of the programmes that have that option, SAP2000 /28/, etc. In this way, the earthquake load can be observed as a time history (acc.), using the step by step method, /13/.

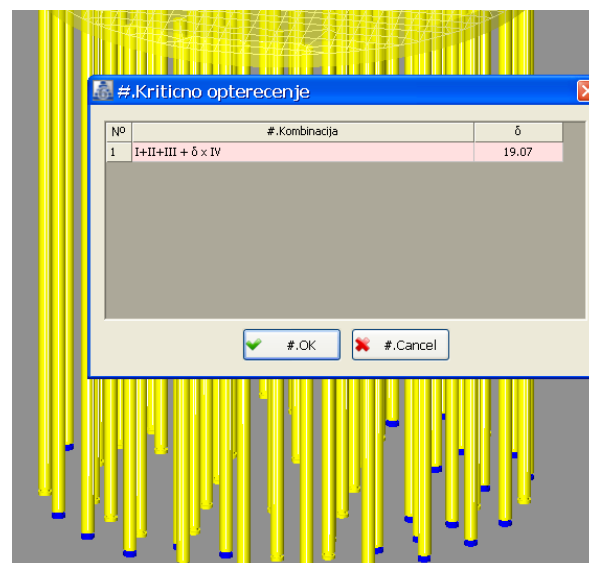


Figure 10. For buckling, the lateral force needs to increase 19 times, while self-weight, live load and stored cement is constant.

Pile failure, in coherent soil

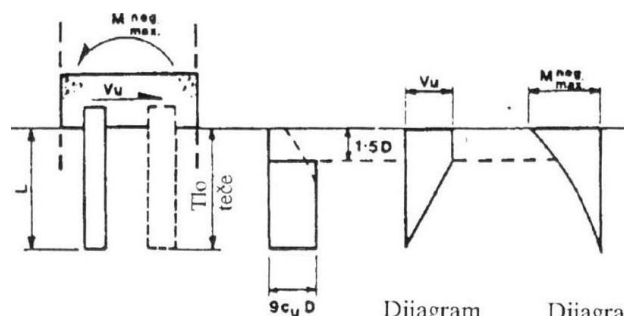


Figure 11. The failure mechanism of a short pile, in coherent soil.

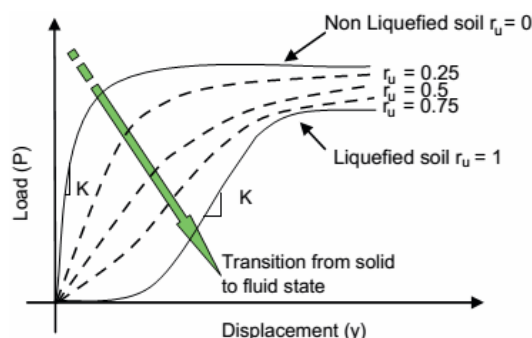


Figure 12. p-y curve for a liquefaction and non-liquefiable sand, depending on the degree of excess pore water pressure, /20/.

CONCLUSION

It should be noted that the extraction of undisturbed samples from soils prone to liquefaction, is almost impossible without freezing, that is why the SPT, CPT, or determination of the velocity of shear waves V_s by cross hole experiments are used as standard, down hole and similar.

Although it is not possible to easily obtain an undisturbed sample, disturbed samples can be extracted from exploratory boreholes, which allow easier identification of soil layers, determination of granulometric composition, etc.

When reconstructing the missing data in the calculation, whether it is the approximate liquefaction method or the

limited equilibrium method, etc., you should use tables and relations and correlations that are generally accepted or have already been used in old proven programmes as Shake, Com 624, all Pile, etc., or reliable literature. Here the problem of the lack of standard SI units can arise, namely in older literature there are diagrams and graphics that only have units of the imperial system IS. One should try to use models that have both SI and IS. If IS is used, a problem may occur when transforming the system, if it is not known whether a pound is a force or a mass, so without a detailed check it is not recommended to use them, especially not for beginners.

It should also be noted that different, but apparently similar, apparatuses are used for SPT, so one should be careful when determining (N1)₆₀.

In various formulas, clearly specify which stresses are involved, whether they are total or effective stresses, or atmospheric pressure.

Isolated liquefiable 'caverns' or a connected larger liquefiable area with sand boiling are possible.

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