Assessment of the Foundation of a Near Shore Power Plant under Earthquake Loading

Hamid Fallah

Mahab Ghodss Consulting Engineers, Iran

ASSESSMENT OF THE FOUNDATION OF A NEAR SHORE POWER PLANT UNDER EARTHQUAKE LOADING

Hamid Fallah
Mahab Ghodss Consulting Engineers
Tehran, 19187, Iran

Abstract

The foundation condition of a power plant in the south of Iran adjacent to Persian Gulf shore is discussed, taking into account the earthquake loading. Because of the high importance of the structures, advanced empirical, semi-empirical, and numerical methods have been employed, in order to verify the plant seismic safety. During empirical studies, the liquefaction potential of sandy interbeds has been evaluated, using the well known method of SPT and taking into account the percentage and type of fines. In semi-empirical studies, the undisturbed clayey samples and reconstituted sandy samples extracted from foundation soils have been tested with stress controlled cyclic triaxial device. The studies have been complemented with the numerical analysis of the structure-foundation system, using pseudo static, linear spectra, and dynamic analyses. All of the results are in good agreement and confirm the safety of the plant complex.

INTRODUCTION

geotechnical investigations have been performed in the site of a high capacity power plant at the south of Iran, in order to determine foundation conditions and geotechnical parameters. These investigations included the drilling of 14 exploratory boreholes with total depth of 740 m, taking disturbed and undisturbed soil samples, and performing in-situ and laboratory tests. In this paper, the seismic safety evaluation of the category I structures foundations is dealt with.

SITE GEOLOGY

In general, the ground surface slopes gently toward the coast from north to south and northeast to southwest, with much of area being quite flat. Surface elevations range from approximately +12 m at the northern corner of the site, to zero at the shoreline.

At the plant site, the Aghajari formation of the Pliocene age and the Bakhtyari formation of the Plio-Pleistocene age are exposed. The upper part of the Aghajari formation at the site is composed of 6 units illustrated on Fig. 1, and explained following.

Foundation soil units are relatively continuous with bedding dipping gently southwestward perpendicular to the shoreline. The uppermost soil layer, unit 1, consists of loose to medium dense silty fine marine sand, ranging between 0.5 to 2.0 meters in thickness. Unit 2 is a caprock formation composed

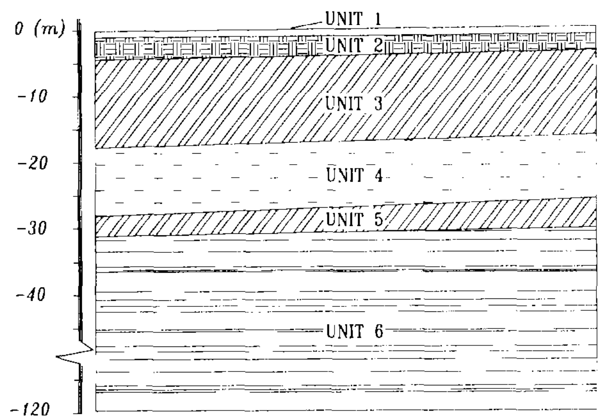


Fig. 1. General profile of the plant foundation.

generally of highly weathered calcareous sandstone and coquina, 3 to 6 meters thick. Below the caprock, unit 3 is predominantly composed of silty clay, together with occasional fine sandy seems and lenses. The consistency of the clay is stiff to hard, and the clay is heavily overconsolidated. Thickness of this unit ranges between 11 and 18 meters. Unit 4 is composed of interbedded silty clay, sandy to clayey silt, and silty to clayey sand strata, totally having the thickness of 8 to 18 meters. Unit 5 is a relatively continuous layer of very stiff to hard silty clay with high strength and plasticity, approximately 2.0 to 4.5 meters thick. Below unit 5, unit 6 is composed of silty clay and clayey silt, with thin layers of silty sand. Predominantly clayey soils exist in unit 6 until the maximum depths explored of 120 meters.

EMPIRICAL STUDIES

The usual approach to evaluate liquefaction potential in a site is to interpret the soil cyclic strength in terms of equivalent cyclic stress ratio (the ratio of cyclic stress amplitude to the effective overburden pressure: CSR). Then the CSR causing liquefaction in a deposit is compared to the CSR which is expected from the probable earthquake.

During the SPT approach used in these studies, the cyclic stress ratio exerted by earthquake was calculated from the following formula (Seed and Idriss, 1971):

$$CSR = \tau_{av} / \sigma'_v = 0.65\ \tau_{max} / \sigma'_v = 0.65 (a_{max}/g)\ r_d (\sigma_v / \sigma'_v) \quad (1)$$

where :
τ_{max} and τ_{av} : maximum and average cyclic stresses,
a_{max} : peak ground acceleration,
σ_v and σ'_v : total and effective overburden pressures, and
r_d : stress reduction coefficient to allow for deformability of the soil column.
Iwasaky et al(1978) recommended the use of the following empirical formula to determine r_d :

$$r_d = 1 - 0.015z, \text{ (z : depth in meters)} \quad (2)$$

Then, the cyclic stress ratio causing liquefaction was obtained using the graph prepared by Seed et al. (1985), and presented on Fig. 2. The N1 value was calculated through the correlation factor C_n defined as (Liao and Whitman, 1986) :

$$N1 = C_n N \quad (3)$$
$$C_n = (1/\sigma'_v)^{1/2}, \ (\sigma'_v \text{ in units of TSF}) \quad (4)$$

Then, after the energy based correction pertaining to SPT

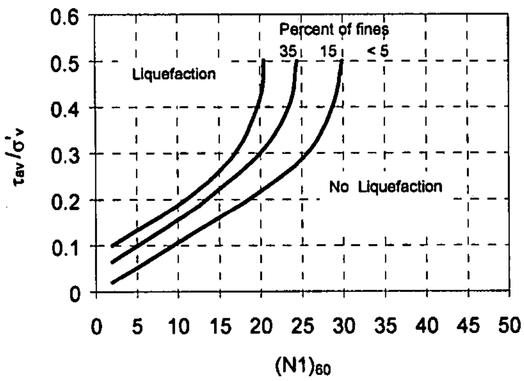


Fig 2. Correlation of cyclic strength with $(N1)_{60}$ (Seed et al., 1985)

apparatus, $(N1)_{60}$ was concluded from N1.

As mentioned before, unit 4 of the BNPP foundation soil contains sandy seems and lenses having the maximum thickness of 1.0 to 1.5 meters, which are the only foundation materials susceptible to liquefaction. Considering high percentage of fines content in these sandy materials, it seemed to be convenient to apply a uniform method to take into account the effect of fines up to about 50 percent and having plasticity indices higher than 10. For this respect, the CSR causing liquefaction was first obtained from the graph in Fig. 2 for fines contents less than 5 percent (i.e., clean sand), then the effect of fines percents up to 50 was considered in terms of the increase in SPT N1 value (ΔN1), based on the chart shown in Fig. 3 (Seed and De Alba, 1986). Also, for the plasticity indices more than 10 further correction was made in terms of a CSR coefficient by use of the chart in Fig. 4 (Ishihara and Koseki, 1989).

The above approach was used for that part of the SPT runs performed at the depths up to 30 m, taking the value of 0.4g for peak ground acceleration (PGA, or a_{max}).

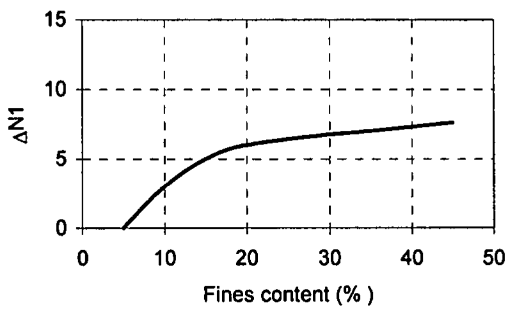


Fig 3. Increment ΔN1 value as a function of fines content (Seed and De Alba, 1986).

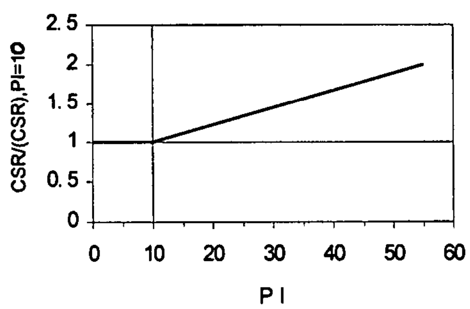


Fig. 4. Chart for modification of cyclic strength allowing for the effects of plasticity index (Ishihara et al., 1989).

SEMI-EMPIRICAL STUDIES

In semi-empirical studies, cyclic resistance ratio is obtained from laboratory tests. In this study, 19 stress controlled cyclic triaxial tests were performed in order to study the behavior of foundation soils under earthquake loading, and also liquefaction assessment of previously mentioned sandy seems and lenses of foundation unit 4. These tests were performed on undisturbed clayey samples and disturbed (reconstituted) sandy samples, extracted from exploratory boreholes.

The samples were selected from the depths of 5 to 40 meters. Cylindrical soil specimens with the height of 10 cm and diameter of 5 cm were first saturated and consolidated until the in-situ effective overburden pressures, then sinusoidal deviatoric stresses with stress ratios (the ratio of the single amplitude cyclic shear stress to the effective confining pressure) between 0.20 and 1.0, and frequency of 2 Hz were exerted. Cyclic loading on the specimens was continued until one of the following conditions were reached:

- excess pore pressure equal to the initial effective confining pressure,
- axial strain more than 15 percent,

and maximum number of loading cycles was fixed to be 500.

The cyclic failure criterion was selected reaching the excess pore pressure values equal to the initial applied effective confining pressure, or producing a double amplitude (peak to peak) axial strain of 5 percent (Ishihara, 1993). Then, some sandy samples tested under the stress ratios more than 0.40 were failed, but clayey samples didn't, as expected. Fig. 5 presents the cyclic failure envelop prepared based on 5% double amplitude axial strain for unit 4 sandy samples. After the selection of the equivalent number of loading cycles equal to 15 regarding the site area seismicity, cyclic stress ratio causing liquefaction can be determined as about 0.48, based on the cyclic failure envelop in Fig. 5.

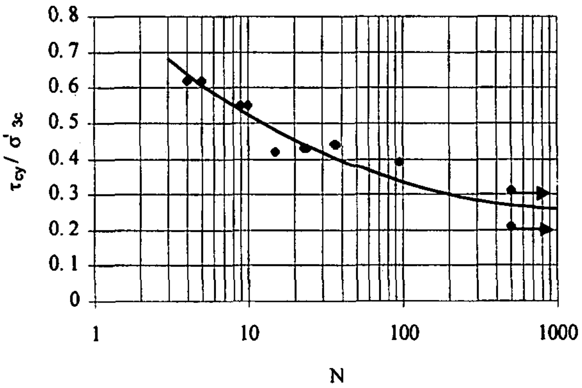


Fig. 5. Cyclic failure envelop for sandy samples.

In order to confirm the results of performed cyclic triaxial tests, these results were compared to the results of previous studies, in terms of the cyclic resistance normalized to the resistance at the 15th cycle. The comparison is summarized in Fig. 6. According to this figure, the results well coincide with the previous ones.

The calculated safety factors against liquefaction using cyclic triaxial test data is illustrated in Fig. 7, for sandy interbeds of the depths of 15 (footing level) to 40 m. For those, cyclic stress ratio is calculated from (1), and cyclic resistance ratio is accepted to be 0.48 for N=15, from Fig. 5. As illustrated, in all depths, safety factors against liquefaction are higher than 1.0.

As mentioned before, none of the clayey samples experienced liquefaction, but some pore pressure build up was observed during cyclic loading. These pore pressures are presented in

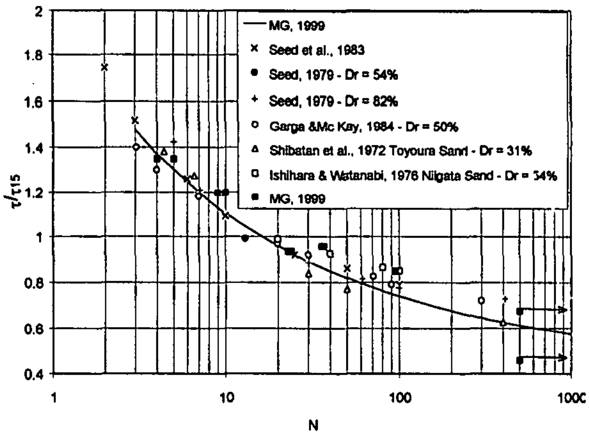


Fig 6. Comparison between the present (Mahab Ghodss or MG) results with previous data.

Fig. 8, in a form similar to Ansal and Erken (1989), in which the excess pore pressure ratio (ratio of the excess pore pressure due to cyclic loading, to the initial effective confining pressure: $\Delta u/\sigma'_{3c}$) is plotted against S.R., for different numbers of loading cycles. Then, the excess pore water pressure generated due to cyclic loading can be assessed for any combination of SR and number of loading cycles.

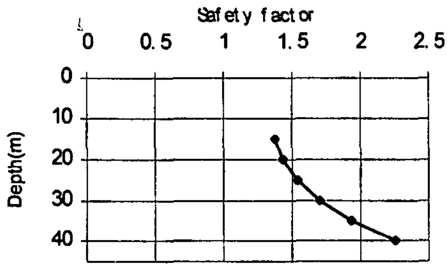


Fig. 7. Safety factor against liquefaction versus depth, based on cyclic triaxial tests.

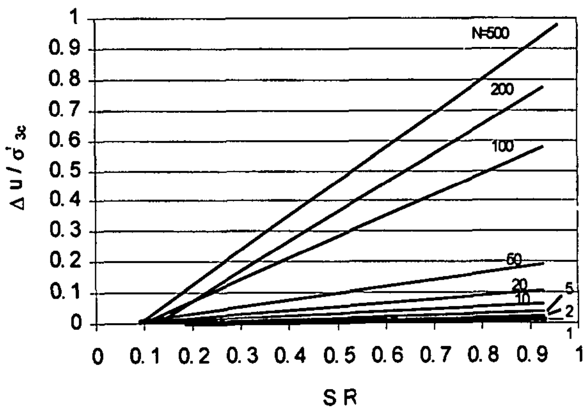


Fig. 8. Excess pore pressure ratio versus SR, for different cycle numbers.

It is evident from Fig. 8 that there is a threshold SR level below which no pore water pressure will develop. For tested clayey samples in the present project, the threshold SR is determined to be about 0.12. Also, for $N_{eq}=15$, and SR about 0.50, excess pore pressure ratio is proved to be not more than 5% at the site seismic conditions. Then, generally for foundation soil predominantly composed of silty clays, excess pore pressures generated during earthquake loading and consequent settlements after loading because of excess pore pressure dissipation, are negligible.

NUMERICAL ANALYSES

For all of the numerical analyses performed, profile of Fig. 9 has been used, including the basements of main structures together with the simplified foundation soil profile. Drained and undrained shear strength parameters of the foundation soil layers in Fig. 9 are presented in Table 1. Following, different analyses are explained.

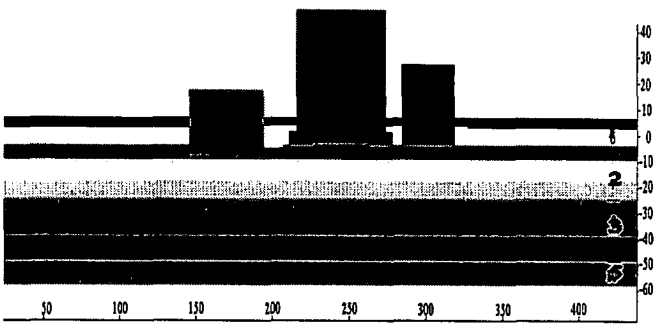


Fig. 9. Foundation profile used in numerical analyses.

Table 1. Shear strength parameters of foundation soil layers.

Layer	Φ'(degree)	C'(KPa)	S_u (KPa)
1	25	50	175
2	27	63	328
3	15	107	350
4	22	144	325
5	24	163	400

Pseudo-static analysis

In order to calculate plant foundation general bearing capacity taking into account both static and earthquake loadings, factors of safety against plane (shallow) and deep shear were determined. The method of slice was used and earthquake loading was exerted to the weight of slices as horizontal coefficients. The coefficients of 0.275g for design base earthquake and 0.4g and 0.45g for safe shutdown earthquake were used in different analyses. The results of calculations are presented in table 2. As it is seen, consequent factors of safety are high enough to confirm foundation general stability.

Then, after confirmation of foundation general stability, local factors of safety in different points of the foundation were evaluated from dynamic analyses.

Table 2. Factors of safety against shear for reactor foundation

Acceleration coefficient	Plane shear	Deep shear
0.275g	1.46	2.07
0.40g	1.19	1.45
0.45g	1.13	1.29

Linear spectral analysis

Linear spectral theory (LST) considers initial earthquake action in a form of acceleration spectra, and intensity of earthquake is given as the value of the peak acceleration of the foundation. This method is based on the eigenform decomposition of the differential equations system of motion. The method is divided into the following steps:

- Solution of a spectral subproblem and determining the main modes of the motion of the foundation–substructure system and related natural frequencies.
- Determination of the modal coefficients and amplification factors (based on acceleration spectrum) for each mode of natural oscillation.
- Calculation of accelerations, displacements, stresses and strains in each mode.
- Superposition of the effect of main modes using the method of SRSS (square root of sum of squares), for each component.

In the modified method utilized here, the above algorithm is repeated with correction of parameters at each step accounting for stress-strain state obtained at the previous, until reaching convergence.

For the present project, LST analysis of reactor foundation was carried out for two seismic loading conditions: earthquake actions from the nearest seismic sources (loading I) with PGA_h equal to 4.0 m/sec^2, and earthquake actions from diffused seismicity (loading II) with PGA_h equal to 4.5 m/sec^2. PGA_v values were 2/3 PGA_h for loading I, and 1/2 PGA_h for loading II.

Factors of safety against liquefaction or shear failure in foundation soils were calculated from the following formula:

$$F.S. = \tau_{cr} / \tau_{lst} \qquad (5)$$

where :

τ_{cr} : critical shear stress, equal to $0.48\sigma'_v$ for sandy soils according to laboratory cyclic triaxial tests, and $0.90S_u$ (undrained shear strength) for clayey soils, taking into account undrained strength loss because of cyclic loading, and

τ_{lst} : shear stress obtained from linear spectral analysis.

Local factors of safety using above approach are plotted in Fig. 10 for the foundation sandy and clayey soils as a result of loading II, which has concluded lower values of F.S. than loading I.

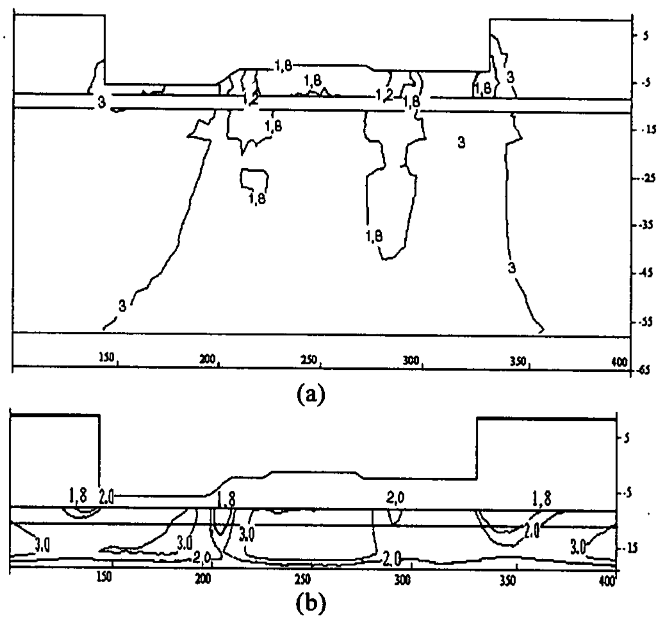


Fig. 10. Factors of safety against cyclic failure, from LST analysis, (a) in clayey soils, (b) in sandy soils.

Dynamic analysis

Dynamic analysis of seismic resistance of the foundation soil substructure system was performed using both linear and nonlinear time domain approaches. The analysis was made by preparing time step form of seismic loading (acceleration history) and then step-by-step integrating of the equations of motion. In linear analyses, constant values of shear modulus and damping were used, whereas in nonlinear analyses, in each time step inner iterations were used to consider strain dependent shear modulus and damping of foundation soils. Static stress-strain state of the foundation soil was taken as initial condition for dynamic analysis.

There are some different accelerograms for different seismic source zones that were prepared from scaling of real records to reach the prescribed values of PGA, predominant frequency, and effective duration. But accelerogram #7 (M_s = 7.3, distance = 36.4Km, 84% probability) yielded the highest stresses in the foundation soil. Horizontal component of accelerogram #7 is illustrated in Fig. 11.

Finite element mesh was prepared for the foundation profile in Fig. 9, including 2375 nodes and 4536 elements. Standard viscous boundary conditions were assumed for the side boundaries of the considered foundation region. Time step for integrating was Δt_1= 0.02sec for accelerogram #7. Parameters of package of layers of the foundation soils used in dynamic analysis are presented in table 3. It must be mentioned that the values of shear modulus and damping presented are the initial values, and in nonlinear analyses, shear modulus values were decreased and damping coefficients were increased with the increase of dynamic shear strains, according to the strain dependency relationships obtained for the foundation materials.

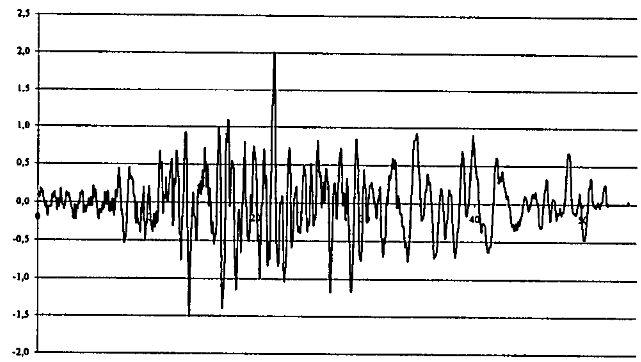


Fig 11. Horizontal component of accelerogram #7 (m/sec^2)

Local safety factors from two dimensional analyses were calculated similar to LST method, from the following formula:

$$F.S. = (SR)_{cr} / (0.65\tau_{dyn} / \sigma'_v) \qquad (6)$$

where:

$(SR)_{cr}$: critical stress ratio causing liquefaction, determined to be 0.48 for foundation sandy soils. For clayey soils, it equals to $0.90S_u / \sigma'_v$.

τ_{dyn} : maximum dynamic shear stress caused by earthquake loading in a point at foundation.

Local safety factors from the above approach using accelerogram #7 (the most critical one) are plotted in Fig. 12.

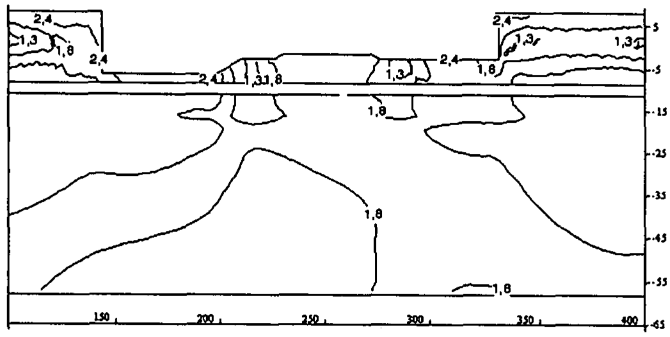
Table 3. Parameters of package of layers of the foundation soils used in dynamic analysis.

No.	Depth of layer from the reactor Basement (m)	Density (Kg/m^3)	Initial shear modulus (Mpa)	Initial damping coefficient (%)
1	0 – 4	1980	200	0.02
2	4 - 12	1990	220	0.02
3	12 – 17	2030	330	0.02
4	17 – 22	2030	440	0.02
5	22 – 37	2060	550	0.02
6	37 – 47	2090	700	0.02
7	47 – 57	2060	1000	0.02
8	> 57	2070	1300	0.02

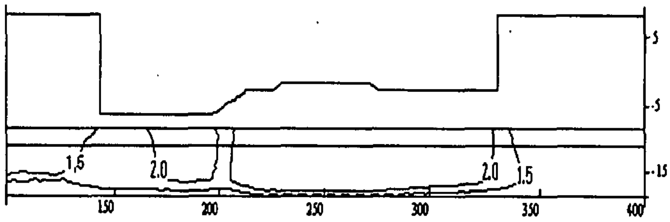
In order to consider uncertainties in determining shear modulus of foundation soils, nonlinear dynamic analysis using accelerogram #7 was repeated with the modulus values multiplied by 1.5 and 2.0, and divided by 1.5 and 2.0. The results of these analyses showed no considerable change in dynamic analysis general remarks.

CONCLUSIONS

Results of studies on the behaviour of the power plant foundation under earthquake loading were presented.



(a)



(b)

Fig 12. Factors of safety against cyclic failure, from nonlinear two dimensional dynamic analysis, (a) in clayey soils, (b) in sandy soils.

According to the empirical approach using SPT results, factor of safety against liquefaction in sandy lenses of unit 4 soils were obtained in all points to be more than 1.0, and most of them were more than 1.5. This is attributed to high percentage of fines and high relative density even up to 90%.

During semi-empirical studies, cyclic failure envelop and critical cyclic resistance ratio (CRR)causing liquefaction were obtained for the foundation sandy soils, that the CRR was determined to be 0.48. This value was used together with the stresses computed from dynamic analyses, in order to calculate local factors of safety in foundation soils. Also, the cyclic excess pore pressure build up model for foundation clayey soils was determined, that concluded excess pore pressure ratio less than 5% for clayey soils at the BNPP seismic conditions.

Pseudo-static analysis confirmed the general stability of the main structures under earthquake effects. Also, local safety factors resulted from linear spectral and dynamic analyses were more than minimum acceptable values in all foundation points. Only in very few points at backfill materials, well far from the category I structures foundations, the factor of safety was slightly less than 1.0 in dynamic analysis with shear modulus multiplied by 2.0, that is negligible. Also, based on dynamic analyses the footings edges differential settlement was determined to be less than 3.5 Cm and consequence inclination less than 0.001, that are less than allowable values.

To sum up, the plant foundation stability under earthquake loading was fully verified as supported by results presented herein.

AKNOWLEDGEMENT

The author thanks Dr. M.H. Baziar and Dr. B. Mahmoodzadegan for their discussions and useful comments on preparing this paper.

REFERENCES

Andersen, K.H., [1988]. Properties of soft Clay Under Static and Cyclic Loadings.@, Proc. Int. Conf. Engrg. Problems of Regional soils, Beijing, China.

Ansal, A. M. and A. Erken, [1989]. Undrained Behaviour of Clay Under Cyclic Shear Stresses.@, J. Geotech. Engrg., ASCE, Vol. 115, No. 7, pp. 968-983.

Ishihara, K. and J. Koseky, [1989]. Discussion of Cyclic Shear Strength of Fines Containing Sands.@, Earthquake Geotechnical Engineering, 12th Int. Conf. Soil Mech. Rio de Janeiro, pp. 101-106.

Ishihara, K., [1993]. Liquefaction and Flow Failure During Earthquakes.@, Geotechnique, Vol. 43, No. 3, pp. 351-415.

Iwasaki, T., F. Tatsuoka, K. Tokida and S. Yasuda, [1978]. A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan.@, Proc. 2nd Int. Conf. Microzonation for Safer Construct. Res. & Application 2, pp. 885-896.

Liao, S.S.C. and R.V. Whitman, [1986]. Overburden Correction Factors for SPT in Sand.@, J. Geotech. Engrg., ASCE, Vol. 112, No. 3, pp. 373-377.

Seed, H.B. and I.M. Idriss, [1971]. Simplified procedure for evaluating soil liquefaction potential.@, J. Soil Mech. Found. Div., ASCE, Vol. 97, SM9, pp. 1249-74.

Seed, H.B., K. Tokimatsu, L.F. Harder and R.M. Chung, [1985]. Influence of SPT Procedures in Soil Liquefaction Evaluations.@, J. Geotech. Engrg., ASCE, Vol. 111, No. 12, pp. 1425-45.

Seed, H.B., and P. De Alba, [1986]. Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands.@, Proc. In-Situ Test, ASCE, pp. 281-302.