A case history for soil improvement against liquefaction, Carrefoursa Shopping Centre-Izmir, Turkey

Une étude de cas d'amélioration des sols et traitement antiliquéfaction pour la fondation d'un centre commercial à Izmir, Turquie

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ABSTRACT: This case study reports the application of dynamic soil replacement as the soil improvement method for the construction of a shopping centre in Izmir, Turkey. The soil profile consists of marine sediments with shallow ground water table. The soils were improved to increase the bearing support and to reduce liquefaction potential considering high seismic activity of the region. This paper summarises and presents geotechnical modelling, seismicity and analyses leading to soil improvement design, "Dynamic Replacement" is applied as the soil improvement method in which rock columns of large diameter (i.e. pillars) are formed based on the techniques developed for Dynamic Consolidation. An extensive total quality control program employing SPT, CPT and pressuremeter testings together with observational methods for monitoring settlements, was performed. This paper describes the design, construction, and performance of dynamic replacement process focusing on the results of quality control tests showing the effectiveness of the applied system.

RÉSUMÉ: Ce rapport fait l'étude de l'application de "Substitution Dynamique" comme une méthode de traitement de sol, pour un centre commercial a Izmir en Turquie. La nappe phréatique se trouve sous le sol composé de sédiments marins. Ce dernier a été traité afin d'améliorer sa capacité portante et de réduire le potentiel de liquéfaction a cause des nombreuses activités sismiques de la région. Ce rapport résume et représente le modèle géotechnique, la sismicité, les analyses et les calculs réalisés pour les travaux de traitement. La méthode de Substitution Dynamique a été utilisée par production de plots ballastés de grand diamètre sur la base de la technique développée pour la Consolidation Dynamique. Un programme complémentaire de contrôle de la qualité a été réalisé en utilisant SPT, CPT ainsi que les tests de PMT par méthode d'observation pour constater les tassements. Cet article étudie l'analyse, la construction et le performance de la Substitution Dynamique et particulièrement les résultats des tests de contrôle qualité qui démontrent l'effectivité de la méthode appliquée.

1 INTRODUCTION

This case study reports a dynamic replacement application for the construction of Carrefoursa Shopping centre in Izmir, Turkey. The soils were improved to increase the bearing support for shallow foundations and reduce liquefaction potential considering the high seismic activity of the region. Total soil improvement area covers an area of about 68,700 m^2 and designed to house three main buildings namely, "Supermarket-Block A", "Commercial Centre-Block B" and "Block C". A total quality control program was performed during the soil improvement works. This case study describes the design, construction and performance of dynamic replacement process. Within the scope of this paper, first geotechnical modelling set by ground investigations and foundation engineering evaluations will be given and then the results of quality control/quality assurance tests performed during the improvement works are to be documented to show the effectiveness of the applied system.

2 SITE DESCRIPTION

2.1 *Local Geology*

The subject site is located at Bostanli in Izmir. The geological units encountered at the site are formed by the deposition of fine grained, equal sized, low energy material transported by the Gediz river. The whole site is characterised by the Quaternary aged, alluvial deposits consisting alternating layers of various sizes. It was proved with geological and sedimentological investigations that the area is the old delta of Gediz River. During the geological time, Gediz River changed its bed, deposited in shallow marine environment and turned into lagoons. The thick deposits include sand, silt, clay sediments and no bedrock is encountered during the subsoil investigations.

2.2 *Facilities*

Within the shopping centre area three structures, namely Block A, Block B, Block C and a car parking lot were planned to be constructed. The area for the proposed one-story supermarket building-Block A contains approximately 16,700 m^2 with the future extension part. Shopping centre-Block B encompasses an area of 15,150 m^2 and the area for Block C is 5585 m^2. No basement structure is planned for the whole commercial centre. Total parking area contains approximately 35,900 m^2. No structures are initially planned for this section, but the subsoil is to be improved in anticipation of a future development. All buildings are designed for steel construction.

Natural ground elevations varied between +0.1 m and +3.0 m with an average value of +0.5 m throughout the site. The final level for the site is planned as at +3.50 m being equivalent to 0.0 level for the architectural design.

3 SUBSOIL INVESTIGATIONS AND SITE CONDITIONS

Subsoil investigations are realised consisting of boreholes, pressuremeter testings and cone penetration testings (CPT). Standard penetration testing (SPT) is also performed systematically during the course of boreholes. The range of the measured ground water depths are between 0.70 m and 1.2 m, with an average depth of 0.85 m throughout the site. The subsoil conditions dominant within the site is the alternation of sand, silt and clay sized material alternating in short distances along horizontal and vertical directions, indicating occasional lenses. A typical data from a CPT and a nearby borehole is given in Figure 1.

Based on the results of boreholes and CPT tests, the subsoil within the site in general, consists of alternation of silty clay, silt and silty sand ranging between soft/loose to medium approximately down to the depth of 24.0 m below the ground surface. Below this depth, a gravelly clay layer with average SPT/N blow count value in the order of 25 is encountered.

In the geotechnical model based on the results of in situ tests, laboratory tests and CPTs, it was observed that, down to approximately 3.5 m depth, the undrained shear strength values are in the order of s_u = 35 kPa. Approximately between the depths of 3.5 m and 12.0 m, a silty, clayey sand layer in loose to medium density is encountered. Between the depths 12.0 m and 24.0 m, a clay layer is present having undrained shear strength value in the order of s_u = 50 kPa while a 2.0 m thick sand layer is found between the depths of 16.0 m and 18.0 m. Based on the results of boreholes and CPTs, the consistency increases from stiff to hard below 24.0 m depth. The identification properties of the soils encountered at the site are determined by laboratory tests and summarised below in Table 1.

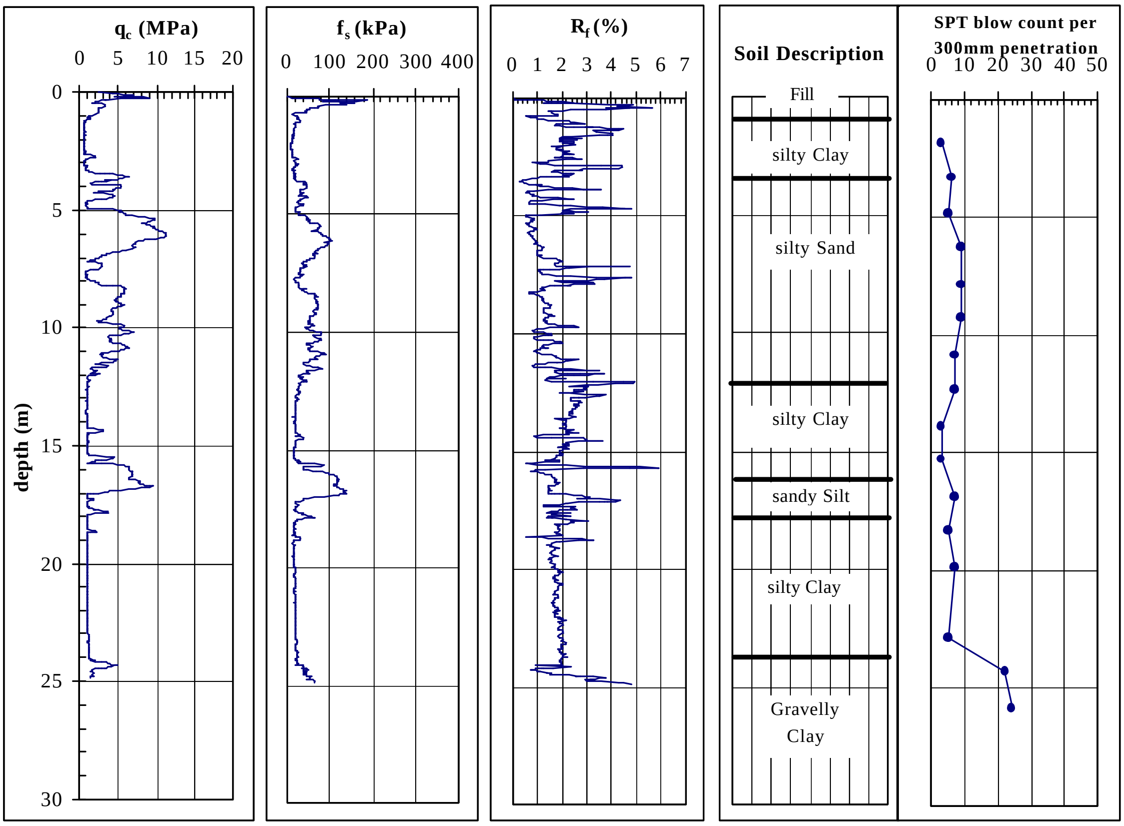

Figure 1. Typical soil profile determined by CPT and boring

Table 1. Identification test results

Property	Sand	Clay
Natural water content, %	*	35-68
% passing #200 sieve	10-43	*
Liquid limit	*	39-83
Plastic limit	*	12-41
Plasticity index	*	16-46

* not available

4 FOUNDATION ENGINEERING EVALUATIONS

4.1 *Bearing capacity*

Based on the shear strength model, the ultimate bearing capacity for shallow foundations and embankment without considering the primary and secondary settlements, is determined by;

$q_{net} \sim 5 \times s_u$

whereas, q_{net} = 5 x 35 kPa = 175 kPa. Therefore the allowable soil pressure is given as;

q_{all} = 70 kPa

considering a safety factor of FS = 2.5 and without taking into account the settlements.

4.2 *Settlements*

The settlement calculations are done both utilising CPT test data which enables detailed layering and also utilising the idealised geotechnical model. The settlement calculations are carried out for a strip foundation width of 4.0 m and a foundation base pressure of 50 kPa and also repeated for a 3.0 m high embankment loading. The results obtained are summarised below in Table 2. It was also recommended in the foundation engineering report that the secondary settlements in the order of 10% of these given values should be considered to be realised in the long term.

Considering the structural and architectural criteria for the planned structures, calculated settlement values are above the allowable limits. The allowable settlements were determined as maximum s = 5.0 cm under the foundations/slabs-on-grade and embankment loading. Ultimately, it was concluded that both foundations and slabs-on-grade and/or embankment loadings should be supported by soil improvement and/or deep foundation systems.

Table 2. Summary of settlement analyses

	Settlement, cm			
Analysis based on	CPTs		Model	
Loading type	Strip foundation	Embankment	Strip foundation	Embankment
Minimum	12.1	20.2	-	-
Maximum	23.8	39.3	-	-
Average	18.7	37.1	16.7	32.9

4.3 *Seismicity and risk of liquefaction*

The subject site is located within the first degree earthquake zone in Izmir with a high seismic activity. The soil liquefaction analyses are performed based on the results of CPT and SPT, and the properties of the encountered subsoil in boreholes. For this purpose, a seismic risk analysis is utilised performed previously for the subject region by Cetinkaya et al. (1993). Accordingly for the subject region,

– Average maximum annual earthquake magnitude: M = 4.8
– Most frequent annual magnitude: M = 4.6
– Maximum magnitude for a return period of 100 years: M = 8.4
– Magnitude with an annual risk of R = 10%: M = 6.3 (design earthquake)
– Bedrock accelerations for an epicentral distance to fault line of 10 km is a = 0.5g, 20 km is a = 0.25g, and 40 km is a = 0.12g

Considering that the subject site is located within 20 km distance to potential fault lines, the maximum bedrock acceleration for the design earthquake was considered as 0.25g for a service life of 100 years. Furthermore, considering the bedrock is at very deep level and the site is covered with weak alluvial deposits, soil amplification is expected during an earthquake and therefore structural units within the facility would be subject to acceleration greater than a = 0.25g. Ultimately, the liquefaction analyses are performed for ground acceleration values of a = 0.3g and 0.4g considering the soil amplification effects. The susceptibility of the subsoil against liquefaction under the earthquake loading was also predicted by the method suggested by Robertson (1988) utilising the CPT parameters. Analyses are done for the cases of PGA (Peak Ground Acceleration) values a = 0.3g and a = 0.4g both using the standard penetration blow counts, and CPT results separately. As a result of these analyses, liquefaction potential was determined in the sand layer existing within the first 12.0 m depth. Below this level, clayey and silty layers are present in general which are not prone to soil liquefaction.

Consequently, in order to prevent liquefaction, soil improvement was recommended to be implemented at the subject site.

5 SOIL IMPROVEMENT WORKS

In the light of ground investigations, dynamic replacement method was chosen to be the best solution among the possible ground improvement techniques applicable, in terms of economy, time and effectiveness. Soil improvement works are realised utilising dynamic replacement method by French company of Menard Soltraitement.

Dynamic Replacement (DR) is a method in which columns of large diameter are formed with granular material based on the techniques developed for Dynamic Consolidation in highly compressible and weak soils. These columns of granular material are called “pillars”. This technique is similar to dynamic consolidation but in this case, pounding is used to form large diameter granular pillars through the soft clayey soils to underlying dense bearing stratum. This method combines the advantages of dynamic consolidation with those of stone columns creating an economical advantage because the excavation of the weak soil can be avoided. Also, high internal shearing resistance is provided within the pillar. These pillars act as oversize vertical drains causing reduction of consolidation time. Schematic principle of the dynamic replacement method is illustrated in Figure 2.

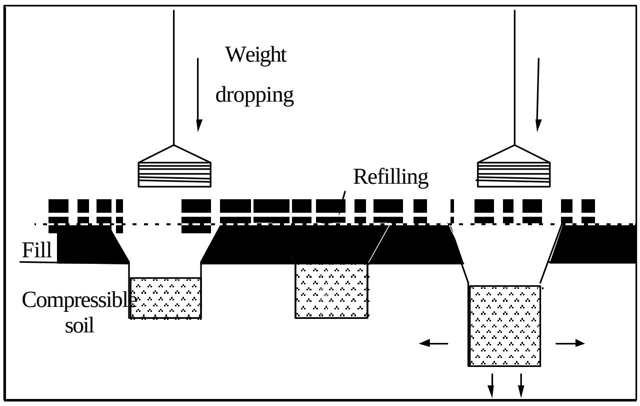

Figure 2. Schematic principle of Dynamic Replacement method

The equipment used for dynamic replacement is similar to the dynamic consolidation equipment i.e. heavy rigs and pounders. However, usually pounder with smaller area is used to facilitate the penetration capacity (Menard Soltraitement 1999). Heavy dynamic replacement (HDR) columns are made with boulders and cobles using energies exceeding 400 tm per blow. The relationship between the effective depth of attained improvement, the pounder weight and the height of the drop is expressed as below in dynamic consolidation:

$$D = (0.3 \text{ to } 0.7)\ \sqrt{W \times H} \qquad (1)$$

where D = maximum depth of improvement in meters; W = falling weight in metric tons; and H = height of drop in meters (Mitchell & Gallagher 1998).

Pre-excavation in dynamic replacement allows to construct deeper columns especially in clays, enabling to obtain very good resistance parameters even in soft compressible soils when combined with HDR method. Construction control methods of dynamic replacement operation at site are similar to those of dynamic consolidation.

Menard designed the soil improvement scheme including the quality control testing for Carrefoursa site in Izmir. For this purpose, the total area was divided into six different zones where different pillar types are constructed. The total area of the site is 64,851 m^2 where the soil improvement is applied. The properties set initially for various types of pillars are given below in Table 3.

Table 3. Details of dynamic replacement scheme

Treatment	DR	HDR				
Pillar Type	Type 1	Type 1	Type 2	Type 3	Type 4	Type 5
Grid, m^2	36	36	36	36	32	54
Drop Height, m	20	20	20	20	20	20
Pounder Weight, t	15	25	25	25	25	25
Blows per Pillar, b/p	20	22	22	22	20	3x22
Energy, tm/m^2	166.7	305.6	305.6	305.6	312.5	611.1

The working platform level was at +1.50 m where the soil improvement application is performed. After the soil improvement works are completed, the level is brought to +3.50 m.

5.1 *Pilot tests in calibration area*

The Pilot Test is part of quality control program which was performed before commencing the soil improvement works and has permitted to adapt the parameters of Dynamic Replacement (DR) and Heavy Dynamic Replacement (HDR) to the mechanical properties of the soil.

The area in which the pilot tests were carried out is a square of 450 m^2 (21.2 m x 21.2 m). Dynamic replacement pillars in this zone were performed according to 6.0 m x 6.0 m grid. Pilot zone testing consisted of initial and final pressuremeter tests, heave and penetration tests, grading and level monitoring. The detailed characteristics of the dynamic replacement equipment used in the pilot area is given below:

- Tamping unit :500 tm crawler crane
- Pounder weight : 25 tons
- Pounder dimension : 1.85 m x 1.85 m – octagonal shape
- Payloader for carrying the material for the pillars
- Surveyor equipment for level measurement

During the calibration works, settlement and heave values are estimated for each penetration. Pressuremeter tests (PMT) on pilot area were realised down to a elevation of –11.0 m, with tests at every meter from the level of +1.5 m to obtain limit pressure (Pl) values and pressuremeter modulus (E_p) values.

Heave and penetration works are realised in order to determine the optimal number of blows for each phase, to determine the optimal volume of fill material to be put into the prints and to check that the pounder penetration is not a volume displacement, but effective compaction of the soil. Following parameters are recorded: (a) diameter of print after each blow; (b) depth of print after each blow; (c) levels of all benchmarks during the test after two consecutive blows; (d) penetration of the pounder. Utilising these parameters, the crater volume is computed and corrected considering the heave volume by plotting effective penetration volume versus the number of blows.

Consequently, the following parameters are adapted for the improvement scheme to be applied at the site.

Full height of drop : 20.0 m
Number of blows : Pillars without pre-excavation (under slabs-on-grade): 15 blows
Pillars with pre-excavation (under footings): 20 blows

Phases of work :

- Phase 1 : pillars under slabs-on-grade without excavation
- Phase 2 : pillars for footing with pre-excavation
- Ironing for improving the first meters of the column
- Partial backfilling of the print with granular material while forming the column
- Complete backfilling of the print up to elevation +1.5 m of working platform

However, a comparison of the results of pressuremeter tests showed that the improvement was not effected with the presence of pre-excavation in clay layer. Consequently the job parameters were modified and adapted as given below :

Full height of drop : 20.0 m
Number of blows : 20 blows
One phase of work (No pre-excavation) with ironing for improving the first meters of the column following the pillar construction.

5.2 *Dynamic Replacement Design*

The design for the Heavy Dynamic Replacement/Dynamic Replacement pillars has been realised by Menard Soltaritement based on the foundation layout. The design is summarised in Table 4.

Table 4. The soil improvement design for Carrefoursa-Izmir site

Area	Structure	Type	Pounder	No of pillars / Energy per blow
Transition	Slab-on-grade	DR	15 tons	1/300 tm
Structural	Slab	HDR	15 t/25 t	1/300 tm - 1/500
	Footing load < 100 t		25 t	1/500 tm
	100 t <Footing load<150 t			2/500 tm
	Footing load > 150 t			3/500 tm

The pillars under footings are performed using a 25 tons penetration pounder. For loads exceeding 100 tons, 2 or 3 pillars are constructed under a single footing. Pillars under the slabs-on-grade are performed utilising alternatively 25 tons or 15 tons pounder depending on the grid configuration. Soil treatment at the transition areas between structural area and open parking lots/roads are realised using 15 tons pounder.

6 RESULTS AFTER TREATMENT – EXECUTION OF QUALITY CONTROL PROGRAMME

Preliminary geotechnical campaign included the pressuremeter testings, boreholes with Standard Penetration Testings, Cone Penetration testings and laboratory testings. For the purpose of comparison, similar tests are realised within the scope of final stage geotechnical campaign following the treatment. The average values of limit pressure, Pl obtained at the same depths from all pressuremeter tests are calculated and plotted in Figure

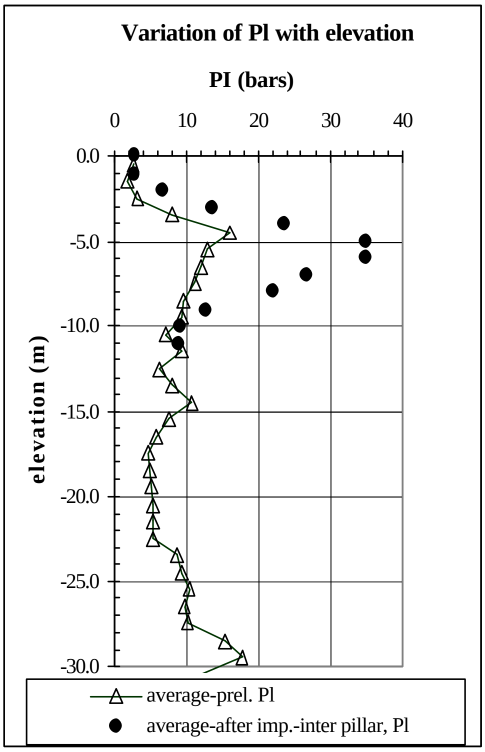

Figure 3. Comparison of average net limit pressure values obtained between pillars before and after soil improvement

3 and Figure 4, separately for the tests done at pillar locations and at the midpoint of pillars to observe the general effect of the soil improvement application.

From these figures, it could be seen that there is minor increase in limit pressure values within the upper clay layer down to the elevation of 3.0 m, between the pillars as expected, while the increase within the sand layer with the effect of dynamic compaction is quite significant. It is seen that the attained levels of improvement between the pillars varied between 30 per cent and +100 per cent within the sand layer. In addition, depth of influence for the dynamic consolidation is estimated as being 11.0 m in between pillars and 13.0 m for pillar locations also taking into account the 1.5 m thick working platform. Therefore the constant given as variable from 0.3 to 0.7 in Equation 1 is estimated approximately as 0.6 for pillar location and 0.5 in between pillars based on the average pressuremeter parameters for the Izmir Carrefoursa site. Maximum improvement occurred within the upper 7.0 m in between the pillars (~0.64 of the effective depth) while scattered levels of improvement was achieved throughout the effective depth at the pillar locations.

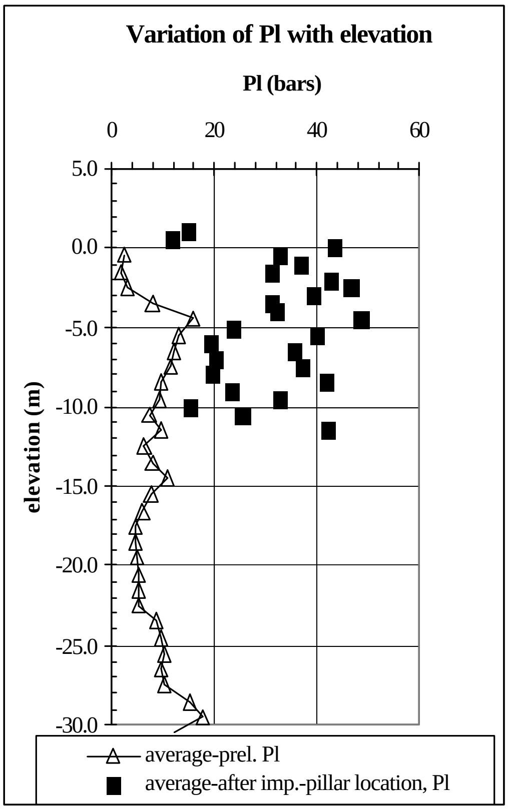

Figure 4. Comparison of average net limit pressure values obtained at pillar locations before and after soil improvement

The control of improvement efficiency with CPTs realised between pillars showed a less pronounced effect compared to the results of pressuremeter tests. Therefore it could be concluded that CPT testing has a more important role in determining the detailed soil profile detecting the thin layers of sands and silts which could be susceptible to liquefaction and silty clays which are not prone to liquefaction.

6.1 *Anti-Liquefaction Criteria*

The liquefaction criterion is checked utilising the basic Seed and Idriss criteria updated in 1997 based on the Standard Penetration Test (SPT) results by Menard. 15 boreholes were performed and according to the testing results, factor of safety values against liquefaction are estimated being greater than one except few levels. These levels are usually the silty and clayey layers as indicated in nearby CPT test points. During the standard penetration tests, representative samples were taken and fine contents of these samples are determined.

The second criterion against liquefaction is set based on the cyclic pressuremeter modulus measuring the sensitivity of the improved soil to repetitive strains cycle. The cyclic modulus is computed as the harmonic mean of the moduli at re-loading. The ratio of cyclic modulus to Menard modulus is calculated for each depth where the test is performed. Fourteen boreholes with cyclic pressurremeter testing were performed.

According to the analyses done by the SPT and pressuremeter modulus approaches, the anti-liquefaction criteria are met for the improved area.

6.2 *Settlement and Bearing Capacity Criteria*

Settlement and bearing capacity criteria are checked utilising the pressuremeter results. The parameters are given below:

- Compressibility parameters for HDR pillars constructed in upper clay layer
- Bearing capacity of HDR pillars and upper clay
- Compressibility parameter for total and differential settlements in sand layer

In general it is concluded that design criteria are met inside the pillars and within the sand layer down to elevation of –11.5 m.

For the purpose of checking the differential settlement criteria for sand layer, finite element analyses are also realised by Menard.

6.3 *Settlement Plates and Related Evaluations*

In order to observe the settlements throughout the site, settlement plates were installed at 18 different points.

Analyses are done to estimate the maximum amount of settlements utilising the results of settlement plates by Asaoka (1978) method. Expected settlements are in the order of 20 cm to 30 cm. Preliminary settlement analyses realised for fill loading were given in Table 2. The estimated average ultimate settlement value is in the order of 32 cm. The settlement plates were installed one month after the construction of first 1.5 m high fill. Therefore results of preliminary settlement analyses realised using the CPT data are in good accordance with the Asaoka analyses.

7 ADDITIONAL LIQUEFACTION ANALYSES USING CPT AND SPT DATA

Additional liquefaction assessments are realised towards the application of the methodology defined by Robertson and Fear (1997) utilising both SPT and CPT data. For this purpose, data from CPT and SPT tests performed between pillars and at the pillar locations following the improvement are used.

The analyses are realised for an earthquake magnitude of M=6.3 and lateral earthquake acceleration of a=0.3g. For the calculation of final factor of safety against liquefaction, magnitude scaling factors suggested by Idriss are also applied. For the CPT analyses, the data from the tests realised between the pillars are utilised. The estimated factor of safety values against liquefaction within the sand layer existing between the depths of 4.0 m and 10.0 m are greater than one in general showing the effectiveness of the soil improvement application. There are levels with factor of safety values less than one between the depths of 10.0 m and 12.0 m at certain CPT locations. However taking into account the overburden effect and the fact that more than 80 per cent of the liquefaction data reported in the literature, belong to the levels within the first 10 m from the ground surface, the levels below 10.0 m depth are considered unlikely to liquefy during the earthquake loading.

In addition to the CPTs, SPTs in between and at the pillar locations were realised and liquefaction analyses are performed using these SPT data (Seed et al.1985). The main purpose of these additional analyses is to compare the results both from SPT and CPT approaches, for the future evaluation of liquefaction for soil improvement works since the application of CPT approach is relatively new compared to SPT approach. The obtained safety factors are variable between 1.0 and 8.0 at the pillar locations giving an average value of 3.0. This average value reduces down to 1.5 to 2.0 for the tests realised between the pillars. Considering that CPTs were also realised in between the pillars, it could be concluded that two methods namely SPT and CPT provided good agreement for this site. The factor of safety values obtained using the SPT/N values, for pillar location and in between pillars, are plotted on the same graph for the purpose of comparison.

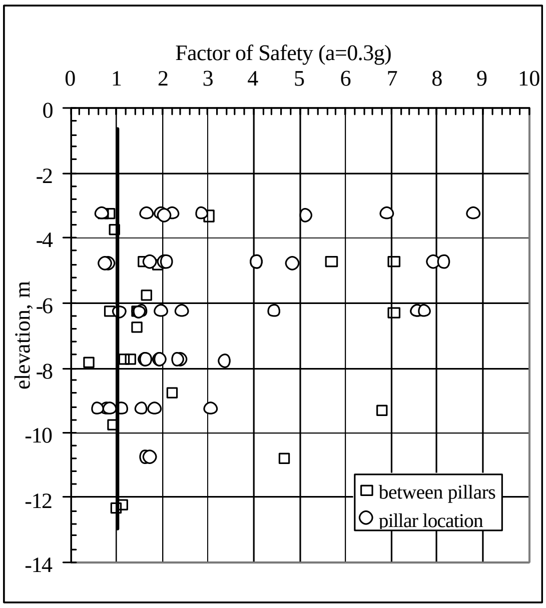

Figure 5. Safety factor values against liquefaction using SPT/N values

8 CONCLUDING REMARKS

Case study of Carrefoursa Commercial Centre construction site in Izmir, is presented where dynamic replacement is applied as the soil improvement method for the first time in Turkey. The Carrefoursa site is located within the first degree earthquake zone with poor ground conditions and shallow ground water table. The improvement scheme was designed both for bearing support under foundations decreasing the post-construction settlement of structures and fill, and for the mitigation of liquefaction risk. The pilot zone tests and subsequent quality control and quality assurance tests consisted of boreholes, pressuremeter testing, cone penetration testing (CPT), Standard penetration testing (SPT), heave and penetration tests, level monitoring and settlement observations.

Liquefaction assessments were realised towards the application of the methodology defined by Robertson and Fear (1997) utilising both SPT and CPT data. The main purpose of these analyses is to compare the results both from SPT and CPT approaches, for the future evaluation of liquefaction risk for soil improvement works since the CPT approach is relatively recent in the geotechnical literature, compared to SPT approach. It was concluded that two methods namely SPT and CPT, resulted in very consistent findings for this site.

Combining the CPTs and SPT borings provided considerably more valuable information for evaluating liquefaction potential than is obtained by either in-situ testing method alone. CPTs identified SPT blow counts affected by overlying or underlying adjacent strata, and enabled more reliable characterisation of relatively thin liquefiable strata. SPT samples however were essential for reliable interpretation of fines content of the analysed soil layers.

9 ACKNOWLEDGMENTS

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