Liquefaction Risk Mitigation — Manchester Airport

J. M. Sobol
Hayward Baker, Inc., Palatine, Illinois

J. I. Baez
Hayward Baker, Inc., Palatine, Illinois

F. J. Swekosky
Hayward Baker, Inc., Palatine, Illinois

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 14.05

Liquefaction Risk Management - Manchester Airport

J. M. Sobol, J. I. Baez and F. J. Swekosky
Hayward Baker, Inc., Palatine, Illinois

SYNOPSIS Densification of loose sandy soil by Vibroflotation was designed and constructed to mitigate the risk of seismically-induced liquefaction for the proposed 15,000 square meter terminal building. The analyses of the geotechnical data and the design of the densification based upon specified parameters is reported. Field installation methods and post-densification results are discussed.

INTRODUCTION

Seed et al. have proposed methodology to determine the factor of safety (FS) against the occurrence of liquefaction. Tokimatsu and Seed's (1987) correlation of SPT values to volumetric strain related to data developed in the Geotechnical Investigation showed that liquefaction-related settlements would be excessive for both life safety considerations and building damage potential. Few projects have been reported in which field data has been analyzed utilizing this conventional theory; then designed, constructed, and verified to the specified criteria. In this paper, we describe one such case history in which the analyses, design, construction, and verification have been successfully documented.

LIQUEFACTION POTENTIAL

Design phase borings at the Manchester (NH) Airport revealed a subsurface profile (Figure 1) of delta-deposited, clean, uniformly graded, saturated, fine to medium sands of loose relative density from depths of 3.7 to 13.7 m. Laboratory gradation and Standard Penetration Tests (SPT) established the potential for seismically-induced liquefaction. Analysis performed in accordance with Seed et al. (1985) indicated a factor of safety (FS) against liquefaction of less than unity under regional seismic design criteria. Liquefaction-related settlements were estimated as proposed by Tokimatsu and Seed (1987) considering cyclic stress ratio (CSR) and in situ SPT (Figure 2). A volumetric strain of 10% of the layer thickness was translated into a potential settlement of 0.3 m below the building footprint.

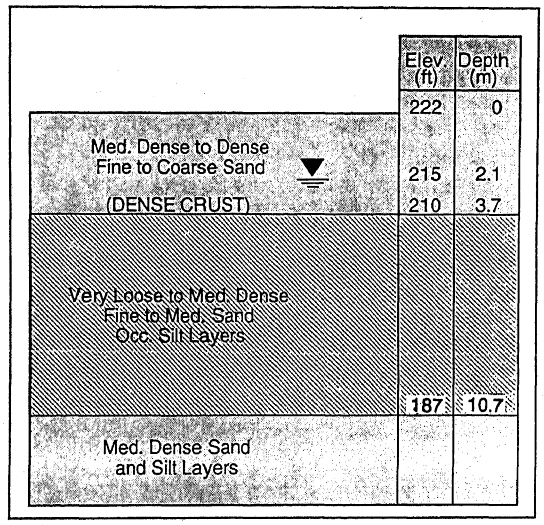


Figure 1: General Subsurface Profile

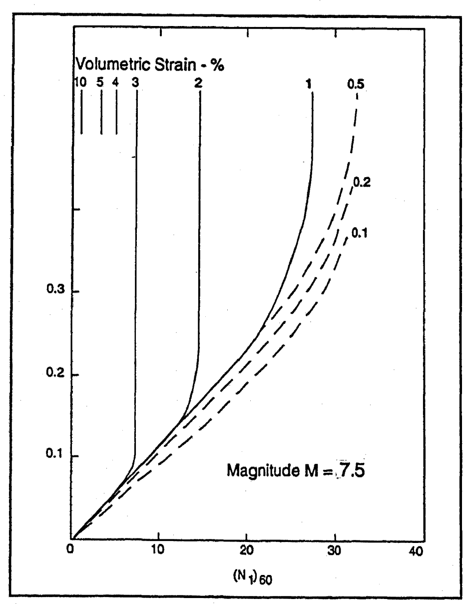


Figure 2: Relationship between CSR, Modified Blow Count and Volumetric Strain for M=7.5 (after Tokimatsu and Seed, 1987)

RISK MITIGATION ALTERNATIVES

To mitigate the risk of liquefaction, the following subgrade preparation/foundation design alternatives were considered:

A. Shallow Foundations
   1. Vibroflotation
   2. Excavation and Replacement
   3. Deep Dynamic Compaction
   4. Deep Blasting
   5. Compaction Grouting

B. Deep Foundations
   1. Piles
   2. Drilled Piers
   3. Pressure Injected Footings

Deep foundation alternatives were eliminated from consideration due to costs and uncertainty of performance during a liquefaction event. Excavation and replacement with dewatering was considered expensive. Dynamic compaction and deep blasting methods presented concerns regarding transmission of vibrations to surrounding properties and uncertainties as to their overall effectiveness to achieve the design D_R criteria. Vibroflotation was selected as a cost-effective soil improvement technique due to its well-documented success in densification of sands and its ability to meet construction schedule requirements.

VIBROFLOTATION

Vibroflotation is used for in situ densification of loose sands to depths of as much as 35 m below surface level. The granular soils are rearranged into a dense condition under influence of specially designed downhole vibrators. The action of the vibrator, often accompanied by water jetting, reduces the intergranular forces between the soil particles, allowing them to move into a more compact configuration. Compaction takes place without setting up internal stresses in the soil, thus ensuring permanent densification, which is achieved above and below the water table.

The vibrator is inserted into the ground to the maximum depth requiring densification and the soil is compacted in lifts from the bottom up. As the soils become more dense, a crater may be allowed to form at the surface around the vibrator, giving visual evidence of the effectiveness of the compaction process. The diameter of influence of the vibrator is up to 4.3 m.

PERFORMANCE CRITERIA/DESIGN SUBMITTAL

Qualified soil improvement specialty subcontractors were identified prior to advertisement of bids to general earthwork contractors. Project specifications required the selected specialty subcontractor to prepare a vibro design to meet the following seismic criteria:

Design Earthquake Magnitude (M) (Richter Scale)	=	6.0
Peak Ground Acceleration (a_g)	=	0.12 g
Minimum FS Against Liquefaction	=	2.0
Allowable Differential Settlement	=	12 mm.
Allowable Total Settlement	=	25 mm.

The design submittal summarized several references regarding evaluation of liquefaction potential and threshold relative density/SPT levels required to theoretically preclude liquefaction. To achieve a design FS = 2.0, the base peak ground acceleration was doubled from 0.12g to 0.24g. Cyclic stress ratios (CSR) were then determined as a function of depth based upon a_g = 0.24g and an appropriate scaling factor of 1.32 (Tokimatsu and Seed, 1987) to translate from M = 7.5 to M = 6.0. This effective CSR was then utilized in Figure 3 (Seed et al. 1985) to determine a design $(N_1)_{60}$ value as a function of depth and fines content for FS = 2.0. To simplify the quality assurance program, the $(N_1)_{60}$ values were converted to uncorrected N-values. The composite soil improvement criteria is developed in Table 1 and plotted in Figure 4.

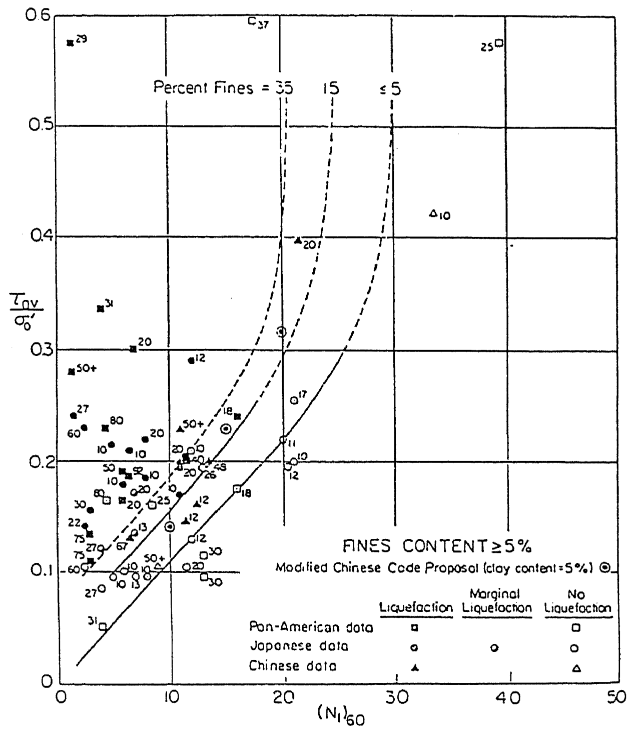


Figure 3. Relationships between stress ratios causing liquefaction and N1-values for silty sands for M=7-1/2 earthquakes (after Seed, Tokimatsu, Harder, and Chung, 1985)

Design phase borings identified potentially liquefiable deposits to depths of 13.7 m. Actual treatment depth selection was based upon studies of several Japanese sites (Ishihara, 1985) where liquefaction has occurred in past earthquakes. These document conditions that relate damaging ground effects (sand boiling or surface cracking) to soil stratigraphy. The relationship between the thickness H_1 of a non-liquefiable surface layer and the thickness H_2 of the underlying potentially liquefiable layer and the likelihood of surface manifestation for peak ground acceleration is shown in Figure 5.

Lateral treatment distance beyond the building exterior was determined to be 4.6 m based upon a 30 degree angle from the vertical extending from the footing grade to the treatment depth. This was based upon experimental Japanese data (Iai et al. 1988) and a survey of liquefaction damage by Mitchell and Wentz (1991) after the 1989 Loma Prieta earthquake.

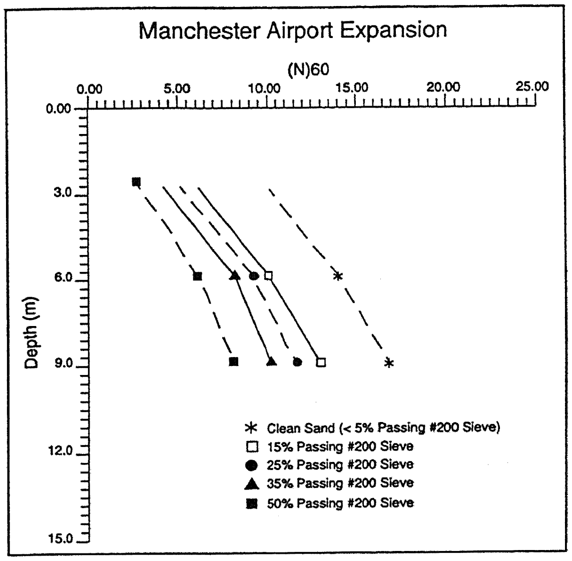


Figure 4: Composite Improvement Criteria

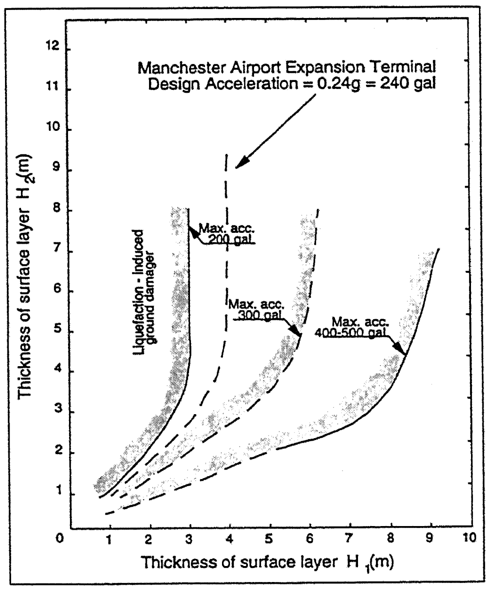


Figure 5: Proposed boundary curves for site identification of liquefaction induced damage (after Ishihara, 1985)

VIBROFLOTATION SITE PROGRAM

More than 2,600 compaction centers were installed to 8 and 11 m depths. A 3 m grid produced the minimum specified D_R in the areas and to the depths where coarse clean sand was present. The compaction grid spacing was contracted to 2 m where post-vibro SPT's and/or the performance of the vibrator indicated that loose D_R conditions remained.

The specialty contractor maintained the elevation of the working platform by providing 17,000 cubic meters of backfill sand and gravel borrow to compensate for the reduction in volume of the densified sand. Phase I of the project was completed within four double-shift weeks with two operating vibrators. Phase II required two weeks operation with a single shift/vibrator operation.

QUALITY ASSURANCE

A total of 34 post-vibro test borings were made typically at 900 square meter intervals at the centroid of compaction points to assess the vibro program. Selection of test boring locations was based upon relative amperage readings recorded for each vibro probe. In general, compliance with project specifications was achieved after initial treatment. Several SPT values at or slightly below specified values were found at depths ranging from 3.7 m to 5.2 m. This condition was apparently caused by the presence of the dense crust of coarse sand overlying the loose, finer sand deposit. It is theorized that the crust was temporarily arching over the loose material below, thus minimizing the effective overburden (confinement) stress, resulting in low N values. Eight (8) borings were subsequently performed in areas where N values were below specified values, in this depth zone, after waiting a period of one (1) to three (3) weeks. In most instances, N values increased after a waiting period to the above specified criteria. Figure 6 depicts adjacent sets of pre-vibro and post-vibro SPT data in terms of depth versus $(N)_{60}$.

TABLE 1

Depth Depth Meter	Total Stress Kg/CM^2	Effective Stress Kg/CM^2	CSR FS=2	CN	$(N_1)_{60}$* 5%	$(N)_{60}$	$(N_1)_{60}$* 15%	$(N)_{60}$	$(N_1)_{60}$* 25%	$(N)_{60}$	$(N_1)_{60}$* 35%	$(N)_{60}$	$(N_1)_{60}$* 50%	$(N)_{60}$
3.0	.61	.52	0.18	1.35	12.3	10	8.1	6	7.3	5.5	6.5	5	4.0	3
6.1	1.15	.75	0.22	1.14	15.3	14	11.0	10	9.7	9.0	8.4	8	6.3	6
9.2	1.68	.98	0.24	1.0	16.3	17	12.2	13	10.9	11.5	9.5	10	7.7	8

*Sub-number 200 sieve fraction.

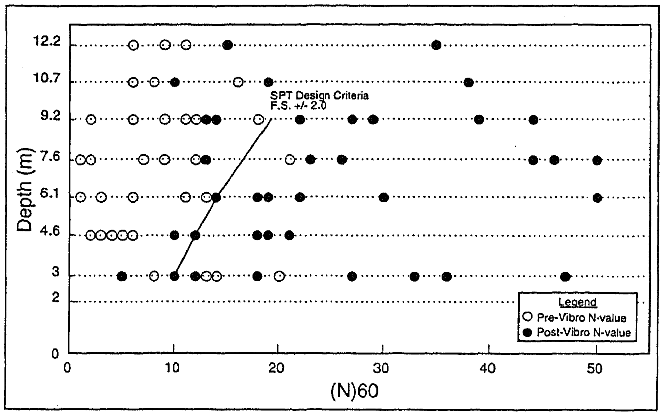


Figure 6: Selected Sand Densification Data
SPT vs. Depth

DENSIFICATION BELOW TREATMENT DEPTH

Comparison of pre- and post-vibro SPT and grain size distribution test results have revealed strong evidence that relative densities were significantly increased in sands and non-plastic silts below the treatment depth. Generally, Vibroflotation is considered effective for granular soils with less than 20% passing the No. 200 sieve to the treatment depth. However, increased relative density was documented on soils with up to 90% fines and to 2 m below the treatment depth.

DENSIFICATION OF SILTY SOILS

Mechanical sieve, hydrometer and Atterberg limit testing performed on silty soil samples from post-vibro borings have revealed these soils to be uniformly graded (C_u between 2.5 and 4.3) and non-plastic. Successful densification of the silty soil by the vibratory action is consistent with Chinese data as presented in Seed et al. (1983) which propose liquefaction vulnerability of fine-grained soils to be based in part upon the following guidelines:

Per cent finer than 0.005 mm.	< 15%
Liquid limit	< 35
Water content	> (0.9) LL

Table 2 presents selected field and laboratory test data to support increased relative density of the uniformly graded, non-plastic silts.

Microphotographs of non-plastic silt size fractions from selected post-vibro boring samples indicate granular texture of the fine-grained soils as shown in Figure 7 and were successfully densified by vibratory action.

SUMMARY

Practical methods to assess the potential for the occurrence of liquefaction and to develop project specifications for risk mitigation are presented for the design and construction of a public facility. The analyses and design methods pioneered by Seed and others have been summarized and adapted for regional seismicity. Selection of seismic design parameters and a prudent factor of safety must be critically assessed for each project. A reliable densification method was chosen based upon subsurface conditions, construction schedule, site constraints, and economy. Verification by standard exploratory methods can assure responsible design criteria have been met.

ACKNOWLEDGEMENTS

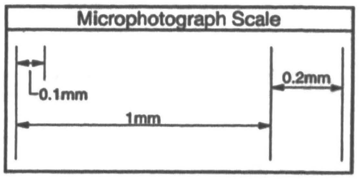
The authors wish to thank the Manchester Airport Authority; O'Brien-Kreitzberg & Associates, Inc. (Program Manager); Hoyle, Tanner & Associates, Inc. (Design Civil Engineers); and the University of New Hampshire Geology Department for their cooperation in preparation of this paper.

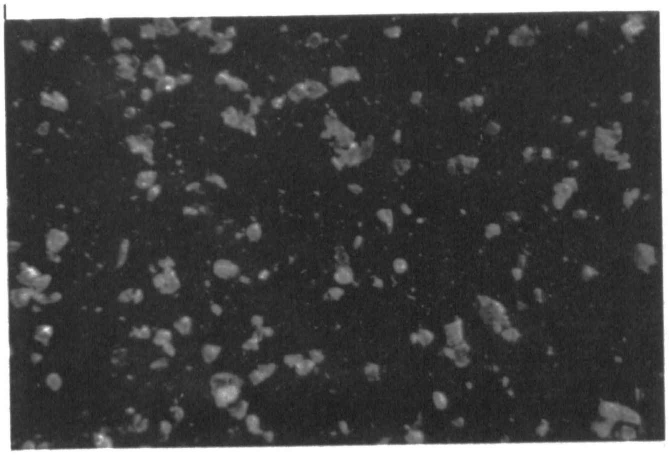
TABLE 2

Pre-compaction Design Boring No.	Sample Depth Meters	$(N_1)_{60}$	% Fines	Post Compaction SPT Test No.	Sample Depth Meters	$(N_1)_{60}$	% Fines	% Clay Size	Coef. of Uniformity (Cu)	Rel. Density Initial	Rel. Density Final	Liquid Limit	Plasticity Index
B5	12.2	14	86	B-122	12.2	33	90	1	3.6	58	≥80	25	0
B11	10.7	11	40	B-117	10.7	27	38	1	4.3	50	75	28	0
B14	10.7	14	61	B-117	10.7	27	38	1	4.3	58	75	28	0
B12	9.2	12	No sample recovered	B-118	9.2	39	55	1	2.5	50	≥80	28	0
B14	9.2	6	97	B-118	9.2	39	55	1	2.5	38	≥80	28	0
B24	9.2	11	No sample recovered	B-504	9.2	22	53	1	3.0	50	70	26	0

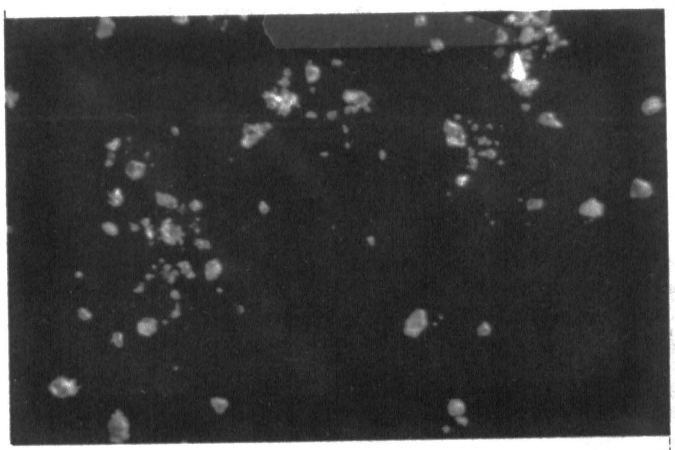
*Treatment depth at locations adjacent to these post-compaction borings = 9.2 meters.

Figure 7:

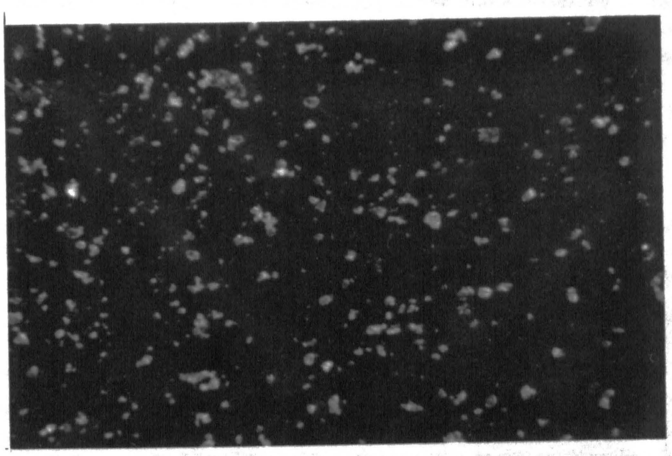




Boring No. B-118 — Depth = 9.2 meters



Boring No. B-504 — Depth = 9.2 meters



Boring No. B-122 — Depth = 12.2 meters

REFERENCES

Iai, S., S. Noda, and H. Tsuchida, (1988). Basic Considerations for Designing the Area of the Ground Compaction as a Remedial Measure Against Liquefaction.

Ishihara, K., (1985). Stability of Natural Deposits During Earthquakes, Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, A.A. Balkema Publishers, Rotterdam, Netherlands.

Mitchell, J. K., and F. J. Wentz, (1991). Performance of Improved Ground During the Loma Prieta Earthquake. UCB/ERRC report 91/12.

Seed, H. B. and I. M. Idriss, (1971). Simplified Procedures for Evaluating Soil Liquefaction Potential, Journal of Soil Mechanics and Found. Div. ASCE, Vol. 97.

Seed, H. B., I. M. Idriss, and I. Arango, (1983). Evaluation of Liquefaction Potential Using Field Performance Data, Journal of Geotechnical Engineering, ASCE, Vol. 109. No. 3, pp. 458-482.

Seed, H. B., K. Tokimatsu, L. F. Harder, and R. M. Chung, (1985). Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, Journal of Geotechnical Engineering, ASCE, Vol. 101, No. 12, pp. 1425-1445.

Tokimatsu, K. and H. B. Seed (1987). Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.