DYNAMIC COMPACTION DENSIFICATION FOR LIQUEFACTION MITIGATION AND IMPROVED FOUNDATION SUPPORT IN THE FRASER DELTA - A CASE HISTORY

by

Ernest Naesgaard[1] and Nelson Beaton[2]

ABSTRACT

A typical Fraser River delta surficial soil profile consists of 3 to 5 metre thickness of stiff to soft clay/silt crust over 15 to 20 metre thickness of loose to dense sands. The loose sands may liquefy during the design earthquake and potentially large settlements and lateral deformations may result.

A procedure which mitigates these liquefaction effects and utilizes relatively economical dynamic compaction (DC) densification has been developed. This procedure consists of dewatering, excavating the clay/silt crust, replacing with sand, densifying with dynamic compaction, and founding the building on spread footing foundations.

This procedure was recently employed, for the first time in the Fraser delta, at the Kwantlen College New Richmond Campus site. Liquefaction resistance and densification requirements were assessed using static cone penetration test tip bearing (CPT Q_t) in accordance with Seed's liquefaction assessment procedure. Menard type pressuremeter tests were also used for quality control.

Extensive vibration monitoring was conducted. Measured vibrations on adjacent structures were below published criteria for damage to structures and cracking of dry-wall or plaster. However the vibrations were within the limits of human perception and were disturbing to some.

INTRODUCTION

The Fraser delta is located in south-western British Columbia, Canada. The delta covers an area of approximately 300 km^2 south of the City of Vancouver where the Fraser River empties into the Strait of Georgia. Seismic considerations are a key factor in foundation design in this area because of the potential for major earthquakes and soft/loose alluvial soil conditions. A typical delta surficial soil profile consists of 3 to 5 metre thickness of stiff to soft clay/silt crust over 15 to 20 metre thickness of loose to dense sands over considerable thickness of soft to firm interlayered fine sands, silts and silty clays. The relatively soft soils may amplify earthquake motions and significant portions of the sands may liquefy, resulting in potentially large settlements and lateral deformations (Byrne & Anderson, 1987;Fraser Delta Task Force, 1991).

The building code requires that foundations in the delta be designed to tolerate, without collapse, the large deformations and settlements which result from

[1]Senior Engineer, Macleod Geotechnical Ltd., West Vancouver, B.C.
[2]Vice President, Geopac West Ltd., New Westminster, B.C.

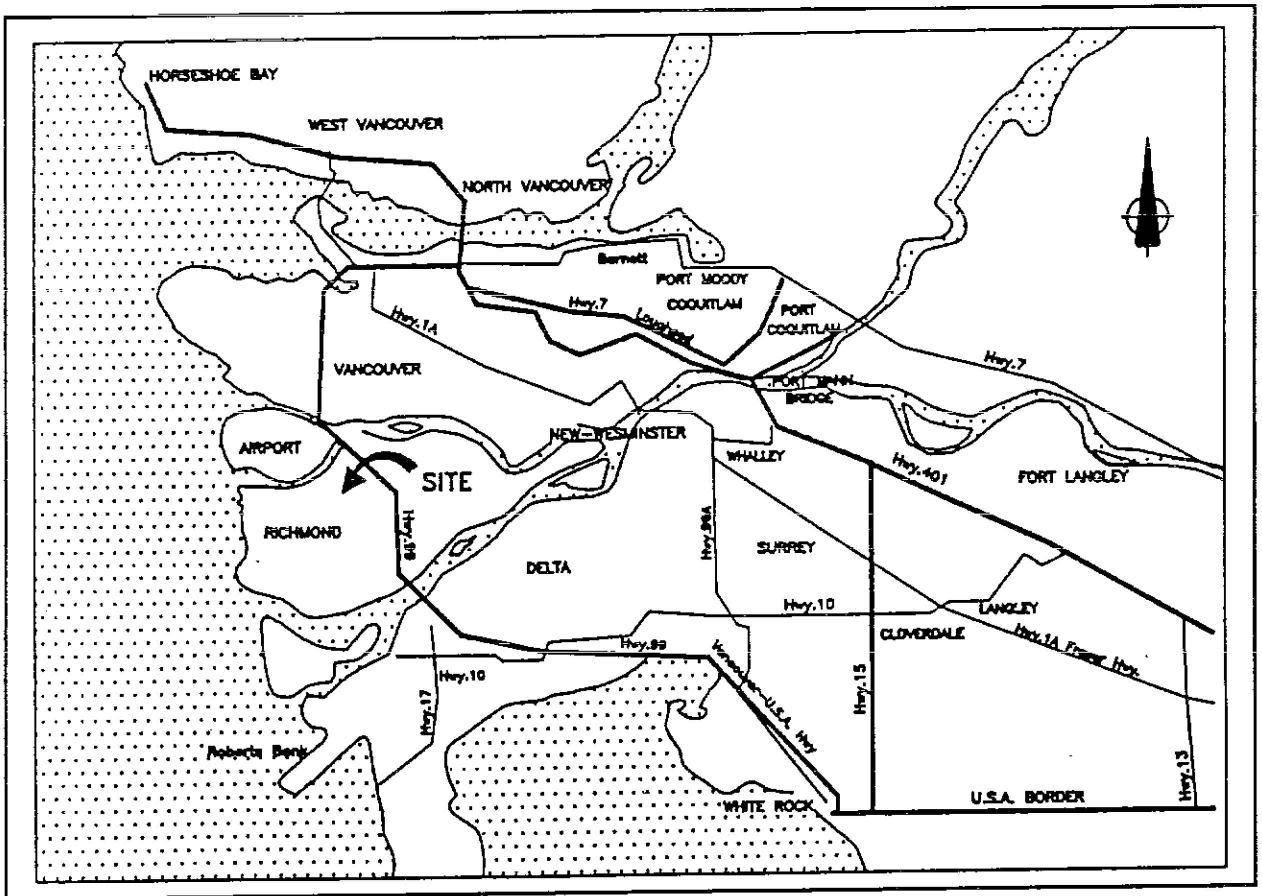

Figure 1 Location of subject site within the Fraser delta

liquefaction, or the ground must be modified to mitigate the liquefaction (NBCC, 1990). Light one to two storey buildings can be designed so they will not punch through the surficial clay/silt crust if the underlying soil liquefies; however, larger buildings with column loads in excess of 750 to 1000 kN may punch through the clay/silt into the liquefied soil. For these buildings the loose sands are often densified to mitigate the liquefaction problem. The most common procedure used to mitigate liquefaction is ground densification. One of the more economical procedures for densifying the ground is Dynamic Compaction (DC). Dynamic Compaction densification is conducted by repeated dropping of a 10 to 20 tonne weight from heights in the order of 15 to 25 m. The impacts both locally compress the ground and generate vibrations and increased pore pressures which result in rearrangement of soil particles into a more compact configuration. This paper describes a new site preparation approach for the Fraser delta, which was recently used for the new Kwantlen college Richmond campus. The procedure uses Dynamic Compaction densification for liquefaction remediation and for improved foundation support.

SITE / STRUCTURE

Figures 1 and 2 show the Fraser delta and the Kwantlen College site location on Lulu Island, Richmond. The site adjoins a large shopping mall complex to the west and multiple residential developments to the east and south. The site is

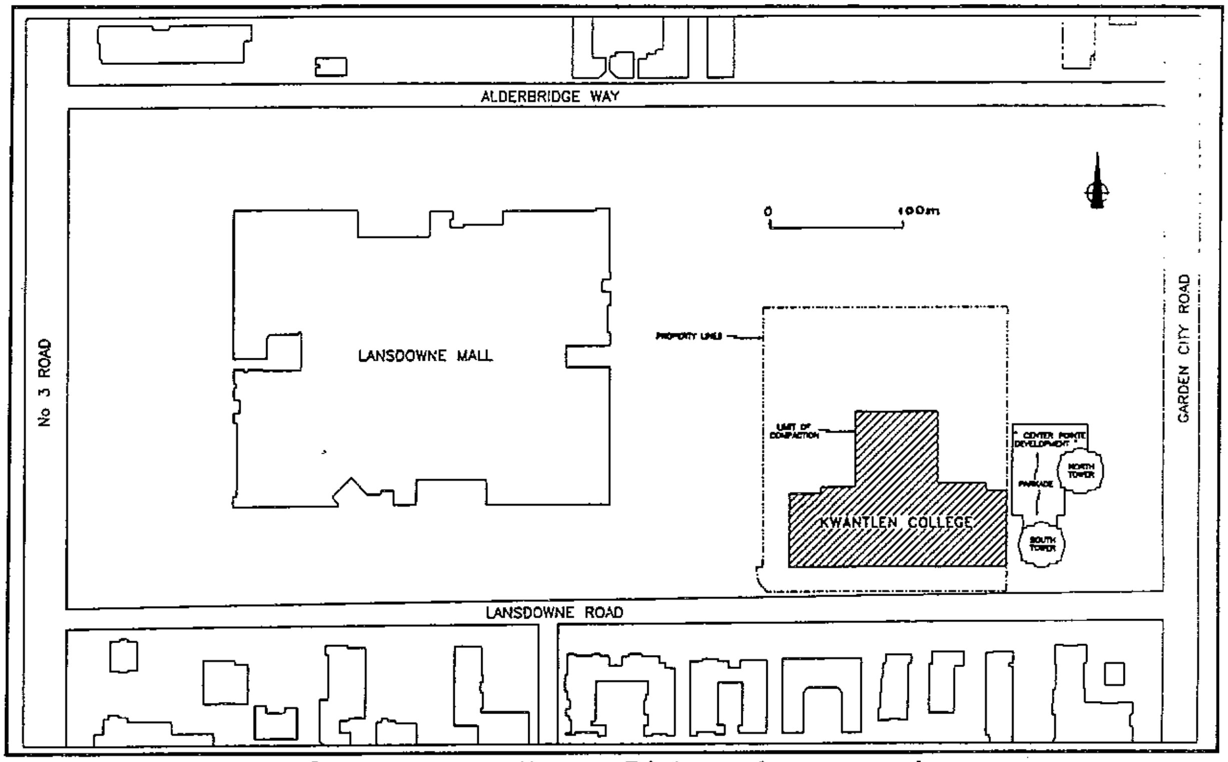

Figure 2 Location of Kwantlen College - Richmond campus site

relatively flat with a grade of approximately 1.5 metres above sea level and a water table at approximately 1.0 metre depth.

The college building is a three storey reinforced concrete and steel structure with column loads of approximately 2000 kN and a footprint of 10,000 m^2. The local seismicity, soil profile and geology of the site is described in more detail by Naesgaard et al., 1992.

SITE PREPARATION / FOUNDATION DESIGN

The site preparation at the Kwantlen site involved installing temporary dewatering, removal of the clay/silt crust, filling with "river" sand, and densification to 10m depth with Dynamic Compaction. This approach reduced the risk of liquefaction, provided a compact bearing surface, and allowed the use of conventional spread footing foundations with slabs-on-grade. Considerable savings were achieved over the alternative vibro-replacement densification and expanded-base-pile foundation design.

The site preparation program is schematically illustrated on figure 3. In addition to the ground improvement, 82 one metre diameter gravel drains were installed at predetermined locations within and around the foundation footprint to reduce the migration of earthquake-induced pore pressures into the densified foundation envelope. These drains were installed using vibro-replacement method.

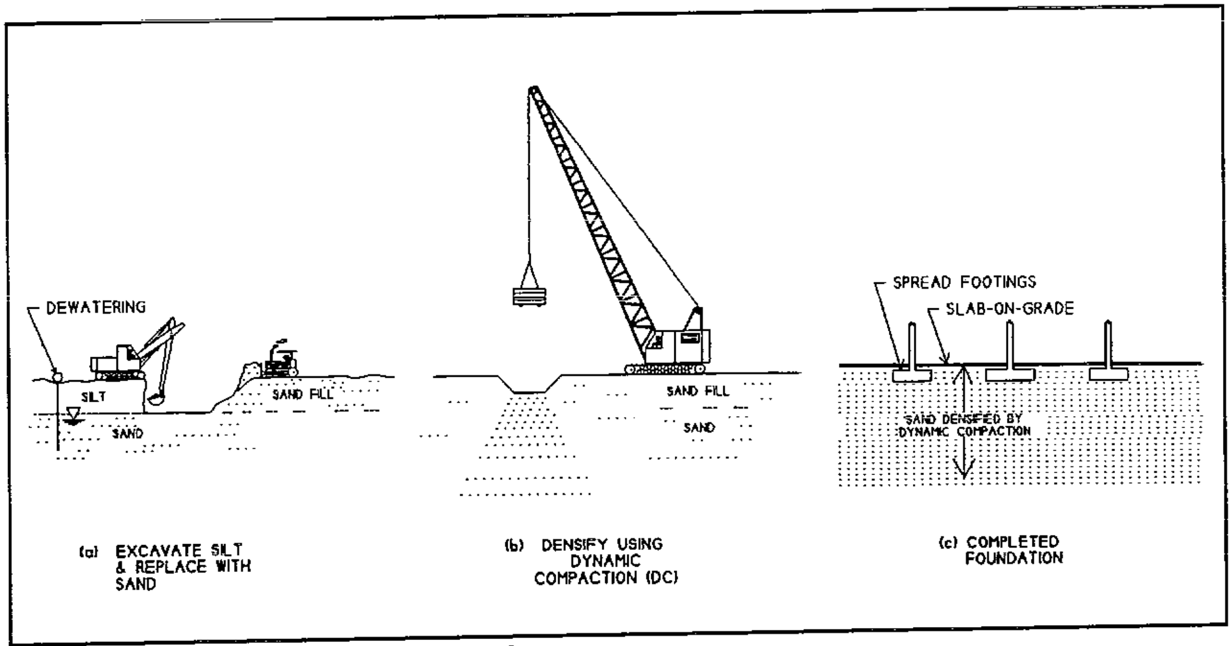

Figure 3 Site preparation procedures

SEISMIC DESIGN CONSIDERATIONS

Liquefaction resistance was assessed using the procedure by Seed and De Alba, 1986, and static cone penetration test (CPT) tip bearing. For design an earthquake magnitude of 7 with a hard ground maximum acceleration of 0.21g was used (NBCC, Sy et al., 1991). Because of the soft soil conditions the hard ground acceleration was assumed to be amplified by a factor of 1.33 to give a surface acceleration of 0.28g (Fraser Delta Task Force, 1991; Sy et al., 1991). A typical soil profile and calculated factor of safety against liquefaction prior to densification are shown on figure 4. The bulk of the liquefiable soils were in the upper 10 to 12 metres. However, there were local zones below this that potentially could also liquefy.

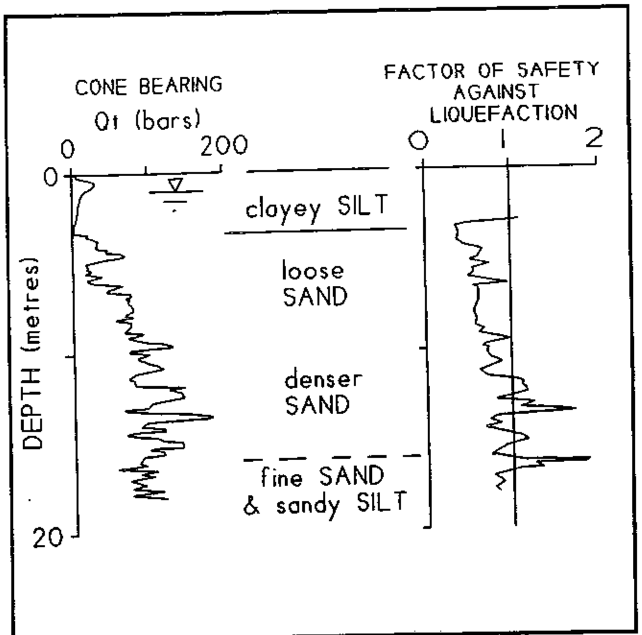

Figure 4 Typical before treatment soil profile and calculated factor of safety against liquefaction

The thickness of densified layer was selected to avoid bearing failure and to reduce post earthquake settlement to tolerable values. Local liquefaction was allowed below the densified zone and potentially would result in minor post earthquake settlement. With the design densification depth of 10 metres, the amount of settlement was estimated to be less than the 50mm design limit. The procedure by Tokimatsu and Seed, 1987 was used to calculate this settlements.

The removal of the clay/silt crust and the densification was extended approximately 7.5m beyond the perimeter of the building foundations. With this 7.5m extension beyond the foundation, the major concentration of the foundation stresses would be within the densified soil. The interface between the densified zone and the adjacent liquefied zone is also an area of potential ground rupture. It is therefore desirable to extend the densification beyond the building footprint to avoid ground rupture within the foundation area.

DYNAMIC COMPACTION DENSIFICATION

A performance criteria based on the CPT Q_t tip bearing and fines content was used for specifying the required densification (figure 5). The fines content was determined by sampling as necessary following the CPT. Improvement was specified to elevation -8.5m (10m depth). The Dynamic Compaction program (ie. size of

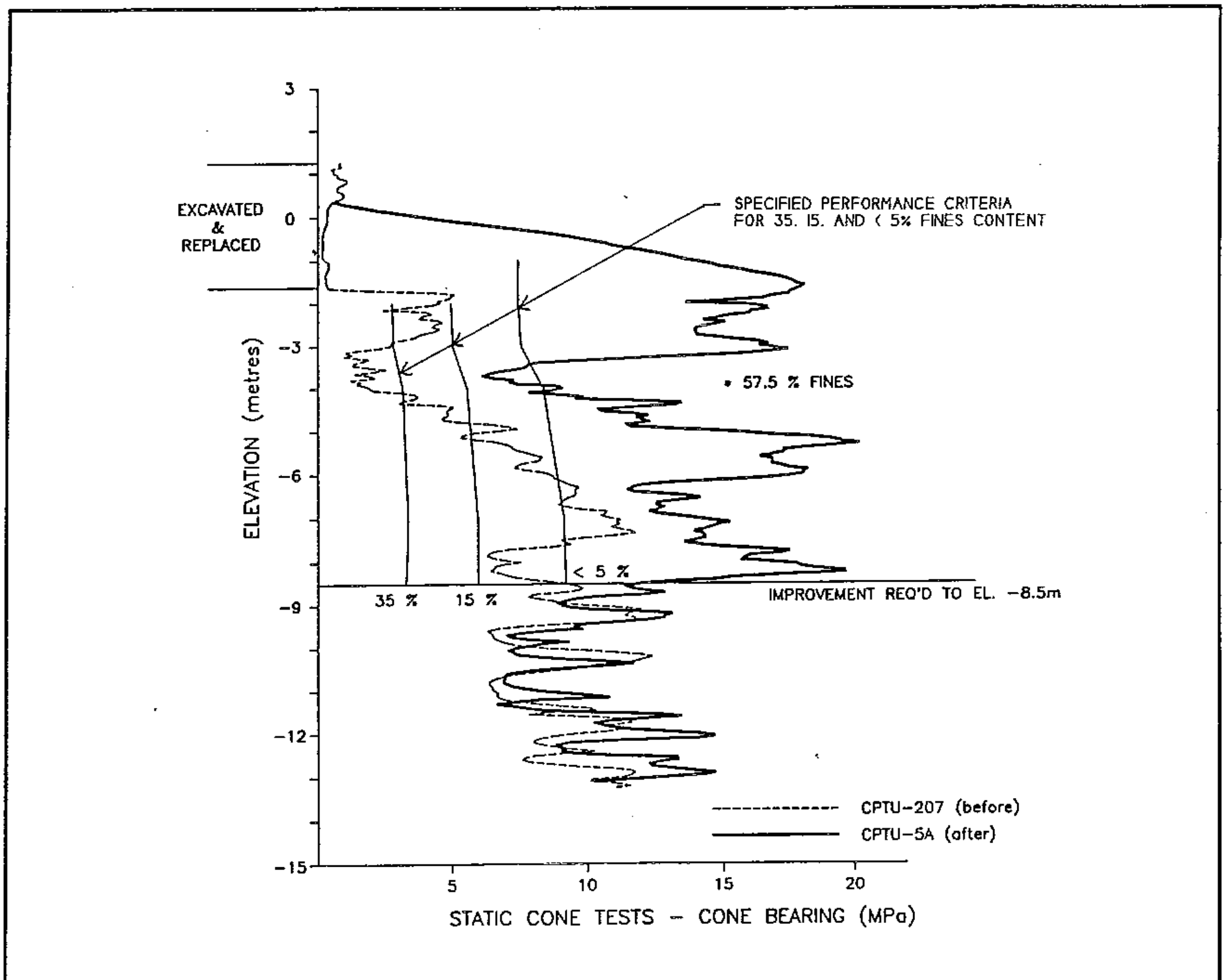

Figure 5 Specified Dynamic Compaction performance criteria and typical before and after CPT Q_t results

weight, drop height, number of drops, and grid spacing) was determined by the ground improvement contractor to achieve the specified criteria. A 2m diameter 15-tonne steel weight and 23m drop height was selected.

Densification was conducted in three high-energy phases followed by a low-energy ironing phase. The first phase consisted of compaction points on a 10m square grid (25 to 35 impacts per grid point), the second phase was on a 10m grid centred between the first phase points (20 to 25 impacts per grid point), and the third phase was a staggered double 10m grid located at points equidistant from the first two phases (5 to 15 impacts per grid point). The ironing phase consisted of a contiguous pattern of impacts using a 13 tonne low energy tamper and a 10 to 15m drop height. Upon completion of the Dynamic Compaction, the upper surface soils were compacted using a large vibratory roller.

Dynamic Compaction work commenced in the north section of the site concurrent with completion of excavation and replacement operations in the south. The contractor treated this area as a test section to assess the level of soil improvement achieved and to establish an efficient compaction design for the overall site. The total applied energy during the production work varied from 365 to 225 tonne metre/metre^2. The energy levels applied to a given area were determined by the depth and degree of improvement required. The lower energy levels were applied to areas where initial CPT test results delineated pre-treatment densities which met the specification in the lower portion of the densification envelope.

Typical before and after CPT Q_t are shown on figure 5. After CPT were located adjacent to before CPT, but between the DC compaction points. The after DC CPT friction ratio showed little or no change, but increased negative CPT pore pressure response was noted throughout the densified zone after treatment indicating increased soil dilatancy. Figure 6 gives the average before and after Q_t profile for the site based on 10 before and 10 after CPT tests and demonstrates significant improvement over the entire treatment depth. The level of uniformity and the degree of excess improvement indicated by figure 6 is related to the effects of averaging and is not representative of the individual CPT profiles which were more variable (see figure 5) as would be expected in an interlayered alluvial deposit. Figure 6 indicates that Dynamic Compaction had an average depth of influence of approximately 11m, which gives an α co-efficient of 0.6 for the empirical relationship (Dumas & Beaton, 1992)

$$d_{max} = \alpha \sqrt{M x H}$$

where d_{max} is the maximum depth of influence, M is the mass of the tamper in tonnes, and H is the height of drop in metres. The α co-efficient can vary from 0.3 to 0.85 depending on soil type and other site-specific factors (Dumas & Beaton, 1992). The 0.6 value represents the upper range for silty sands/sandy silts and the lower range for free-draining granular soils. Since the Kwantlen soils consist of interlayered soils ranging from clean sand to sandy silt, this α value agrees well with previous data.

The induced settlement generated by Dynamic Compaction over the densified footprint varied from 0.64m to 0.48m. This represents a volume reduction of 5 to 6% over the specified 10m treatment depth and compares well with the 5 to 7% volume reductions reported by Dumas and Beaton, 1992, for alluvial soils.

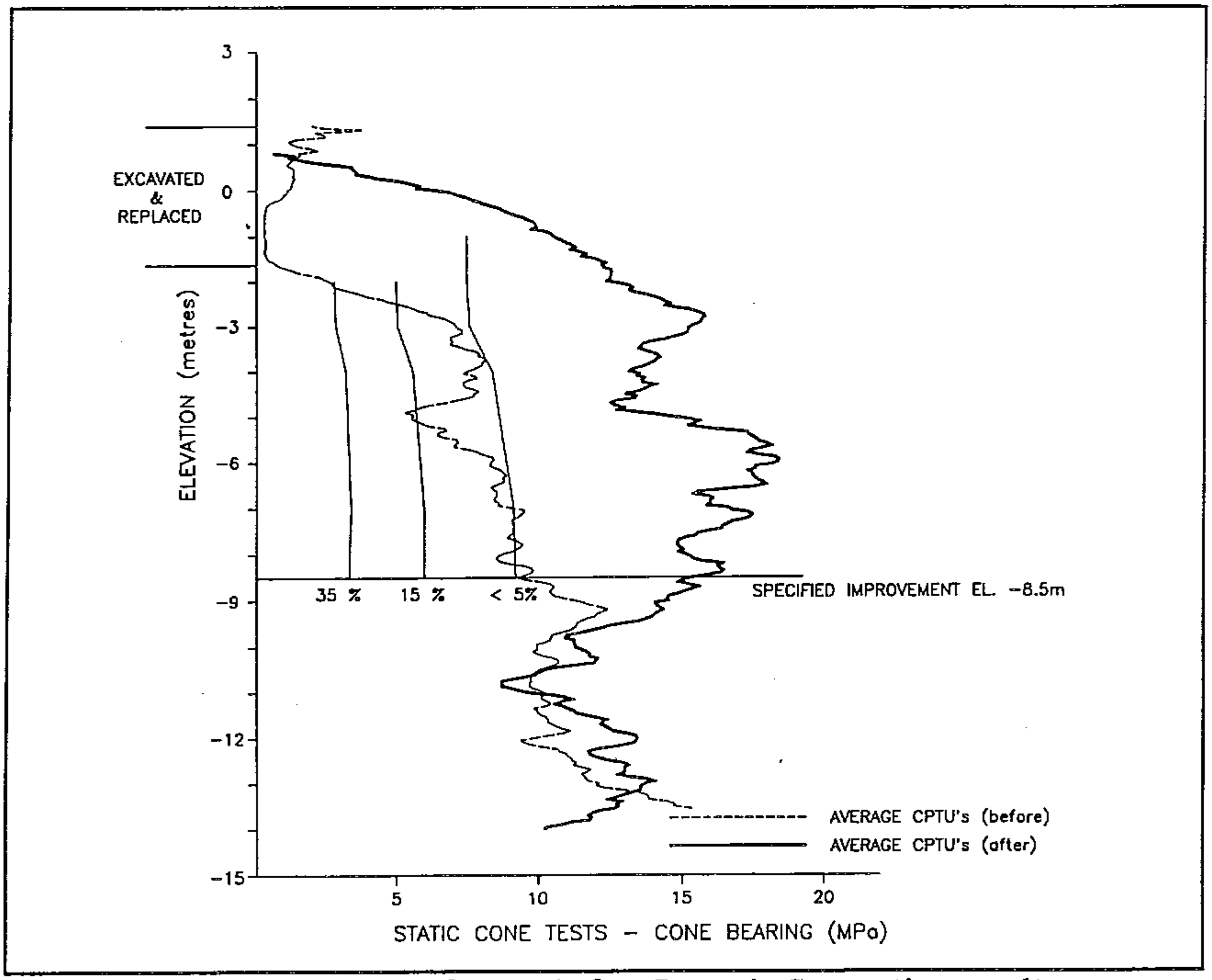

Figure 6 Averaged CPT Q_t before and after Dynamic Compaction results

PREDICTED FOUNDATION PERFORMANCE

In addition to the CPT testing program, the contractor carried out pressuremeter testing at eight (8) locations on the site using Menard type pressuremeter equipment. Settlement and bearing capacity calculations carried out using these data and the procedure by Menard, 1975, give allowable bearing capacities of between 450 and 630 kPa (using a factor of safety of 3 on ultimate) and predicted settlement of between 3mm and 7mm for the design bearing pressure of 200kPa and footings up to 3m x 2.4m size. No footing settlement data is available, however general foundation performance has been satisfactory.

EFFECTS ON ADJACENT DEVELOPMENTS

Potential concerns regarding the described site preparation procedure are the vibrations resulting from the Dynamic Compaction work and potential settlements related to water-table draw-down.

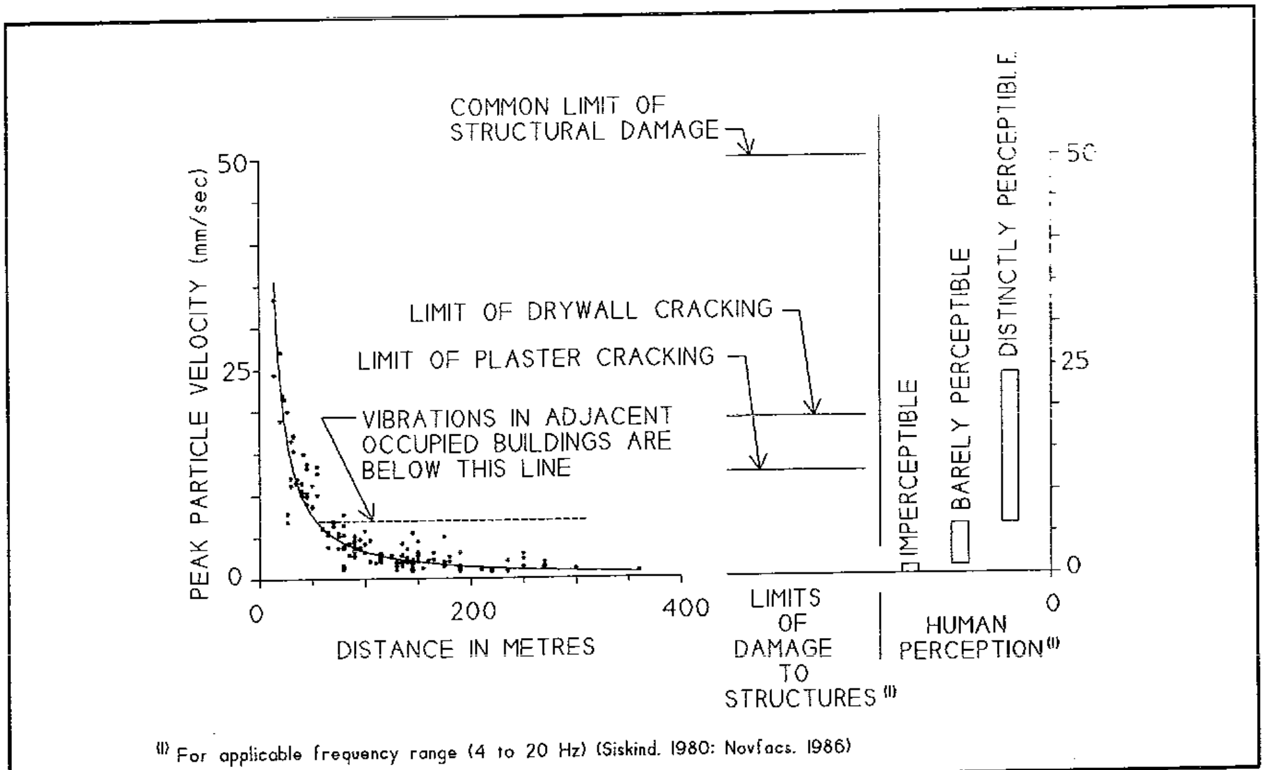

Figure 7 Attenuation of ground vibrations from point of impact and damage and perception thresholds

On this site, located in an urban setting, the vibrations were disturbing to some of the residents in adjacent apartment buildings to the south. This resulted in complaints and allegations of damage. Completion of the work was delayed to permit assessment of the claims. Data from vibration monitoring, tiltplate monitoring, crack movement monitoring, settlement survey elevations, and pre-construction survey photographs of adjacent buildings were reviewed. Analysis of the collected data indicated that the work was not adversely affecting the neighbouring properties. Following this technical review, the Dynamic Compaction work was completed.

Generally the peak particle velocity is used as an indicator of building damage potential and human response to vibrations (Siskind, 1980). The vibrations from the Dynamic Compaction had a predominant frequency in the range of 6 to 12 Hz and attenuated rapidly away from the drop point. Figure 7 shows measured maximum peak particle ground velocities with distance from the compaction source. Vibrations at the apartment structures south of Lansdowne Road were less than 7 mm/sec and generally in the order of 3 to 4 mm/sec. As indicated on figure 7 these levels are well below published levels which would cause structural damage to buildings and cosmetic cracking in plaster or drywall (Siskind, 1980). The vibrations are however within levels which can be easily felt by persons, with the higher levels being disturbing to some (NAVFAC DM 7.03, 1986).

Two reinforced concrete high-rise towers were under construction just 10m east of the site at the time of the Dynamic Compaction work. Interior finishing had not been completed, and peak particle velocities up to 33mm/sec were measured with no indication of damage. Compliance with a minimum 10m buffer zone for Dynamic Compaction operations necessitated that a small portion of the eastern compaction boundary be treated using vibro-replacement methods. The contractor carried out this work in conjunction with the installation of the gravel drains following completion of the Dynamic Compaction work. The maximum vibration measured at the high-rise during vibro-replacement operations was 2.5mm/sec.

Soil consolidation and settlement due to water-table draw-down can be a concern for adjacent services and structures. At this site, dewatering-induced settlements were expected to be minimal because the adjacent structures were pile supported and the clay/silt crust overconsolidated. Elevation surveys on adjacent Lansdowne Road and tiltmeter plate data on structures indicated no measurable settlement. Where necessary, recharge wells could be considered to mitigate the effects of water table draw-down.

CONCLUSIONS

The foundation approach described proved to be an economical, efficient and effective solution to the static and seismic design requirements of this Fraser delta project. Concerns regarding the application of Dynamic Compaction in urban areas and the consequences of large scale dewatering should be addressed when planning such projects. A well organized program including public notification and information, pre-construction condition surveys of adjacent structures and continuous monitoring during the work, should be considered an integral part of similar ground improvement applications.

Acknowledgements

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