WELLINGTON REGIONAL STADIUM
SITE ASSESSMENT

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SUMMARY

The Wellington Regional Stadium is one of a number of projects planned or under construction along Wellington's reclaimed waterfront. Each of these projects has had to consider ground stability with respect of liquefaction and lateral spreading, taking into account Wellington's relatively high seismic activity and the weak and variable nature of the reclaimed land.

The proposed stadium site straddles two distinct areas of reclamation; one of hydraulic fill and the other of end tipped gravel fill. The potential for liquefaction and lateral spreading of the site has been assessed. A relatively high potential for liquefaction of the hydraulic fill sands has been concluded. On the basis of an approach proposed by Newmark [4] a potential for upto 1 m of lateral movement of the site has been predicted under the design earthquake (600 to 800 year return period). Ground improvement options have been considered to mitigate these predicted movements. The formation of gravel columns by vibro-replacement has been concluded to be an appropriate form of ground improvement.

INTRODUCTION

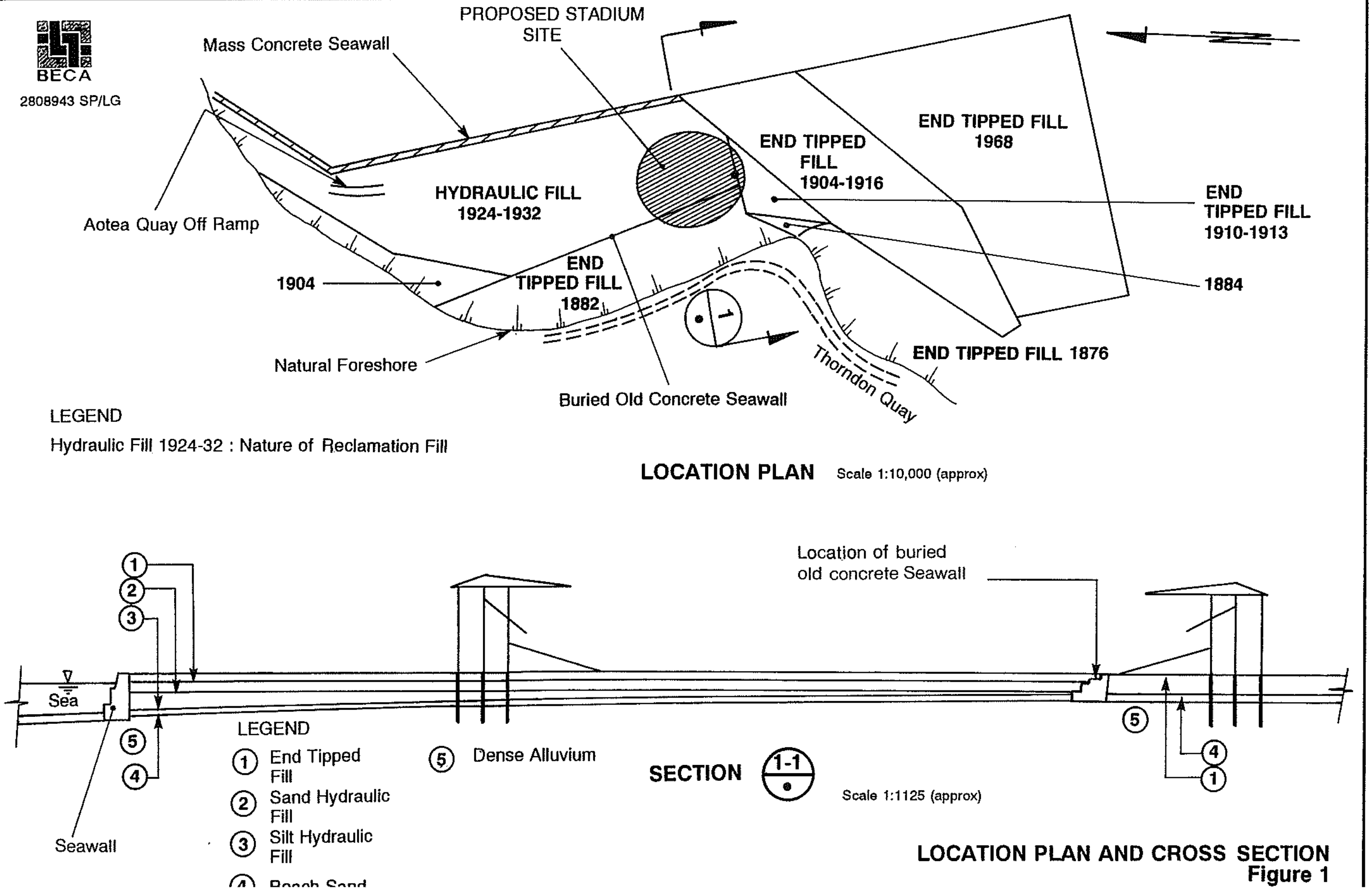
The boarder between Wellington's central business centre and the foreshore has been formerly occupied by railway and port facilities and carparks. Now with this land being surplus to railway and port requirements a unique opportunity is offered for its redevelopment, linking the city to the foreshore with public facilities and space. This waterfront land comprises reclamations varying from end tipped quarried rock to dredged and hydraulically placed marine silts. The reclamations were constructed in stages over a 100 year period. With these ground conditions and Wellington's relatively high seismic activity the redevelopment has presented geotechnical engineers with some special challenges. Each development has had to consider ground stability with respect to liquefaction and lateral spreading and develop appropriate ground improvement and/or foundation systems considering the specific site ground conditions and the importance and public risk associated with the development.

This paper discusses the site assessment and conceptual design of ground improvements for one of these Wellington waterfront developments; the Wellington Regional Stadium. The proposed stadium is to provide an outdoor sports and cultural venue with two levels of covered grandstand seating for 35,000 people.

SITE DESCRIPTION

The proposed stadium site is located on reclaimed land lying immediately to the west of the junction of Aotea Quay and Waterloo Quay, Wellington. The site is approximately level with a ground surface level 2.0 m to 2.5 m above mean sea level. Available information relating to the construction of the reclamation has been researched [2] and is summarised on Figure 1.

As can be seen from Figure 1, the majority of the site was reclaimed by a hydraulic filling operation during the period 1924 to 1932. The western and southern sections of the site were reclaimed by end tipping of rock fill during the periods 1882 and 1910 to 1913 respectively. The old concrete sea wall forming the boundary between the 1882 and 1924-32 reclamations remains buried beneath the site. The location of this old seawall has been determined from historical legal survey data and confirmed by excavation.



LOCATION PLAN AND CROSS SECTION
Figure 1

The reclamation fill beneath the site varies between 4 m and 7 m in thickness and compact beach sands and dense alluvial deposits underlie the fill. The old seabed surface dips beneath the site at a gradient of typically 5% to the east. A mass concrete wall of approximately 10 m in height forms the edge of the reclamation approximately 80 m to the east of the stadium site.

As indicated on Figure 1 there are two distinct areas of reclamation within the site. Namely the 1924 to 1932 hydraulic fill area and the earlier end tipped fill areas. Borehole investigations revealed fill materials within the hydraulic fill area to typically comprise 1 m thickness of compact gravel over 3 m thickness of loose sand over 0.5 m to 3.5 m thickness of very soft silt. The fill materials in the end tipped fill areas typically comprise 1 m thickness of compact gravel over 3 m to 4 m thickness of loose gravel.

The sites location is 500 m south-east of the Wellington fault.

LIQUEFACTION

The liquefaction potential of the site was assessed using empirical correlations developed by Seed [6] which relate normalised SPT blowcounts to ground accelerations causing liquefaction in the field.

The influence of particle size grading on liquefaction potential was also assessed for the various materials encountered at the site. In particular the silt content of the sands and the susceptibility of the loose gravels to liquefaction were considered. Work by Evans [3] has demonstrated by the way of triaxial tests that gravelly soils can be susceptible to liquefaction.

From these assessments it was concluded that the loose sand hydraulic fill could be expected to liquefy as a consequence of an earthquake with a 200 year return period (magnitude M=7.4, peak ground acceleration a max = 0.2g).

The end tipped gravel fill could be expected to temporarily lose strength as a result of pore water pressure build up initiated by the shaking of a major earthquake. Build up of pore water pressures in this material to the extent that it liquefies cannot be discounted but is considered unlikely. Such shaking could also be expected to cause up to 200 mm of settlement in the end tipped fill.

The cohesive nature of the silt hydraulic fill is such that liquefaction potential of this material is assessed to be low.

Due to the density of the beach sand, indicated by the SPT results, this soil is expected to have a relatively low liquefaction potential.

LATERAL SPREADING

Introduction

Lateral spreading is the permanent displacement of ground along a weak or liquefied soil layer toward a free edge, such as a reclamation edge or river, as a result of earthquake ground accelerations.

The phenomena of lateral spreading has been observed as a consequence of a number of recent major earthquakes including the 1987 Edgecombe, 1989 Loma Prieta and 1995 Kobe earthquakes.

The very soft and potentially liquefiable soil layers in the sites hydraulically filled areas provide weak layers along which lateral spreading could be expected to occur. It is postulated that severe earthquake shaking could cause the collapse of the seawall which forms the edge of the reclamation 80 m to the east of the stadium site. Lateral spreading type shear movements could then occur along a series of scallops extending inland, resulting in permanent lateral surface displacements reducing with distance from the collapsed seawall. The magnitudes of these potential displacements were predicted using the approach proposed by Newmark [4].

Method of Analysis

In the approach proposed by Newmark [4] the horizontal ground acceleration that reduces the factor of safety against sliding along critical shear plains to 1.0 is first assessed. This acceleration level is the threshold "yield" acceleration for that shear plain, beyond which accelerations will result in permanent displacements. Displacements are computed by integrating over the portion of the acceleration time history of the design earthquake where the applied ground acceleration exceeds the assessed threshold yield acceleration. Newmark [4] and later Ambraseys and Menu [1] analysed the acceleration time history of a number of earthquakes in the magnitude range of 6.6 to 7.3. For each of these acceleration time histories they computed maximum displacements for various ratios of yield acceleration to peak ground acceleration. From this analysis they prepared charts relating the acceleration ratio to maximum permanent displacement. The two cases of symmetrical resistance, or a level shear plane, and unsymmetrical resistance, or a sloping shear plane, were considered by these researchers. In the case of the unsymmetrical resistance it was assumed that the yield acceleration in the upslope direction was high such that no upslope movements occur. Charts were produced for both of these cases.

These charts were applied in assessing the potential displacements by lateral spreading at the proposed stadium site.

The Analysis and Conclusions

The undrained shear strength of the hydraulic fill silt was assessed on the basis of 75 mm diameter insitu shear vane tests to be 15 kPa. The residual undrained shear strength of the liquefied hydraulic fill sand was assessed by correlations with SPT results proposed by Seed and Hander [5] and Stark and Mesri [6]. These correlations suggest a residual undrained shear strength of 15 kPa.

Simple block stability analyses assuming these shear strengths gave yield accelerations of 0.1 g.

To be consistent with the design of the structure peak ground accelerations as specified by the Loadings Code NZS4203 [7] for both the ultimate limit state and the serviceability limit state were considered. The ultimate limit state design earthquake has a peak ground acceleration of 0.6 g (estimated to be a 600 to 800 year return period event for this site) while the serviceability limit state design earthquake has a peak ground acceleration of 0.1 g.

Calculating the yield to peak ground acceleration ratio for the ultimate limit state case and applying this to the Newmark [4] and Anbraseys and Menu [1] charts, the potential maximum permanent displacement was predicted for the hydraulic fill area of the site to be of the order of 1 m. Under the serviceability limit state earthquake liquefaction of the hydraulic fill sand and yielding along this plain would not be expected. The estimated yield acceleration for shear of the soft silt equalled the serviceability limit state case peak ground acceleration, thus any yielding of the soft silts in this case could be expected to be small (<25 mm).

In the end tipped fill reclamation areas the magnitude of lateral displacement would depend on the degree of pore water pressure build-up and weakening in the loose gravel layer. This layer would not be expected to become as weak as the silt and sand layers in the adjoining hydraulic fill area thus any lateral displacement of the end tipped fill could be expected to be less than that of the hydraulic fill. Differential lateral displacements along the boundary between hydraulic fill and end tipped fill (the buried seawall) could therefore be expected. These differentials could be up to the full displacement of the hydraulic fill (in the case of no yield of the end tipped fill).

Potential total and differential lateral ground movements of the order of 1 m were thus predicted as a consequence of the design earthquake. Ground movements, of this order could be expected to leave the stadium in an irreparable condition and (depending on structural form) the risk of a collapse type failure of the stadium, could be high. Ground improvement options were investigated to mitigate these ground movements and other consequences of liquefaction.

GROUND IMPROVEMENTS

The ground improvement options of dynamic compaction and vibro-replacement were considered in detail while other methods including soil grouting, excavation and replacement, and driven gravel columns were quickly discounted on technical and cost grounds. Isolating the structure on piles from the potential ground movements was found to be impractical because of the high lateral loads imposed on the piles by the potential ground movements.

Dynamic Compaction

Ground improvement by dynamic compaction has been successfully applied at three Wellington reclaimed sites in recent years. The process involves repeatedly dropping a large weight (say 15 tonnes) from height (say 15 m) on a specified grid pattern to compact the reclamation fill. The hydraulic fill area of the stadium site differed from the Wellington sites recently compacted in that a substantial proportion of the fill comprised soft silt. It is unlikely that the silt hydraulic fill will compact or consolidate to the required degree under dynamic compaction. It is expected that the energy of the dynamic compaction would be absorbed in displacing the silt and thus the overlying sand hydraulic fill is also unlikely to be improved to the required degree.

The end tipped fill can be expected to be adequately improved by dynamic compaction. Similar materials have been successfully compacted at other Wellington sites. The Department of Survey and Land Information (DOSLI) house vibration sensitive equipment in their building 180 m south-west of the site boundary. The vibrational effects of dynamic compaction were assessed at this distance on the basis of monitoring data from other sites improved by dynamic compaction. It was concluded that it was probable that dynamic compaction would adversely affect the operation of DOSLI's equipment.

Because of dynamic compactions predicted poor performance on the hydraulic fill and its probable adverse affect on DOSLI's equipment this method of ground improvement was not considered further.

Vibro-Replacement

The process of vibro-replacement involves probing the full depth of the ground to be improved with a vibrating probe on a specified grid basis. On extraction of the probe gravel is introduced and compacted to expand into the void left by the probe. The final product is a series of dense gravel columns of 0.8 m to 1.2 m in diameter on a grid at spacings of 2 m to 3.5 m.

Vibro-replacement was last used in New Zealand in Queenstown in 1986 but the equipment is readily available from Australia where it is commonly used to improve Australia's west coast sands.

The hydraulic fill sand and end tipped gravel fill can be expected to be readily compacted by vibro-replacement, mitigating their potential to liquefy and laterally spread. The improved drainage provided by the free draining gravel columns will also reduce liquefaction potential.

Little or no improvement in the strength of the hydraulic fill silt is expected by vibro-replacement, however the replacement of the silt with the stronger gravel columns will improve the average shear strength along this potential shear plane reducing the potential for lateral spreading.

The conceptual design set the size and spacing of the gravel columns to be produced by vibro-replacement such that the average shear strength along the critical failure plain through the silt would be increased to 35 kPa. With this shearing resistance the Newmark [4] approach suggests lateral spreading under the ultimate limit state design earthquake would be limited to 200 mm. It is considered feasible to design the stadium's foundations and structure to tolerate these movements with any damage being relatively minor and repairable.

The conceptual design has selected ground improvement by vibro-replacement for the stadium site.

CONCLUSION

There is no standard solution of ground improvements and foundations for developments on Wellington's reclaimed waterfront. The reclamations were formed in stages over a 100 year period employing a variety of techniques and a variety of materials. Appropriate ground improvement and foundation systems must be developed for each project considering the specific site conditions and the importance and public risk associated with the specific development.

With the variable and weak ground conditions at the proposed stadium site, vibro-replacement has been concluded to be a necessary and appropriate form of ground improvement for this site.

The Museum of New Zealand is another Wellington waterfront development. This building is currently under construction and since it is to house the nations treasures its design attracts a high importance factor. The ground conditions at the site contrast the stadium site in that they predominantly comprise end tipped granular fills allowing effective ground improvement by dynamic compaction.

ACKNOWLEDGEMENTS

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