New Zealand Experience on Economical Design of Reinforced Earth Embankments on Liquefiable Ground

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Abstract:

New Zealand is situated within a seismically active region. The country covers an area of 270,000 square kilometres of land and is occupied by 3.5million of people, with over 50% of its population residing in three major city centres. The design of infrastructure over liquefiable ground in low population areas presents a major challenge to the engineers, especially when ground improvement works could form such a significant portion of a project cost as to make it uneconomical.

This paper presents two case histories of the innovative and economical design of structures over seismically liquefiable ground in low population areas. The first case is a sports stadium and the second a pair of bridge abutments, both of which involved a minimum of ground improvement works. The designs accept that the structures may partially lose their serviceability under the design earthquake. However by ensuring their ductility by the use of the geo-reinforcement, the integrity of the structures is maintained. Following a severe earthquake, some repair works will be required to reinstate the serviceability structures. This paper presents the design considerations and methodology adopted to address the seismic risk to the structures.

1 Introduction

The big eruption of Mt Ruapehu volcano in New Zealand in 1995 and the massive Richter magnitude 7.8 Napier earthquake in 1931 provide evidence of the active tectonic nature of the country. The country sits astride the boundary of the Pacific and Australian tectonic plates. Apart from seismic activity arising from the sub-surface flow of volcanic materials in volcanic regions, all New Zealand's earthquakes are the result of its locations across the Pacific and Australian plate boundary. Earthquakes and volcanic and geothermal activity, which have been caused by the plate movements, have continuously occurred in the geological past. The country has more than 150 known active geological faults and many of them twitch repeatedly. Around 200 earthquakes a year are big enough to be felt. Generally, New Zealand's current level of earthquake activity is not as high as in Japan but it is similar to that in California.

The country covers 270,000 square kilometres of land area (approximately 7.5 times of land area of Taiwan) with over 50% of its population living in three major city centres, Auckland, Wellington and Christchurch, Figure 1. The combination of the non-uniform population distribution and active geology environment of the country has provided significant challenges to engineers to design infrastructure, such as roading and sports

Figure 1 New Zealand Map

stadiums, over poor ground in low population areas where large-scale ground stabilisation work may not be economically justifiable.

New Zealand structural design codes are based on the Limit State and Ductility design concepts where under the design seismic event structural damage is acceptable provided brittle failure does not occur. Unfortunately, a similar design code has not been finalised for geotechnical structures and foundation soils mainly because of the strong debate within the geotechnical and structural communities on whether or not the ultimate limit state design method is appropriate or convenient for geotechnical design.

This paper presents an innovative and economical design approach for (i) a sport stadium and (ii) a pair of bridge abutments over seismically liquefiable deposits in low population areas. In both projects, geo-reinforcement with a minimum of ground stabilisation work were employed to ensure the integrity and ductile behaviour of the structures, within the framework of ultimate limit state design, during and after seismic liquefaction. One of the most significant design features of the method is that the designs accept that the structures may partially lose their serviceability under the design earthquake and will require some repair work to reinstate them. The adoption of this design concept has made the two projects economically justifiable. Otherwise both projects may not have proceeded due to unaffordably high ground stabilisation costs.

2 Geology and Seismology

New Zealand comprises the North Island and South Island. The landmass was part of a larger submerged micro-continent that originally formed on the eastern margin of the Gondwana super-continent about 200 million years ago. It separated from the Gondwana and become isolation from other places about 110-120 million years ago.

Today New Zealand straddles the boundary between the Australian and Pacific plates, Figure 2. Broadly speaking, the Australian plate is heading north while the Pacific plate is heading west. The combination of these motions means that the Pacific plate, which includes much of the South Island, is moving relative to the Australian plate at a rate of about 40 millimetres each year in a southwest direction. As a result the North Island is moving away from South Island at about 4 metres every century.

During the relatively short human occupation of New Zealand, the country has experienced a number of large earthquakes. Epicentres of earthquakes with estimated local magnitude, M_L, (Richter magnitude) in excess of M_L 6.5 since mid 1800's are shown in Figure 3. Generally, the earthquakes sources deepen from the east to the west under the North Island. This reflects the westward dip of the subducting Pacific Plate as shown in Figure 2.

Significant seismic research has been undertaken in New Zealand in the last 100 years. Figures 4 and 5 present the seismic hazard maps of New Zealand in terms of peak ground acceleration for 475 and 2500-year return periods, Dorwick et al (1995). The maps have essentially formed the basis of seismic design codes. The seismic hazard maps are for general design purposes and do not exclude the requirement of site-specific seismological studies for major structures.

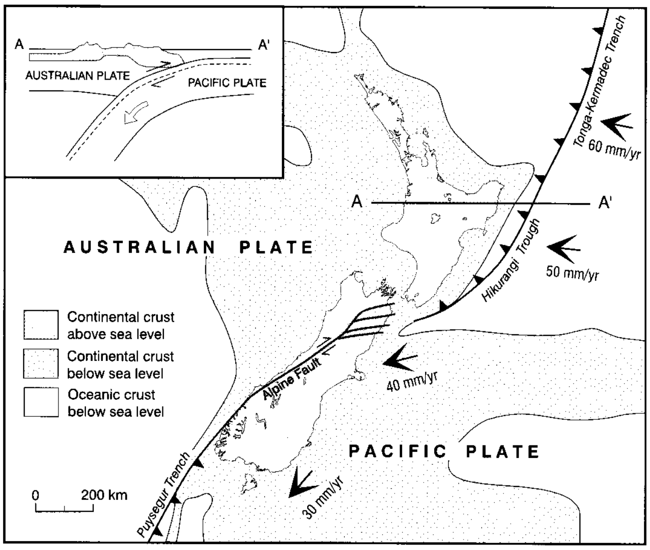

Figure 2 Australian Plate and the Subducting Pacific Plate Under North Island

3 Seismic Design Codes

The Limit State design concept forms the basis of relevant codes for seismic design of structures in New Zealand. All new structures have to be designed to function satisfactorily without major repairs under the specified serviceability limit state loadings. The serviceability limit state is reached when the structure becomes unfit for its intended use through deformation, vibratory response, degradation or other physical aspects. Structures also have to satisfy ultimate limit state requirements when subjected to ultimate limit state loadings. The ultimate limit state is reached when the structure ruptures, becomes unstable or loses equilibrium.

The earthquake effects specified for the ultimate limit state are based on the 475-year earthquake return period similar to the one shown in Figure 4. Basic requirements for the ultimate limit state earthquake effects are that the deflection of the structure shall not endanger life, or cause loss of structural integrity. The later requirement ensures that the structures are designed and detailed to exhibit ductile behaviour and not brittle failure when the specified ultimate limit state seismic loading condition is exceeded. Obviously the codes accept permanent deformation and loss of function of the structures caused by an earthquake in excess of the design earthquake.

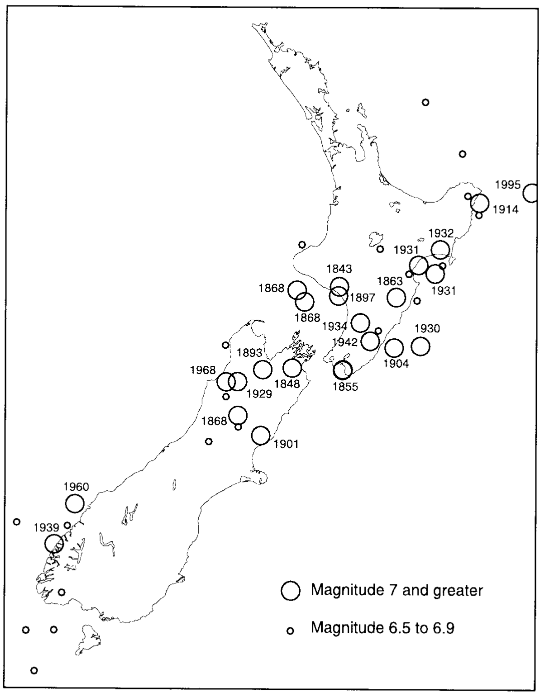

Figure 3 Epicentres of Earthquakes in Excess of M_L 6.5 Magnitude Since 1840

The codes are almost silent on the seismic design of geotechnical structures and foundation soils subject to large earthquake loading. However, the Transit New Zealand Bridge Manual specifically requires that the bridge abutments, if they form part of the bridge structure, should perform elastically during the design seismic event even though the bridge structure itself may require

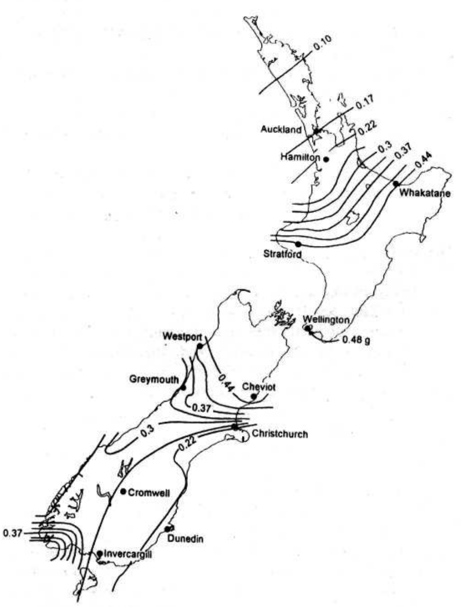

Figure 4 Seismic Hazard Map of Peak Ground Acceleration for 450-year return period

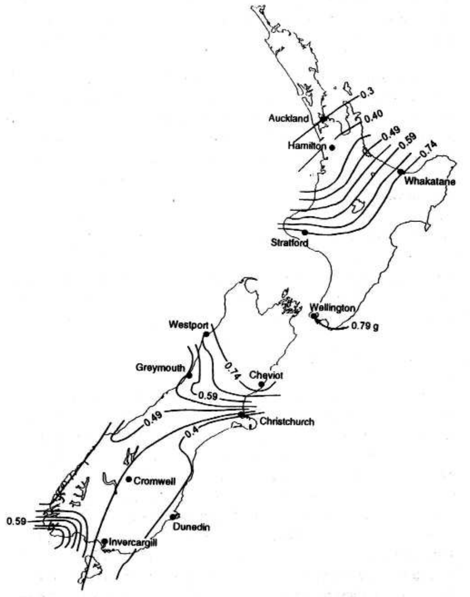

Figure 5 Seismic Hazard Map of Peak Ground Acceleration for 2500-year return Period

temporary repairs to make it usable by emergency traffic. Further, where a bridge is located at a site which is susceptible to earthquake induced liquefaction, designers are required to mitigate the effects of the large movements that may result from settlement, rotation or translation of the substructures. These requirements have placed a significant challenge to geotechnical engineers to ensure that the foundation and geotechnical structures perform satisfactorily even when the ultimate limit state seismic loading condition is significantly exceeded. However this may not be realistic and achievable for most alluvial deposits in some ground conditions without major ground improvement works.

4 Infrastructure Development Challenges in Low Population Areas

Auckland is the largest city in New Zealand and has its highest population with over 1.3 million people. It is within a relatively low seismicity area as shown in Figures 4 and 5. The area is generally underlain by volcanic rock and ash, and residually weathered to unweathered mudstone, siltstone and sandstone. Seismic liquefaction of alluvial deposits within Auckland is generally not a major issue for infrastructure development works.

Wellington and Christchurch are the second and third largest cities of the country and are located within the highest seismic zone covering the lower half of the North Island and the upper half of the South Island. However the population of each of the cities is less than 350,000 people. In other cities, populations are generally less than 150,000. Some of the sites in the highest seismic zone that have been identified and subjected to liquefaction in historic seismic events have been investigated and reported in Fairless (1984) and Cheung et al (1995).

For infrastructure development work in low population areas, ground improvement works to improve the seismic liquefaction resistance of the ground to relevant code requirements may form a very significant portion of the overall project cost. This has placed a significant demand on geotechnical engineers to develop innovative and economical solutions so that the projects can be economically constructed.

The ultimate limit state design concept has recently been extended from structural design to geotechnical design in two of our recent projects including a sport stadium (embankment stand) and a bridge structure although no such provision was allowed for in the design codes. In the projects, integrity of the stadium and the bridge and its abutments during and following seismic liquefaction are maintained by the use of geo-reinforcement and careful detailing to ensure that structures perform in an entirely ductile manner. The two design case histories are presented below. Prediction of liquefaction behaviours of soils is generally based on NCEER (1997) and Stark (1998).

5 Case Study 1 - Jade Stadium South Stand

5.1 Background

Jade Stadium in Christchurch is one of the major stadiums in New Zealand hosting important sporting events in the South Island. The south stand was designed to provide about 6500 seats and covers a plan area of 100m x 50m. The seating area comprises a sloping reinforced concrete slab-on-grade structure with a single storey elevated structure behind. They are both founded on a high tensile strength geotextile and HDPE geogrid reinforced embankment up to 4m high with a near vertical rear face and a 3H:1V(horizontal:vertical) sloping front.

Because of the relatively low utilisation of the stadium with less than 10 major sporting events annually, the project team decided to deviate from the relevant New Zealand design code requirements for the stand structure. The most significant deviation from the code requirement is that the design accepts that the structure may loss its serviceability under the 150-year recurrence interval earthquake loading applicable to the serviceability limit state due to the 500mm settlement and 200mm lateral movement, which would occur due to liquefaction. However the structure is designed to perform as a ductile structure and maintain its integrity thereby allowing it to be repaired after a severe earthquake. On this basis no subsurface ground improvement work was carried out.

5.2 Challenges

The major challenges for the geotechnical engineers on the project are summarised below:

- Seismically Liquefiable Ground Conditions - The site is underlain by a liquefiable sand layer up to 15m depth and is located within one of highest seismicity areas of New Zealand. The current seismicity model indicated that the site is likely to be subject to seismic liquefaction in earthquakes with less than a 150-year return period.
- Limited Budget - Ground stabilisation by means of stone columns or E~Quake Drains to mitigate the seismic liquefaction potential of the ground was estimated to cost over $1.5million and exceed 50% of the available budget for the entire project. The cheapest dynamic compaction option was not acceptable because the stadium is immediately adjacent to the residential area.
- No Published Information on Performance of Geogrid Reinforced Embankment on Liquefied Ground - During the scheme assessment stage, an extensive literature research was undertaken to determine the seismic performance of geogrid reinforced embankments on liquefied ground. Unfortunately, no published information was found. Therefore, a first principal theory was developed to understand the liquefied sand flow beneath the embankment during seismic liquefaction.
- Earthquake Resistant Design - Historical evidence generally indicated that seismic liquefaction causes large deformations and significant damage to earth embankments. Therefore the embankment had to be designed to exhibit ductile behaviour to avoid any brittle failure and to maintain the integrity of the entire embankment based on the framework of the Limit State Design concept.

5.3 Design Methodology

During the scheme assessment stage, four ground stabilisation techniques and three structure options were evaluated. After submission to client, the embankment stand option was selected. In the design, the combinations of the following techniques were adopted to overcome the difficulties mentioned above.

- The entire perimeter of the embankment was reinforced by HDPE (high density polyethylene)

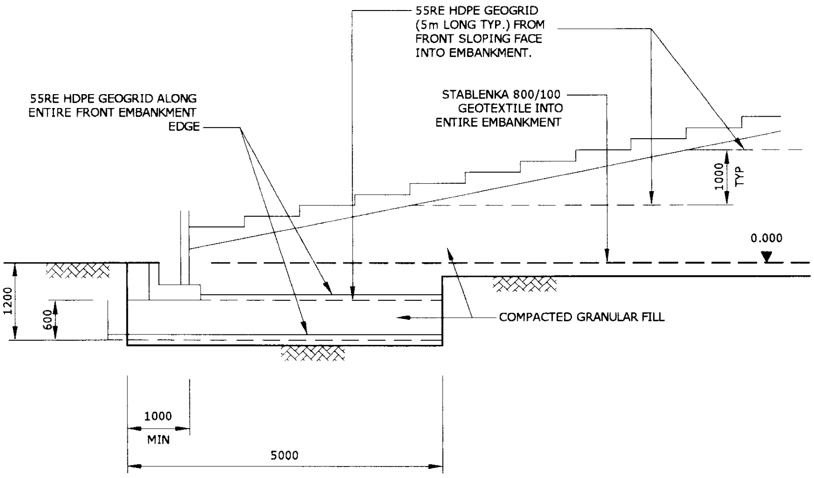

Figure 6 Jade Stadium Geogrid and Geotextile Reinforced Embankment Slope Fronting

geogrid, Figures 6 and 7. The reinforcement forms a geogrid reinforced soil block of 5m wide around the edge of the embankment.

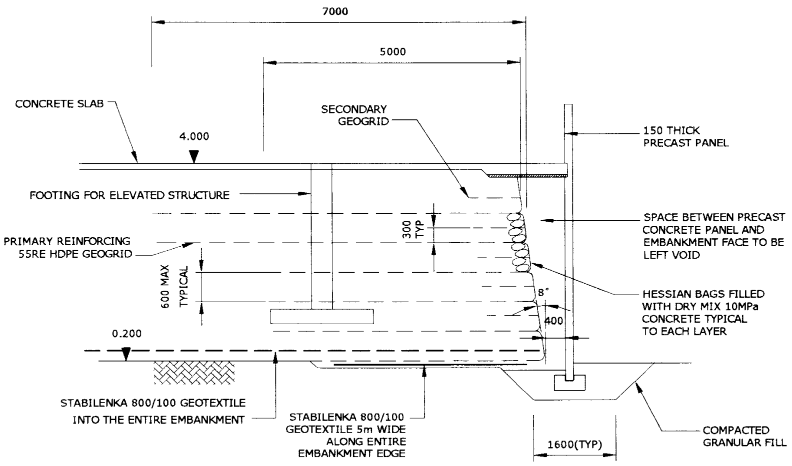

Figure 7 Jade Stadium Geogrid and Geotextile Reinforced Embankment Rear Face

- Geogrid reinforced soil layers were placed in a brick laying pattern, Figure 8, to ensure the ductility and integrity of the embankment around the perimeter during seismic liquefaction and to avoid weak butt jointing planes as employed in common geogrid construction practice.

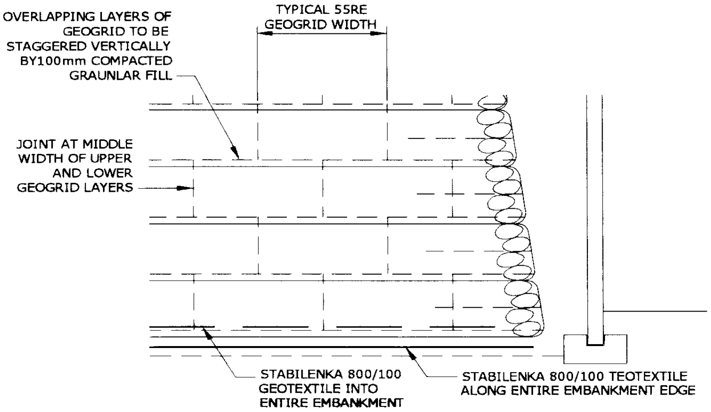

Figure 8 Face Geogrid Pattern and Basal Geotextile Reinforcement

- As an additional safety measure to ensure the ductility and integrity of the embankment perimeter, a 5m wide high-tensile strength geotextile of 800kN/m was placed along the base of the embankment edge. The combination of the geogrid and geotextile is intended to ensure that the embankment perimeter edge performs as an elastic beam during seismic liquefaction.

- The entire base of the embankment was reinforced by high tensile strength geotextiles of 800kN/m in both the longitudinal and lateral directions to ensure that no tension cracks could form at its base during a seismic event. The formation of tension cracks at the base could result in large lateral deformations of the embankment.

- Careful detailing of the placement of geotextile and geogrids was undertaken, especially in areas such as corners, stairways and openings, to avoid weak zones within the embankment.

- The reinforced embankment has been designed to perform as a non-liquefiable reinforced soil raft with all the footings of the stand structure founded on the embankment. This is to ensure that liquefaction of the ground beneath and outside the embankment will not cause loss of foundation support to the stand structure.

- The footings of the stand structure are tied together by ground beams of up to 800mm x 800mm in size and a slab-on-grade on the top of the embankment. This is to ensure the integrity of the stand structure during a seismic event.

- Up to 500mm liquefaction induced settlement was estimated from FLAC analyses and a review worldwide databases on the performance of shallow founded buildings on sandy soils following seismic liquefaction. The predicted settlement is considered acceptable to the design

- All wall face panels are for aesthetic and protection of the geogrid facing purposes only.

The design was rigorously peer reviewed by an independent geotechnical consultant appointed by the City Council. During the construction period, our geotechnical engineers and the geogrid supplier provided extensive input to assist the contractor with the construction. The embankment was satisfactorily completed within the allowable 3-month construction time frame and within the available budget.

5.4 Design Significance of Project

- First Known Application of a Geotextile Reinforced Embankment on Liquefiable Ground - To the authors' knowledge, it is the first time in New Zealand that a geotextile and geogrid reinforced embankment was employed to deal with the large deformation and ground break-up problems associated with seismic liquefaction.

- Unique Geogrid and Geotextile Detailing - We are not aware of the previous employment of a similar geogrid reinforced soil laying pattern. The design maximises the available seismic ductility of the embankment along the perimeter edge at no extra construction cost, an aspect that is not addressed in common geogrid embankment construction practice.

6 Case Study 2 - SH1 Tahuna Road Overbridge

6.1 Background

The highway authority, Transit New Zealand, continuously monitors the vehicle accident rate on the state highway network. High accident and poor geometry potions of the state highway network are scheduled for upgrade and realignment. The priority of the works is based on a cost and benefit ratio or the so-called B/C ratio. Currently a B/C ratio of over 4 is required for a project to be proceeded with. In the low population areas outside the major city centres, the B/C of many accident and poor geometry areas of the highway could be low because of the relatively low traffic volume.

The Tahuna Road Overbridge is part of a NZ$ 15 million State Highway SH1 realignment project which is situated at about 100km south of Auckland. The overbridge is a slightly skewed two span bridge designed to carry 2 lanes of rural traffic over the new four lane SH1 currently under construction. The site is underlain by an alluvial deposit over 20m thick and is susceptible to seismic liquefaction. The bridge and its abutments have been economically designed at a total cost of under NZ$750,000 excluding any ground improvement work. However, if ground improvement work is to be undertaken to satisfy the Transit New Zealand Bridge Manual requirement that the bridge abutments have to perform elastically under the design loading conditions, the cost of the ground improvements alone would be in the order of NZ$ 1million. The improvement works would exceed the value of the bridge works and make the bridge economically unjustifiable.

We therefore decided to deviate from the Transit Bridge Manual design requirement, and undertake only those ground improvement works necessary to limit the bridge abutment permanent movement to acceptable limits under the design seismic loads. The utilisation of this limited permanent displacement approach developed for the project significantly increased the design effort and required close liaison between the bridge and geotechnical designers so that the entire structure could maintain its integrity during seismic liquefaction. The bridge structure is also designed to be repairable following the 450-year ultimate limit state design earthquake event.

6.2 Challenges

The major challenges in the project are summarised below:

- Design-Construct Project - The project was an open tender design-construct project with five tenderers bidding for the project. The contractor's consultants had very little room to build any conservatism into the design without jeopardising the contractor’s chances of winning the tender.
- Seismically Liquefiable Ground Conditions - The site is underlain by a seismically liquefiable sand layer of over 20m thick. Although the cost of the bridge is relatively low, a specialist engineering seismologist was called in to develop a site-specific seismic model to optimise the design. The seismic model was used to determine the seismic induced porewater pressure within the sand for various earthquake magnitudes, ground accelerations and return periods.
- Low Cost and Benefit ratio - The project has a marginal B/C ratio. If large-scale ground improvement work was adopted, the project may not have proceeded.
- Development of a limited displacement method - The design of the bridge and its abutments was based on acceptable permanent seismic induced displacements under various earthquake loading conditions. Both the bridge and the abutments are designed to maintain their integrity during a severe seismic event and be able to be repaired after a major earthquake event.

6.3 Design Methodology

Two bridge options and three ground improvement techniques were considered. It was found that the two span option with the bridge deck founded on shallow strip foundations located immediately behind the reinforced earth abutments and a central pier founded on a strip foundation was the cheapest option, Figure 9. The significant design features comprise the following:

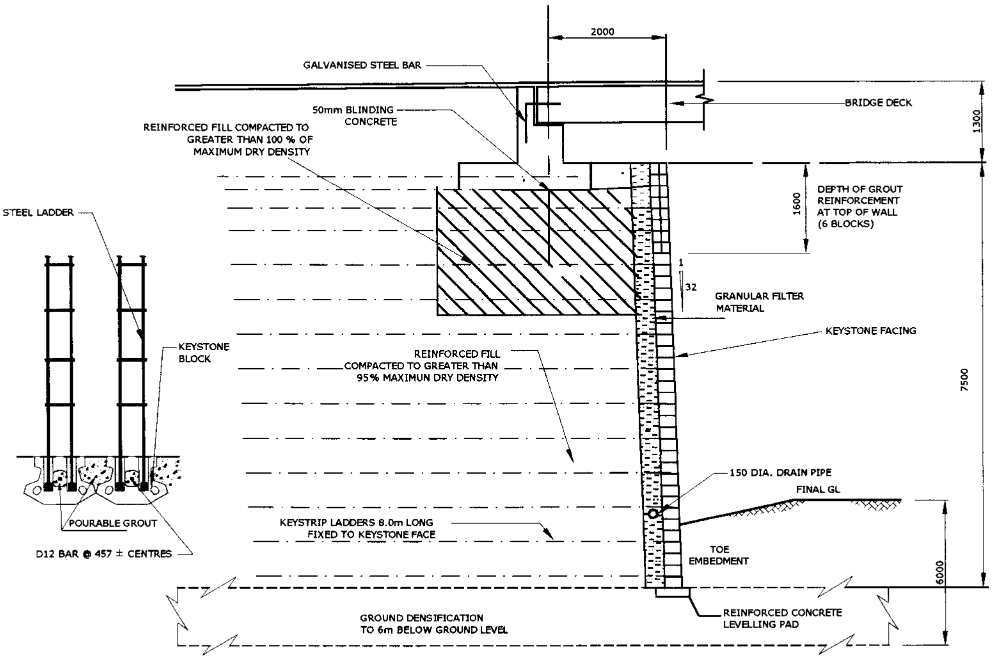

Figure 9 Tahuna Bridge Steel Ladder Reinforced Abutment with KeyStone Facing

- Mechanically Stabilised Earth (MSE) Retaining Abutments - MSE walls have demonstrated excellent seismic performance, exhibit extremely good ductile behaviour also have a pleasing appearance. KeySystem(TM) steel ladder reinforcement with KeyStone facing was selected in the project. High tensile strength geotextiles were also employed to enhance the structural ductility of the abutments.

- Ground Improvement - Extensive analyses indicated that a ground improvement depth of 6m was sufficient to limit the permanent horizontal bridge abutment displacement to less than 200mm even when the ultimate limit state design earthquake was exceeded. The minimum 6m deep improvement was also required to maintain the static stability of the abutments following severe earthquake events during the period that the seismic induced pore water pressure had not been fully dissipated.

- Resonant Vibro-Compaction ground improvement - The insitu sand generally has a fines content of less than 3-5%. Resonant vibro compaction is suitable for this soil and is the cheapest option because the vibro-probe, comprising three plates of 500mm wide and 16mm thick at 120 degree and 10m long, can be made locally without the need to import special equipment from overseas. The contractor had a suitable vibration hammer readily available. The resonant frequent of the ground was continuously monitored to optimise the densification efficiency.

- Bridge foundations - The bridge is founded directly on strip footings at the two MSE abutments and the central pier. The bridge footings at the abutments have approach slabs and will be tied into the abutment fill with steel ladders to ensure that the bridge responds elastically under the design seismic loading. Transverse seismic loads are be resisted by friction between the abutment footings and approach slabs and the underlying granular material in the abutment fill.

- Bridge Deck - The deck comprises prestressed double hollow core beams connected with transverse prestressing designed to distribute traffic and parapet impact loadings. The two spans and the central pier and the abutment footings are tied together by galvanised reinforcement.

- Limited Permanent Displacement Design Concept - The bridge and abutment design extended the ductility concept within the framework of the Ultimate Limit State design for structures to include foundations on seismically liquefiable ground. Significant investigation of the bridge deck and seat interface was undertaken to evaluate the acceptable level of seismic induced movement of the abutments and their influence on the bridge deck. First, below the 450 years Ultimate Limit State earthquake loading condition, the acceptable abutment movement was set to 50mm. The bridge deck has been detailed to tolerate the movement without damage. Second, if the Ultimate Limit State design earthquake loading is significantly exceeded, the abutment movement is limited to less than 200mm. The passive soil wedge behind the bridge seat is designed to fail and act as an energy absorption mechanism to dissipate the earthquake energy transferred from the bridge deck to the abutment so that the integrity of the entire bridge structure is maintained.

Although the cost of the bridge is relatively low, relatively significant analytical effort was employed in the design for the evaluation of the ground and the bridge response. FLAC analyses were also employed to verify the soil-structure interaction response.

6.4 Design Significance of Project

- The development of "Limited Permanent Displacement" design concept - To the knowledge of the authors, the concept was used for the first time for bridge design in New Zealand. The adoption of the concept is a reasonable approach for the design for a low traffic volume bridge where potentially high ground improvement costs dictated a need to deviate from the bridge code requirements.

- Significant Cost Saving - The introduction of the "Limited" seismic induced permanent displacement design concept has reduced the ground stabilisation work for the project from over NZ$1million to NZ$100,000 (90% cost saving) by extending the Ultimate Limit State design method for the design of structures to bridge abutments and foundation soils.

7 Conclusions

New Zealand structural design codes and the Transit Bridge Manual are based on the Limit State design concept. They require structures to remain fit for purpose under serviceability limit state loadings but allow ductile failure of structural elements at the ultimate limit state. The codes and the bridge manual are essentially silent on the foundation performance requirements although the bridge manual requires that bridge abutments perform elastically under the design ultimate loading conditions where the abutment forms part of the bridge. The bridge manual requirements are understandable because they prevent distress occurring below ground level where repairs may be difficult or impractical. Nevertheless designs based on the codes and bridge manual may not be economically justifiable, especially for structures with low utilisation in low population areas.

By accepting limited seismic induced displacement in the design and provided that the structures are detailed to provide sufficient ductility, substantial amounts of up-front ground stabilisation costs can be saved. Some repair work may be required to fully reinstate structures following a severe earthquake, however the overall cost could still be significantly lower than that required where substantial ground stabilisation is necessary to satisfy normal code requirements.

By means of geo-reinforcement, ductile structures can be ensured with careful detailing. This paper presents case history studies of geotextile, geogrid and steel ladders applications in geotechnical structures over seismic liquefiable ground in New Zealand for low population areas.

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