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ASSESSMENT AND RESTORATION OF AN EARTHQUAKE-DAMAGED SPORTS STADIUM IN NEW ZEALAND

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Abstract

The Lancaster Park (formerly AMI) stadium spectator stands in Christchurch, New Zealand were of modern reinforced concrete design and construction, with the two larger stands supported on stone-column foundations. This paper summarises the damage that occurred as a result of the Canterbury earthquake sequence (CES) of 2010-2011 and the engineering assessments that were carried out. The Canterbury earthquake sequence included several strong and damaging ground motions, with the most severe shaking being from the 22 February 2011 magnitude M_w6.3 event at an epicentral distance of only 6km from the stadium. That event resulted in peak ground accelerations at the stadium site of approximately 0.5g horizontally and 0.75g vertically. Damage to the stands included bulging, loosening and contamination of the stone column foundations, global and differential settlement of the structures, and structural damage including extensive cracking of concrete, yielding and strain damage (and in some cases fracture) of reinforcement. Engineering assessments included mapping of observed damage, material testing and analytical damage predictions. The earthquake damage was assessed to have left the structures and foundations with a loss of capacity, reduced performance and uncertain future. One of the smaller stands was considered to be unsafe and the authorities required it to be demolished. The likely scope of repairs for the three remaining stadium structures and their foundations is conceptually described, but the cost and viability of such repairs have not yet been established.

Keywords: earthquake damage assessment; stadium structures; settlement and shaking damage; reinforced concrete; ground improvement; stone columns

1. Introduction

The Canterbury Earthquake Sequence (CES) of 2010-2011 caused widespread significant damage to buildings and infrastructure across Christchurch City, as well as at the Lancaster Park Stadium. The most severe shaking was due to the 22 February 2011 magnitude M_w6.3 earthquake with an epicentre approximately 6km from central Christchurch. It caused very strong shaking, resulting in the deaths of 185 people and widespread damage to buildings and infrastructure. Lancaster Park Stadium was the largest sporting and events venue in Christchurch with a seated capacity of approximately 40,000. The stadium sustained substantial damage to foundations and structure as a result of the earthquake sequence, and in particular the February 2011 earthquake, and, as at 2016, the facility remains closed. Engineering damage assessments following the earthquakes included site inspections/observations, testing of various materials in the building and foundations and theoretical predictive analysis of foundations and structures due to shaking and ground movements. Some consideration of repairs required for re-use of the facility has been made but the financial viability of carrying out such repairs has not yet been established. Although all of the facilities were damaged to varying degrees, this paper concentrates only on the two largest stands, Deans and Paul Kelly. The Hadlee Stand, which was the oldest and one of the smaller stands, was severely damaged and ordered to be demolished by the authorities. The other smaller Tui Stand, although damaged, is not discussed in this paper.

2. Lancaster Park stadium

2.1 Background and history

Lancaster Park stadium has considerable history, dating back to the 1880s when, as swamp farmland owned by Benjamin Lancaster, it was purchased and a cricket ground established. Circa the 1910s, the Canterbury rugby union became co-owners, with the stadium's sporting use primarily shared between cricket and rugby through to current times.

Spectator stands constructed early-mid 20th century were progressively demolished and replaced as the stadium underwent significant development between 1995 and 2010. At the time of the Canterbury earthquake sequence Lancaster Park Stadium comprised four main spectator stands and associated access structures including ramps, stair towers and link bridges, refer Fig. 1. The Hadlee (north), Tui (south), Paul Kelly (west) and Deans (east) stands are named after either prominent local sports people or commercial sponsors. The oldest existing Hadlee Stand opened in 1995, Tui and Paul Kelly Stands were opened in 2000, and Deans stand in 2010. The facility is owned by VBase Limited, itself wholly-owned by Christchurch City Council.

Fig. 1 – Lancaster Park Stadium prior to earthquakes looking south (left), and post-earthquake aerial taken 24 February 2011 (right - from Koordinates.com)

The facility had been intended to be used as a venue for the 2011 Rugby World Cup, but due to the earthquake damage all Rugby World Cup matches were relocated away from the city, and the stadium was closed for assessment.

2.2 Foundation and structural form

Foundation and structural construction type and form, and design details for the two main stands are described in the following sections. Fig. 2 shows cross sectional diagrams of the two main stands.

2.2.1 Paul Kelly Stand (west)

The Paul Kelly Stand, designed in 2000, is a four level partially covered structure, curved in plan, with an overall seated capacity of around 17,000 spectators plus corporate boxes and hospitality lounges. Associated ancillary structures include a multi-level access ramp hall, and north and south stair towers.

The stand structure comprises twenty-four grids of precast and in situ reinforced concrete radial shear wall/frame structures supporting precast concrete bleachers and hollowcore floor units with in situ topping. Radial structure is tied together longitudinally by a curved precast and in situ reinforced concrete moment resisting frame. Roof structure comprises cross-braced cantilever steel rafters with lightweight metal cladding. Radial structure provides the primary gravity load support with bleachers and floor structure spanning between radial grids. Transverse/radial loads are largely resisted by shear walls, moment framing and raker 'bracing' structure on each radial grid. The radial structure was designed for limited ductility, with hinge zones expected within the raker or moment frame beams. In the curved longitudinal direction, seismic forces are resisted by shear walls between Ground and Level 1 (Concourse level), and above Level 1 by a three-level circumferential moment resisting frame running the length of the structure. The original designers adopted a weak-beam, strong-column system with plastic hinge zones at the ends of beams at Levels 2, 3 and 4, and also at the base of the columns just above Level 1. The frame had been designed as fully ductile in accordance with relevant New Zealand design standards. The radial and circumferential structures work together for stability and to resist seismic lateral loads and torsional effects.

The foundations are a composite system comprising reinforced concrete shallow strip footings, for the lighter loaded field side structure, and a continuous raft beneath the main heavier loaded part of the superstructure. To help protect the structure against the effects of liquefaction induced foundation failure and settlement, ground improvement was constructed under the stand. This comprised vibro-replacement using a triangular grid of stone columns extending to a depth of approximately 9m below ground and covering the footprint of the stand and the associated structures. The liquefiable ground below the stand was only treated to partial depth, thereby creating a non-liquefying crust supporting the building.

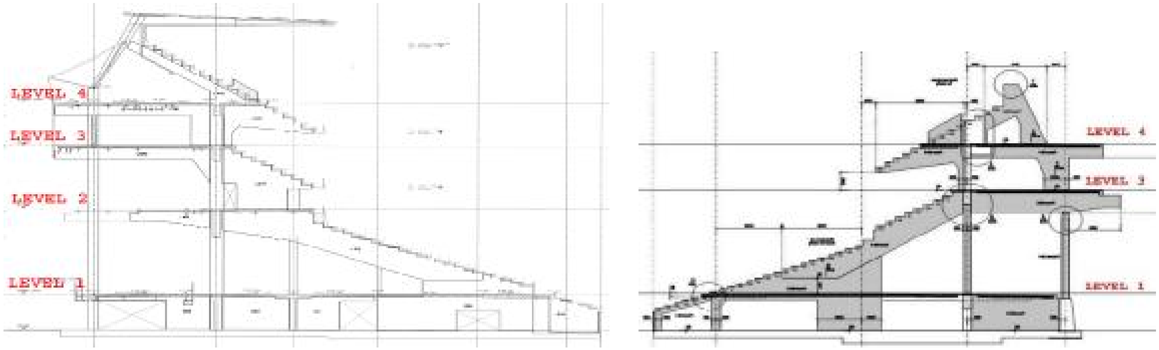

Fig. 2 – East-west cross-sections through main stands (Paul Kelly – left, Deans – right)

2.2.2 Deans Stand (east)

Deans Stand, designed in 2008, is a three level covered stand, curved in plan, with an overall seated capacity of around 13,500 spectators. Associated ancillary structures include a multi-level access ramp hall, north and south stair towers and an attached corporate hospitality space (Deloitte Lounge) at level 4. The stand superstructure is of a similar arrangement to Paul Kelly, comprising radial shear wall/frame structures and a circumferential moment frame along the length of the stand. Concrete construction is both precast and in situ. The roof structure comprises cross-braced cantilever steel rafters with lightweight metal cladding. Transverse/radial loads are resisted by a mixture of structures on the radial grids; beam and column moment framing, shear walls and raker 'brace' members. According to the original design documents the designer intended the radial seismic system and seismic mechanism to include a fully ductile foundation beam. In the longitudinal direction, lateral loads are primarily resisted by a three level moment resisting frame following the longitudinal curve of the stand. Similar to Paul Kelly a fully ductile weak-beam, strong-column system was again adopted with plastic hinge zones to form at the ends of beams at Levels 1, 3 and 4 and also at the base of the columns at foundation level. Foundations beneath the main stand are a composite system comprising shallow reinforced concrete ground and tie beams on stone columns. As for the Paul Kelly Stand, to protect the structure against liquefaction induced ground and foundation failure, ground improvement was undertaken, but this ground improvement was constructed only beneath the footprint of the main part of the Deans stand structure. This comprised vibro-replacement - a grid of stone columns extending to a depth of approximately 9m below ground and concentrated in bands centred under the structural grids. The liquefiable ground below the stand was only treated to partial depth, with the intention of creating a non-liquefying crust or soil raft. The foundations for the adjacent access ramp hall, stair towers, link structures and Deloitte Lounge comprised screw piles extending to a depth of approximately 15m, with reinforced concrete pile caps and tie beams.

2.2.3 Seismic design basis

Based on the age of each structure, only the newest Deans Stand would have been designed to the latest structural design Standards applicable at the time of the Canterbury earthquakes. From limited information available to the authors, both the major stands were designed using capacity design methods to the material and design standards applicable at the time of their design. The design of each stand appears to have adopted a ductile design philosophy for the reinforced concrete structures. The designer selected designated "plastic hinge zones" and the remainder of the structure was intended to remain elastic and undamaged. Physical damage expected to occur in the ductile plastic hinge regions under large design earthquake events included reinforcement yielding, concrete cracking and spalling.

3. Canterbury earthquake sequence

The Canterbury earthquake sequence of 2010 and 2011 included several strong and damaging ground motions including; 4 September 2010 (Moment magnitude M_w=7.1), 22 February 2011 (M_w=6.3), 13 June 2011 (M_w=5.6 and M_w=6.3) and 23 December 2011 (M_w=5.8 and M_w=6.0). Numerous aftershocks exceeding M_w=5 also occurred. The most severe shaking at Lancaster Park Stadium occurred during the 22 February 2011 earthquake. Although the earthquake was of relatively moderate magnitude, and the duration of strong shaking was only a few seconds, the earthquake epicentre was shallow and very close – approximately 6 km south-east of the stadium. That event resulted in estimated peak ground accelerations (PGA) at the stadium of approximately 0.5g horizontally and 0.75g vertically. According to the Canterbury Earthquakes Royal Commission reports [1], response spectra derived from ground motions measured at stations around the central city exceeded the 1 in 2,500 year acceleration response spectra given by NZS 1170.5:2004 [2]. Vertical accelerations were also particularly severe and may have contributed to the severity of damage observed at the stadium.

4. Observed damage to structures

There were a few people present in the facility at the time of the 22 February 2011 earthquake, but no reported injuries. Widespread damage, a result of both earthquake shaking and foundation/ground settlement occurred to all of the Lancaster Park Stadium structures. Damage to the foundations included bulging, loosening and contamination of stone columns, plus global and differential settlement. Damage to the primary structures included extensive cracking and spalling of concrete, and yielding and strain damage (and in some cases fracture) to reinforcement. There was also extensive damage to secondary structural elements such as concrete bleachers and their connections, building fabric, fit-out and disruption of contents.

4.1 Ground damage

The area around the stadium experienced considerable liquefaction due to the 22 February 2011 earthquake. Large amounts of ejected sand (sand boils) were observed, together with differential settlement of commercial and residential buildings and uneven land surfaces. The extent of surface expressions of liquefaction within the confines of the stadium was generally less than for the surrounding area.

Ground level changes and building settlements were measured by level surveys and by reference to LiDAR data from before and after the earthquakes. Total settlement of between 200 and 600mm was recorded at the four stands following the February 2011 earthquake. Marked differential settlement was apparent between elements of structures on different foundation types, and across the structures themselves. The Paul Kelly Stand tilted away from the playing field fairly uniformly by around 100mm at roof level. The main body of the Deans Stand settled by between 200 and 500mm (refer Fig.3), with each end tilting backwards by around 100mm and the central portion settled approximately 300mm relative to the ends and tilted towards the field by 150mm. It is unclear whether these settlements occurred during or after earthquake shaking. Examples of surface expressions of ground damage are shown in the photographs in Fig. 4.

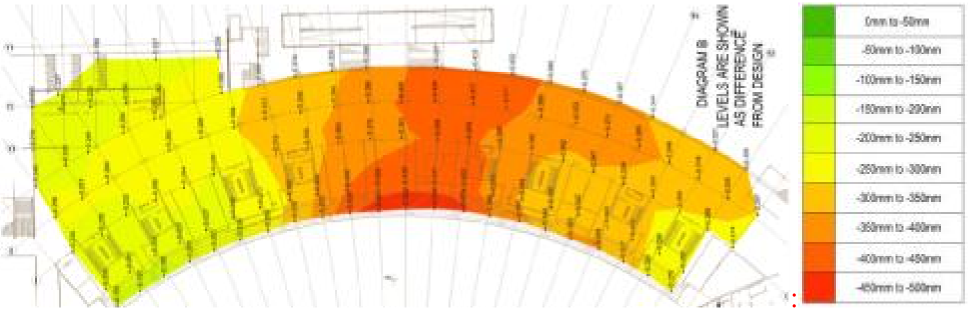

Fig. 3 – Deans Stand surveyed settlement contours

Fig. 4 – Settlement and liquefaction damage – ground surface disruption (left) and pavement bulging (right)

Subsequent physical and numerical investigations showed the stone columns to have bulged and loosened in response to the earthquake shaking and seismic loading from the structure above. The columns were also found to have been contaminated with liquefaction ejecta – fine silty sand - from liquefied ground between and below them. Limited cone penetration testing of the ground between the stone columns in Deans Stand indicated that the densification of that soil achieved by stone column construction may have been lost, with the material returning to its pre-construction state.

4.2 Structural damage

Structural damage occurred to all stands and ancillary structures. Photos of typical types of structural damage observed are provided in Figs. 5 to 7. A large proportion of the structural elements were exposed and readily observable without the need to remove linings or fit-out. Cracking to the reinforced concrete elements was widespread, with residual crack widths ranging from hairline cracking up to 12 mm. While residual crack widths that were observed often appeared small, it was expected that maximum crack width opening during the earthquake shaking may have been several times larger. Visual observations of residual cracks were not able to give any indication of strain damage to reinforcing bars, therefore steel testing and analysis work, described later, was necessary.

4.2.1 Paul Kelly and Deans Stands

Damage sustained by the two main stands, Paul Kelly and Deans, included the following:

- Global and differential settlement across the stands and their associated structures as described in 4.1.
- Damage to the primary longitudinal load resisting structure in each of the stands included cracking and spalling in shear walls, beams and columns, and, in the Paul Kelly Stand, fractured and buckled beam reinforcement (refer Fig. 5). As expected from the design engineer’s reports, the longitudinal moment frame damage was typically concentrated at the ends of the beams, in intended plastic hinge zones.
- Damage to the radial structure of each stand included cracking and spalling of shear walls, floor beams, raker beams and columns. This cracking was evident on multiple radial gridlines and over multiple levels of the superstructure.
- Precast concrete bleacher units were damaged throughout the stands including; spalling to radial raker beams at bleacher support steps, damage to bleacher connections, and spalling and cracking in

bleacher units. The damage in Paul Kelly was more severe with large pieces of spalled concrete falling on to the main public concourse.

- Other damaged elements of the superstructures included; foundation beams and slab on grade structure, suspended floor slabs, secondary structure such as lift shafts, vomitories, precast panel elements - typically concrete cracking, localised areas of spalling and residual distortion. In the Deans Stand a severe tearing failure occurred over a length of approximately 10m between the Level 1 floor and its supporting radial wall structure near the middle of the stand, refer Fig. 6.
- The adjoining ramp halls were both left visibly distorted and tilted, with significant residual displacements/movements at seismic joint locations. The Paul Kelly ramp hall suffered damage to connections to the main structure including failure of link bridge connection pins, bolts and steel bracing. Connections between the Deans Stand and its ramp hall also failed; at Level 1 the concourse slab connecting the main stand and ramp hall fractured/failed (refer Fig. 7) and at Level 3 the bracing connections to the main stand floor slab/diaphragm also failed.

Fig. 5 – Paul Kelly Stand longitudinal frame cracking (left), and fractured reinforcement (right)

Fig. 6 – Deans Stand shear wall cracking (left) and failure of slab support (right)

Fig. 7 – Deans Stand fracture of slab (~100mm horizontal separation) between stand and ramp hall (left), movement between adjacent bleacher units (right)

5. Physical testing of foundation and structure damage

5.1 Foundation investigations and testing

Investigation of stone column damage comprised:

- Digging of test pits to expose the upper portion of stone columns beneath and adjacent to structures.
- Machine boreholes through the stone columns.
- Laboratory testing of clean and contaminated stone column material.
- Cone penetration testing of ground between the stone columns and in the playing field, well beyond the stone column treatment area.

Stone columns work by stiffening, strengthening and densifying the ground they pass through. They also provide a groundwater drainage path. These effects all act to improve the resistance of the surrounding ground to liquefaction. Stone columns are stiffer than the ground around them, so they carry a large proportion of the load from the overlying structure. They improve shallow bearing capacity and reduce settlement under static loads.

The February 2011 earthquake exceeded design ground shaking levels, and liquefaction was evident from field observations of ejecta and from the settlement which occurred between, beneath and beyond the stone columns. Calculations and field investigations supported this conclusion. These investigations demonstrated that liquefaction had affected and damaged the stone columns in the following ways:

- The stone columns were contaminated with fine silty sand as liquefaction ejecta flowed in to and upwards through them during and following earthquake shaking (refer Fig. 8). It was estimated that this has reduced their drainage performance by approximately 50%, and they were left less able to relieve excess pore water pressure build-up (the precursor of liquefaction) in future earthquake shaking.
- Liquefaction between the stone columns reduced the confinement of the columns markedly. Following dissipation of excess pore water pressures, the locked in lateral confining pressure developed during stone column construction had been lost, with confinement recovering from the liquefied condition to the equivalent of normal consolidation. It was estimated that around 30 to 35% loss of confinement of the stone columns had resulted.
- The stone columns beneath the stands had bulged into the surrounding liquefied soils resulting in them dilating and losing strength and stiffness. The strength reduction was compounded by contamination with fine grained soils, and the net effect was estimated to be in the order of 15 to 20% loss of original strength. As a result, they were considered to be less able to support structural loads under a future liquefaction event. This loss of capacity is expected to inevitably lead to additional settlement of heavily loaded stone columns during future earthquake shaking, particularly under the Deans Stand.
- The perimeter stone columns and the lower portion of the stone columns directly under the stands have been subjected to pore water pressure migration from the surrounding and underlying liquefied soils. When combined with earthquake induced shearing, dilation and loosening of these columns is expected to have occurred. This reduces the ability of the perimeter columns to protect the internal columns from the effects of liquefaction beyond the stands. It also reduces the ability of the lower portion of all of the columns to prevent liquefaction between them and thus increases the potential for bulging of the lower part of the columns in future earthquakes.
- Liquefaction that has occurred between the stone columns has led to loss of the densification of the surrounding ground achieved by stone column construction. While the loss of densification may not be vital for static performance of the foundation system, it is assessed to have reduced performance during future earthquakes.

Fig. 8 – Stone column material samples: with liquefaction contamination (left) and clean (right)

Investigations concluded that the stone columns had suffered significant loss of function during the February 2011 earthquake and would need repair or alternative liquefaction measures installed if the stands were to be reused. Some of the heavily loaded stone columns beneath the Deans Stand were assessed to be unable to safely support the stadium loads under future liquefaction conditions. The capacity of the columns directly beneath the Paul Kelly Stand had been similarly reduced, although less severely because of the thick concrete foundation slab under that structure.

5.2 Reinforcing steel damage testing

A targeted programme of testing of reinforcing steel was carried out in order to identify and quantify damage to reinforcing steel in cracked areas of reinforced concrete elements. Testing methods included insitu hardness, laboratory hardness and monotonic and cyclic tensile testing.

Holmes Solutions carried out non-destructive physical testing using a methodology they developed to estimate strain damage to reinforcing bars using insitu Leeb hardness testing. The method correlated increased surface hardness against stress, for strain hardened bars, and then inferred maximum imposed strain and residual strain capacity. Calibrations for various field conditions were included and normalised to baseline undamaged material stress-strain behaviour. Strain damage predictions based on Leeb hardness were found to be useful indications of where further laboratory testing should be carried out. Additional testing was carried out by Holmes Solutions to ascertain strain ageing damage, cyclic strain and fatigue damage to a small number of samples. The testing confirmed that Grade 300 (mild steel) and Grade 430 reinforcing steels used widely in the potential plastic hinge regions of the stadium structures both showed strain ageing effects. Micro-alloyed Grade 500E reinforcing steel used in the Deans stand (but not generally in potential hinge zones) showed some but much less strain ageing. For steel susceptible to strain ageing it means that, following plastic straining and a period of ageing, the steel shows different mechanical behaviour, with a re-emerged yield plateau at a higher yield stress than before, increased ultimate tensile capacity and reduced ultimate strain capacity.

Professor Milo Kral from the University of Canterbury tested reinforcing from the stands using his laboratory Vickers hardness testing methodology [3] to quantify plastic strain damage to reinforcing bars including the effects of strain ageing. Fig. 9 shows the stress-strain behaviour of undamaged (blue) and earthquake damaged (red) bars and the correlation of residual strain capacity to measured Vickers hardness for Grade 300 reinforcing steel from the Deans Stand. Testing for the stadium also included physical tensile testing to failure of damaged and undamaged steel samples to compare actual (tested) residual strain capacity against predictions based on hardness measurements. The testing methodology and application to Lancaster Park structures is possibly a world first in quantifying damage to reinforcing steel in earthquake damaged buildings. Testing indicated that, in a number of critical locations of both stands, 30-50% of the original strain capacity of reinforcing steel had been lost, leaving significantly reduced residual strain capacity.

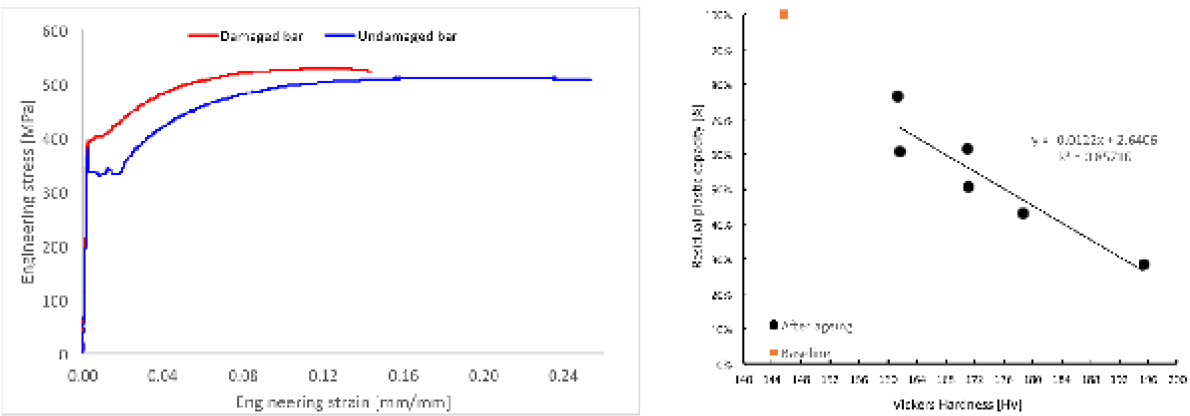

Fig. 9 – Stress-strain behaviour for undamaged and earthquake damaged bars (left), and hardness vs residual plastic strain capacity (right) for 25mm diameter Grade 300 bars from Deans Stand

It was also apparent from the results that residual crack width is a broadly useful indicator of strain damage to reinforcing bars inside a concrete member. Only 11% of samples from locations with residual crack widths below 0.5 mm showed detectable damage (plastic strain > 0.01), whereas 100% of samples from locations with cracks above 1 mm clearly showed significant strain damage from the tests.

6. Prediction of foundation and structural damage by computational analysis

6.1 Stone Column analysis

The stone columns are formed from gravel which is compacted into a dense state. Dense granular soils dilate (rather than contract) when sheared, resulting in an overall loosening, especially at large strains. The stone columns have been subjected to large shear stresses as a result of the earthquake shaking and would have likely dilated (increased in volume), resulting in a reduction in their shear stiffness and hence effectiveness at liquefaction mitigation by stress concentration. Furthermore, it was expected that the columns under Deans and Paul Kelly had bulged as the confinement provided by the surrounding ground reduced as a result of earthquake shaking (and particularly once the surrounding ground liquefied). This bulging would also, inevitably, have led to a degree of loosening. The greater the degree of liquefaction in the soils surrounding the stone columns, the greater the extent of bulging would have occurred to the point where the effectiveness of the stone columns to carry vertical loads or to resist shear stress was severely reduced.

Detailed modelling of stone columns under earthquake conditions was carried out by Earth Mechanics from California and reviewed by Professor Geoffrey Martin. Fig. 10 shows some graphics of the axisymmetric numerical analysis of the dilation and softening effects. The following conclusions were made about the performance of the stone columns beneath the radial grids of the Deans Stand:

- The reduction in lateral support of the granular stone column material resulting from liquefaction both between and below the stone columns led to significant shearing and bulging of the columns, and loss of ability to support vertical structural loads.
- The infiltration of finer grained sand and silt particles into the stone columns, along with the loosening of the gravels due to shearing and bulging, led to reduced stiffness and strength of the granular stone column material.
- The degraded post-earthquake strength and stiffness left the stone columns with adequate capacity to support the applied load (from the ground beam) under non-liquefied conditions, consistent with observations that the stadium remained standing.
- Under liquefaction conditions for a future design-level earthquake, the load carrying capacity of a stone column would be reduced to the point where it would be unable to safely support the stadium.

Results for the Paul Kelly stand analyses were similar.

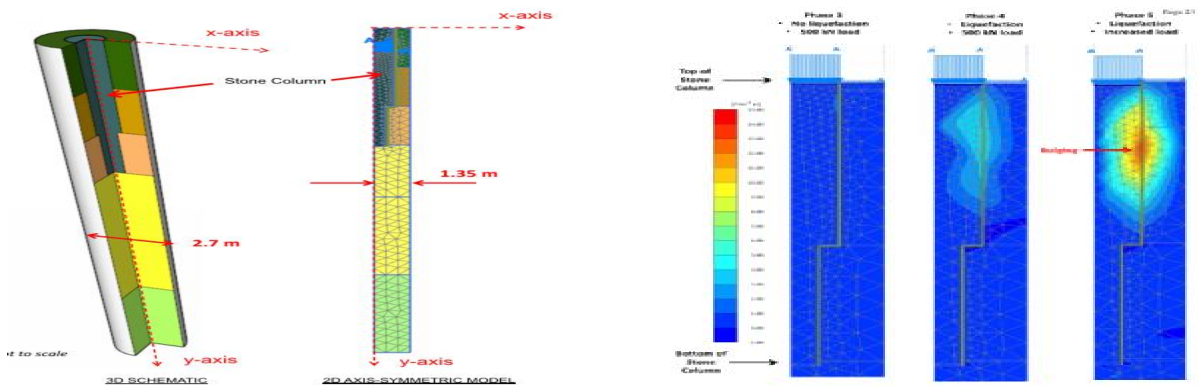

Fig. 10 – Stone Column numerical analysis Plaxis 2012 graphics: axisymmetric finite element model (left) and horizontal bulging displacement contours (right)

6.2 Nonlinear structural analyses

Nonlinear static "pushover" and nonlinear time history analyses (NLTHA) were carried out on the Paul Kelly and Deans Stands using earthquake records from the 22 February 2011 earthquake. The effects of surveyed differential settlement on each of the stands were also separately analysed. Three-dimensional models of the structures (refer Fig. 11) were developed and analysed using analysis software SAP2000.

In both stands the behaviour of the primary north-south longitudinal moment resisting frames were of particular interest, as they were designed for significant ductility (force reduction) compared with the radial structural elements. It was possible to compare behaviour against the original design philosophy (typically designed as weak-beam, strong-column) and also to provide a theoretical assessment of the extent of yielding and damage to the structures when subject to earthquake records from the strongest earthquake shaking. Some comparisons were then made against site observations of damage and results from steel reinforcing testing.

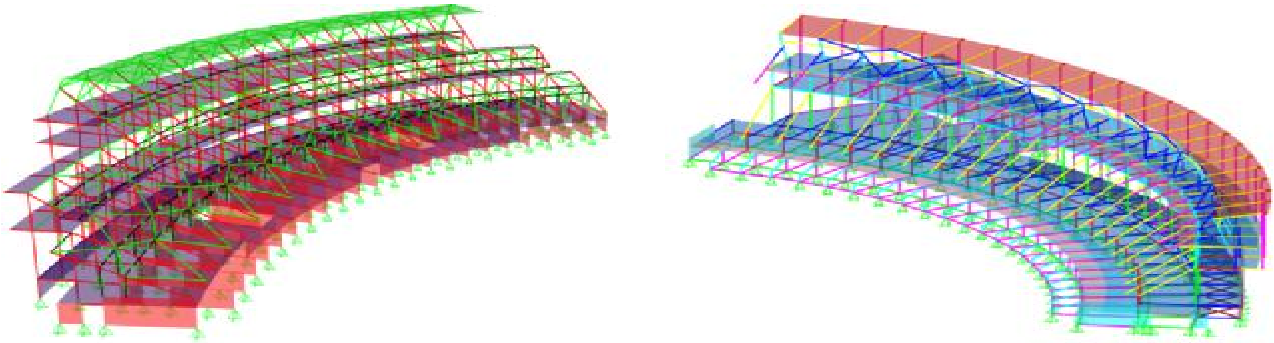

Fig. 11 – Structural Modelling (Paul Kelly – left, Deans – right)

The analyses showed that, as expected, both structures were significantly stiffer and stronger transversely/radially than longitudinally. The predicted behaviour of the structures under longitudinal earthquake shaking was generally consistent with what the original designer assumed. In particular, the analyses suggested weak-beam strong-column mechanisms for the longitudinal moment resisting frame in each of the stands, with hinges forming at the ends of the beams at each level and at the base of the columns. This prediction also matched site observations of concentrated areas of damage evident at the ends of the beams, with lesser damage noted at the base of columns.

NLTHA outputs for the Paul Kelly Stand indicated that reinforcement in the longitudinal moment frame beams would have yielded during the 22 February 2011 earthquake. Maximum reinforcing steel strains in the order of 20 times yield strain were predicted, approximately 20% of the original ultimate strain capacity (not including reductions due to strain ageing). Analysis of the Paul Kelly stand using the surveyed differential settlements indicated localised areas of large stresses in the ground floor shear walls, and also yielding of reinforcement in the longitudinal frame beams. The residual inelastic strain effects predicted at the end of the ground shaking could sensibly be combined with the effects of differential settlement.

Analysis for the Deans Stand also indicated yielding in beam reinforcement due to earthquake shaking but of lesser magnitude compared with the Paul Kelly Stand. Differential settlement analysis for the Deans stand indicated reinforcement in the longitudinal frame beams would have yielded, and also large stresses in the radial structure on selected grids. The high stress locations in the model corresponded reasonably well with the locations where extensive cracking was observed in the structure.

NLTHA results for both stands also indicated that the beam reinforcement in the longitudinal frames was subject to several reversals of plastic (inelastic) strain at the locations where yielding or hinging occurred.

The analyses indicated there was a complex interaction between radial and circumferential lateral load resisting structure in both stands, due to the curved nature of the buildings, and also potentially influenced by the sloping diaphragm effects of the seating bleacher structures.

6.3 Fatigue Damage Assessment

Professor John Mander of Texas A&M University provided assistance in developing and applying his state-of-the-art methodology [4] for the assessment of reinforcement fatigue damage for the Deans and Paul Kelly stand structures. The methodology considered inputs such as; earthquake ground motions, overall seismic capacity of the structures based on an assumed plastic collapse mechanism, structural geometry, plastic hinge rotations and reinforcing bar stress-strain behaviour. These analyses indicated that substantial fatigue damage including fracture of some bars occurred at observed and predicted areas of concentrated damage in the stands. Fatigue damage fractions of 20% and 100% of the as-new fatigue capacity were predicted at critical plastic hinges in the moment frames of the Deans and Paul Kelly stands, respectively. The high damage fraction prediction for the Paul Kelly seemed plausible since several fractured bars had been observed at critical sections of the longitudinal ductile moment frame.

7. Discussion

All four spectator stands, and their ancillary structures, were assessed to have been subject to earthquake loads in excess of those which they were originally designed for. While there were some elements that suffered structural failure, no collapse occurred and the stands generally performed quite well and as would be expected for modern capacity-designed reinforced concrete structures.

For the two main stands, Paul Kelly and Deans, physical damage observations, nonlinear time history analysis outputs and results from steel testing showed reasonable correlation. The behaviour was consistent with the original designer's documented seismic design philosophy and failure hierarchy intent, in particular a weak-beam strong-column mechanism for the longitudinal moment frames. Earthquake damage was concentrated at the ends of the beams and included concrete cracking and spalling, and plastic straining of reinforcing steel including some bar fractures.

Precast concrete bleacher units and their supports were extensively damaged, as a result of seismic shaking, diaphragm behaviour of the seating tiers and primary structure displacements - but no loss of support was observed. Bleacher unit detailing and displacement compatibility with primary structural displacements appear to be items requiring particular design consideration to avoid unsatisfactory behaviour.

It appears that the majority of the superstructure damage in the Paul Kelly stand was due to seismic shaking, with less due to the effects of differential settlement. Deans stand had considerably more damage to primary and secondary structure around its centre third suggesting the effects of differential settlement were more pronounced in that portion of the stand. Paul Kelly had a more extensive continuous foundation raft structure than Deans and also significantly stronger and stiffer structure longitudinally between ground and Level 1 (an essentially continuous shear wall). The lesser differential settlement sustained by Paul Kelly was perhaps a result of its ability to redistribute loads more effectively. Deans stand did not have the benefits of a raft foundation or longitudinal shear walls at ground floor and appears to have been more susceptible to the effects of liquefaction and differential settlement.

The residual capacity and performance of the structures were assessed to be significantly reduced due to the earthquake damage sustained. Substantial repairs would be necessary to restore the damaged structures to a condition suitable for re-use and with reliable and acceptable levels of performance.

8. Repair of the damaged stadium stands

Repairs required to damaged elements of the stadium are expected to range from relatively minor through to extensive, including for some elements demolition and reconstruction.

The scope of the repairs to the stadium is likely to include the following:

- Detailed investigation of the condition of the existing foundations and ground improvement.
- Further inspection and testing of a wide range of structural elements and materials.
- Construction of alternative ground improvement such as contiguous grout columns installed around the perimeter and in a cellular pattern under the stands.
- Re-levelling of parts of the building to bring them within acceptable functional tolerances.
- Replacement of structural elements with extensive cracking or strain damage to steel. This would likely include; the longitudinal frame of the Paul Kelly stand, localised areas of floor slab and shear walls in both stands, some foundation and secondary structure elements.
- Removal of approximately half of the precast bleacher units to access and repair damaged supports.
- Replacement of many damaged precast bleacher units and repair of others.
- Rebuilding of both access ramp halls, two of the access stair towers and various other structures.
- Rebuild of the demolished Hadlee stand.
- Replacement of drainage infrastructure and other underground utilities.
- Reconstruction of the sports field.

Based on the advice of the owner's construction advisors, it is likely that repairs would take two to three years to complete. The cost and viability of undertaking such repairs is yet to be determined. Independent peer review of the geotechnical and structural engineering for the foundation and structural repairs will be required.

9. Conclusions

The Lancaster Park sports stadium facilities sustained substantial damage, mainly as a result of severe shaking from the M_w6.3 earthquake of 22 February 2011. Considering the intensity of shaking and the basis for which the buildings were designed, they performed well and the damage is generally as expected.

Engineering damage assessments included making physical observations, testing of various materials in the building and foundations and computational predictive analysis of foundations and structures due to the effects of ground shaking and ground movements.

Level and verticality surveys showed that substantial earthquake settlements and out of vertical movements occurred, including total level change of up to 600 mm and differential settlements of up to 300 mm across the footprints of each building.

Extensive liquefaction occurred under the buildings and liquefaction material was ejected from the ground surface around the foundations. The stone column ground improvement constructed beneath the two major stand structures suffered damage due to bulging, loosening and contamination with silty sand. This damage was investigated using physical investigations and theoretical assessment. It was concluded that the ground improvement was no longer effective or reliable under strong earthquake shaking.

Damage to structures due to direct earthquake shaking and differential settlements included extensive cracking of reinforced concrete elements, separation of adjacent structures and widespread plastic strain damage to steel reinforcing including fracture of bars at a small number of locations.

A targeted programme of testing of reinforcing bar damage was carried out using in situ hardness and also laboratory testing of extracted steel samples. The methodology may be a world first in quantifying damage to reinforcing steel in earthquake damaged buildings. The methodology for estimating irreversible fatigue damage to the structures may also have been a world first.

Nonlinear structural analyses using static pushover and dynamic response history methods, and using strong motion records obtained from stations near the stadium site, were used to predict damage to the structures.

The repair works necessary to re-open the facility are extensive and would be likely to take two to three years to complete. The cost and viability of such repairs have not yet been established.

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