UNDER	THE COMMISSIONS OF INQUIRY ACT 1908
IN THE MATTER OF	ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES
AND IN THE MATTER OF	THE CTV BUILDING COLLAPSE

BRIEF OF EVIDENCE OF DAVID HARDING IN RELATION TO THE CTV BULDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

SAUNDERS & CO
SOLICITORS
PO BOX 18
CHRISTCHURCH
PH: 03 379 7690
FAX: 03 379 3669
Email: jmk@saunders.co.nz
(Solicitor acting: James Michael Kirkland)

BRIEF OF EVIDENCE OF DAVID HARDING IN RELATION TO THE CTV BULDING

INTRODUCTION

1. My name is David Harding.

Part of my evidence involves my professional opinion. I have read the Code of Conduct for expert witnesses. I agree to comply with that code.

Further, in preparing this Brief of Evidence, I have not read any brief of evidence already filed before the Commission.

I have deliberately refrained from doing so, on the basis of wishing to provide this Brief of Evidence from my own recollections of events, calculations and professional opinion.

Following the "*filing*" of my Brief of Evidence, I shall read other relevant briefs. I propose to file a supplementary brief, if there is any point arising that should be addressed to assist the Commission.

From November 1985 to November 1988, I was an employee at Alan Reay Consultants, Consulting Engineers of Christchurch. At the early stage of this employment I worked on the structural design of the CTV building. I also later had a role in visiting the site during its construction.

QUALIFICATIONS - EXPERIENCE

2. At that time my qualifications were:

    2.1. Bachelor of Engineering (Civil) with second class Honours from Canterbury University,1 May 1973

    2.2. Post graduate Diploma in Business Administration from Canterbury University, 2 May 1984

    2.3. Member of Institute of Professional Engineers New Zealand, 25 November 1985

    2.4. Registered Engineer May 1976

3. My experience at that time was as a Structural and Civil Engineer.

After graduation I was employed by Hardie and Anderson, Consulting Engineers between February 1973 and 1977. Work at that firm included the design of domestic buildings and foundations, site levelling surveys, stormwater design and structural design of single storey factories, offices, warehouses and school buildings, structural strengthening of brick buildings and full scale testing of fibreglass structures.

1978 – MAY 1980

4. I was employed by Alan Reay Consultants Limited between 1978 and May 1980 as a Civil and Structural Engineer, designing structural elements of residential buildings, and single or two storey industrial and commercial buildings particularly of precast concrete construction. This also involved site levelling surveys and site soils investigations for low rise buildings. I left in May 1980 to gain experience in Civil Engineering.

5 MAY 1980 – 22 NOVEMBER 1985

5. I was employed by the Waimairi District Council as their Design Engineer between 5 May 1980 and 22 November 1985. This position was to run the Design Office, and included responsibility for a Traffic Engineer, three Civil Engineering Cadets and two draughtsmen. The work included principally Civil Engineering, including the design of roundabouts, the reconstruction or shape correction of District Roads, and the reconstruction or diversion of main roads such as Johns Road and Prestons Road. I carried out Cost Benefit analyses for the Northcote New Brighton Expressway, and designed the first stages between Mairehau High School and Travis Road.

   I designed the landfill access road through Bottle Lake Forest, and carried out preliminary designs and cost benefit studies for Fendalton Road four laning.

   There was some structural engineering relating to the annual survey of bridges, and the maintenance of bridges. I carried out preliminary investigations and then designed the Jellie Park waterslide and associated platforms and swimming pools.

AUGUST 1985 – CONTACT FROM ALAN REAY CONSULTANTS LIMITED

6. In about August 1985 I was offered an opportunity to return to Alan Reay Consultants Limited. Alan "*Alan*" advised me that since my earlier employment with his firm that he had expanded the firm in order to design multi-storey buildings, and at that time had designed a number of multi-storey buildings. He said that he had engaged a structural engineer and structural draughtsmen previously employed by Holmes, Wood Poole and Johnstone, particularly because of their experience in the design and draughting of such buildings. His

current Structural Engineer, John Henry, was shortly due to leave the firm and Alan offered me that position. Alan understood that I had no experience in the design of multi-storey buildings which required the use of the ETABS computer program for the dynamic analysis of such buildings. He advised me that I had the opportunity to gain that experience in his office, and to become an Associate with his firm in the near future.

7. While I was working out my three months' notice at Waimairi, Alan rang me to see if I could reduce that notice period, as it appeared that John Henry was leaving imminently, and he was left short staffed for current projects. These included a low rise residential building for the Hospital Board. I was unable to shorten my notice at Waimairi, but on enquiry of Alan it appeared that the proposed Hospital Board building was a regularly proportioned concrete masonry building which was relatively low rise, so it did not require a dynamic analysis using ETABS. Accordingly it was possible for that building to be designed using the Equivalent Static Method, with which I was experienced. I offered to prepare the calculations for this building for him while I was working out my notice. This I did, and I then forwarded my calculations to his office for his draughtsmen to complete the structural drawings.

NOVEMBER 1985 – EMPLOYMENT WITH ALAN REAY CONSULTANTS LIMITED

8. This second period of employment with the firm started in November 1985 and finished in November 1988. The nature of my responsibilities was initially the same as it had been during my earlier employment with the firm. Alan would have contact with the Clients and with the Architect, and Alan would then determine the structural concept for the building, and prepare preliminary calculations. At this time Alan decided which materials to use in the building construction, the location of major structural elements, and the likely method of construction.

After I was shown the preliminary drawings, I would prepare the final structural calculations, and provide information to the structural draughtsmen in order for them to complete their final drawings. These drawings were typically prepared to a standard format, in many cases using standard details which had been developed within the firm over time. Alan would decide which materials were to be used, such as concrete, structural steel or timber, and if concrete, which elements of the building were to be constructed with precast concrete or insitu poured elements, the type of proprietary floor system to be used, and whether a mobile crane or a tower crane was to be used to construct the building. He would also direct whether steel mesh or bar reinforcement were to be used in the wall panel construction.

Alan would have separate discussions with the structural draughtsmen, and would specify the layout of the drawings, and the purpose and extent of the drawings. This would vary between minimal drawings for regular design build clients with specific requirements, more extensive drawings for less regular clients, and shop drawings for each element for other clients.

The draughtsmen would prepare carcase drawings, and then I would provide information to them from my structural calculations. Alan would regularly look over the shoulder of the draughtsmen during the drawing production, and if he saw any structural details on the drawings which did not comply with his requirements then he would request changes accordingly from myself and the draughtsman.

After completion of the drawings, Alan would review them, and present them to the client for approval prior to Building Permit Application.

9. After Building Permit approval, I would be involved in the site observation of the structural elements of the building construction. Typically I would visit the site prior to each concrete pour to inspect the reinforcement placement, and after pours to view the concrete after form removal and to view the supplier documents for the ready mix concrete as supplied to site.

10. Toward the end of my employment, after I had built up relationships with some of the firm's clients and Architects, and had become more aware of the firm's standard procedures, I had input to the preliminary design of structures. I worked for Tony Scott, from Williams Construction, who I had met during the construction of the CTV building, by carrying out the preliminary design of a number of buildings, and I provided him with preliminary construction details to facilitate costing of other design build projects.

CTV BUILDING

11. One of the first projects I was involved with, and the first multi-storey building requiring an ETABS analysis, was at 249 Madras Street, now known as the CTV building. Alan consulted with the client and the Architect, prepared the preliminary calculations and the concept design, and arranged for the preliminary Architectural drawings to be amended to meet his requirements.

12. These drawings were then presented to me, and Alan advised me of the reasons for the building layout shown on the drawings. He explained that the client had seen an existing building at 299 Durham Street, which was at the north west corner of the intersection of Durham Street and Armagh Street. This is now referred to as the Contours Building. The client had been impressed by the look of the Contours building, and he wanted the CTV building to have a similar layout of the services core and the same facade treatment. In order to repeat the look and detailing of the Contours building the client engaged the same Architect to design the CTV building as had designed the Contours Building. I understood this to be Alun Wilkie Architects.

The features of the Contours Building which were to be replicated included the services core arrangement, where the core was to be located to the north of the main building, and offset away from the main building so that the service core walls were visually separate from the main building office areas. It also required the same facade details, including the same diameter external columns, precast concrete spandrel panels, glazing setback behind the perimeter columns and layout of internal columns. The CTV building would be higher and of different overall size, but this would just involve more repetition of the same details.

13. It was considered that the shear core walls were too high to be constructed as precast concrete panels, and that the shape of the building and its proximity to other buildings would make it impractical to construct using mobile cranes. Accordingly the shear core walls were to be constructed from insitu poured concrete, and it was expected that the building would mainly be constructed using a tower crane.

14. It was agreed that the building could not be designed using the Equivalent Static Method, as it did not meet the criteria set out in NZS4203:1984 for the use of this method. As the building was more than four stories high, and had an irregular layout of walls, it was decided to design the building using the modal response spectrum method, using the ETABS computer program.

15. Alan was aware that I had not used ETABS before, and that I was relying on him for guidance in the use of the program and the resultant method of building design. Accordingly Alan provided me with a set of structural calculations and computer input and output sheets which had been prepared by his previous structural Engineer, John Henry. These calculations were for a recently constructed building designed in Alan's office, which was located at 287 Durham Street, at the north west corner of Durham Street and Gloucester

Street. This is now referred to as Landsborough House. I was to use these calculations as a method template for modelling the CTV building on the computer.

These calculations appeared to be well set out, and clearly done by a person experienced in multi-storey building design. The calculations for the gravity elements, seismic resisting elements and foundations were in separate sections, and the assumptions which had been made in terms of the separation of gravity elements and seismic elements, and the calculation of section properties were evident. The computer input sheets and the output files were in the form of concertina pattern continuous sheets. I do not recall seeing a set of drawings for Landsborough House, and I was not expected to copy any details of the design of that building.

I recall no contact with the client or the Architect during the design process. Alan told me that he did not want me to contact John Henry as I prepared my calculations, but to ask Alan if I had any queries, and to keep him apprised of my progress with the design. I accepted this requirement, and acted accordingly.

16. It was determined that the gravity elements and seismic resisting elements for the CTV building were to be separated in the same way as for the template calculations. The reinforced concrete walls would be designed to resist all lateral wind and earthquake loads, and these would cantilever from a foundation beam. The circular columns were to be the same size as the Contours building, and would be designed as gravity resisting elements only. They would be detailed in such a way that they were not to attract any significant lateral seismic shear loads. This appeared to be a sensible arrangement as the walls were to be made much stiffer than the round columns, and this would avoid the complication of designing the floor beams and columns as ductile frames.

17. The computer to be used was located at the University of Canterbury. This was at a time when the ETABS program was too large to be run on a desktop computer. By removing the internal frames from the seismic model of the building, the model became simpler and easier for the computer to manage. The calculated lateral sway of the building would be calculated using the strength and stiffness of the wall elements only, without any assistance from the columns.

It was understood that the ETABS program included the assumption that the cantilever walls were rigidly fixed at the ground level, and the suspended floor diaphragms were rigid with no deformation within these diaphragms.

I recall a number of visits to the University to provide input data for the seismic analysis. I would leave my input data at the University, where it would be loaded onto the University mainframe computer by the staff. About a week later I would pick up the output data sheets and return to the office to assess them. The main design criterion which was checked on each output run was the interstorey deflection of the building under the most severe combination of design loading. This was to be kept below the limit in section 3.8.3 of NZS4203, which is equivalent to 0.83% of the interstorey height. For the first computer runs I recall that the interstorey deflection was excessive, and I tried unsuccessfully to correct this by increasing the wall thickness.

18. I recall discussing this with Alan, and I recommended that we should add an additional shear wall on the south face of the building to help to resist the torsional rotation of the building. Alan was concerned that a wall in this location was not present on the Contours building, so the addition of this wall on the CTV building may not be acceptable to the client. I believe Alan then discussed this with the owner and the Architect, and it was agreed by them, and relayed to me, that this wall could be added if it were limited in size such that it would be concealed behind the external egress stair on the south wall. This required the south wall to be constructed as a coupled shear wall, with holes in the centre of the wall at each floor level to facilitate egress to the stair landings.

This wall was added to the seismic model of the building, and the wall thicknesses were adjusted again including the additional south wall. By this means the building was made stiff enough to reduce the interstorey deflection to below the limits set out in the Building Code.

19. After the wall thickness had been so determined, the detailed design of the structural elements of the building was completed. This included the design of the reinforcement in the structure.

At this time I do not recall the existence of a block wall on the west boundary. This may have been added later in the design after it became evident that a fire rated wall was required on that side of the building. The western block wall was to be designed as non structural partition, which would be isolated from the floor diaphragm at the top of the wall within each storey.

20. The structural draughtsmen would have prepared carcase drawings for the building according to directions from Alan, and based on their earlier experience with drawing multi-storey buildings. I would have provided details of reinforcement in order to complete the drawings. Alan would regularly visit the draughtsmen to monitor progress of the work, and to view the details and information which I had given to them. If he had any concerns as to the type of detailing, my failure to use standard details, or my use of non preferred products, then he would advise me of the changes he required.

21. My assessment as to whether the building complied with the relevant codes would be based on my previous experience with each element. In areas where I was inexperienced, I would refer to the template calculations provided. Prior to the submission of the documents for a building consent, Alan would review the drawings and calculations. In some cases Alan may redesign elements himself, or refer them back to me for amendment or redesign.

22. The documents required for the building permit application were prepared by various people in the office. Normally the draughtsmen would prepare the drawings, and I would prepare the structural calculations. Alan would normally complete the Design Certificate as the Principal of the firm.

23. Any correspondence from the Christchurch City Council would normally be viewed by Alan upon receipt, and then referred to me or to the draughtsmen for action. If there was any contentious element then Alan may become involved, but otherwise the drawings or calculations would be amended to comply with City Council requirements.

24. I do not recall discussions with the City Council on the CTV project. I have recently read the letter received from Mr Tapper dated 27 August 1986. I do not recall answering this letter myself, but it appears to be typical of the kind of letter which is expected from a structural checking engineer. If I had replied to the letter, I expect that I would have prepared a written reply, but I have not seen such a letter at this time. I expect that all of Mr Tapper's requests would have been complied with. Mr Tapper was an experienced and thorough Engineer with a good eye for detail. He had assessed drawings for many office designs. He would not have approved any documents which had an omission or contained a detail which he was not satisfied with. My normal reaction would be to welcome his ability to note such details and to amend them accordingly.

25. I have been asked if I recall any contact with Bryan Bluck of the City Council at this time. I do not. In any case, Bryan would not have short cut Graham Tapper

by talking to me directly, as I recall Bryan was more involved with policy than details of specific building permits. I cannot remember talking to Bryan Bluck about detailed design on any project I have done.

26. In specific regard to the floor connection to the wall system, it appears that the connection of the precast beams to the ends of the shear walls, and the provision of slab ties between the walls and the mesh reinforcement were accepted by Graham Tapper as an accepted connection.

At that time mesh was an accepted form of floor reinforcement. In this case the steel Hi Bond decking would have contributed additional strength to the floor diaphragm. It was effectively made continuous by the use of saddle bar reinforcement over the precast beams.

27. The remark in Mr Tapper's letter about the restraint of the HiBond refers to his concern regarding the fire rating of the floor system. The floors were required to have a 1.5 hour fire rating between the floors. Where the floor is to be constructed with a Hi Bond steel tray system, there is concern that the floor is exposed to a fire from below.

28. If no other measures are taken, then the heat of a fire could cause the Hi Bond to overheat, so that the steel loses strength leading to failure of the floor due to the fire. In order for the floor system to have the required one and a half hour fire rating, one option is to longitudinally restrain the HiBond so that it can support load by membrane action. Clearly that was not possible in this case, as the steel is cast into the side of a concrete beam using end caps to avoid grout loss, and without continuity of the HiBond through the beam. This appears to be the concern Mr Tapper has.

29. A second alternative for fire rating the floor is to coat the underside of the floor with an insulating layer, which may be a specialised sprayed coating or an additional ceiling made of fire resistant material. This option was not preferred in this case.

30. A third option, which was chosen on the CTV building, was to provide fire emergency reinforcement in the floor in addition to the Hi Bond and the mesh. This reinforcement is located within the concrete floor above the level of the Hi Bond, at such a depth that it is insulated from the heat of the fire by a suitable thickness of concrete. In this case, if the fire should weaken the exposed HiBond, then the midspan tension loads would be carried by the fire emergency reinforcement, and the floor would retain sufficient strength and integrity to be

able to support a reduced loading for the required fire rating period. The provision of this reinforcement appears to have been accepted by Mr Tapper at the Building Permit stage.

The letter to Williams Construction dated 19 August 1987 advises Williams Construction of the agreement that had been reached between the City Council and Alan Reay Consultants at the time of Building Permit application with regard to the method chosen to fire rate the floor. It appears that Mr Bluck thought we were using the first alternative described to fire rate the floor, when we were actually using the third alternative. I appear to have met with Mr Tapper in August 1987 to confirm that my understanding of the Building Permit correspondence was correct.

WILLIAMS CONSTRUCTION

31. The first time I met the agents of Williams Construction was at the beginning of the building construction. At that time I met the site foreman, the Quantity Surveyor, Tony Scott, and the Construction Manager, Gerald Shirtcliff.

32. Prior to each pour of concrete, the Site Foreman would have advised me, and I would have visited the site in order to inspect the reinforcement placement and the formwork construction. I would also have inspected the concrete supply dockets from previous pours to confirm that the specified strength of concrete had been supplied to the site. The practice was that a hand written record of each site visit would be left on site. This would record any instructions to the builder for remedial action and may request or authorise variations to the building contract. A carbon copy of such instructions was then returned to the Engineer's office, where it was typed and distributed to affected parties.

33. For this project I was requested by the builder to design a base for the tower crane which was to be used on the site in order to construct the building.

34. After the CTV building was constructed I had no further involvement with it while employed at Alan Reay Consultants Limited. I left the firm in November 1988. My only involvement after I left was a telephone call from Alan in about 1990, to say that the building had been peer reviewed by another engineer as part of due diligence prior to leasing. This report had queried the connection of the building to the shear core. Alan asked me if I had authorised the inclusion of any additional reinforcement, such as drag bars, during the building construction. I replied that I could not recall having requested anything

additional. I had no further contact with Alan, and no contact with Geoff Banks or the reviewing engineer.

HYLAND REPORT

35. Section 3 Description of CTV Building

35.1. I generally agree with the description of the building in the Hyland Report.

35.2. On page 43 the internal beams are referred to as moment resisting frames. I would not call the internal beams moment resisting frames. The beams were designed to be continuous beams and as such were designed to be moment resisting only between adjacent beams for gravity loading. The columns were not intended to be a part of a moment resisting frame, and the ends of the columns were designed as pin joints. Consequently the beam column joints were not designed to carry any bending moment from the columns, and any contribution which these columns may make toward the building lateral stiffness was not relied upon.

35.3. On page 46 the construction of the concrete masonry wall on the west boundary is speculated upon. I do recall that the top course of this wall was left unfilled during construction. While the builder was constructing the wall below an existing beam in the floor above, he noted that it would practically be difficult to fill the top course of the masonry.

The wall was being constructed from the inside of the building, as access to the outside was restricted by the presence of a neighbouring building. I consider it unlikely that the outside of the vertical isolation joints could have been filled with mortar at that time. This joint may have been filled from the outside at a later date after the adjacent wall was removed, as it may then have become a waterproofing issue for this building.

I have no knowledge of any works carried out after construction of the building.

36. Section 8 Collapse Scenario Evaluation

36.1. On page 88 it is acknowledged that other collapse scenarios to the four postulated in the report cannot be discounted. I believe that the scenarios in the report make insufficient acknowledgement of the effect of vertical acceleration on the building.

VERTICAL ACCELERATION

The report acknowledges on page 92 that the vertical ground accelerations may have increased the axial loads on the columns and thereby reduced the column drift capacity. I believe this is the key to why the building failed as it did.

36.2. A photograph taken by Ross Becker, through a window of the adjacent IRD building, shows books stacked up on steel trestle tables. Most of the books are still sitting on the tables after the earthquake, as it appears that the lateral design forces have not been sufficient to dislodge them. However the tables have a major sag in the middle, as if the books have at some time tripled in weight. This shows the effect of vertical acceleration, and it shows that this site was subject to very high levels of vertical acceleration.

36.3. The loadings code of the day, NZS4203:1984, makes no provision for vertical accelerations on a building. The commentary to the code states at page 17:

"*Although significant vertical acceleration components of ground motions have been recorded during earthquakes(for example 0.2 to 0.3g in the San Fernando earthquake) no vertical acceleration load terms have been included in the design loads of this standard except for parts such as horizontal cantilevers and anchorage of machinery because there is at present no certainty about the damage potential of combined dynamic effects*"

The design loadings for Christchurch in NZS4203:1984 appear to be predicated on an earthquake epicentre maybe 100 kilometres away and maybe 6 kilometres deep. The radiated seismic energy arrives at the site close to horizontal, and causes predominantly horizontal shaking. The vertical component of the ground acceleration is relatively small and is insignificant compared to the vertical live loading the building is designed for. Consequently this is not normally a critical case that will determine a building's design.

However for the Christchurch earthquake in February 2011, the epicentre was about 8 kilometres away and 6 kilometres deep. It can be expected that the vertical acceleration will be of a similar magnitude to the horizontal acceleration. The vertical acceleration will be highest closer to the epicentre, and certainly the CTV building is closer to the epicentre than the three centres at which records are used in the Hyland report.

36.4. The magnitude of vertical accelerations at various sites in Christchurch are described in a report titled "Near-Source Strong Ground Motions Observed in the 22 February 2011 Christchurch Earthquake" by Brendon A Bradley and Misko Cubrinovski. **ANNEXURE "A".**

Table 1 shows peak ground accelerations at the CCCC and CHHC sites, which were both used in the Hyland Report (page 233). These show vertical accelerations of 0.79g and 0.62g respectively. These are nearly double the horizontal accelerations at each site. The closest site to the west, which is not in the Hyland report, is the Pages Road pumping station, PRPC. This recorded a vertical acceleration of 1.88g, or three times the horizontal acceleration at that site.

36.5. The report, at page 193, states that

"*The ratio between vertical and horizontal ground motion amplitude is strongly dependent on source-to-site distance, and weakly dependent on source magnitude or faulting style*"

36.6. In hindsight, the structural design of the CTV building was vulnerable to the effects of severe vertical acceleration. The floors are relatively heavy, weighing 4.0kPa. The live load which the building was designed for is 2.5kPa, but for the cumulative effect of large areas on many floors this load is reduced to close to 1.25kPa due to the low probability that all floors will be fully loaded at the same time.

The effects of vertical acceleration not only act on the mass of the building itself, but also on the live load or building contents. A 1g vertical acceleration doubles the load on a column, and a 2g acceleration trebles it.

The following calculation illustrates the effect of vertical acceleration on a typical internal column on lines 2 and 3.

The loads on the internal columns on lines 2 and 3 are derived from the figures in the design calculations, pages G36 to G39, as set out below at ground floor level.

Dead Load of building structure	D	1522 kN
Reduced live load	Lr	352 kN
Total unfactored load	D+Lr	1874 kN
Design Load on Column	1.4D+1.7Lr	2729 kN
Column Capacity as designed		3100 kN

36.7. The actual loads on this column with the associated vertical acceleration are as below:

Column load with 0.65gVertical	1.65(D+Lr)	3100 kN
Column Load with 0.79g (CCCC)	1.79(D+Lr)	3354 kN
Column Load with 1.88g (PRPC)	2.88(D+Lr)	5397 kN

36.8. It is noted that the column capacity, with no drift, is reached with 0.65g vertical acceleration. The axial loads induced by the vertical accelerations recorded at the two strong motion stations adjacent are shown to be well above this figure, and are high enough to initiate column failure.

36.9. It is accepted that the actual column capacity will be lower than the figure shown where the concrete strength is less than specified, and may also be lower if there is drift up to the code specified limit.

36.10. Other columns in the building will have a similar factor of safety in their design, as the column diameter is the same throughout the building, being limited to 400 millimetres diameter by Architectural considerations. The concrete covers are limited by durability concerns for the exposed columns. At the time of design, the column strengths were been adjusted to match the design loading by adjusting the concrete strength and the vertical column reinforcement. These figures in the 1982 concrete code are independent of the transverse reinforcement in the columns.

36.11. Consequently it is possible that failure could have been initiated at any level, as the design factors of safety were similar. If one particular floor level had been affected by a bad batch of concrete, then all columns at that level, poured at the same time, would have been affected, and that is the level at which failure may have been initiated. Once one floor failed, then the floors would have pancaked and affected all floors, leading to vertical collapse of the floors, the south wall falling into the middle, and the floors rotating away from the lift shaft.

36.12. As stated on page 88 of the Hyland Report, the four collapse scenarios are postulated with a common thread that collapse of an internal column triggers progressive collapse. The investigation then seeks to find capacity issues within the building which support those scenarios.

To be sustainable, these scenarios require that the building did collapse progressively. They also appear to require that the building may have prematurely separated from the shear core, that the south face of the building suffered excessive sway in the east west direction, and that the spandrel panels interacted with the columns to increase ductility demand on the columns.

With regard to the progressive collapse, the overriding common thread among the witness statements in the Hyland report is that the collapse happened quickly. It appears to have been a sudden collapse rather than a progressive one. Many witnesses referred to a bolt upwards at first, which is the manifestation of upwards vertical acceleration. The reported sensations are in my view consistent with vertical acceleration effects overloading all of the columns on one storey at a similar time, causing sudden and rapid collapse rather than progressive collapse, but with one side of the floor holding up on the connection to the shear core.

36.13. In regard to the separation of the floors from the shear core, it appears from the description of the building debris as set out in the Hyland report on page 89 that the slab connection did not fail prematurely. To quote that report:

"Review of the physical collapse evidence indicated that failure may not have occurred at the Drag Bar connections to the North core at levels4,5 and 6 prior to slab pulling away. The slabs at level 3 and 4 were seen to have hung up on the North core with their line 3 ends resting on the ground after the collapse, as seen in figure 95. This would not be expected to have occurred if they had first lost their support adjacent to the North Core. It was therefore concluded that the slab failures

observed at levels 4,5 and 6 had most likely occurred due to the floors losing their support along lines 2 and 3 as those columns collapsed"

36.14. On page 264 it is noted that the slab appeared to rotate downwards and prying the drag bars off the wall. This failure is more consistent with vertical acceleration affecting the internal columns.

36.15. With regard to the excessive sway of the south wall in an east west direction, witness 6 had the impression that the building moved in that direction. The overriding consensus appears to be that it just went in on itself, collapsing vertically within its own footprint. Witness 12 (in the Hyland report) reported that the facade of the building appeared to come off prior to the collapse. This is consistent with the spandrel panels separating from the building due to vertical accelerations, and then rotating outward from the edge of the floor slab. This may have been closely followed by failure of the columns at once, also due to the vertical acceleration.

36.16. If it is accepted that the western block wall was in fact isolated effectively from the structure during construction, then the torsional eccentricity of the building for a north south earthquake is relatively small. For an east west earthquake, the torsional loads on the building are resisted substantially by the northern east-west wall of the shear core, and the coupled shear wall on the south face. Given the disparity in stiffness of the two walls, it would be expected that if there had been significant east west sway, then the south wall would have moved the most, and that there would have been significant damage to the coupling beams in that south wall.

However, as noted on page 78 of the Hyland Report, there was very little cracking of the coupling beams, not the extensive cracking that would be expected. This would indicate that the south wall was performing its function of restricting the lateral east west sway, and was providing adequate lateral support to the gravity columns. The failure condition of the south wall is consistent with it being dragged down and inwards by the floor slabs, following failure of the internal columns due to vertical acceleration.

36.17. With regard to the effect of the spandrel panels coming in contact with the columns, and reducing the effective length of the columns, this appears to be speculation based on the acceptable construction tolerances as set out in the Concrete construction code. There appears to me little evidence to support this theory. The cracks in the south wall columns which were reported in the September earthquake damage report in Appendix K were not linked to contact with the spandrels. It would have been evident to the inspecting Engineer at that time if they had been linked.

The method used to construct the building was such that the columns were constructed first, and then the spandrel panels were site measured to fit between the columns as built. Where the contractor is expected to manoeuvre a large, wide and heavy precast element into place between two rigid columns, where the operator is in the top of a tower crane and probably unsighted of the location of his load, then the contractor is going to make sure that he has more tolerance available to him, not less. He would have site measured the gap between the columns and would have constructed the length of the spandrel panels accordingly. It was known during construction that a gap of up to 16mm wide each side between the column and the spandrel panels could be filled by a fire rated sealant which would still maintain the fire resistance of the completed spandrel construction. The builder would have used all of this tolerance, and would have located the spandrel centrally between the columns with an equal gap each side. There was no reason for him to leave a small gap on either side, as that gap would then be impractical to seal and make waterproof.

37. Design Issues

37.1. In Section 9 a number of issues are raised in terms of the compliance of the building, which are discussed as follows.

37.1.1. Building Interstorey Drift limits

The building was required by NZS 4203:1984 to comply with a drift limit of 0.83% of the interstorey height. The building did comply with this requirement, as set out in the structural calculations provided, on pages S15 and S16. This is accepted in the Hyland report, in appendix F. It is inferred in the Hyland report that closer tolerances on drift are required by the commentary clauses to NZS4203 for secondary frames. It was considered at the time of design that the commentary clauses were not mandatory, and that in any case the internal beams were not designed to be moment resisting frames, and were detailed accordingly.

37.1.2. **Drift Capacity of Columns**

The drift limit for the building as required by the code was achieved due to the strength and stiffness of the wall elements, and with no contribution from any concrete frames. The internal beams are not considered to be frames, rather continuous beams with simple props below, so detailing as for a frame is not required. If this building had been similarly constructed using structural steel rather than concrete, the columns would have been fixed with a simple bolted connection. In this case the bottom bars in the beams were made discontinuous, and shear reinforcement in the beam column joints was omitted to achieve the same purpose.

37.1.3. Minimum Shear Capacity of Columns

As previously noted, the columns were designed to be pin ended, with no contribution to the horizontal shear capacity of the building. Accordingly shear reinforcement was not considered to be necessary.

37.1.4. **Spandrel Panel Separation**

There is no evidence that adequate separation between the columns and the spandrels was not provided. It is noted on page 110 of the Hyland report that a minimum gap of 7mm would have been required. The specified gap was 10mm, and the most likely construction gap would have been closer to 16mm on both sides of the columns.

37.1.5. **Beam Column Joints**

The internal supports are not considered to be moment resisting frames. Accordingly reinforcement of the beam column joints to provide moment resistance is not required. The horizontal reinforcement in the top of the floor beams is continuous through the beam column joints, which would have ensured that the beams could not become separated from the columns.

37.1.6. **Plan Asymmetry and Vertical Irregularity**

The then Building Code, NZS 4203, provided limits on plan irregularity in so far that a building may not be designed by the Equivalent Static method if certain regularity criteria are not met. If a building exceeds those criteria, then it shall be designed by another method such as the Modal Response Spectrum Analysis. Accordingly, the building was designed by the Modal Response Spectrum Method, using the ETABS program.

The internal lines of support were not considered to be secondary moment resisting frames, and were not included in the seismic analysis.

37.1.7. **Wall on Line A**

The top course of these walls was seismically separated from the floors above. It is accepted that this requirement was not adequately shown on the drawings, but this factor was rectified on site and the walls were constructed in accordance with the design assumptions. With regard to the existence of mortar in one of the vertical construction joints rather than flexible sealant, it has been explained that the outside face was inaccessible during construction, so it would have been difficult to provide flexible sealant in that position at that time. There was no structural connection between the masonry wall and the previously poured column such as by horizontal reinforcement. Vertically placed mortar on the far face of a wall would have been very weak and it is not expected that it would have made a significant connection between the two elements.

37.1.8. **Diaphragm Connection**

The floor diaphragm was connected to the shear wall in the core in a number of ways. Three of the precast beams on line 4 are built in to the end of two of the outstanding walls. The four bottom bars of each beam pass between the vertical bars in the ends of the shear walls, and the four top bars also pass continuously through the joints. There are a number of slab ties cast into the face of the shear walls which lap with the mesh in the topping.

The steel Hi Bond tray decking forms an effective floor diaphragm, as it is well restrained against buckling by the adjacent concrete. Where the Hi Bond is discontinuous at the precast beams, there are additional saddle bars over the beams which provide effective continuity. There is fire emergency reinforcement in the Hi Bond trays, as well as mesh throughout the floor.

The beams on the perimeter of the building were constructed with exposed stirrups, and the mesh in the floor was effectively lapped with that reinforcement prior to pouring the topping.

It is accepted that current best practice recommends that bar reinforcement be used in floor slabs in preference to welded wire mesh. However the codes at the time allowed the use of mesh in this location, as did the current code until very recently.

As previously noted, the inspection of the floor slab connection to the shear core suggests that the connection between the floor diaphragm and the shear core did not fail in tension, and it performed its function as designed until the columns failed, in my opinion under excessive axial loading.

37.1.9. **Robustness**

This is a difficult concept to quantify. The building was designed such that the shear walls would provide lateral shear resistance to the building, with no assistance from the internal columns or beams. I believe that the walls and their foundations were designed adequately to carry out that function. The main shear walls remained standing with little damage, and the south wall was substantially undamaged until it was brought down by the floor slabs following failure of the columns due to vertical accelerations.

I am not aware of any section in the current building code which prohibits a design concept with similar features to the CTV building.

37.1.10. **Documentation**

Construction Joints. The preparation of construction joints is included in the concrete construction standard, along with many other details for concrete construction. It is contended that it is not necessary to repeat all details of such a standard in the specifications for a specific project.

Block Masonry Separation. It is accepted that the top course of the block walls was not detailed to be isolated on the drawings. However, this was rectified on site during routine pre pour inspections, and the top joint was isolated as required to comply with the design assumptions.

Starter bars in beams. Starter bars were not required in the beams 18 and 22, as the stirrups in these beams were exposed, and were lapped with the mesh in the floor before pouring the floor. This provided an adequate connection to the beams.

Spandrel Panel Separation. An adequate gap between the columns and the spandrel panels was shown on the drawings, and as previously discussed, it is likely that this gap was constructed wider than this.

STANDARDS AND CODE ISSUES

38. I agree with many of the recommendations in the reports.

Following observations of a number of buildings following the earthquakes, it appears that many buildings have experienced lateral sway which is somewhat larger than expected from the design calculations and computer analysis.

This may be partly due to assumptions made during the modelling of the building, such as the assumption that shear walls are rigidly fixed at the base; no allowance being made for foundation flexibility; the flexibility of the soil below the foundations in liquefiable soils; the degree of cracking in the concrete; the state of the concrete and reinforcement following earlier earthquakes; and the reinforcement content of the walls and columns.

There was no provision for vertical acceleration on buildings in NZS4203:1984. Even the current building code, "NZS1170.5:2004 Structural Design Actions-

New Zealand" only requires that the vertical acceleration be assessed at 0.7 times the horizontal acceleration.

As previously noted, the paper written by Bradley and Cubrinovski, titled "Near Source Strong Ground Motions Observed In The 22 February 2011 Christchurch Earthquake" reports on the observed vertical accelerations. The vertical accelerations were three times the horizontal acceleration at the Pages Road Pumping Station. That report goes further to report, at page 189, relative to both the 22 February Christchurch earthquake, and the 4 September 2010 Darfield earthquakes,

" *it can be clearly seen that V to H ratios above 1.0 are frequently observed for distances up to Rrup =40km in both these events (as well as other historical earthquakes worldwide), and hence the code prescription of 0.7 is, without question, significantly un-conservative.*"

39. It would be my hope that, due to the excessive lateral movement which takes place in an earthquake, that the present Code be amended to also require that all columns be detailed for ductility, irrespective of the calculated lateral sway of the structure.

D Harding

David Harding

Date: 5/6/12

NEAR-SOURCE STRONG GROUND MOTIONS OBSERVED IN THE 22 FEBRUARY 2011 CHRISTCHURCH EARTHQUAKE

Brendon A. Bradley[1], Misko Cubrinovski[1]

SUMMARY

This manuscript provides a critical examination of the ground motions recorded in the near-source region resulting from the 22 February 2011 Christchurch earthquake. Particular attention is given to reconciling the observed spatial distribution of ground motions in terms of physical phenomena related to source, path and site effects. The large number of near-source observed strong ground motions show clear evidence of: forward-directivity, basin generated surface waves, liquefaction and other significant nonlinear site response. The pseudo-acceleration response spectra (SA) amplitudes and significant duration of strong motions agree well with empirical prediction models, except at long vibration periods where the influence of basin-generated surface waves and nonlinear site response are significant and not adequately accounted for in empirical SA models. Pseudo-acceleration response spectra are also compared with those observed in the 4 September 2010 Darfield earthquake and routine design response spectra used in order to emphasise the amplitude of ground shaking and elucidate the importance of local geotechnical characteristics on surface ground motions. The characteristics of the observed vertical component accelerations are shown to be strongly dependent on source-to-site distance and are comparable with those from the 4 September 2010 Darfield earthquake, implying the large amplitudes observed are simply a result of many observations at close distances rather than a peculiar source effect.

INTRODUCTION

On 22 February 2011 at 12:51pm local time, a moment magnitude M_w6.3 earthquake occurred beneath the city of Christchurch, New Zealand, causing an unparalleled level of damage in the country's history, and the largest number of causalities since the 1931 Hawkes Bay (Napier) earthquake. Compared to the preceding 4 September 2010 M_w7.1 Darfield earthquake, which occurred approximately 35 km to the west of Christchurch, the close proximity of the 22 February event lead to ground motions of significantly higher amplitude in the densely populated regions of Christchurch. As a result of these significantly larger ground motions, structures in general, and commercial structures in the central business district in particular, were subjected to severe seismic demands and, combined with the event timing structural collapses accounted for the majority of the 182 causalities [1].

The following section provides a brief overview of the tectonic and geologic setting of the Canterbury region in order to provide context for the observed ground motions which are discussed in subsequent sections on the basis of source, path and site effects, and comparisons with empirical prediction models, design guidelines, and those of the 4 September 2010 Darfield earthquake.

TECTONIC AND GEOLOGIC SETTING

New Zealand resides on the boundary of the Pacific and Australian plates (Figure 1) and its active tectonics are dominated by: (i) oblique subduction of the Pacific plate beneath the Australian plate along the Hikurangi trough in the North island; (ii) oblique subduction of the Australian plate beneath the Pacific plate along the Puysegur trench in the south west of the South island; and (iii) oblique, right lateral slip along numerous crustal faults in the axial tectonic belt, of which the 650-km long Alpine fault is inferred to accommodate approximately 70-75% of the approximately 40 mm/yr plate motion [2, 3].

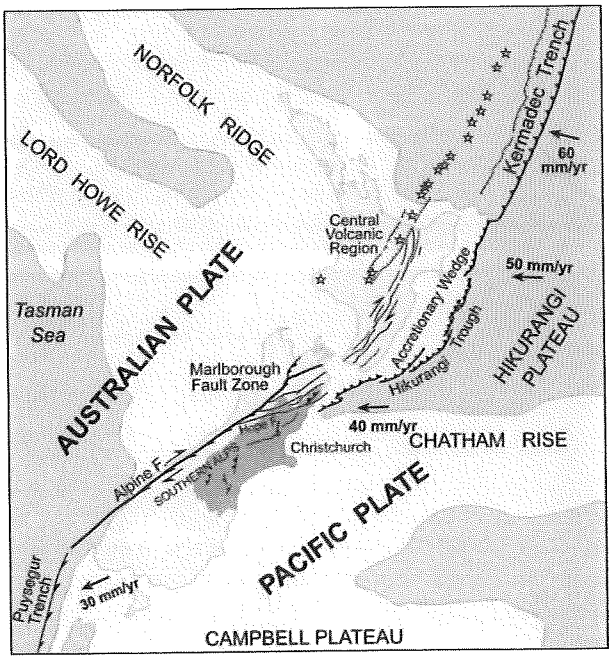

Figure 1: Tectonic setting of New Zealand.

[1] *Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch (member)*

There are numerous identified faults in the Southern Alps and eastern foothills [4] and several significant earthquakes (i.e. $M_w > 6$) have occurred in this region in the past 150 years, most notably the 4 September 2010 M_w7.1 Darfield earthquake [5]. The M_w6.3 Christchurch earthquake occurred at 12:51pm on Tuesday 22 February 2011 beneath Christchurch, New Zealand's second largest city, and represents the most significant earthquake in the unfolding seismic sequence in the Canterbury region since the Darfield earthquake. Herein, a moment magnitude of 6.3 is used with reference to this event, however it is noted that reported values range from M_w6.3 for a geodetic finite fault model [6], 6.2 for regional moment tensor solutions (J. Ristau, pers. comm.), and 6.1 for the USGS teleseismic moment tensor solution. The M_w6.3 event occurred on a previously unrecognised deeply-dipping blind fault, which trends north-east to south-west (the location relative to Christchurch is presented in the context of observed ground motions subsequently). Figure 2 illustrates the inferred slip distribution on the fault obtained by Beavan *et al.* [6]. It can be seen that slip on the fault occurred obliquely with both significant up-dip and along-strike components (average rake, $\lambda = 146°$). For the purpose of the subsequent engineering analysis of strong ground motion, the Beavan *et al.* finite fault model was 'trimmed' using the methodology of Somerville *et al.* [7], which resulted in the removal of 1 km from the Northeast and Southwest extents of Figure 2. The resulting 'trimmed' fault therefore has dimensions of 15 km along-strike and 8km down-dip, giving a total area of 120km^2.

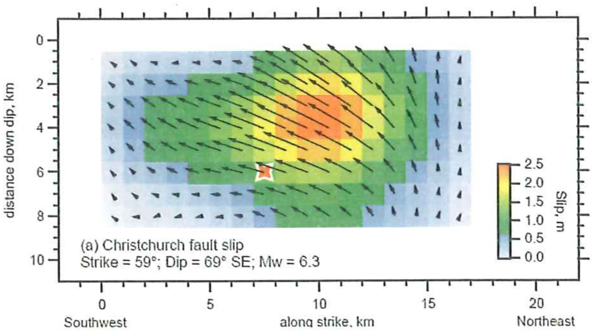

***Figure 2:** Distribution of fault slip inferred in the 22/02/2011 Christchurch earthquake [6]. Arrows indicate the slip vector and the inferred hypocenter is indicated by a star.*

Christchurch is located on the Canterbury plains, a fan deposit resulting from the numerous rivers flowing eastward from the foothills of the Southern Alps [8]. In the vicinity of Christchurch, the Canterbury plains are comprised of a complex sequence of gravels interbedded with silt, clay, peat, and shelly sands. The fine sediments form aquicludes and aquitards between the gravel aquifers, and with the nearby coastline to the east, result in the majority of Christchurch having a water table less than 5 m depth, with the majority of the area including, and to the east of, the central business district having a water table less than 1 m from the surface [8]. The postglacial 'Christchurch formation' created by estuarine, lagoonal, dune, and coastal swamp deposits (containing gravel, sand, silt, clay, shell and peat) is the predominant surface geology layer in the Christchurch area which outcrops up to 11 km west of the coast and has a depth of approximately 40 km along the coast itself [8]. At the southeast edge of Christchurch lies the extinct Banks Peninsula volcanic complex.

SUMMARY OF OBSERVED STRONG MOTIONS

Volume 1 ground motion records were obtained from GeoNet (www.geonet.org.nz/) and processed on a record-by-record basis. Table 1 presents a summary of the ground motions in the wider Christchurch region that were recorded within a source-to-site distance of $R_{rup} = 20$ km, including: station site class (*SC*) according to the current New Zealand loading standard, NZS1170.5:2004 [9], peak ground acceleration (*PGA*), and peak ground velocity (*PGV*) for geometric mean horizontal component; and peak vertical ground acceleration (PGA_V). It can be seen that significant ground motions were recorded in this event with ground motions of up to 1.41g in the horizontal component (at Heathcote Valley, HVSC), and 7 and 16 records having PGA's exceeding 0.4g and 0.2g, respectively. To put such numbers in context it is noted that prior to the Darfield earthquake the maximum recorded PGA in New Zealand was 0.39g [10]. Figure 3 illustrates the spatial distribution of fault-normal, fault-parallel, and vertical ground motions observed in Christchurch City. The subsequent sections elaborate on the salient features which can be observed in Figure 3 and Table 1.

Table 1: Summary of observed ground motions at strong motion stations in the 22 February 2011 Christchurch earthquake.

Station Name	Code	*SC*	R_{rup} (km)	*PGA* (g)	PGA_v (g)	Station Name	Code	*SC*	R_{rup} (km)	*PGA* (g)	PGA_v (g)
Canterbury Aero Club	CACS	*D*	12.8	0.21	0.19	Lyttelton Port Naval Point	LPOC	*C*	6.6	0.34	0.39
Christchurch Botanic Gardens	CBGS	*D*	4.7	0.50	0.35	North New Brighton School	NNBS	*E*	3.8	0.67	0.80
Christchurch Cathedral College	CCCC	*D*	2.8	0.43	0.79	Papanui High School	PPHS	*D*	8.6	0.21	0.21
Christchurch Hospital	CHHC	*D*	3.8	0.37	0.62	Pages Rd Pumping Station	PRPC	*E*	2.5	0.63	1.88
Cashmere High School	CMHS	*D*	1.4	0.37	0.85	Christchurch Resthaven	REHS	*D*	4.7	0.52	0.51
Hulverstone Dr Pumping Station	HPSC	*E*	3.9	0.22	1.03	Riccarton High School	RHSC	*D*	6.5	0.28	0.19
Heathcote Valley School	HVSC	*C*	4.0	1.41	2.21	Rolleston School	ROLC	*D*	19.6	0.18	0.08
Kaipoi North School	KPOC	*E*	17.4	0.20	0.06	Shirley Library	SHLC	*D*	5.1	0.33	0.49
Lincoln School	LINC	*D*	13.6	0.12	0.09	Styx Mill Transfer Station	SMTC	*D*	10.8	0.16	0.17
Lyttelton Port	LPCC	*B*	7.1	0.92	0.51	Templeton School	TPLC	*D*	12.5	0.11	0.16

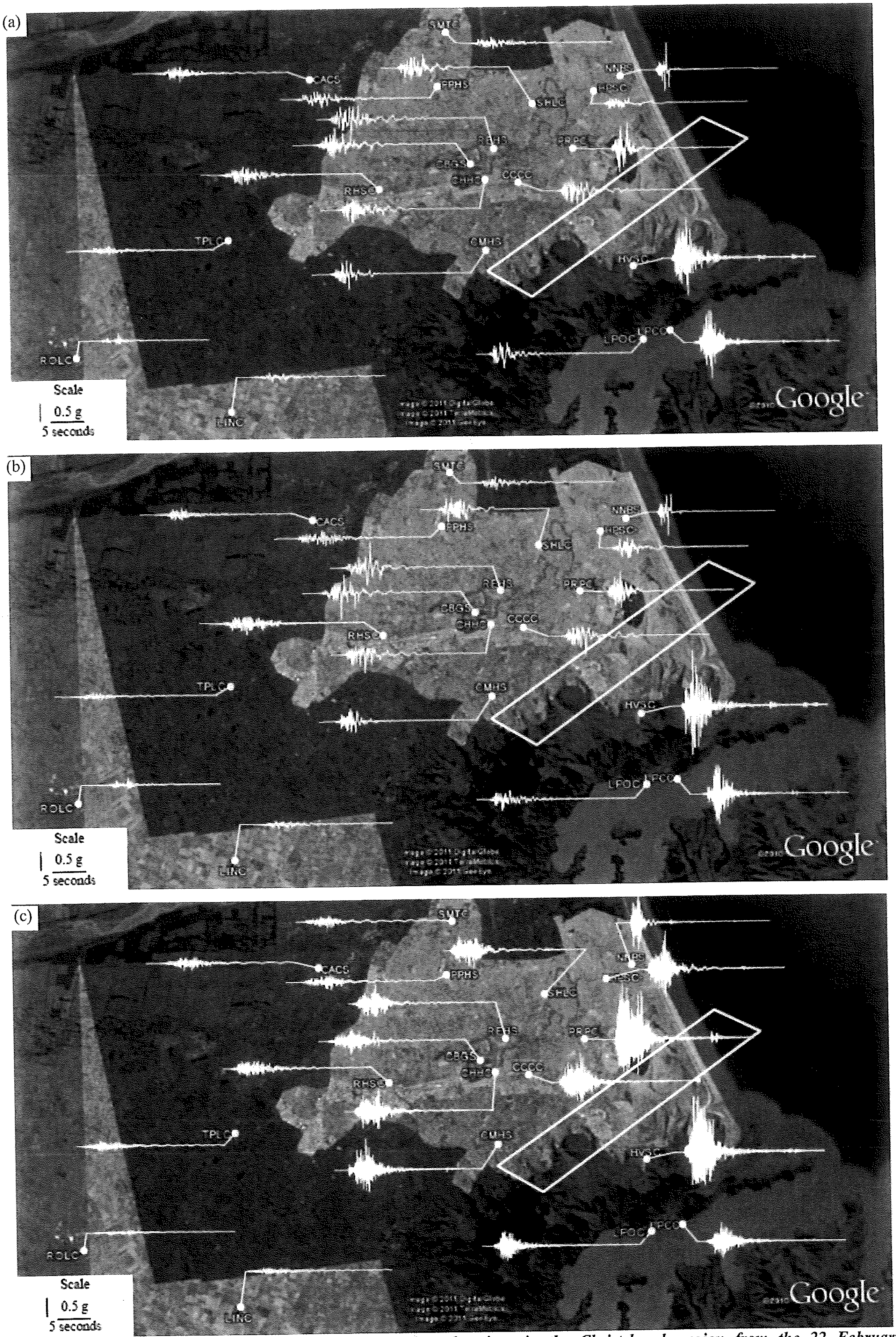

Figure 3: Observed acceleration time histories at various locations in the Christchurch region from the 22 February earthquake: (a) fault-normal horizontal; (b) fault-parallel horizontal; and (c) vertical components.

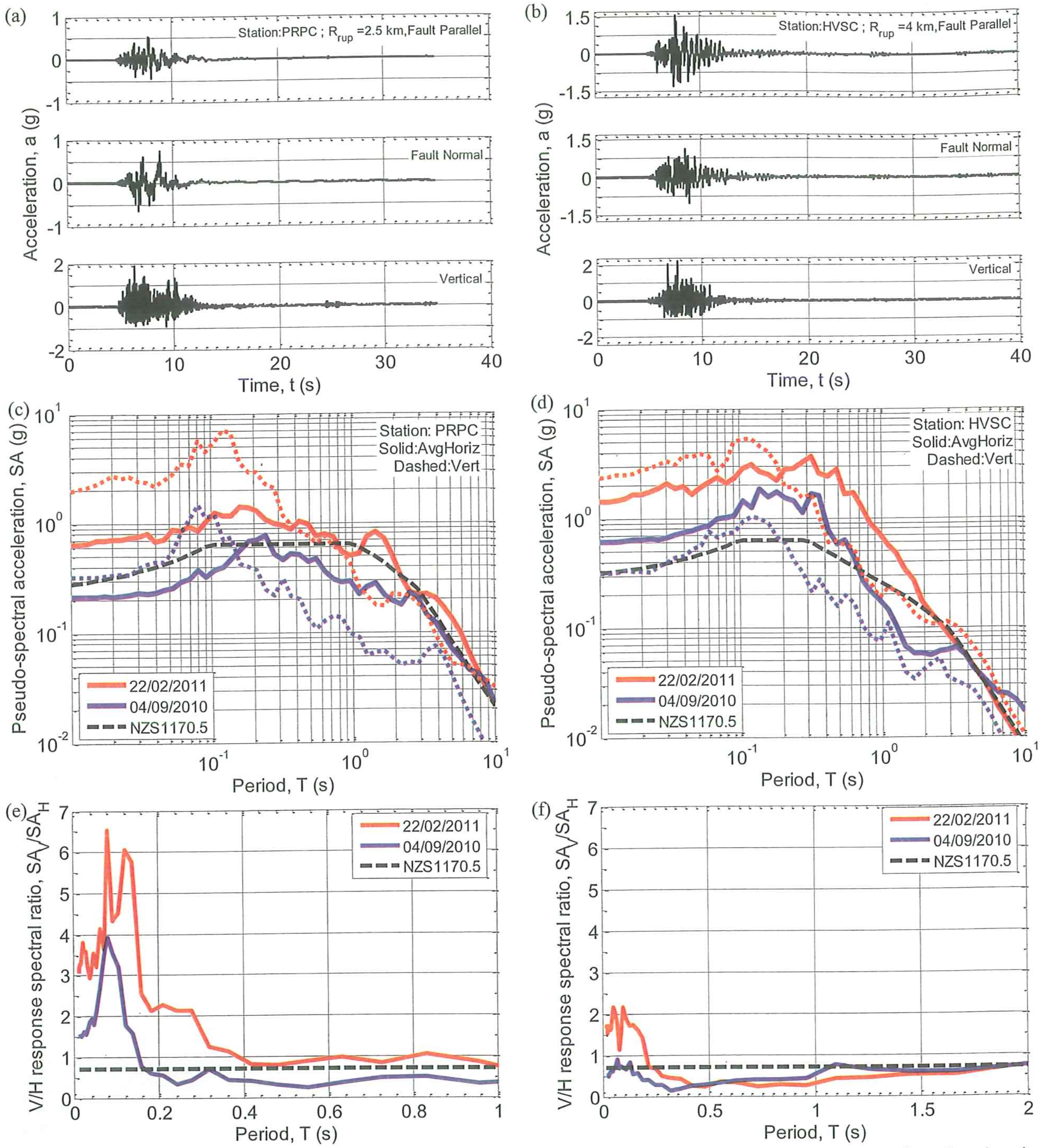

Figure 4: ***Extreme ground motions observed at Pages Road (PRPC) and Heathcote Valley (HVSC) in terms of acceleration time history, pseudo-acceleration response spectra, and vertical to horizontal spectral ratios. Note the different scale used for vertical acceleration time histories in Figure 4a and Figure 4b with that of the horizontal.***

EXTREME GROUND MOTIONS

Examination of Figure 3 illustrates that very significant ground motion amplitudes were recorded in both the horizontal and vertical components at Pages Road (PRPC) and Heathcote Valley (HVSC), which are reproduced at a larger scale in Figure 4a and Figure 4b, respectively. In particular, maximum PGA's in the vertical component of 2.21g and 1.88g were observed at HVSC and PRPC, respectively. The vertical acceleration time histories at these two sites are also inferred to exhibit the so-called 'trampoline effect' [11, 12] caused by separation of surficial soil layers in tension, limiting peak negative vertical accelerations to approximately -1g. As discussed subsequently, the ground motion at PRPC also experienced significant forward directivity effects which are evident in the long-period content of the fault normal component in Figure 4a.

Figure 4c and Figure 4d illustrate the geometric mean horizontal and vertical pseudo-acceleration response spectra at PRPC and HVSC during both the Christchurch and 4 September 2010 Darfield earthquakes, and Figure 4e and Figure 4f illustrate the vertical-to-horizontal spectral ratios at these two sites in these two events. It can be clearly seen that the nature of the surface ground motion at each of these sites is similar in each of the two events, but fundamentally different between the two sites. For example, the response at PRPC is dominated by a relatively 'flat' response spectrum for high frequencies, indicative of nonlinear response in soil soft deposits. Furthermore, the vertical ground motion amplitude at high frequencies is particularly large (i.e. Figure 4e), indicating a soil deposit with high compressibility, that is, low

P-wave velocity (e.g. clay, silt, peat). In contrast, the response at HVSC is characterised by large short period (i.e. $T < 0.4$s) ground motion with a rapid fall-off in spectral ordinates at longer periods (the exception being the increase for the Darfield earthquake at long periods due to the forward directivity pulse [13]). The vertical-to-horizontal spectral ratio is also notably lower than that at PRPC and only larger than 1.0 for very high frequencies. In-depth analysis of the strong ground motion at HVSC indicates a strong basin edge effect at this site due to its location near the Port Hills, resulting in constructive interference between direct S-waves propagating through the underlying basin, and diffracted Rayleigh waves induced at the basin edge [13].

NEAR SOURCE FORWARD DIRECTIVITY

In the near-source region ground motions may exhibit forward directivity effects due to the rupture front and direction of slip being co-aligned with the direction toward the site of interest. While the finite fault model in Figure 2 does not provide information on the temporal evolution of rupture, based on the central location of the inferred hypocenter, the direction of slip is not well aligned with an elliptically inferred rupture front. As a result, it is expected that rupture directivity effects will only be important over a small area of the earth's surface, relative to other possible rupture scenarios [14]. This is in contrast to the 4 September 2010 Darfield earthquake, in which strike-slip rupture occurred bilaterally on the Greendale fault and forward directivity effects were significant for all locations in Christchurch city [13].

Figure 5a illustrates the three component velocity time history at Pages road (PRPC), where forward directivity effects can be seen in the fault-normal component manifested as the large ground velocities of low frequency which cause a PGV of approximately 100 cm/s in the fault-normal component, while the fault-parallel component PGV is approximately 40 cm/s. This is further evident in the polar plot of the velocity trajectory at PRPC in Figure 5c. Figure 5b illustrates the three component velocity time history at Christchurch Hospital (CHHC) where a velocity pulse in the fault normal component is not clearly evident (although there is some evidence in the fault-parallel component indicating complex rupture), and the large velocity amplitudes are the result of surface waves (elaborated upon subsequently). Again the lack of a strong forward directivity effect is evident in the velocity trajectory shown in Figure 5d, in which no clear polarity of large amplitude velocity is observed in the fault normal direction, and in fact the peak velocity is observed in the fault parallel component.

Figure 6 illustrates the observed and empirically predicted pseudo-acceleration response spectra at CHHC with and without the consideration of directivity effects. The empirical directivity effect was estimated using the model of Shahi and Baker [15]. It can be seen that the predicted effect of forward directivity is relatively small (compared to the basin depth effect discussed subsequently) because of the small propagation distance from the hypocenter along the fault plane toward the site (which gives a low probability of observing a velocity pulse in the model of Shahi and Baker [15]), and also the lack of alignment between the inferred rupture front and the slip vector (which isn't considered in the model, but obviously physically affects the magnitude of forward directivity).

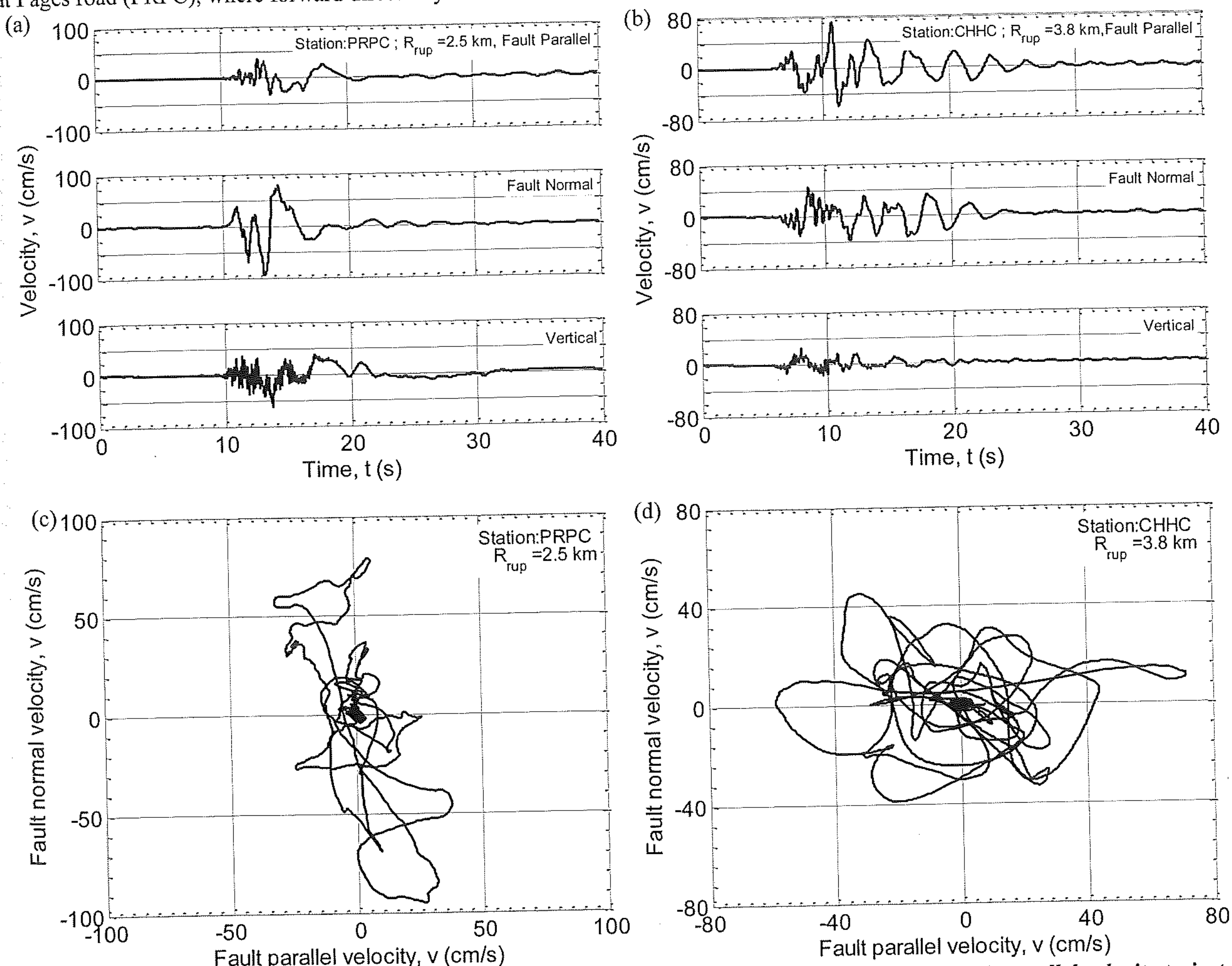

Figure 5: Velocity time histories and corresponding horizontal trajectory of fault normal and fault parallel velocity trajectory at Pages Road (PRPC) and Christchurch Hospital CHHC).

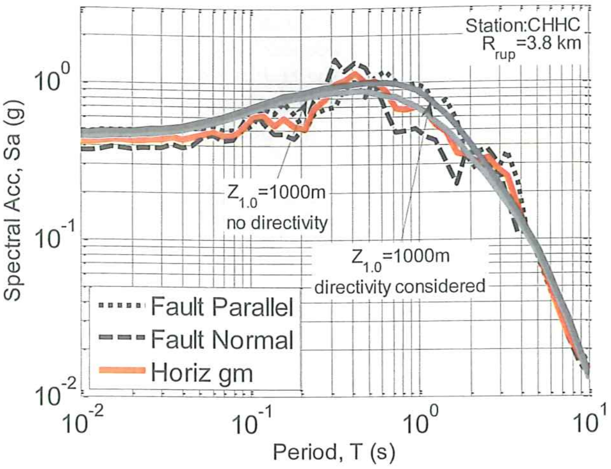

Figure 6: Empirically predicted effect of directivity on spectral amplitudes at Christchurch Hospital (CHHC). The prediction shown is for the horizontal geometric mean.

BASIN-GENERATED SURFACE WAVES

As previously mentioned, Christchurch is located on a sedimentary fan deposit with the volcanic rock of Banks peninsula located to the south east. While specific mechanical and geometrical details of the predominant sedimentary basin layers are not well known, previous investigation has revealed the depth of gravel layers is in excess of 500 m, with basement rock inferred to be at depths in excess of 2.0 km at various locations [13, 16].

Figure 7a provides a schematic illustration of the deep geology of the region along a plane trending south east to north west. Figure 7a also illustrates one possible ray path from the M_w6.3 rupture in which seismic waves propagate up-dip and enter the sedimentary basin through its thickening edge. The large post-critical incidence angles of such waves cause reflections which lead to a waveguide effect in which surface waves propagate across the basin resulting in enhanced long period ground motion amplitudes and shaking duration [17]. Figure 7b illustrates the fault-normal, fault-parallel, and geometric mean horizontal pseudo-response spectra at Christchurch Hospital (CHHC), located at a source-to-site distance of R_{rup} = 3.8 km on the footwall. Also shown in Figure 7b is the predicted median response spectra for the site using the Bradley [10] empirical model for two different values of a proxy for basin depth. The Bradley [10] model is based on the Chiou and Youngs [18] model with New Zealand-specific modifications. Basin effects are accounted for in the model through the use of the parameter $Z_{1.0}$, which represents the depth to sediments with shear wave velocity, V_s = 1.0 km/s. For site class D conditions (a nominal 30-m average shear wave velocity of $V_{s,30}$ = 250 m/s) the default value of $Z_{1.0}$ is on the order of 300 m [18]. Figure 7b illustrates that spectral amplitudes at CHHC for periods greater than 0.3 seconds are under-predicted using this default $Z_{1.0}$ value. Given the thickness of gravels in the Christchurch basin is known to be greater than 500 m implies that $Z_{1.0}$ would be significantly greater than 500 m. Figure 7b also illustrates the predicted spectral amplitudes, using a value of $Z_{1.0}$ = 1000 m, where it can be seen that the empirical prediction of long period spectral amplitudes is significantly increased, compared with those using $Z_{1.0}$ = 300 m, in line with the observed amplitudes.

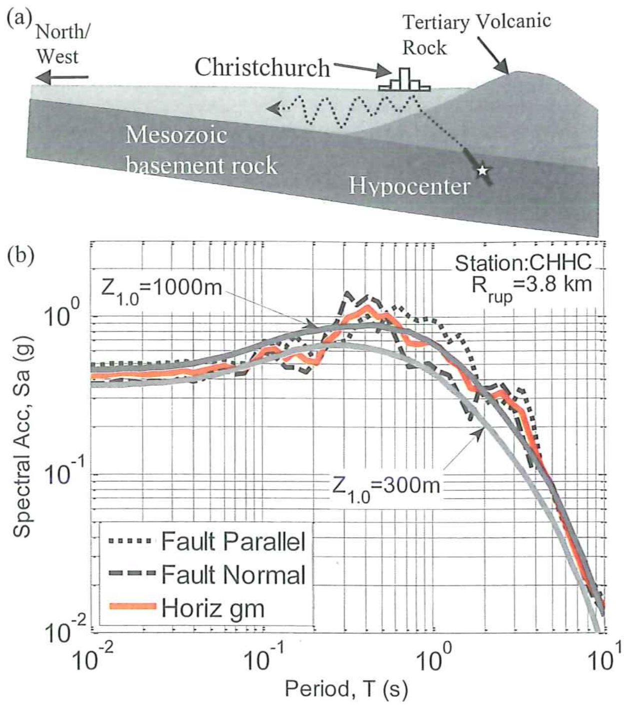

Figure 7: (a) Schematic illustration of waveguide effects occurring in the sedimentary basin underlying Christchurch (not to scale); and (b) influence of basin depth on pseudo-spectral acceleration ordinates predicted empirically compared with that observed at Christchurch Hospital (CHHC). The prediction shown is for the horizontal geometric mean.

The increase in amplitude of horizontal ground motion at long periods illustrated at Christchurch hospital (CHHC) was also observed at numerous other locations in the region as depicted at four locations in Figure 8. At close source-to-site distances clearly discerning surface wave contribution is not trivial due to the overlap in time of the first surface wave arrivals and scattered S-waves. Both Papanui (PPHS) and Styx Mill (SMTC) however illustrate several long period oscillations subsequent to the majority of S-wave arrivals. The significant amplitude Rayleigh surface waves in the vertical component at SMTC are particularly noticeable, and are also observed at other strong motion stations (i.e. Figure 3c). The significance of basin-induced surface waves becomes more visible and predominant as the distance from the .causative fault increases, both as a result of the different wave propagation velocities of the body and surface waves (so they arrive at different times and are easier to visually bracket), and also because of the fact that body waves geometrically attenuate at a higher rate (R^{-1}) than surface waves ($R^{-1/2}$) with distance. As a result it can be seen in Figure 8 that, at both Templeton (TPLC) to the west of Christchurch, and Kaiapoi to the north, the duration and also amplitude of the surface waves relative to body waves significantly increases. At KPOC in particular, it can be seen that despite being 20 km from the causative fault, high frequency ground motion occurs followed by significant surface wave amplitudes with PGV's up to 20 cm/s. The large amplification of high frequency ground motion followed by surface waves was also observed at KPOC during the Darfield earthquake [13], and combined with the very loose soil deposits, indicates how liquefaction occurred in this region during both the earthquakes, despite source-to-site distances of R_{rup} = 27.6 km and 17.4 km, respectively.

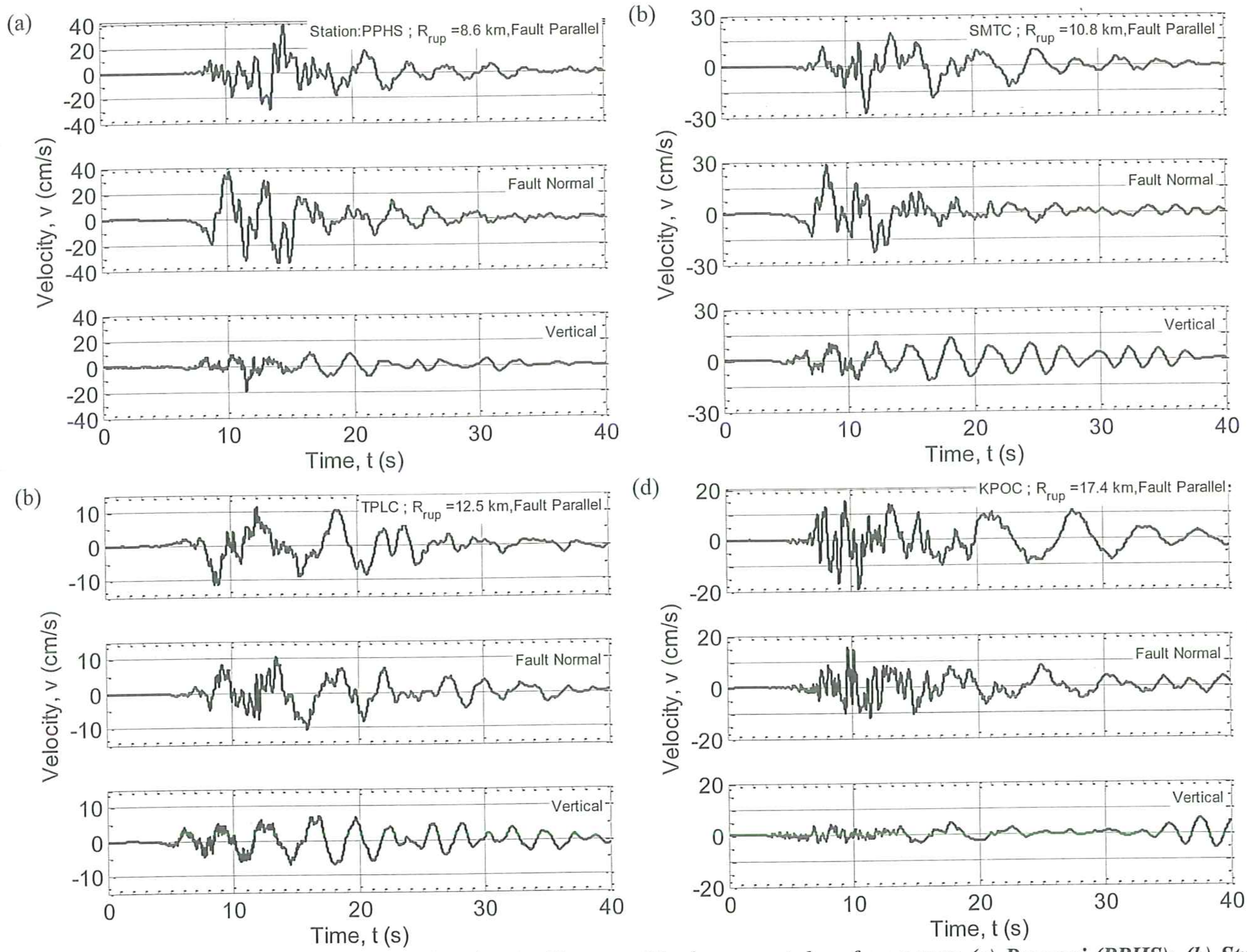

Figure 8: Velocity time histories illustrating the significance of basin-generated surface waves: (a) Papanui (PPHS); (b) Styx Mill (SMTC); (c) Templeton (TPLC); and (d) Kaiapoi (KPOC).

NONLINEAR NEAR-SURFACE RESPONSE AT SOIL SITES

Near-surface response at Lyttelton Port

When interpreting the observed ground motions in Figure 3, it is worth recalling that only the Lyttelton Port (LPCC) station to the southeast of Christchurch is located on engineering bedrock (i.e. site class B). Stations HVSC and LPOC located near the edge of the Port Hills rock outcrop are site class C, while all remaining stations are situated on the Christchurch sedimentary basin and are predominantly site class D, with those having (identified) soft soil layers deemed site class E. Unfortunately at present the site characterisation of strong motion stations in the Christchurch region, and New Zealand in general, is relatively poor with the above site classes determined from geological maps, and details such P- and S-wave velocity, SPT, and CPT data not available. Clearly, obtaining such information is a high priority to rigorously understand the site-specific features of observed ground motions, and is the focus of immediate studies. Nevertheless, a wealth of insight can still be obtained from inspection and analysis of the observed ground motions.

Direct observation of the difference between soil and rock sites, and the impact of nonlinear response can be made by comparing the ground motions observed at LPCC and LPOC located at Lyttelton Port approximately 1 km apart. The LPCC instrument is located on engineering bedrock, and the site conditions at LPOC are inferred as a relatively thin (~30 m) colluvium layer comprised primarily of silt and clay (J. Berrill, pers. comm.). In addition to a comparison of the acceleration time histories in Figure 3, Figure 9 illustrates the pseudo-acceleration response spectra of the geometric mean horizontal and vertical ground motion components at the two sites. It can be seen that the observed horizontal ground motion at the LPOC site has significantly lower high frequency ground motion amplitude, longer predominant period (Table 1), larger peak ground velocity, and larger significant duration, relative to LPCC, inferred as the result of nonlinear response of the surficial soils. In contrast to the significant difference in horizontal ground motion, it can be seen that there is relatively little difference between the vertical ground motion at LPCC and LPOC, with peak vertical accelerations of 0.51g and 0.39g, respectively.

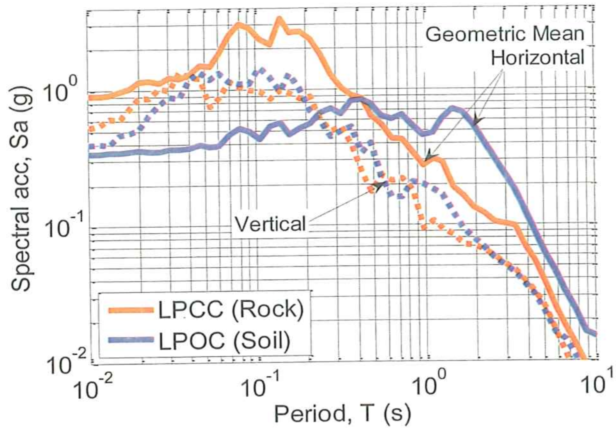

Figure 9: Comparison of geometric mean horizontal and vertical response spectra observed at two stations in Lyttelton Port, one on outcropping rock (LPCC), the other on soil (LPOC).

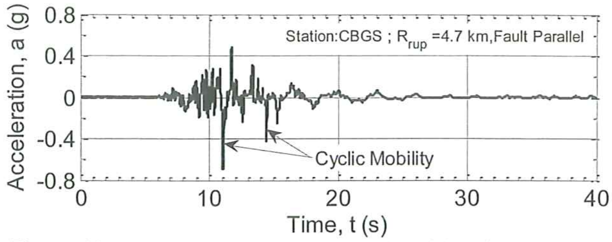

Figure 10: ***'Spikes' in acceleration time histories resulting cyclic mobility in underlying liquefied soils.***

Evidence of liquefaction

One of the major causes of damage in the M_w6.3 Christchurch earthquake resulted from the severity and spatial extent of liquefaction in residential, commercial and industrial areas. The horizontal components of acceleration depicted in Figure 3a and Figure 3b show clear evidence of liquefaction phenomena in the central business district and eastern suburbs which are located in the near-source region beyond the up-dip projection of the fault plane. For clarity, an example ground motion for Canterbury Botanic Garden (CBGS) is shown in Figure 10, for which the acceleration 'spikes' due to cyclic mobility are explicitly annotated. Such phenomena occur as a result of the rapid increase in shear stiffness and strength during large shear displacement in soils as a result of volumetric dilation, which consequently allows for the propagation of high frequency ground motion.

In the central business district (i.e. REHS, CBGS, CHHC, CCCC), Cashmere (CMHS) and Shirley (SHLC), evidence of liquefaction at depth is inferred based on the manifested reduction in high frequency content of ground motion following several seconds of S-wave arrivals, and the subsequent acceleration 'spikes'. In the eastern suburbs (i.e. PRPC, HPSC, NNBS), the picture is somewhat more complex. The ground motion at Pages road (PRPC) also has some of the characteristics discussed above, but in addition exhibits very high accelerations in the fault-normal and vertical directions, which likely result from both surficial soil and source effects, due to its proximity to the up-dip projection of the slip asperity (as previously noted). The ground motion at North New Brighton (NNBS) exhibits several seconds of cyclic mobility before an abrupt reduction in acceleration amplitude resulting in a very short significant duration of 2.4 seconds (Table 1). The ground motion observed at Hulverstone Drive (HPSC) is also of interest due to the relatively small horizontal component acceleration amplitudes compared with what might be expected at such a near-source location (including observed shaking at nearby stations), and relative to its high vertical accelerations.

No significant signs of liquefaction are evident in the ground motions recorded to the west of those discussed above, which results from three factors: (i) a reduction in amplitude of ground shaking; (ii) a change in surficial soil characterization; and (iii) an increase in water table depth as noted previously. Given the observed spatial extent of liquefaction in the Darfield earthquake [19], in which the majority of this western region was unaffected by liquefaction, despite been subjected to generally stronger shaking than the eastern regions (where liquefaction was prevalent), it can be logically concluded that the character and in-situ state of the soils are the predominant reason for the absence of liquefaction in the western Christchurch region [8].

VERTICAL GROUND MOTION

As previously noted with reference to Figure 3c, large ground motions were observed in the vertical component at various locations in this earthquake. Such large vertical accelerations can be understood physically, because the majority of strong motion stations are located on soil sites, and for soil sites in sedimentary basins large vertical accelerations at near-source locations can result from the conversion of inclined SV-waves to P-waves at the sedimentary basin interface which are subsequently amplified and refracted towards vertical incidence due to the basin P-wave gradient [20]. Secondly, the relatively steep dip of the fault plane ($\delta = 69°$), and up-dip rupture propagation also likely resulted in a large component of fault slip oriented in the vertical direction.

Figure 11 illustrates the ratio of peak vertical acceleration and peak horizontal acceleration observed at the near-source strong motion sites in the Christchurch earthquake. For comparison, the empirical model of Bozorgnia and Campbell [21] is also shown. It can be seen that peak vertical-to-horizontal ground acceleration ratios of up to 4.8 were observed. The peak vertical-to-horizontal ground acceleration ratios show a rapid decay with source-to-site distance and it can be seen that the observed ratios compare favourably with the Bozorgnia and Campbell empirical model for source-to-site distances beyond 5 km, but significantly under-predict the ratios at closer distances. In Figure 11, data are also differentiated by whether liquefaction was observed (as discussed previously). It can be seen that almost all strong motion records at distances less than 5km show liquefaction evidence (the exception being HVSC). At the aforementioned sites (with source-to-site distances are less than 5 km), the large peak vertical-to-horizontal ground acceleration ratios observed are interpreted to be the result of significant non-linear soil behaviour (including liquefaction) which generally results in more of a reduction in peak horizontal accelerations than peak vertical accelerations (e.g. as seen in Figure 9).

To explore the results in Figure 11 in more detail, and provide addition insight, Figure 12a illustrates the geometric mean horizontal pseudo-acceleration response spectra at PRPC, CHHC and RHSC, and Figure 12b the corresponding vertical-to-horizontal ratios. As has been commonly observed in numerous other studies, it can be seen that the vertical-to-horizontal (V-to-H) spectral ratio is largest at high frequencies with values that can be significantly greater than 1.0, and tends to reduce rapidly for vibration periods greater than $T = 0.1$s, and as a function of source to site distance (i.e. from Table 1, $R_{rup} = 2.5$ km, 3.8 km, and 6.5 km for PRPC, CHHC, and RHSC, respectively). Figure 12c-Figure 12f illustrate the V-to-H spectral ratios for four different vibration periods, $T = 0.0$, 0.1, 0.2, and 0.3s as a function of source-to-site distance for both the 22 February 2011 Christchurch and 4

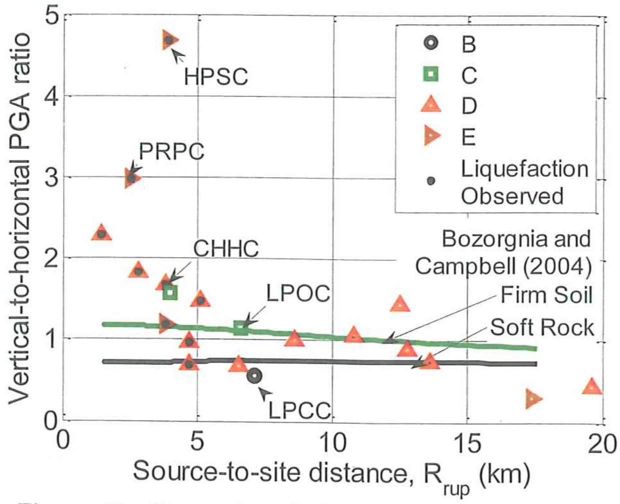

Figure 11: ***Observed vertical-to-horizontal peak ground acceleration ratios as a function of source-to-site distance in comparison with the empirical equation of Bozorgnia and Campbell [21]. Data are differentiated by site class as well as evidence of liquefaction.***

September 2010 Darfield earthquakes. Also shown for comparison is the empirical model of Bozorgnia and Campbell [21], and the prescribed ratio of 0.7 for the development of vertical design spectra in NZS1170.5 [9]. Firstly, it can be clearly seen that V-to-H ratios above 1.0 are frequently observed for distances up to $R_{rup} = 40$ km in both these events (as well as other historical earthquakes worldwide [21]), and hence the code prescription of 0.7 is, without question, significantly un-conservative. Secondly, it can be seen that while there is significant scatter in the observed ratios, the Bozorgnia and Campbell empirical model is able to capture the overall trends in the observations, except for $R_{rup} < 10$ km for which it underestimates the observed ratios. Comparison of the observations from the Darfield and Christchurch earthquakes also illustrates that the ratios, on average, are principally a function of source-to-site distance and there is no evidence for a systematic differences between the two events due to their different magnitude and style of faulting. This lack of average dependence the seismic source features is consistent with that of Bozorgnia and Campbell [21]. Comparison of the ratios observed at the same station in the two different events (annotated in the figures for PRPC and HPSC) illustrates that there is some systematic site effect, for example, HPSC is always above the average prediction, but

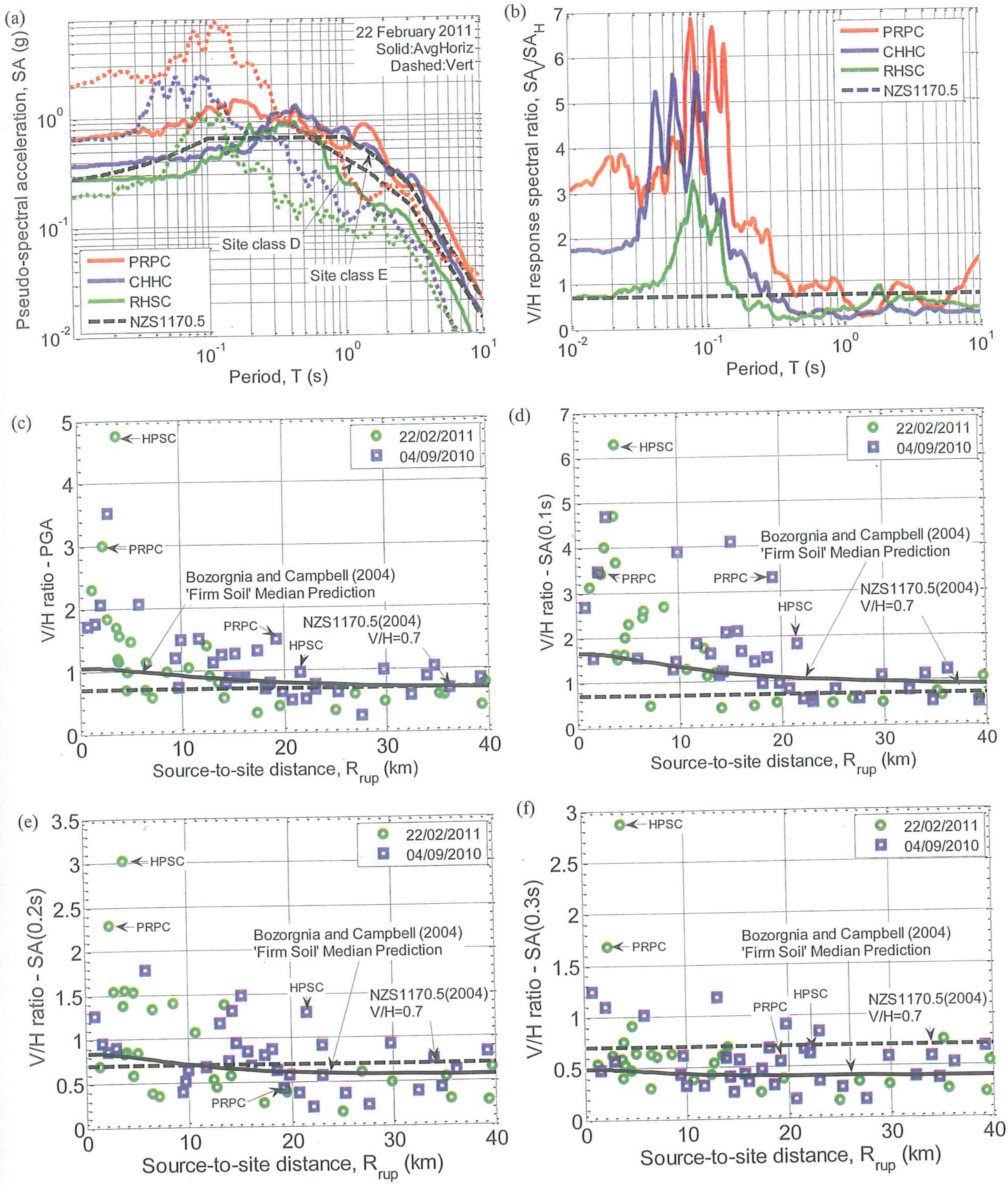

Figure 12: Vertical ground motion response spectral amplitudes observed: (a)-(b) Example geometric mean horizontal and vertical response spectra and their vertical-to-horizontal ratio; (c)-(e) vertical-to-horizontal response spectral ratios for T = 0.0-0.3 s as a function of distance observed in the 4 September 2010 Darfield and 22 February 2011 Christchurch earthquakes and comparison with the empirical prediction of Bozorgnia and Campbell [21].

this is not always the case for PRPC with the ratio for $T = 0.2$s well above the prediction in the Christchurch earthquake, but below the prediction in the Darfield earthquake. Given that vertical ground motion is only significant at very high frequencies, then it is expected to be strongly correlated with near-surface P-wave velocity structure, and some of the fluctuations observed in Figure 12 are likely the result of variability in the amplitude of the horizontal ground motion on the V-to-H ratio (due to nonlinearities for example).

The above discussions serve to illustrate that the large number of observed strong vertical ground motions in the 22 February 2011 Christchurch earthquake is simply a result of a larger number of recordings at very small source-to-site distances relative to the Darfield earthquake (e.g. 15 records within 10 km in the Christchurch earthquake as compared with 8 in the Darfield earthquake), rather than any specific source effect during rupture in the Christchurch earthquake. Finally, as horizontal ground motion amplitudes within Christchurch city in the Christchurch earthquake were larger than those from the Darfield earthquake (elaborated upon subsequently), then nonlinear shear deformation of soils which results in a reduction of tangent shear modulus, and therefore the ability to propagate high frequency ground motion, was more significant in the 22 February event. Nonlinear shear deformation on the other hand does not have as significant an effect on the compressibility of soil, which is related to P-wave velocity, and hence vertical ground motion amplification. The significant effect of nonlinear site response on horizontal ground motion, yet minor effect on vertical ground motion, was clearly illustrated in Figure 9.

COMPARISON OF OBSERVATIONS WITH EMPIRICAL GROUND MOTION PREDICTIONS FOR HORIZONTAL COMPONENTS

To provide a more complete analysis of the ground motions discussed in the previous sections with respect to physical phenomena this section compares the observed ground motions with empirical ground motion predictions. A rigorous assessment of the efficacy of various empirical ground motion prediction equations (GMPEs) is not attempted, and the aim is merely to identify ground motions which have intensity measures deviating from such GMPEs, and subsequently an attempt to explain such deviations based on previous physical phenomena-oriented discussions.

Pseudo-acceleration response spectra

Figure 13 illustrates the pseudo-acceleration response spectra (SA) amplitudes of ground motions recorded within 50km of the casual faults in the Darfield earthquake at periods of $T = 0.0$, 0.2, 1.0 and 3.0 s. The observations are compared with the empirical SA GMPE developed by Bradley [10], which is a NZ-specific modification of the Chiou and Youngs [18] and Chiou *et al.* [22] models. For each of the different vibration periods considered, the median, 16th and 84th percentiles of the prediction for site class D conditions is shown. Mixed-effects regression [23, 24] was utilized in order to determine the inter- and intra-event results for each vibration period. The value of the normalized inter-event residual (η) is also shown in the inset of each figure.

The results of Figure 13 illustrate that the Bradley [10] GMPE is able to capture the source-to-site distance dependence of the observations with good accuracy. The inter-event term, which can be viewed as an overall bias of the amplitudes predicted relative to those observed, indicates that the model has very small bias for vibration periods of T= 0.0 and 0.2 s (i.e. η = 0.034 and -0.037, respectively), but that there is a an under-prediction of SA(1.0) amplitudes for a handful of ground motions at source-to-site distances less than 10 km, and also a notable under-prediction of SA(3.0) amplitudes for all distances (i.e. η= 1.283). The good prediction of high frequency ground motion (i.e. PGA and SA(0.2)) indicates that the source rupture didn't have a significantly different stress drop than what would be expected for such events. Hence, based on the previously discussed observations it can be logically concluded that the under-prediction at medium-to-long vibration periods is likely primarily a result of the fact that the model does not explicitly account for the large long-period ground motion resulting from basin-generated surface waves (as previously noted the basin depth parameter, $Z_{1.0}$, is presently set based on the near surface shear wave velocity, *Vs30*, due to a lack of data on basin depths for various locations in New Zealand), or near-source forward directivity. As was previously noted with reference to Figure 6 and Figure 7, the explicit consideration of these effects can help to improve the prediction of the model at long periods, which is an active area of current research.

Another possible reason for the under-prediction of ground motion at long periods is the additional amplification of long period motion resulting from highly nonlinear soil behaviour. While the empirical model attempts to account for soil nonlinearity, clearly this is achieved in a highly simplified manner, and there is a limited number of strong motions previously recorded on soft soil deposits. While it is often noted that highly nonlinear behaviour also results in an increase in hysteretic damping it should be borne in mind that because of the short duration of shaking (as elaborated below), there was generally not a large amount of time for hysteretic damping to have a significant effect on the peak response amplitude.

Finally, Figure 13 also annotates various strong motion stations which lie outside the 16th and 84th percentiles of the empirical prediction, and which have been mentioned in previous sections. It can be seen, for example, that the short period spectral amplitudes observed at Heathcote Valley (HVSC) are significantly above those predicted (for site class C, even though only the site class D prediction is shown) as a result of basin edge effects [13]. For SA(1.0) and SA(3.0), in particular it can be seen that all of the notable under-predictions occur for ground motions within 10 km, and for which as previously noted, significant basin effects were evident.

Significant duration

The duration of strong motion is also important if strong motion amplitude is sufficient to cause nonlinear response of soil deposits and/or structures. Figure 14 illustrates the 5-75% and 5-95% significant durations (D_{s575} and D_{s595}, respectively) of ground motion observed at stations within 50 km of the causative fault. It is worth noting that anecdotally the 5-75% and 5-95% definitions of significant durations can be considered to approximately represent the durations the majority of energy associated with body-wave arrivals and body- plus surface-wave arrivals, respectively [25].

The empirical prediction of Bommer *et al.* [26] was utilized in the comparisons with the observed durations. It can be seen in Figure 14 that for both measures of duration, the observations are on average in good agreement with the observations, with inter-event residuals of η = -0.064 and -0.179 for D_{s575} and D_{s595}, respectively. However, for D_{s575} in particular, it can be seen that for ground motions within approximately 10-15 km, the ground motion duration at site class D sites (which the prediction is shown for), tend to be larger than the median of the prediction, although less than the 84th percentile, while in contrast the durations tend to be below average beyond this distance.

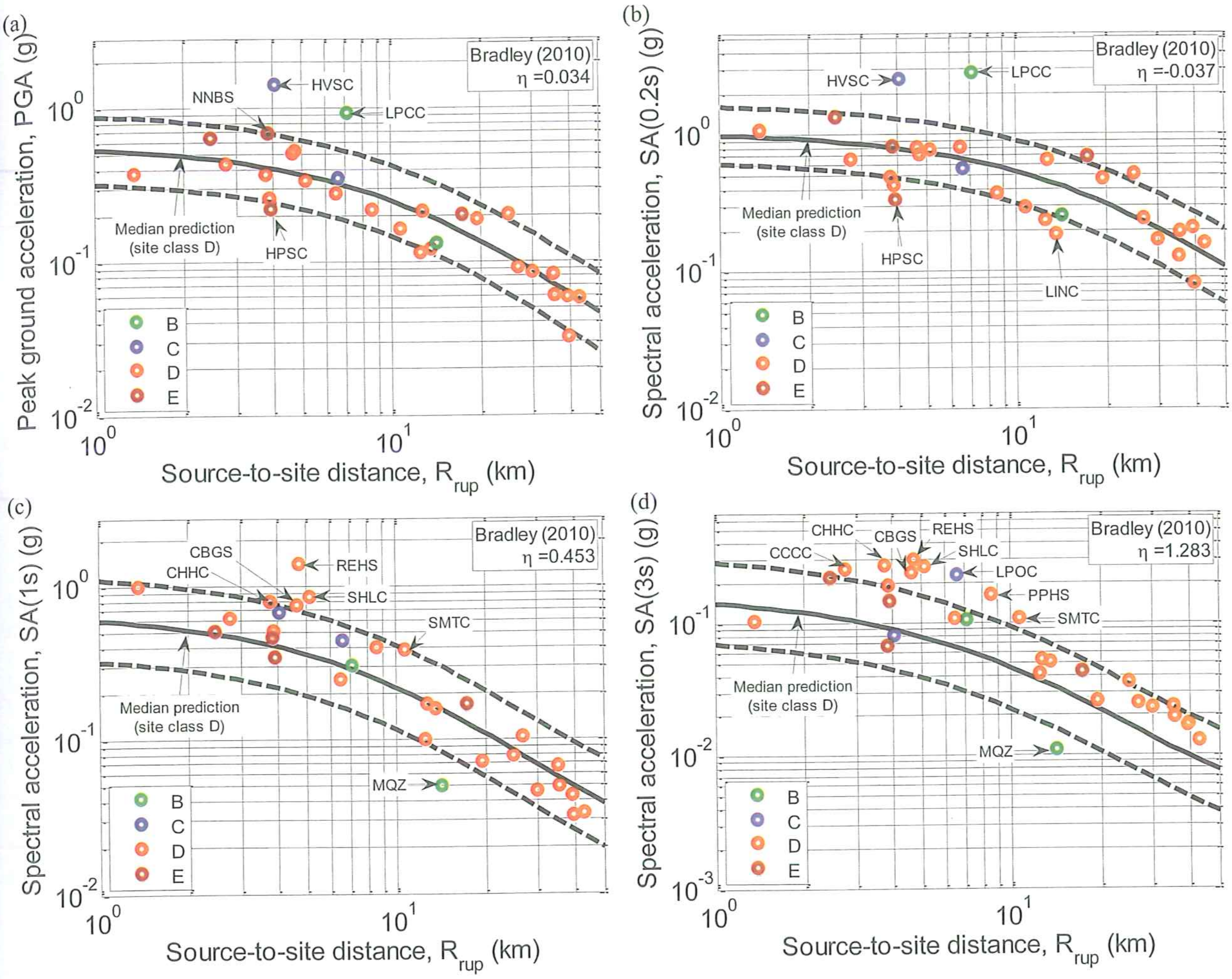

Figure 13: ***Comparison of pseudo-acceleration response spectral amplitudes observed with empirical prediction equations: (a) PGA; (b) SA(0.2s); (c) SA(1s); and (d) SA(3s).***

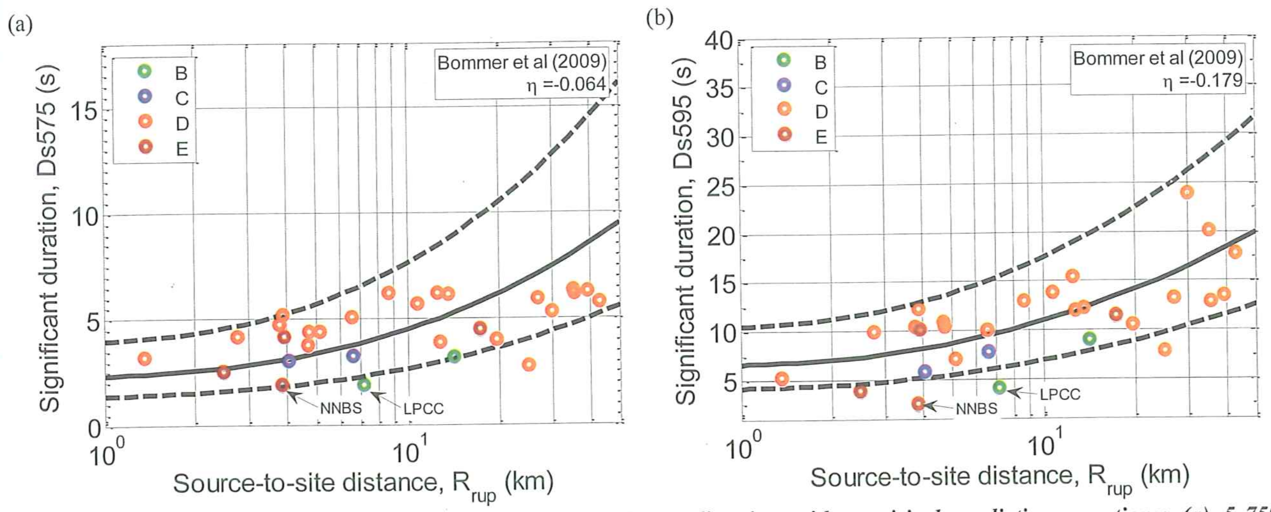

Figure 14: ***Comparison of observed ground motion significant direction with empirical prediction equations: (a) 5-75% significant duration; and (b) 5-95% significant duration.***

It is speculated that this maybe the result of the rupture having a shorter than typical source duration (which would mean that motions at all distances, on average, would be below the D_{s575} prediction), but that within the near-source region (in this case R_{rup}< 15 km) significant nonlinear behaviour leads to an increase in long period nature of the surface motion and consequently strong motion duration. For the 5-95% duration it can be seen that there is no clear bias at the larger source-to-site distances, likely a consequence of the basin-generated surface waves (as discussed with reference to Figure 8).

GROUND MOTION INTENSITY IN THE CENTRAL BUSINESS DISTRICT (CBD)

The Christchurch earthquake caused significant damage to commercial structures in the CBD, with a large portion still (at

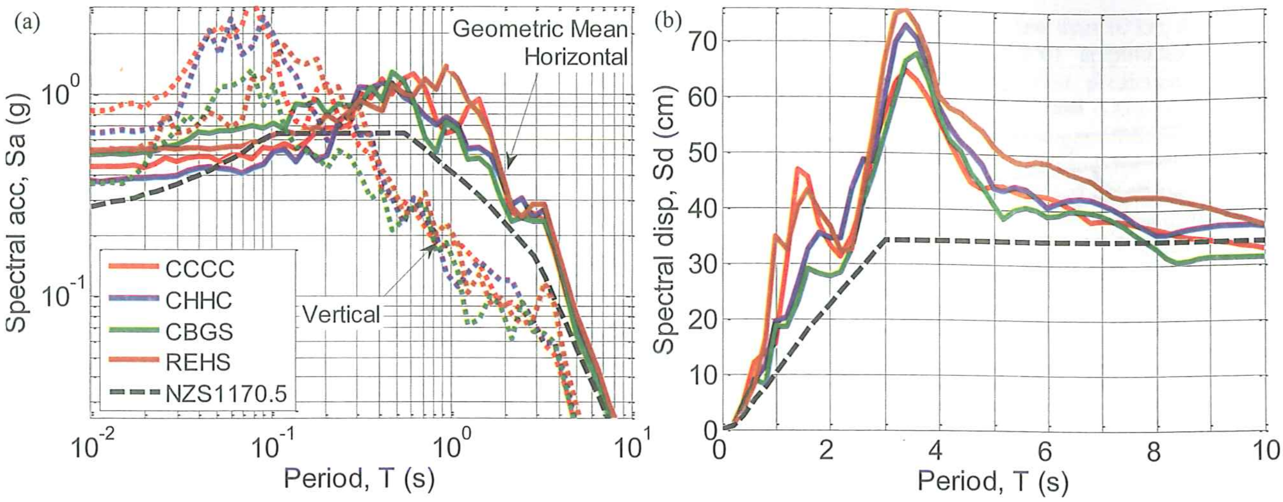

Figure 15: Comparison of response spectra from four strong motion stations located in the Christchurch central business district: (a) horizontal and vertical pseudo-acceleration response spectra; and (b) horizontal displacement response spectra.

the time of writing) prohibited while an estimated 1,000 structures (of various typologies, construction materials and age) are being demolished. The complete collapse of the Pine Gould Corporation (PGC) and Canterbury Television (CTV) buildings also lead to the majority of the 182 casualties [1].

Figure 15a and Figure 15b illustrate the pseudo-acceleration and displacement response spectra of four strong motion stations (CCCC, CHHC, CBGS, REHS) located in the CBD region. Despite their geographic separation distances (relative to their respective source-to-site distances) it can be seen that the characteristics of the ground motion observed at these locations is relatively similar. This is particularly the case for long-period ground motion amplitudes, which have longer wavelengths and therefore are expected to be more coherent. On the other hand, at short vibration periods there is more of a discrepancy in seismic intensity due to a shorter wavelength and therefore lower wave coherency, and probably more importantly due to the nonlinear response of significantly different surficial soil layers [27]. Figure 15a, in particular, illustrates that the strong long period ground motion previously discussed with respect to CHHC (i.e. Figure 7b) was observed at all four CBD stations and both Figure 15a and Figure 15b illustrate that the seismic demands were above the 475 year return period design ground motion for Christchurch site class D as specified by the New Zealand loading standard, NZS1170.5 [9]. Furthermore, Figure 15b illustrates that for structures whose secant period at peak displacement is in the region of 1.5 or 3.5 seconds, the displacement demands imposed by the ground motion were in the order of two times the seismic design level.

COMPARISON WITH GROUND MOTIONS OBSERVED IN THE 2010 DARFIELD EARTHQUAKE AND DESIGN SPECTRA

The M_w6.3 Christchurch earthquake was the second event in approximately six months to cause significant ground motion shaking in Christchurch, having been preceded by the 4 September 2010 Darfield earthquake [5]. In this section comparison is made between the ground motion intensities in these two events at various locations, and also with respect to seismic design spectra.

Figure 16 illustrates the geometric mean horizontal and vertical pseudo-acceleration response spectra of ground motions at various strong motion stations in Christchurch resulting from both the Christchurch and Darfield earthquakes, in addition to those that have been already presented for PRPC and HVSC in Figure 4. It can be immediately seen that for the majority of vibration periods of engineering interest the spectral amplitudes are larger for the Christchurch earthquake. The primary exception of the above statement is the spectral amplitudes at long vibration periods (i.e. $T > 2$s) due to both the longer duration of shaking and forward directivity effects in the Darfield earthquake [13]. Strong long-period spectral ordinates associated with these phenomena in the Darfield earthquake can be clearly seen at CCCC, RHSC and CACS stations. Figure 16a illustrates that at Christchurch Cathedral College (CCCC), which is located in the Christchurch CBD, spectral amplitudes in the Christchurch earthquake were approximately twice that of the Darfield earthquake for vibration periods less than $T = 1.5$s. It can also be seen that at CCCC station, spectral amplitudes resulting from the Darfield earthquake were notably below the design spectra for $T < 2$s. Figure 16c-Figure 16d also illustrate that spectral amplitudes from the Darfield earthquake were below the design spectra at short periods throughout the majority of Christchurch, with exceptions being Heathcote Valley (HVSC), Lyttelton Port (LPCC), and several western suburbs (i.e. TPLC, ROLC, LINC) not shown here [13].

Another notable feature illustrated in Figure 16 is the similarity of the response spectral shapes at a given site from these two events. In such an examination it is important to note the markedly different source locations of these two events, with the Christchurch earthquake occurring to the south-east, and the Darfield earthquake approximately 30km west of, central Christchurch. Hence, the source and path effects of the ground motion at a single site are expected to be significantly different in both events. For example, Figure 16b and Figure 16c illustrate the similarity of response spectral shapes, for vibration periods less than $T = 2$ s, of both horizontal and vertical ground motion components at Riccarton (RHSC) and Canterbury Aero Club (CACS). At vibration periods larger than $T = 2$ s, the aforementioned source effects from the Darfield earthquake become significant (as well as 3D basin structure) and the response spectral shapes at a given site from these two events deviate. These observations clearly point to the importance of local site effects on surface ground motions, particularly at high to moderate vibration frequencies, and hence the benefits that can be obtained via site-specific response analysis as opposed to simple soil classification (recall that most of the sites in the Christchurch basin are assigned as site class D [9]). It should also be noted that the RHSC and CCCC sites discussed above, while experiencing significant ground motions, are founded on soils which did not exhibit liquefaction (which obviously causes a notable change in the stiffness and strength of the affected soils and hence modifies the near-surface site response).

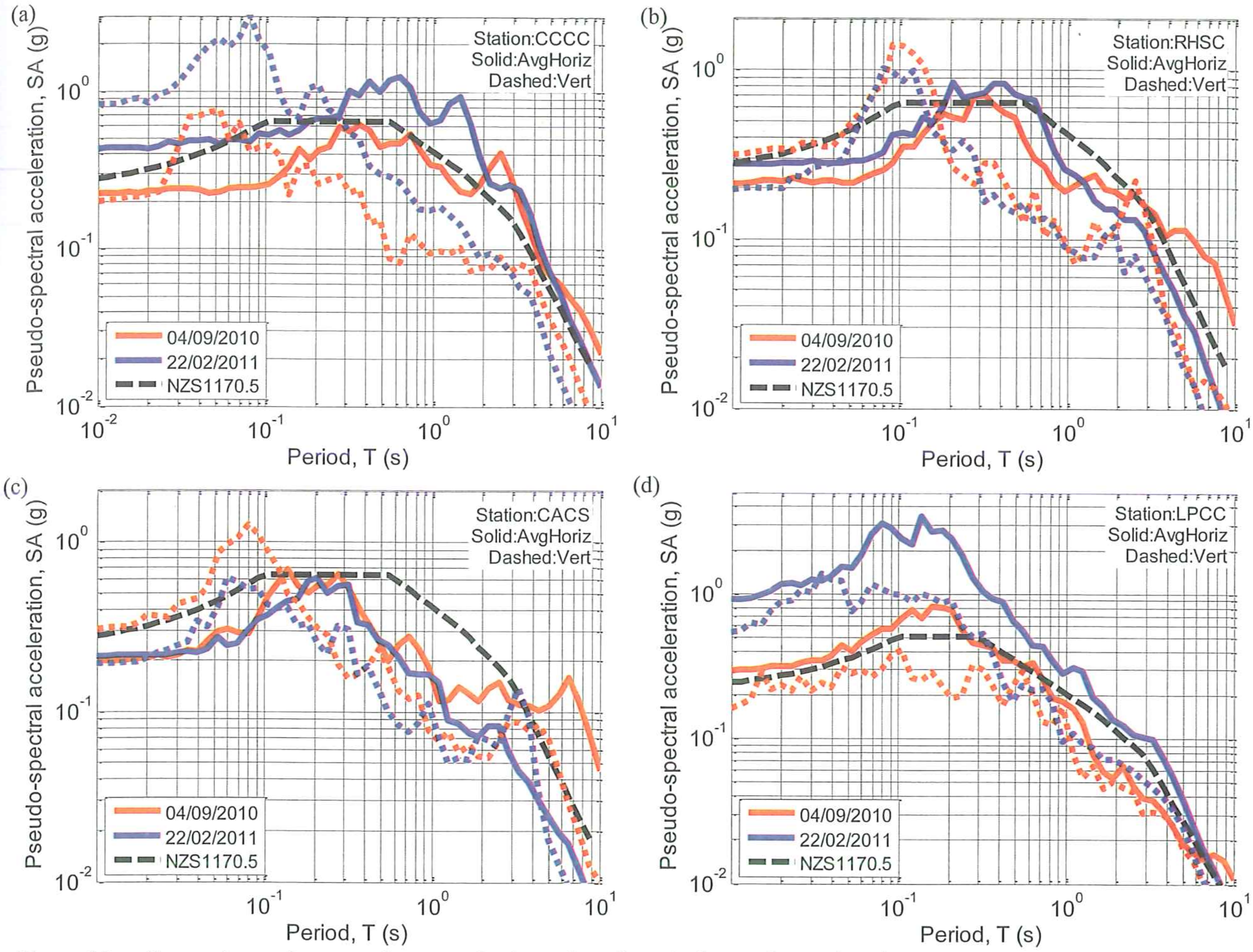

Figure 16: ***Comparison of geometric mean horizontal and vertical pseudo-acceleration response spectra observed in the 22/02/2010 Christchurch and 04/09/2010 Darfield earthquakes at various strong motion stations.***

CONCLUSIONS

The 22 February 2011 M_w6.3 Christchurch earthquake imposed severe ground motion intensities, which were in excess of the current seismic design spectra and those experienced in the 4 September 2010 Darfield earthquake, over the majority of the Christchurch region.

The dense set of near-source ground motions enable a detailed examination of salient features of the earthquake source, path and local site characteristics. It was seen that forward directivity due to the rupture propagation was evident at Pages Road (PRPC), however, such effects were not predominant over the region due to the inferred misalignment between the rupture font and slip vector. The large velocity contrast between the Christchurch sedimentary basin and underlying rock likely lead to a waveguide effect in which seismic waves were ‘trapped’ and propagated across the basin, principally resulting in an increase in long period response spectral amplitudes and ground motion durations. The severity of the ground motion intensity in the near-source region resulted in significant nonlinear soil behaviour and severe and widespread liquefaction which were evident in recorded acceleration time histories. The ratio between vertical and horizontal ground motion amplitude is strongly dependent on source-to-site distance, and weakly dependent on source magnitude or faulting style. It was seen that the vertical-to-horizontal response spectral ratios were similar for the Darfield and Christchurch earthquakes and hence the large vertical ground motions observed were simply a result of the significant number of near-source recordings rather than any event-specific features.

On average, the observed ground motion amplitudes were seen to be consistent with empirical predictions for high frequencies, and the under-prediction for long periods is a likely result of the pronounced basin-generated surface waves, forward directivity and significant nonlinear soil behaviour observed. Discerning the relative contribution of each of these effects at various locations is the subject of ongoing work using more sophisticated methods of analysis.

The Christchurch earthquake produced ground motions in the majority of the eastern and central Christchurch region which had pseudo-acceleration response spectral amplitudes that were generally above the 475-year routine seismic design spectra, and also larger than those of the 4 September 2010 Darfield earthquake. At a single strong motion station, the similarity of response-spectral shapes of the ground motion observed from the Christchurch and Darfield earthquakes, for which source and path effects were largely different, also illustrated the significance of site-specific response for short and moderate vibration frequencies and hence that clearly more detailed subsurface investigations and modelling are needed to adequately infer the performance of soil and overlying structures in future earthquakes than simply using alphabet-based site classifications.

ACKNOWLEDGEMENTS

Dr. Weng Yuen Kam provided constructive comments on a draft of the manuscript, and discussions with Dr. John Beavan and Dr. John Berrill are greatly appreciated.

REFERENCES

[1] New Zealand Police. (2011) "Christchurch earthquake: List of deceased".

[2] DeMets, C., Gordon, R.G., Argus, D.F. and Stein, S. (1994) "Effect of recent revisions to the geomagnetic time scale on estimates of current plate motion". *Geophysical Research Letters*; **21** 2191-2194.

[3] Sutherland, R., Berryman, K. and Norris, R. (2006) "Quaternary slip rate and geomorphology of the Alpine fault: Implications for kinematics and seismic hazard in southwest New Zealand". *Geological Society of America Bulletin*; **118** 464-474.

[4] Stirling, M.W., Gerstenberger, M., Litchfield, N., McVerry, G.H., Smith, W.D., Pettinga, J.R. and Barnes, P. (2007) "Updated probabilistic seismic hazard assessment for the Canterbury region". 58.

[5] NZSEE. (2010) "Special Issue: Preliminary observations of the 2010 Darfield (Canterbury) Earthquakes". *Bulletin of the New Zealand Society for Earthquake Engineering*; **43**(4): 215-439.

[6] Beavan, J., Fielding, E.J., Motagh, M., Samsonov, S. and Donnelly, B.S. (2011) "Fault location and slip distribution of 22 February 2011 Mw 6.3 Christchurch, New Zealand, earthquake for geodetic data". *Seismological Research Letters, Focused Issue on the 2011 Christchurch New Zealand Earthquake*.

[7] Somerville, P.G., Ikikura, K., Graves, R.W., Sawada, S., Wald, D., Abrahamson, N.A., Iwasaki, Y., Kagawa, T., Smith, N. and Kowada, A. (1999) "Characterizing crustal earthquake slip models for the prediction of strong ground motion". *Seismological Research Letters*; **70**(1): 59-80.

[8] Brown, L.J. and Weeber, J.H. (1992) "Geology of the Christchurch urban area", Geological and Nuclear Sciences 110.

[9] NZS 1170.5. (2004) "Structural design actions, Part 5: Earthquake actions - New Zealand". *Standards New Zealand:* Wellington, New Zealand, 82.

[10] Bradley, B.A. (2010) "NZ-specific pseudo-spectral acceleration ground motion prediction equations based on foreign models" Report No.2010-03, Department of Civil and Natural Resources Engineering, University of Canterbury: Christchurch, New Zealand. 324.

[11] Aoi, S., Kunugi, T. and Fujiwara, H. (2008) "Trampoline Effect in Extreme Ground Motion". *Science*; **322**(5902): 727-730.

[12] Yamada, M., Mori, J. and Heaton, T. (2009) "The Slapdown Phase in High-acceleration Records of Large Earthquakes". *Seismological Research Letters*; **80**(4): 559-564.

[13] Bradley, B.A. (2012) "A critical analysis of strong ground motions observed in the 4 September 2010 Darfield earthquake". *Soil Dynamics and Earthquake Engineering*; (submitted).

[14] Aagaard, B.T., Hall, J.F. and Heaton, T.H. (2004) "Effects of Fault Dip and Slip Rake Angles on Near-Source Ground Motions: Why Rupture Directivity Was Minimal in the 1999 Chi-Chi, Taiwan, Earthquake". *Bulletin of the Seismological Society of America*; **94**(1): 155-170.

[15] Shahi, S.K. and Baker, J.W. (2011) "An Empirically Calibrated Framework for Including the Effects of Near-Fault Directivity in Probabilistic Seismic Hazard Analysis". *Bulletin of the Seismological Society of America*; **101**(2): 742-755.

[16] Hicks, S.R. (1989) "Structure of the Canterbury Plains, New Zealand from gravity modelling", Geophysics Division, Department of Science and Industrial Research: Wellington.

[17] Choi, Y., Stewart, J.P. and Graves, R.W. (2005) "Empirical Model for Basin Effects Accounts for Basin Depth and Source Location". *Bulletin of the Seismological Society of America*; **95**(4): 1412-1427.

[18] Chiou, B.S.J. and Youngs, R.R. (2008) "An NGA Model for the average horizontal component of peak ground motion and response spectra". *Earthquake Spectra*; **24**(1): 173-215.

[19] Cubrinovski, M., Green, R.A., Allen, J., Ashford, S.A., Bowman, E., Bradley, B.A., Cox, B., Hutchinson, T.C., Kavazanjian, E., Orense, R.P., Pender, M., Quigley, M. and Wotherspoon, L. (2010) "Geotechnical reconnaissance of the 2010 Darfield (Canterbury) earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*; **43**(4): 243-320.

[20] Silva, W.J. (1997) "Characteristics of vertical strong ground motions for applications to engineering design, FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities, Burlingame, CA, Proceedings" Earthquake Spectra, National Center for Earthquake Engineering Research: Buffalo, New York.

[21] Bozorgnia, Y. and Campbell, K.W. (2004) "The vertical-to-horizontal response spectral ratio and tentative procedures for developing simplified V/H and vertical design spectra". *Journal of Earthquake Engineering*; **8**(2): 175-207.

[22] Chiou, B., Youngs, R., Abrahamson, N. and Addo, K. (2010) "Ground-Motion Attenuation Model for Small-To-Moderate Shallow Crustal Earthquakes in California and Its Implications on Regionalization of Ground-Motion Prediction Models". *Earthquake Spectra*; **26**(4): 907-926.

[23] Pinheiro, J., Bates, D.M., DebRoy, S., Sarkar, D. and the R Core team. (2008) "nlme: linear and nonlinear mixed effects models".

[24] Abrahamson, N.A. and Youngs, R.R. (1992) "A stable algorithm for regression analyses using the random effects model". *Bulletin of the Seismological Society of America*; **82**(1): 505-510.

[25] Bradley, B.A. (2011) "Correlation of significant duration with amplitude and cumulative intensity measures and its use in ground motion selection". *Journal of Earthquake Engineering*; (in press).

[26] Bommer, J.J., Stafford, P.J. and Alarcon, J.E. (2009) "Empirical Equations for the Prediction of the Significant, Bracketed, and Uniform Duration of Earthquake Ground Motion". *Bulletin of the Seismological Society of America*; **99**(6): 3217-3233.

[27] Cubrinovski, M., Bray, J.D., Taylor, M.L., Giorgini, S., Bradley, B.A., Wotherspoon, L. and Zupan, J. (2011) "Soil liquefaction effects in the Central Business District during the February 2011 Christchurch earthquake". *Seismological Research Letters, Focused Issue on the 2011 Christchurch New Zealand Earthquake*.