Report to the Canterbury Earthquakes Royal Commission

on the

Inelastic Response Spectra for the Christchurch Earthquake Records

by

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1 Introduction

The Institute of Geological and Nuclear Science (GNS) have made available the earthquake acceleration records and published elastic acceleration response spectra for all of the major Canterbury earthquakes. These records have shown the levels of acceleration to be much higher than the levels seen in the current and past New Zealand seismic design standards or codes.

Seismic analysis of the data recorded during the 22 February 2011 Christchurch earthquake at the Rose School, University of Pavia, Italy, and at the University of Canterbury, New Zealand, identified a potential issue in the assumptions made in virtually all of the current seismic design standards, that is, that the deflections of the elastic and inelastic frames should have been similar.

Due to the findings of these analyses, this report has been prepared to review the earthquake acceleration records from the Christchurch Central Business District (Christchurch CBD) in all of the Canterbury earthquakes to provide input to the review of the assumptions made in the New Zealand seismic design standards.

1.1 Revised Report.

In mid-September 2011 a researcher using the program Inspect reported an apparent discrepancy between the computed Spectral Displacement and the displacement obtained from the computed yield displacement multiplied by the ductility factor. This yield displacement can be obtained by dividing the computed yield force by the stiffness. The stiffness is a function of the natural period of free-vibration. On investigation it was realized that the Spectral Displacement and Spectral Accelerations were being enveloped over all the iterations used to match the target ductility instead of just over the final iteration. This has now been corrected and all the spectra reported herein have been recomputed. In most cases this has slightly reduced the spectral values but all of the original observations are unchanged. Comparison with a set of results from Inspect and the non-linear time-history analysis program Ruaumoko2D are shown in Appendix F. These results show very close agreement.

2 Displacement Ductility

2.1 General

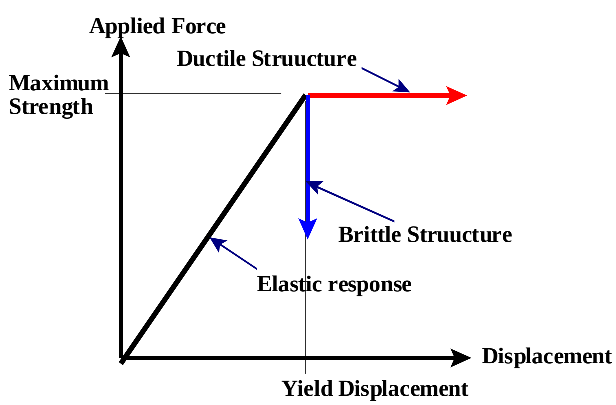
A linear elastic building or building component is one where when the load is removed the member or building returns to its original un-deformed position. When a structure, or a member in a structure, reaches the point where the maximum strength is reached then the failure that follows may be classed as brittle failure or a ductile failure.

2.2 Brittle behaviour

A brittle failure is that seen in the fracture of materials like cast iron, glass or wood. For buildings or parts of buildings, examples of brittle failures occurred in un-reinforced masonry buildings as demonstrated in the recent Christchurch earthquakes. In a brittle system, once the fracture load is reached the result is, in a component, a complete break into two pieces and the complete loss of its load carrying capacity, and in a building, shedding of parapets or even catastrophic building collapse.

2.3 Ductile behaviour

A ductile member, or structure, continues to deform with permanent deformation and resulting damage but without the loss of the ability to carry the load that was present when the yield load was reached. This is shown in Figure 1.



$$\text{Ductility } \mu = \frac{\text{Inelastic Displacement}}{\text{Yield Displacement}}$$

Figure 1. Ductile and Brittle Structures

An example of a ductile system would be a piece of mild steel wire which when bent to a point less than the yield load will return to the original shape. After it reaches the maximum, or yield load, it can be bent to considerably larger displacements though will not now naturally return to its un-deformed position. It is permanently bent. In a reinforced concrete building this post-yield behaviour will be shown as large cracks and the structure will also show permanent displacements, i.e. the columns will no longer be vertical.

The displacement ductility is defined as the maximum displacement divided by the yield displacement. The yield displacement is the displacement when the material yields. For an elastic-perfectly plastic (elasto-plastic) material it is also the maximum load. Any displacement beyond this point involves permanent, or non-recoverable, displacement.

A fully ductile building or building component should be able to be deformed to 4 to 6 times the yield displacement (ductility 4 to 6) without loss of strength. The ductile response does involve damage to the building and this damage may be compounded by the number of times that the building displaces beyond the yield displacement.

There is a limit to, not only the maximum displacement, or ductility, but also to the accumulated damage that a building can sustain before loss of its load carrying ability will occur. The accumulated damage is a function of the number of times the building or building component exceeds the yield strength and the level of ductile displacements in each of these inelastic excursions.

For example, if a piece of wire is bent backwards and forwards beyond the yield point enough times, the wire will eventually break due to low cycle fatigue. For buildings designed to resist earthquake excitation, the duration of strong shaking can become a deciding factor as to whether the building will be repairable or not, as low cycle fatigue type effects may occur.

A building or building component that has a ductile response, but can at the best achieve a ductility of less than 2, is often referred to as a building or building component of limited ductility.

Modern seismic design attempts to prevent such brittle system behaviour, and the current seismic design in New Zealand aims to achieve ductile buildings. The behaviour of the building beyond the yield point is referred to as the inelastic response of the system.

3 Response Spectra

The acceleration and displacement response spectra are the maximum absolute acceleration and displacement of a single mass structure subjected to the earthquake ground acceleration history. Figure 2 shows an example of a simple structural model with a mass M, a stiffness $K=3EI/L^3$ and the displacement of the mass x due to the applied load P.

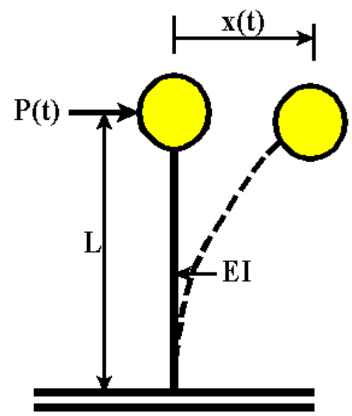


Figure 2. Single Mass Structure.

All buildings, have a natural frequency of free-vibration which is a function of the mass and stiffness of the structure. This can be thought of analogous to that of a tuning fork except that in buildings the frequency is much lower and is usually measured by the inverse of the frequency as the period of free-vibration T, in seconds. A multi-storey building has a very large number of natural frequencies.

In this report only the lowest, or fundamental, natural frequency has been considered. Designers for tall buildings do consider the higher natural frequencies and the mode shapes of free-vibration associated with these frequencies. The lowest natural frequency of free-vibration has the longest (largest) natural period of free-vibration.

Any vibration in a building is observed to die away with time. This decay of motion is referred to as a damped response [Clough, 1975] and all buildings have an inherent amount of damping. The majority of the world's seismic design standards assume that this amount of damping is 5% of critical viscous damping. This 5% critical viscous damping means that a vibratory motion, once started, will halve in amplitude in 2 cycles.

A building with a specified natural period of free vibration and a specified level of viscous damping is subjected to the selected ground acceleration time history record. An example of a selected ground acceleration time history record is shown in Figure 3.

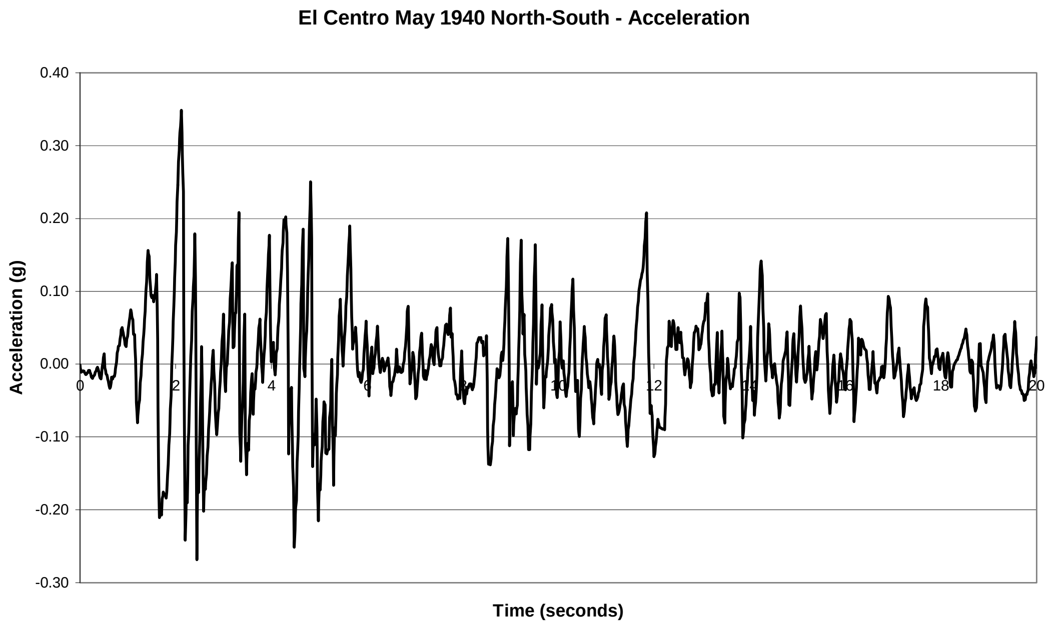


Figure 3a. Ground Acceleration History
El Centro May 1940 – North-South Component

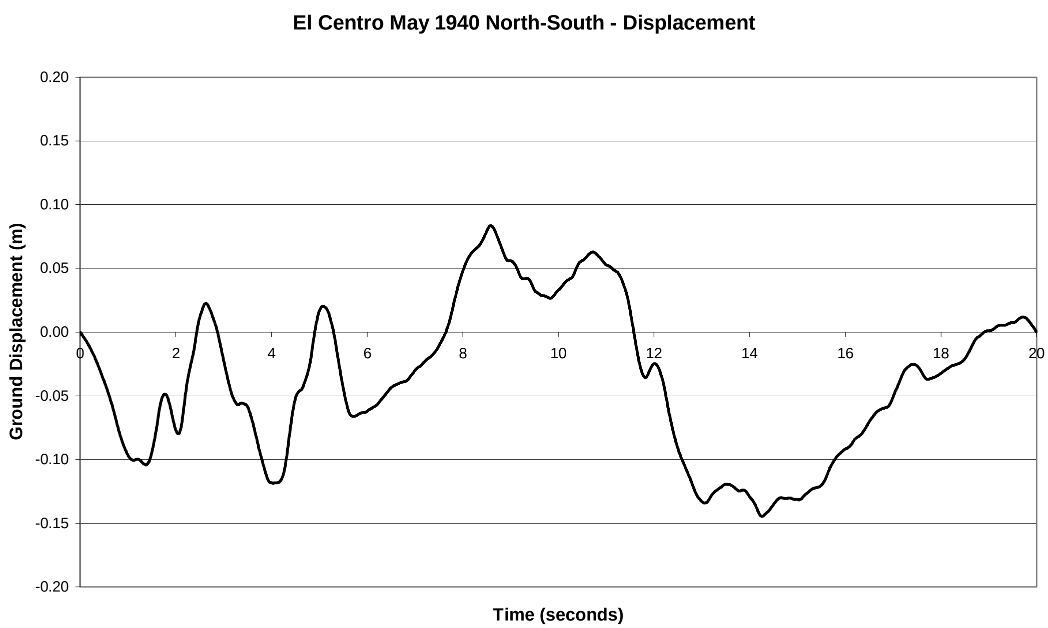


Figure 3b. Ground Displacement History
El Centro May 1940 – North-South Component

The equations of motion are integrated with time to compute the displacement and acceleration of the building due to this ground acceleration. Over the duration of the analysis, the maximum absolute value of the displacement of the mass relative to the foundation and the maximum absolute value of the acceleration of the mass are recorded.

These values provide the spectral displacement and spectral acceleration for that particular natural period of free-vibration [Clough, 1975]. The analysis is then repeated for a range of natural period of free-vibration, usually covering the range from about 0.1 seconds to about 5 seconds with small steps in the natural period. The whole process is then repeated for different levels of viscous damping. These results are then plotted to produce the response spectra graphs, examples of which are shown in Figures 4 and 5.

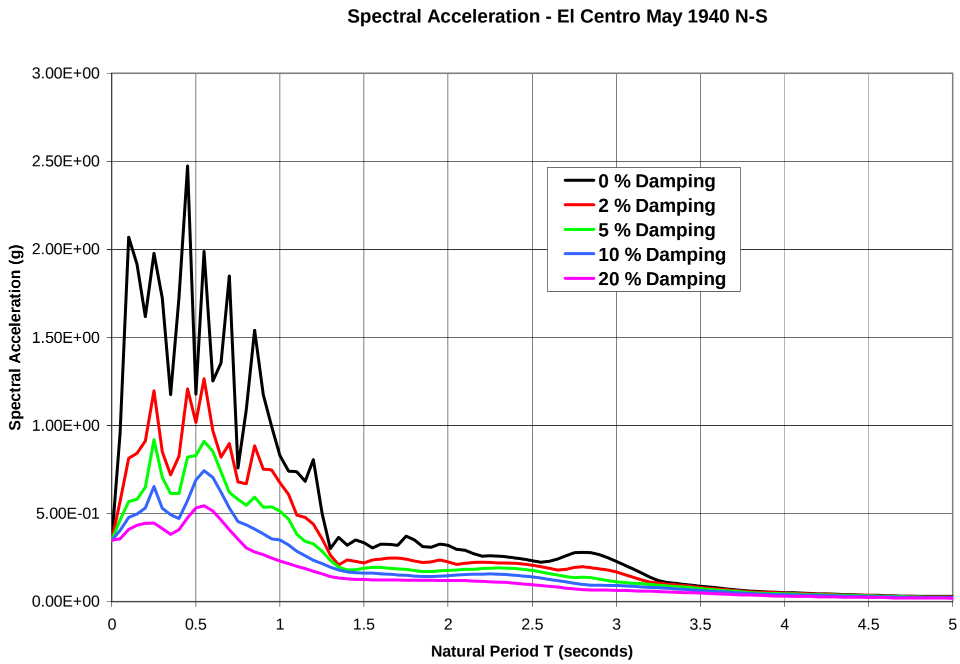


Figure 4. Acceleration Response Spectra
El Centro May 1940 – North-South Component

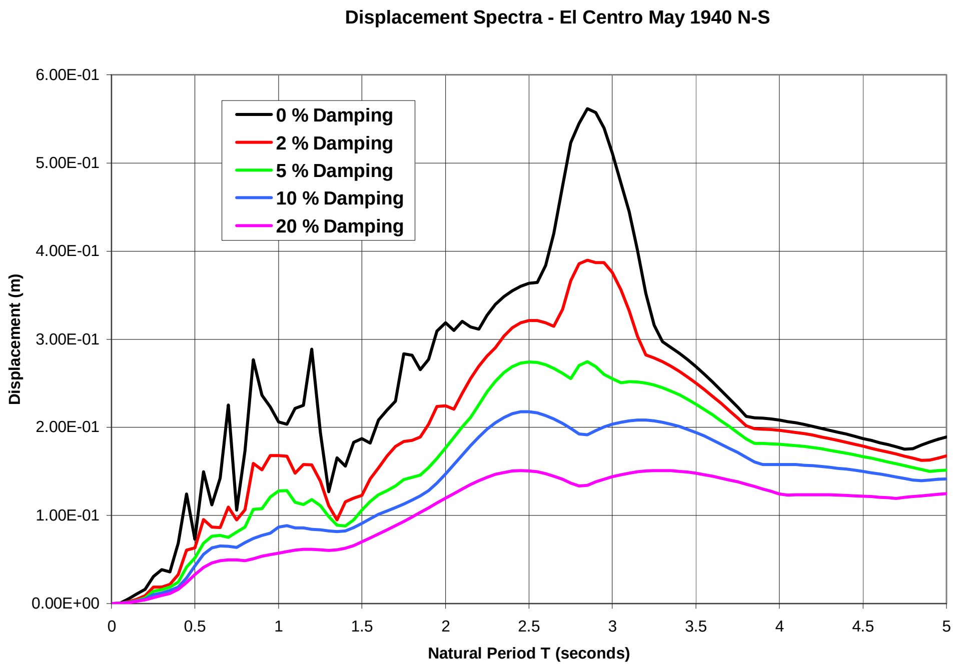


Figure 5. Displacement Response Spectra
El Centro May 1940 – North-South Component

The response spectrum shown in figures 4 and 5 are specific to the selected earthquake acceleration history records. It must be noted that this is not a spectral analysis of the acceleration record itself. The spectra represent the response of a building with a

specified natural period of free-vibration and a specified level of viscous damping to that acceleration history or record.
Each of these acceleration history records is unique. Records from different recording sites recording the same earthquake have noticeably different acceleration histories. A recording site will have different acceleration histories from different earthquakes. These variations may be seen by studying the ground acceleration records for the Canterbury earthquakes shown in Appendix A. There are four different sites in the Christchurch CBD and three different earthquake events and the variations are easy to see.

These computed response spectra are those of a linear elastic structure whose properties, including the stiffness and hence natural period of free-vibration, do not change during the period of the earthquake excitation.

If the acceleration response spectrum is known, then for that particular earthquake, the design engineer needs to know the natural period of free-vibration of the building and the percentage of critical viscous damping to obtain the maximum acceleration and displacement of the building. The design engineer does not need to perform a time-history integration for the response of the building. From Newton's Second Law, which states that the force is proportional to the mass of the structure (a function of the weight of the building), and the known acceleration the maximum inertia force induced in the building by the earthquake and hence the displacement caused by that inertia force can be determined. The maximum displacement can also be obtained directly from the displacement response spectrum if it is available.

The response spectra illustrate that buildings with a short natural period, i.e. stiff and generally low buildings, attract high accelerations and therefore high inertia forces. Buildings that have a longer natural period, i.e. flexible and generally taller buildings, attract larger displacements but lower inertia forces.

Greater amounts of viscous damping reduce both the acceleration and displacement responses of a building. It is possible to add damping devices to buildings to increase the level of damping. This has been done to a limited extent in parts of Japan to reduce the potential effects of large earthquakes. Damping devices are expensive and require ongoing maintenance. At the present time there are few guidelines available to designers to size viscous dampers and assess the damper locations in a building.

Another way of achieving the equivalent of this damping is to allow an inelastic response of the building as energy can also be dissipated by plastic work. In traditional design, this is done in allowing parts of the building to yield with a ductile response thus reducing the accelerations in the building. However, this is at the expense of structural damage to the building which will need to be repaired afterwards.

A more recent option is to use base-isolation. The plastic work takes place in devices, such as lead-rubber bearings, at the base of the structure. If necessary, these devices can be replaced after a major earthquake without any effect on the building integrity. However, base-isolation only is really useful for short natural period buildings [Andriono, 1991a, 1991b]. Base-isolation has been used in New Zealand and overseas to protect stiff brittle structures. Base-isolation has also been used for bridges in New Zealand since 1975 and some buildings [Boardman, 1983].

In the last few years research in New Zealand has also considered using energy dissipater devices within buildings such as at the base of rocking walls and also at beam-column joints. If the deformations in the devices from the seismic event were severe enough the devices would need to be replaced. The building integrity and use, however, would not be compromised.

4 The use of Acceleration and Displacement Response Spectra in Building Design.

Structural engineers designing buildings to resist earthquake excitation use response spectra to determine the design loads and structural displacements.

In the simplest approach, the first step is to determine the natural period of free-vibration of the building. This fundamental property of the structure has a significant influence on how markedly the building will be excited by the earthquake and hence the magnitude of the building's response to the earthquake excitation. Once the natural period T and the percentage of critical viscous damping (most seismic codes assume 5% of critical viscous damping) is known, then the spectral acceleration is obtained and the inertia force acting on the mass of the structure is determined. These forces are then applied to the building and the maximum displacements are computed.

These displacements could also be obtained directly from the spectral displacement, but in most current seismic standards, or codes, including the New Zealand Standard NZS1170.5:2004, only the spectral acceleration is supplied. In most seismic codes the spectral acceleration is supplied as a lateral seismic coefficient C which is the spectral acceleration expressed in units of the acceleration of gravity so that the inertia forces are obtained by multiplying the weight of the building by the seismic lateral coefficient C rather than the mass of the building multiplied by the spectral acceleration. The spectral displacement can be derived from the spectral acceleration as they are related by the mass and stiffness of the building. The displacement of a building is, in a general sense, the applied force (which is the spectral acceleration multiplied by the mass) divided by the stiffness of the building

$$\mathbf{Sd} = \frac{\mathbf{Sa * M}}{\mathbf{K}} = \mathbf{Sa} * \left(\frac{\mathbf{M}}{\mathbf{K}}\right) = \mathbf{Sa} * \left(\frac{\mathbf{T}}{\mathbf{2\pi}}\right)^2$$

where **Sd** is the spectral displacement, **Sa** is the spectral acceleration, **M** is the mass of the structure, **K** is the stiffness of the structure and **T** is the natural period of free-vibration. This equation is not exactly true as there is also a damping force but this only of the order of 10% of the inertia forces and the structure forces for 5% critical damping and it is also 90 degrees out of phase with the inertia forces and the structure forces. The equation is used in most response spectra programs to obtain the pseudo acceleration spectra from the displacement spectra. The spectral acceleration and the pseudo spectral acceleration are exactly the same for un-damped systems and differ by usually less than 1% for lightly damped systems.

As shown in Figure 6, the earthquake forces applied to the building are intended to provide similar displacements to those that would be caused by the earthquake ground accelerations and the induced inertia forces in the building.

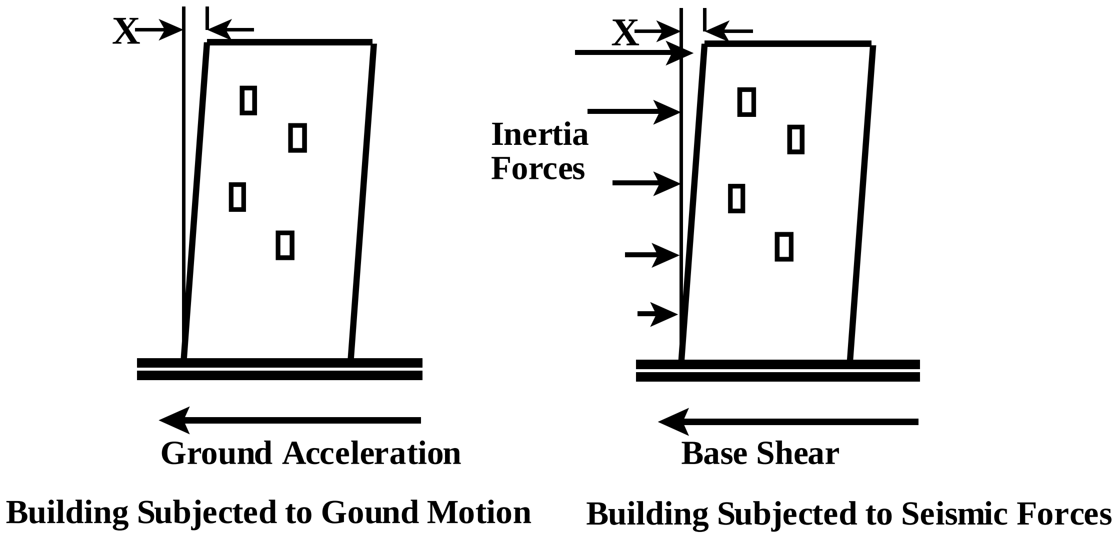


Figure 6. Building subjected to ground motion and the "equivalent" seismic actions the Base Shear is the sum of the Inertia Forces.

In recent years there has been a move from a force-based design to a displacement-based design [Priestley, 2007] where the spectral displacement is used as the starting point rather than the spectral acceleration. The design displacement spectra are derived from the acceleration spectra using the relationships which are described above.

The response spectra used for design are not the spectra associated with a particular earthquake as those particular ground motions will never be seen again. The response spectra for many earthquakes have been computed and scaled to account for the relationships between the likelihood of that level of excitation and the effects of attenuation due to distance from possible epicenters.

The type of excitation (both near field and far field) from the many different sources for a particular area are assessed and a suitable design spectra is produced. The response spectra from a particular earthquake have very irregular amplitudes and small differences in the natural period T could lead to major variations in the spectral values. The design acceleration spectra are usually a smoothed function which makes the spectral values much less sensitive to such uncertainties. Figure 7 illustrates the design spectra shape from the current New Zealand Standard NZS1170.5:2004.

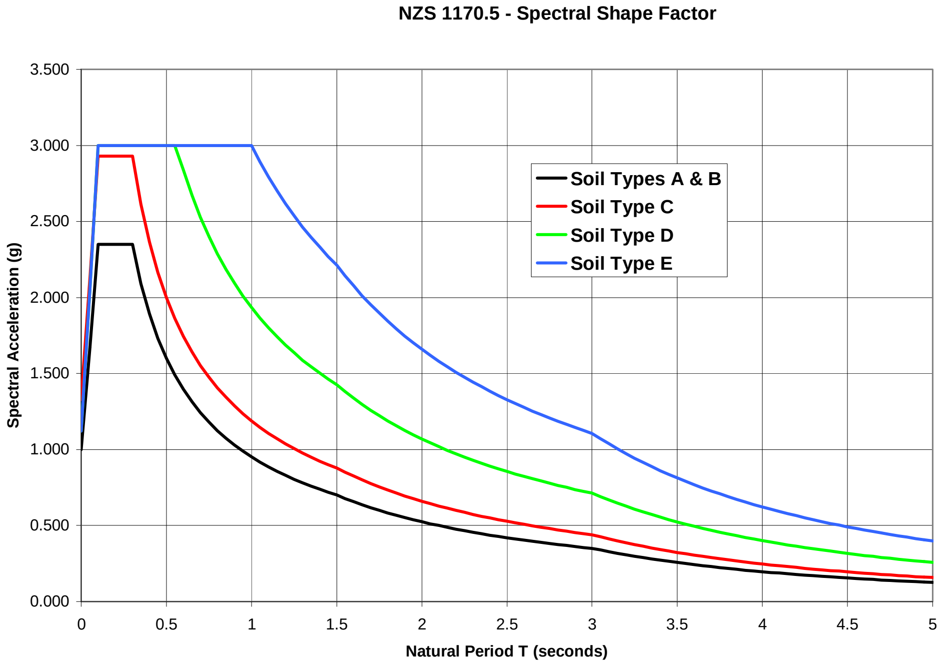


Figure 7. Seismic Lateral Force Response Spectra From NZS1170.5:2004

The computed natural period T is not known to a great degree of precision. The first uncertainty is the mass of the building at the time of the earthquake. The floor loadings vary with time and most standards take a fraction of the maximum design load as the long term likely load. Other uncertainties are the elastic properties of the materials (the elastic modulus of concrete usually increases with age but may also deteriorate with effects of weathering, moisture content and the effects of poor maintenance) and in reinforced concrete structures, the degree of cracking within the beam and column members (caused by other loadings or earlier small earthquakes etc.).

Another factor, unfortunately, is that a large number of design structural engineers do not consider the effect of foundation compliance (the effects of the flexible foundation and soils underlying the building etc) into their calculations for the natural period of free-vibration. Most structural engineering text books only show calculation examples where the building is assumed to be built on a rigid foundation. This ignoring of the effects of foundation compliance means that most of the natural periods computed are much smaller than the real values. The argument supporting this omission is that this shorter natural period usually result in larger design forces, providing a degree of conservatism in the design strengths, but it does also mean that the computed displacements may be under-estimated.

The development of design spectra for New Zealand has followed the trends of overseas practice but with several New Zealand innovations over the years leading to the current design standard NZS1170.5:2004. The coverage of these aspects of the development of the New Zealand Standard is beyond the scope of this report.

5 Factors that Influence Response Spectra Values

The factors that influence response spectra values for a building at a specific location are:

- The natural period of free-vibration;
- The percentage of critical viscous damping;
- The type of underlying soil;
- The Zone Factor;
- The distance from the epicenter of the earthquake;
- The distance and orientation of the fault line for structures near the source fault;
- The required earthquake return period appropriate for the building;
- The design ductility chosen for the building;
- Vertical earthquake components.

5.1 Natural period of free-vibration

As shown in Figures 4, 5 and 7 above the natural period of free-vibration is the most important factor in the magnitude of the spectral acceleration or spectral displacement for the building. The first step in a design for seismic actions is to determine the natural period of free-vibration for the building.

5.2 Percentage of critical viscous damping

The percentage of critical damping has a major effect in the magnitude of the spectra values. A damping of 5% of critical viscous damping approximately halves the spectral values. To halve the values of response again requires approximately 20% damping.

Most codes assume that the level of damping in a normal building as approximately 5% of critical damping. Research has shown bare steel frames may have less than 2% of critical damping. This implies a considerable increase in the response spectral values. The NZS1170.5 spectral shape shown in Figure 7 does not show variations for damping as the spectral shape is that for 5% damping. If the level of damping in the building was known to be markedly different from 5% then the designer should modify the spectral shape to allow for the variation in damping.

NZS1170.5 does not give any guidance on the changes associated with variations in the level of damping but some help can be found in some of the earthquake engineering literature.

5.3 Type of underlying soil

The current New Zealand seismic design standard NZS1170.5 provides design acceleration spectra that give different spectral shapes for different underlying soil types (refer Figure 7). These different responses are to take into account that the underlying soils change the motion that arrives at the ground surface.

A stiff foundation soil will transmit the high frequency components of the earthquake ground motion. A softer soil tends to amplify the ground motion. This is more evident for longer period motions as the soft soil is not able to transmit the high frequency accelerations associated with the bedrock motion. This is shown in the greater amplification for the longer periods (lower frequencies) than for the shorter periods.

The soil underlying a building is a structure in its own right and responds to the bedrock motion in the same way that a building on the surface responds to the ground motion at the top of the soil profile. This was demonstrated in the 1985 Mexico City earthquake where buildings with a natural period of free-vibration similar to the natural period of the underlying soils suffered very severe damage even though the epicenter of the earthquake was approximately 475 km from Mexico City.

5.4 The Zone Factor

The Zone Factor is a multiplier on the spectral values used for design. The Zone Factor is aimed to provide similar earthquake risk for any site in New Zealand. In locations where earthquakes are more likely to occur and the level of shaking is expected to be greater then the Zone Factor is higher. In areas where the risk is perceived to be lower the Zone Factor is smaller.

For Christchurch the Zone Factor presented in NZS1170.5 is 0.22. On 19 May 2011, the Department of Building and Housing issued a compliance document to revise the Zone Factor for Christchurch to 0.30 to account for the increased risk of significant earthqaukes for some time following the February 2011 earthquake.

5.5 The distance from the epicenter of the earthquake

The further the building site is from a likely earthquake epicenter the lower the observed level of shaking. This effect is largely built into the Zone Factor which takes into account the distance from likely earthquake sources such as active faults, and the likelihood of significant earthquakes occurring on those faults.

Earthquakes with an epicenter close to the site will show a greater content of the high frequency motion whereas for distant epicenters the longer period motions will be more evident. The high frequency content attenuates more rapidly with distance than does the low frequency content.

5.6 The distance and orientation of the fault line for structures near the source fault

Where the building is located near the fault that is the source of the earthquake motion, there has sometimes been observed a strong long acceleration pulse particularly in the direction normal to the fault. This may cause larger displacements in structures with longer natural periods of free-vibration. This effect was first observed in 1971, but 1994 Northridge (California) earthquake highlighted the consequences of this effect.

Design standards include a Near Fault Factor which is a multiplier to the design spectral acceleration value. In New Zealand it only applies if the building is less than 20 km from the source fault. NZS1170.5 lists the faults that are to be considered as candidates and their distances from the major towns and cities.

A factor of 1.0 is used for buildings with a natural period less than or equal to 1.0 second, and the factor increases to 1.72 for buildings with a natural period equal to or greater than 5 seconds. This maximum value holds for buildings within 2 km from the fault line and is linearly scaled down to a value of 1.0 as the distance increases from 2 km to 20 km.

5.7 The required earthquake return period appropriate for the building

The Return Period Factor, **R,** influences the magnitude of the spectral values used for design of buildings. This factor is a means of changing the level of design force and protection for the building.

For ordinary buildings designed for the 475 year return period event, i.e. having a 10% chance of the design forces being equaled or exceeded in the 50 year design life, the **R** factor has a value of 1.0. If the building is a critical structure, such as a hospital, which is required to be operational after a major earthquake, then it may be designed to have a similar level of damage for an earthquake with a return period of 2250 years, i.e. a 2% chance of the design forces being equaled or exceeded in the 50 year design life, in this case **R**=1.8. For temporary structures which have a short design life the value of **R** may be less than 1.0, the standard quoting a value of 0.6.

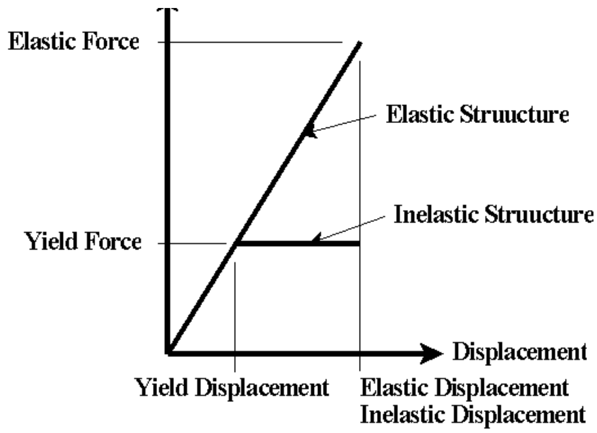
5.8 The design ductility chosen for the building

For over half a century it has been recognized that it is expensive to design a normal building to remain elastic, or undamaged, for the design level earthquake. The current design life of a building designed to New Zealand Standards is 50 years. The design level earthquake excitation is for an approximately 475 year return period so that it is assumed that there is a 10% chance that during the life of the building that design strength will be reached or exceeded.

There is a great risk in assuming elastic response for the design level earthquake, as is the pattern in certain parts of the world, in that no consideration is given to the behaviour of the building should that level of excitation be exceeded which may result in the building having a brittle failure mechanism. The current New Zealand Standard penalizes buildings designed without a minimum level of available ductility. Since 1975 New Zealand designers have used the concept of Capacity Design [Paulay, 1992]. Here the design is such that if the seismic excitation exceeds the elastic strength of the building the design ensures that only desirable ductile failure mechanisms are permitted. The aim is to prevent brittle mechanisms such as shear failure in the beams, failures in the beam-column joints and soft-storey mechanisms in the building as a result of hinging in columns. Even where structures are designed to meet higher strength requirements such that the structure should sustain no damage in the design level earthquake, the aim is still to ensure that the failure mechanism for a greater than design level earthquake would still be ductile.

Most design standards allow for inelastic design and the degree of inelasticity is usually measured in terms of the building ductility. The difficulty with design for buildings exceeding the yield strength of the building components is that the analyses are much more difficult and are, in general, impossible to carry out until the design is complete. This is certainly true for a non-linear, or inelastic time-history analyses. Therefore, the analyses carried out for design are usually modified elastic analyses. There is a problem in defining exactly what is meant by the yield displacement of a building but that is beyond discussion in this document.

Current design practice, world-wide, makes use of an observation made by Newmark in 1960 [Veletsos, 1960] where it was found that the displacements of an inelastic structure were similar to the displacements of an identical structure but which was assumed to remain linearly elastic. This has become known as the “equal displacement” concept.



$$\text{Ductility}\ \mu = \frac{\text{Inelastic Displacement}}{\text{Yield Displacement}}$$

Figure 8. Equal Displacement Concept.

If this concept is true, and the building, after yield, is assumed to be perfectly plastic (having no stiffness) then as can be seen using similar triangles in Figure 8, that for a building ductility factor μ then the yield force is the elastic force divided by μ. This means that if the designer assumes that the building will be designed for a ductility of four then the design forces are a quarter of the elastic design forces. The designer will have to ensure that the building can sustain the displacements associated with being displaced to four times the yield displacement. If the ductility is greater than 1.0 it means that for the design level earthquake there will be damage. The higher the ductility factor the greater the damage to the building. The result is lower initial construction cost and expected greater repair costs should the earthquake occur during the life of the building.

Increasing the design forces means increasing the strength of the building which increases the cost of the building. The design standards set out the minimum strength requirements. Most owners want the cheapest possible structure which means that most buildings are built to the minimum, or standards, requirements.

Newmark also noted that for buildings with short natural periods of free-vibration the energies, or velocities, were similar and this is shown in Figure 9 where the areas under the force-displacement curves are similar. This implies that the inelastic building has larger displacements than the elastic structure and the force reduction is smaller than that associated with equal displacement. As the natural period tends to zero then the accelerations of the elastic and inelastic structures are similar so that the forces are similar.

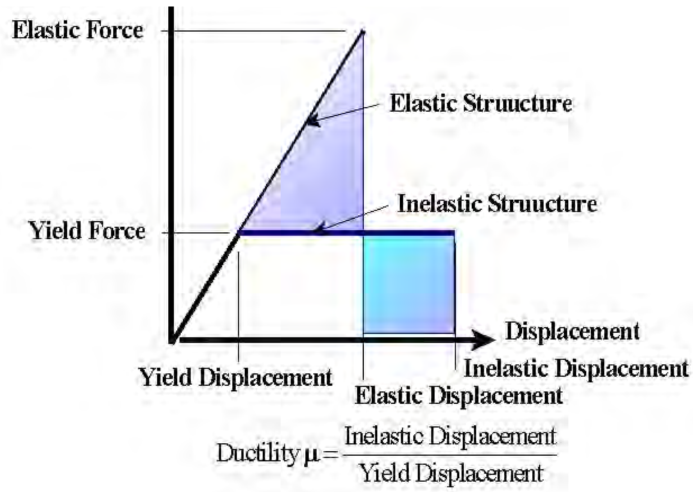


Figure 9. Equal Energy Concept.
Shaded Areas are Equal

To avoid the steps that would occur in the inelastic spectra when one moved from one natural period region to another, NZS1170.5 uses a linear interpolation with natural period, from using a force multiplier of 1.0 at zero natural period (equal acceleration) to a force multiplier of **1/μ** (equal displacement) at a natural period of 0.7 seconds. The inelastic displacements are always **μ** times the elastic displacement.

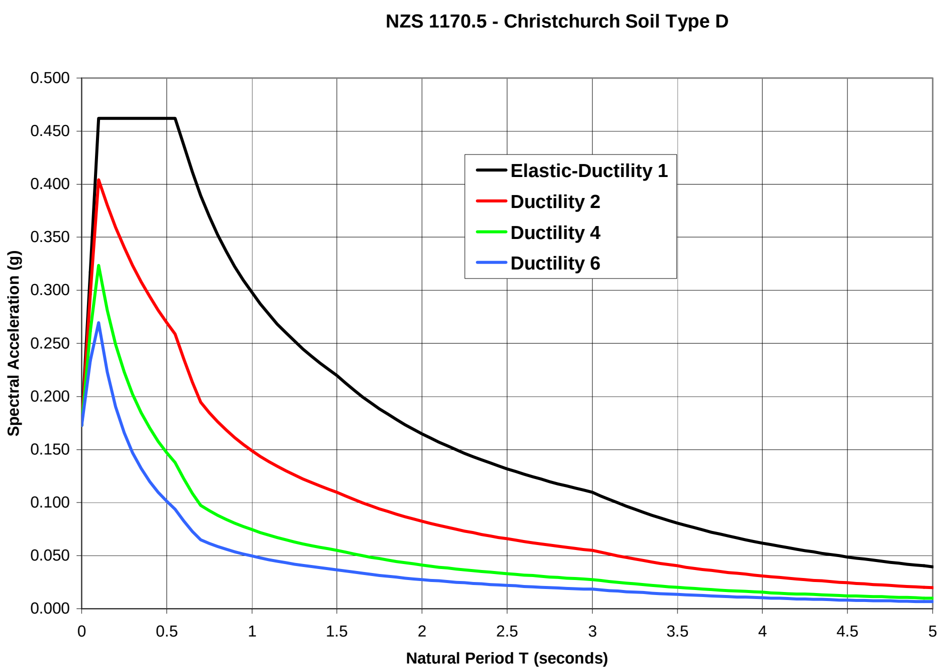


Figure 10. Inelastic Acceleration Response Spectra for Soil Type D using NZS1170.5 relationship between the elastic and inelastic spectra

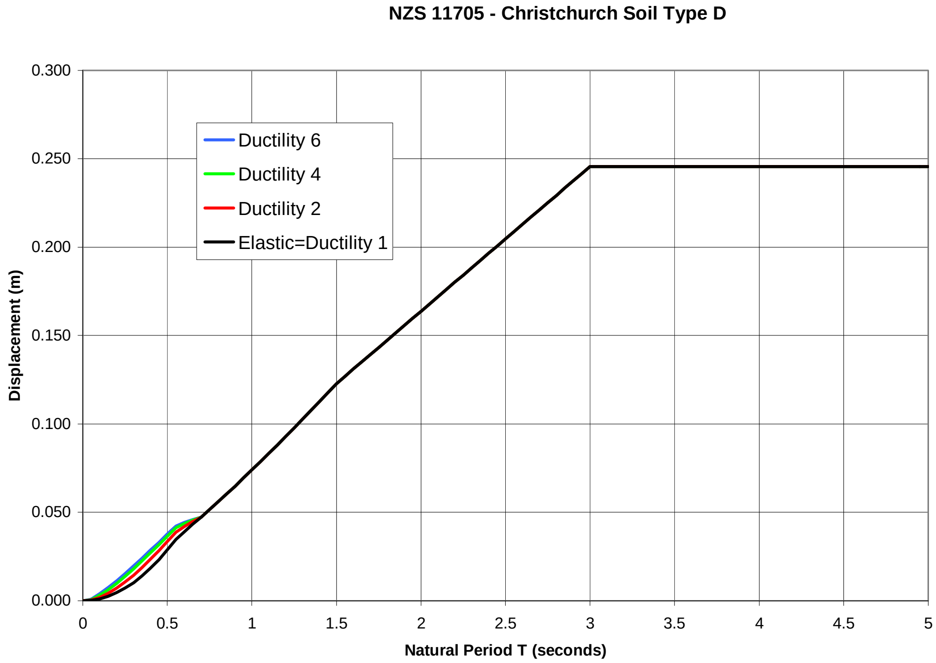


Figure 11. Inelastic Displacement Response Spectra for Soil Type D using NZS1170.5 relationship between the elastic and inelastic spectra

Figures 10 and 11 show the inelastic design spectra for ductility 1 (elastic), ductility 2, ductility 4 and ductility 6 derived from the NZS1170.5:2004 elastic spectra. It can be seen that for natural periods greater than 0.7 seconds equal displacement is assumed. It will be noted that although the inelastic displacements are multiplied by **μ** at very small natural periods they do not seem to be large as the elastic displacement tend to zero as T tends to zero.

It was the observed marked deviations from these concepts in a series of inelastic analyses using the Christchrch earthquake records that led to the commissioning of this report.

5.9 Vertical earthquake components

In design for vertical accelerations the design spectra are taken as being the same as for the horizontal accelerations but multiplied by a factor of 0.7. Traditionally, design for vertical forces is governed by the maximum design live loads which have relatively large factors of safety and assume an elastic design. The vertical earthquake components are usually felt to be within the envelope of these vertical loads as there is the assumption that the long term live loads (40% of the maximum) are more appropriate than the maximum design live loads at the time of the earthquake..

6 Acceleration and Displacement Response Spectra for the Christchurch Central Business District

There are four stations in the Christchurch Central Business District (CBD) at which strong motion records were recorded during the Christchurch earthquakes. These sites were at Christchurch Hospital (near Antigua Street in the South-West part of the CBD), Christchurch Botanical Gardens (near the Rose garden to the West of the CBD), Christchurch Cathedral College (near Barbados Street to the South-East of the CBD) and Resthaven Retirement Home (near Peacock Street in the North-West of the CBD). All instruments provided records for the 4th September 2010 earthquake and the 22nd February 2011 earthquake but there are no records for Christchurch Cathedral College for the 13th June 1011 earthquake. Some of the acceleration records show recording errors for which GNS is not yet able to explain but these are not of any significance to this report as they occur in the later parts of the records long after the significant shaking had died away. All acceleration records are recorded at a time-step of 0.005 seconds.

The inelastic spectra shown in this report are generated using the computer program INSPECT [Carr, 2003,2011] developed in 2002 at the Department of Civil and Natural Resources Engineering at the University of Canterbury. The program is part of the Ruaumoko suite of non-linear structural analysis programs developed over the past 35 years. Further information on the capabilities of Ruaumoko can be found at www.ruaumoko.co.nz.

INSPECT was initially designed to generate inelastic response spectra because it was felt that the 1960 Newmark equal displacement concept used by seismic codes to obtain the design acceleration and displacement spectra should be reviewed in light of the many developments that had taken place in earthquake engineering, dynamic structural analyses and computing power since 1960. During the past 50 years there have been a very large number of strong ground motion accelerations recorded in many places around the world, A large number of these acceleration records show levels of shaking much greater than those known in 1960. When Newmark made his observations, computers were relatively slow with very small memories, programs for structural analysis were almost non-existent and the total number of recorded earthquake motions was very small.

This INSPECT program initially computes the elastic acceleration spectrum for the earthquake. For a given target ductility ratio **μ** the program estimates the level of the yield force using equal displacement for long natural periods and equal energy for the shorter natural periods. The program then iterates by varying the yield force to achieve a ductility to within 0.5% of the target ductility. The maximum number of iterations is limited to 200.

The following points may be noted:

- In the examples the damping was assumed to 5% of critical viscous damping
- The target ductility was 2, 4 and 6.
- Newmark's original paper assumed that the inelastic building was elastic-perfectly plastic and this model was used for these analyses. Analyses for the February 22nd 2011 earthquake were repeated for a more realistic Modified Takeda model which better represents a reinforced concrete buildings [Carr 2010.

- Most analyses used only a small number of iterations but there were some natural periods where the 0.5% tolerance was not achieved in 200 cycles. As other researchers have found, for some natural periods and some earthquake records, the displacements vary widely with only very, very small changes in the yield force. This is particularly noticeable in the inelastic displacement spectra for the Resthaven accelerogram in the February 22nd earthquake when using the elasto-plastic (elastic-perfectly plastic) hysteresis model.
- The spectra were computed at 0.05 second steps of the natural period.
- All analyses used a time-step of 0.001 seconds which provides 50 steps per cycle for the smallest natural period considered. This should enable high accuracy in modeling the inelastic response
- The analyses used 40 seconds for the duration of the earthquakes except for the September records when 60 seconds was used. The analyses could have ignored the first parts of the record where nothing is happening but, for simplicity, the GNS records were used without deleting the initial parts of the records.
- All the spectra shown for the earthquakes have been plotted using the same scales so that the amplitude difference between the different recording sites and different earthquakes are easily observed.

In particular, if Newmark's Equal Displacement concept applied then the displacement spectra for all ductility levels should show lines that are close together for the longer natural periods and show greater than the elastic displacements as the natural period becomes small. Theory requires the relative displacement to be zero when the natural period is zero. Similarly, the ductile acceleration spectra for a building ductility of $\boldsymbol{\mu}$ should be $\mathbf{1/\boldsymbol{\mu}}$ of the elastic acceleration spectra and tend to the same value as the natural periods become small. Theoretically they must be the same as the maximum ground acceleration when the natural period T is zero.

All of the earthquake acceleration records used in this report are shown in Appendix A.

In the following section a selection of computed inelastic displacement spectra and inelastic acceleration spectra are shown. The results of the analyses have been grouped into three sections, one for each of the September 2010, February 2011 and June 2011 earthquakes.

All the inelastic spectra are available for study in Appendices, B, C, D, and E.

7 4 September 2010 Earthquake Horizontal Components

In this earthquake the strongest shaking is seen in the North-South direction.

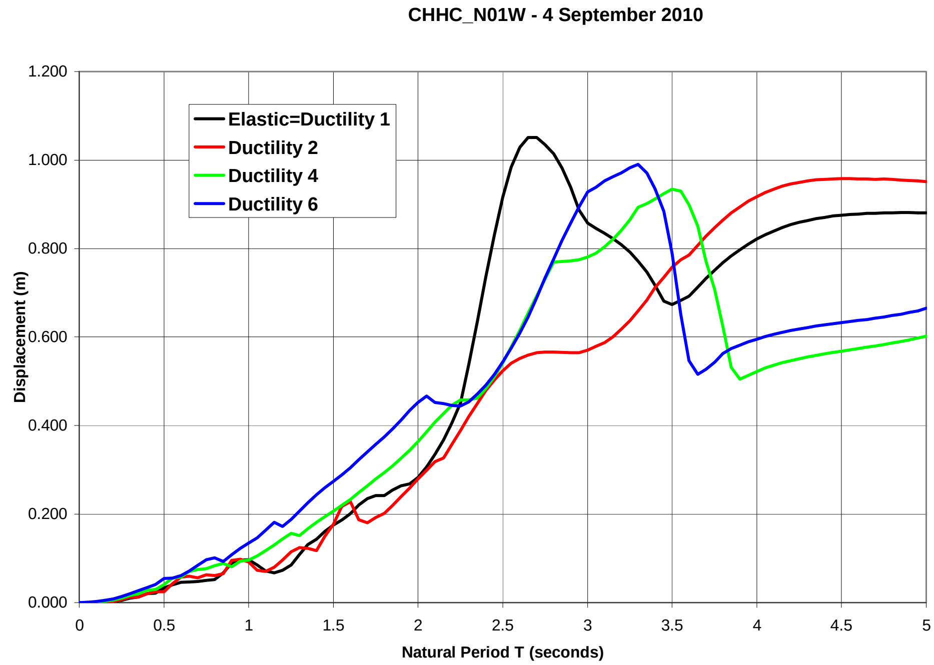


Figure 12. Displacement Response Spectra
Christchurch Hospital, 4 September 2010- North 01° West Component

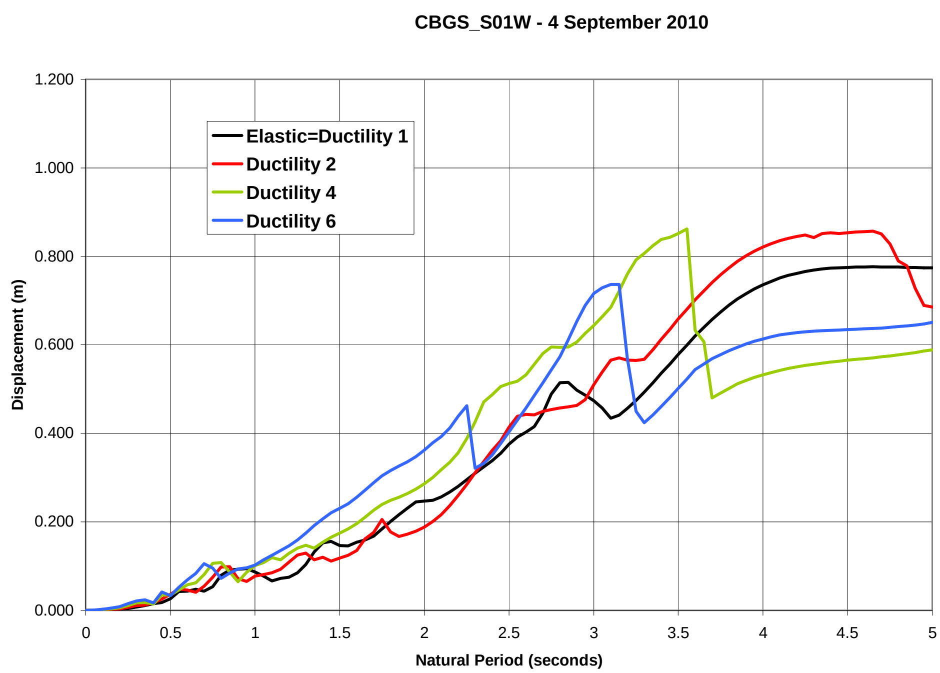


Figure 13. Displacement Response Spectra
Christchurch Botanic Gardens. 4 September 2010- South 01° West Component

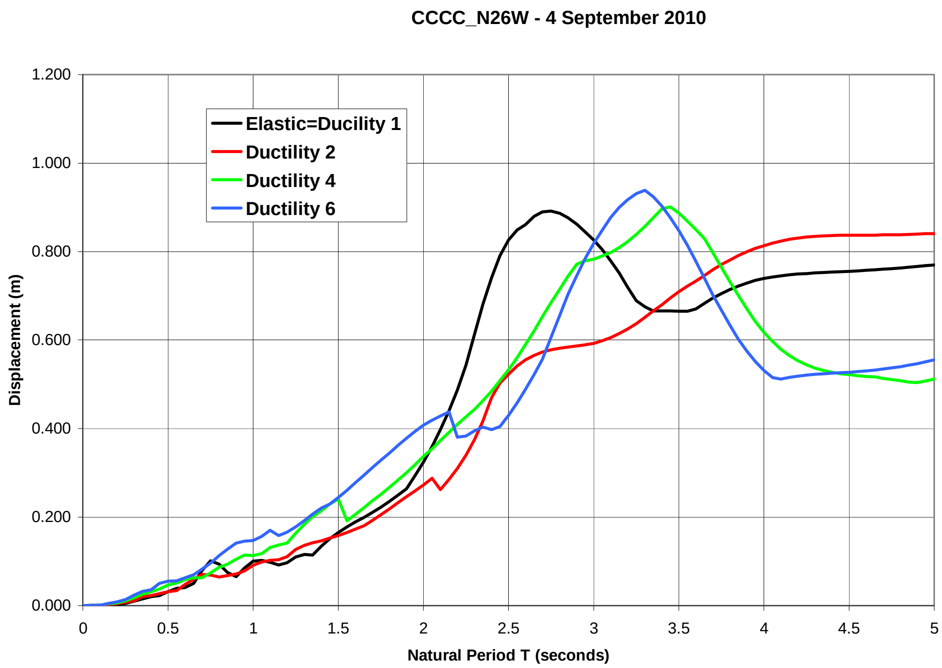


Figure 14. Displacement Response Spectra
Christchurch Cathedral College. 4 September 2010- North 26° West Component

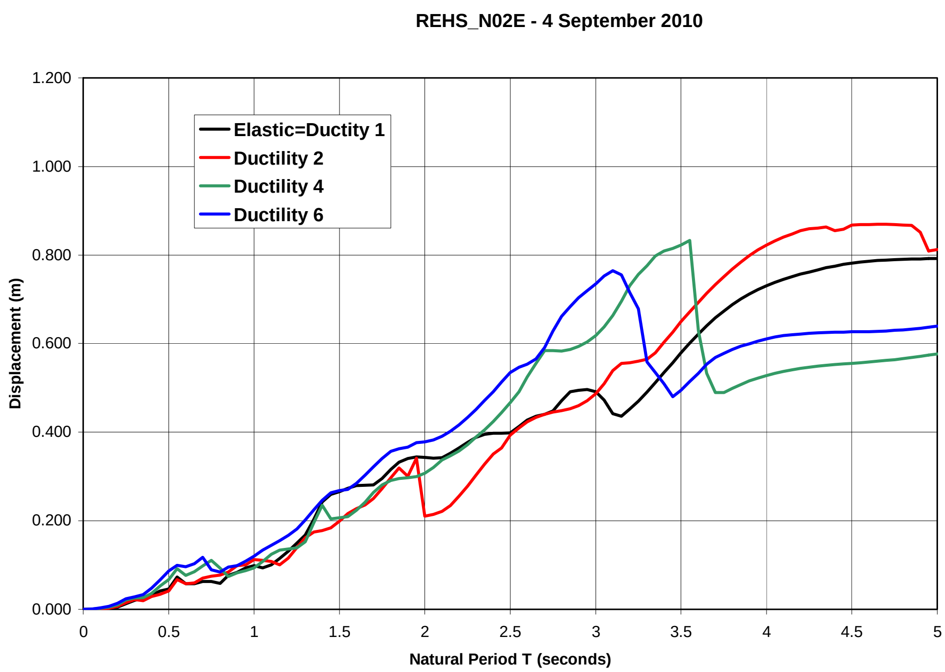


Figure 15. Displacement Response Spectra
Resthaven. 4 September 2010- North 02° East Component

It can be seen that none of these spectra fit the equal displacement concept. The inelastic displacements are greater than the elastic displacements for nearly all natural periods of free-vibration with only reductions for large ductility displacements for natural periods greater than about 3.5 seconds.

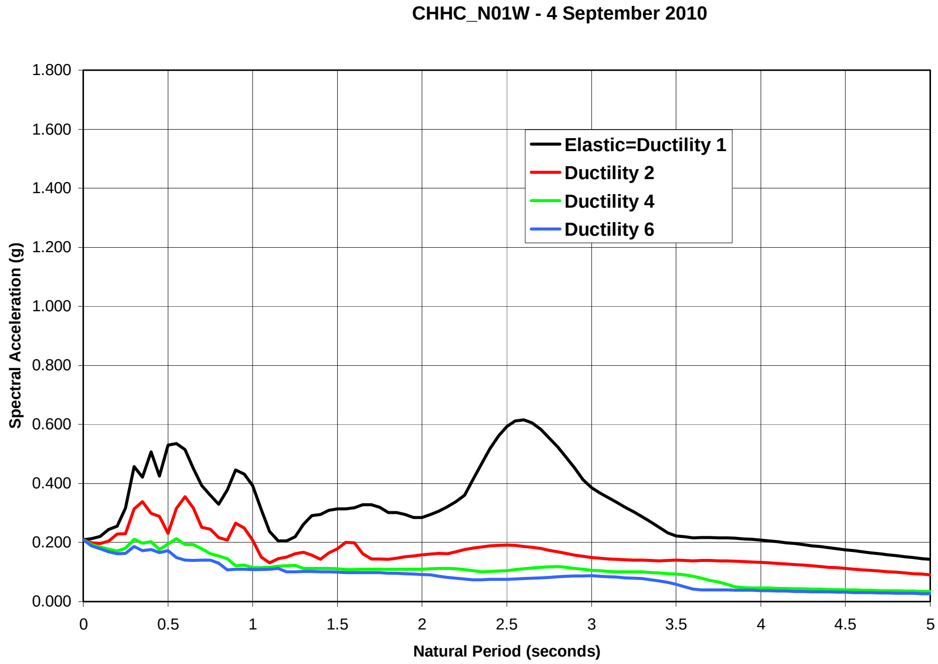


Figure 15. Acceleration Response Spectra
Christchurch Hospital 4 September 2010- North 01° West Component.

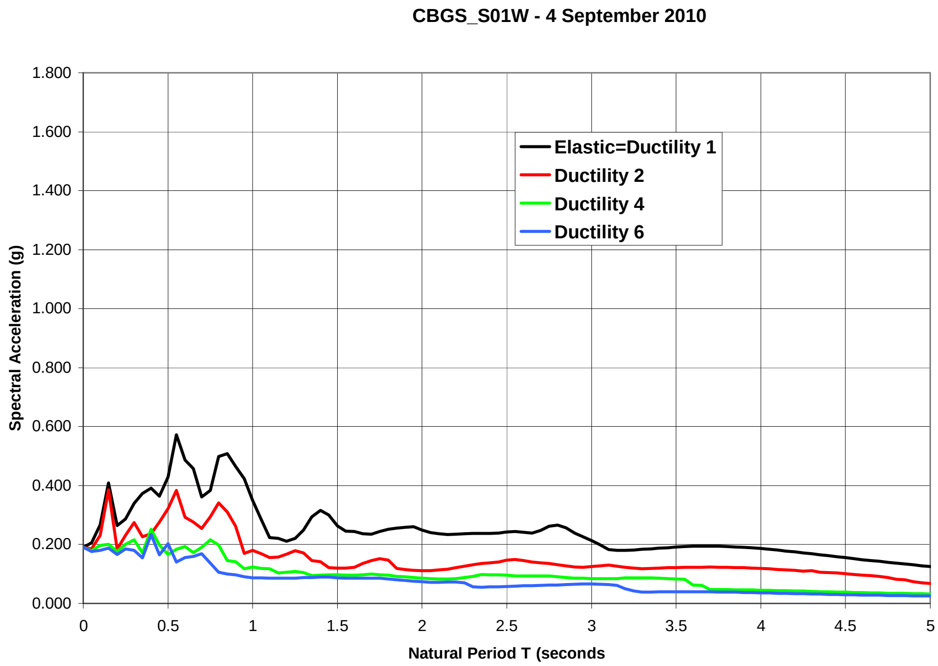


Figure 16. Acceleration Response Spectra
Christchurch Botanic Gardens. 4 September 2010- South 01° West Component

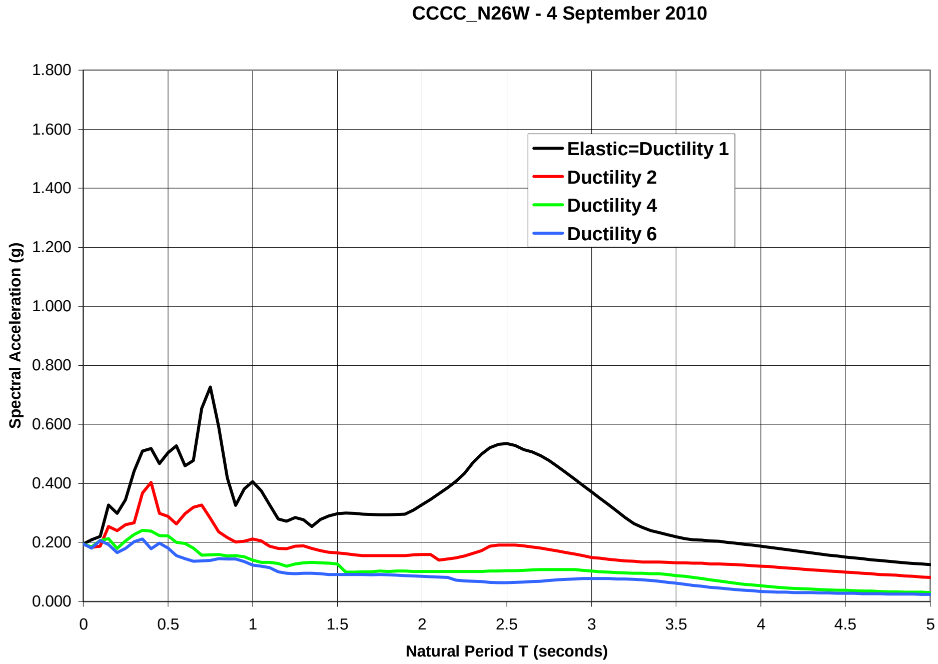


Figure 18. Acceleration Response Spectra
Christchurch Cathedral College. 4 September 2010- North 26° West Component

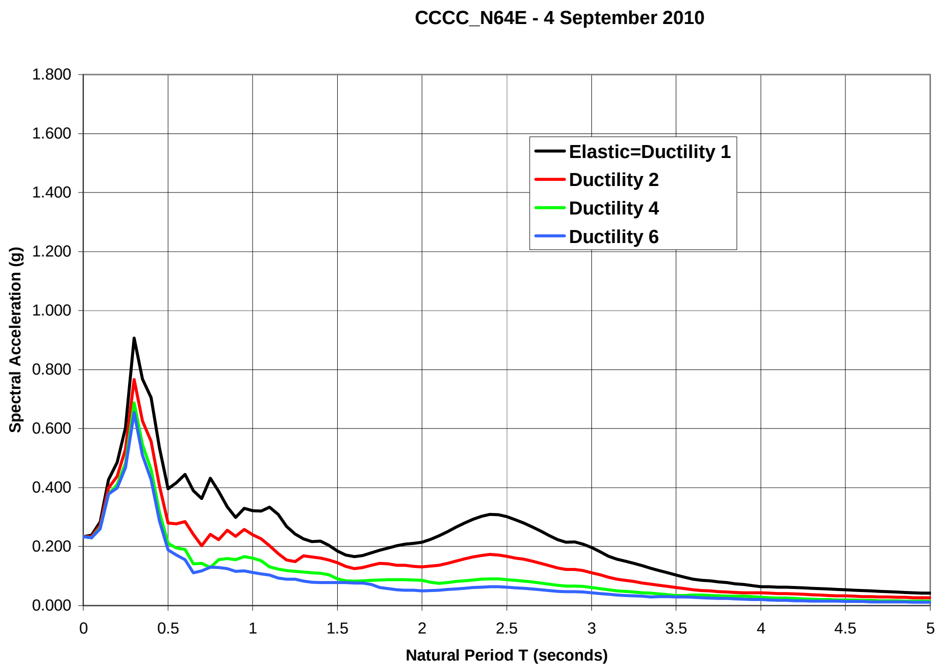


Figure 19. Acceleration Response Spectra
Christchurch Cathedral College. 4 September 2010- North 64° East Component

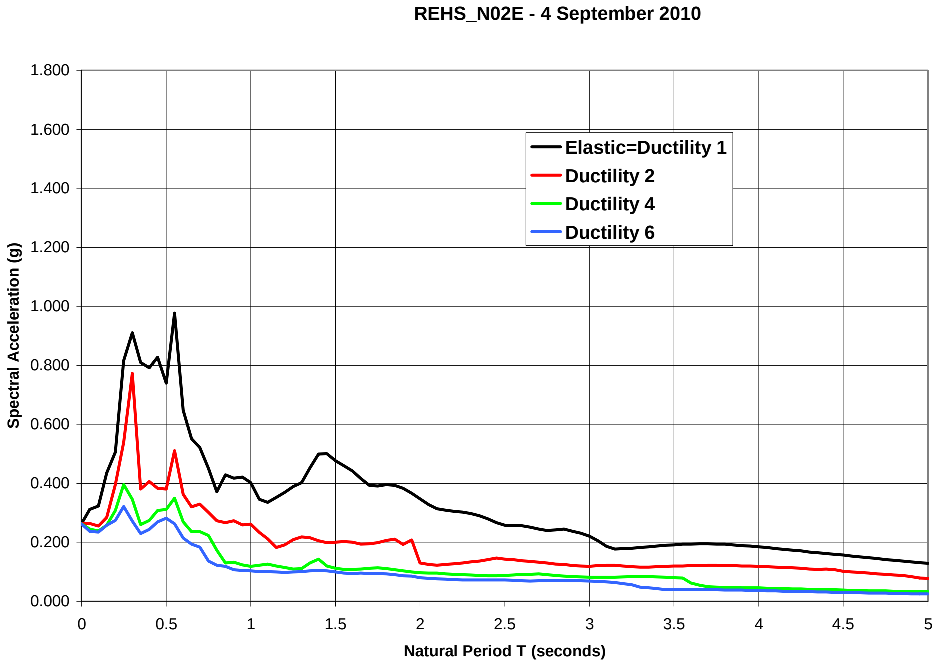


Figure 20. Acceleration Response Spectra
Resthaven. 4 September 2010- North 02° East Component

The acceleration records, in general, show large accelerations for the longer natural periods of free-vibration. This is probably the effects of attenuation with distance for the higher frequencies (short periods) and also the amplification of the responses in the longer natural periods caused by the great depth of gravels and other soft material underlying the city. Two spectra for Christchurch Cathedral College are shown. One clearly shows the amplification of the long period response whilst in a direction at right angles the motion is predominantly in the higher frequencies (shorter periods).

It must be noted that the yield forces as a function of the elastic forces (accelerations) are not reduced by dividing by the ductility factor as expected from the equal displacement concept but by numbers equal to, or greater, than the ductility factor. The ratio is sometimes greater than twice the ductility factor.

8 22 February 2011 Earthquake Horizontal Components

This earthquake indicated greatest shaking in the East-West direction.

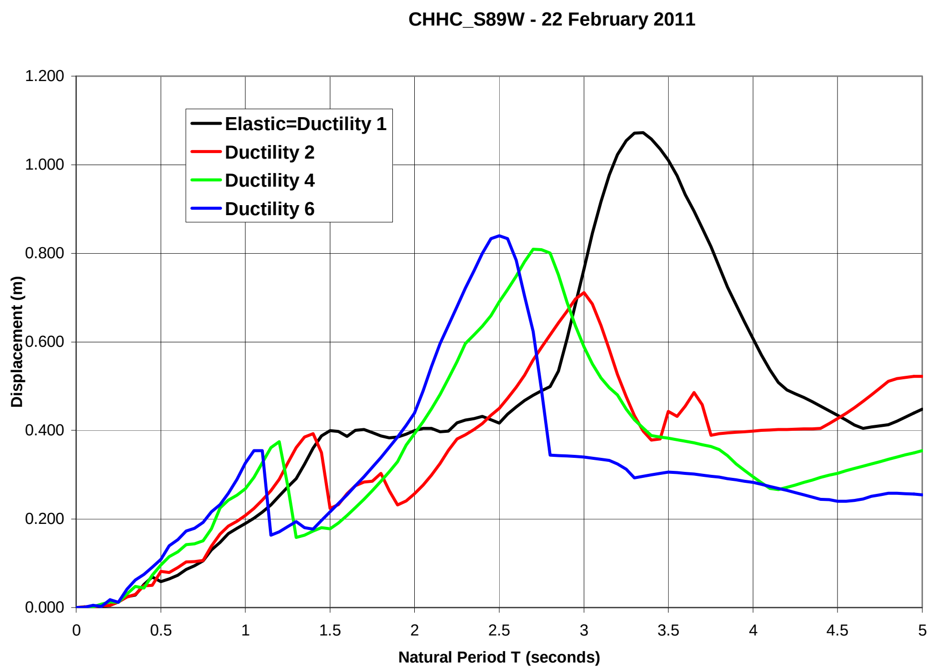


Figure 21. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component

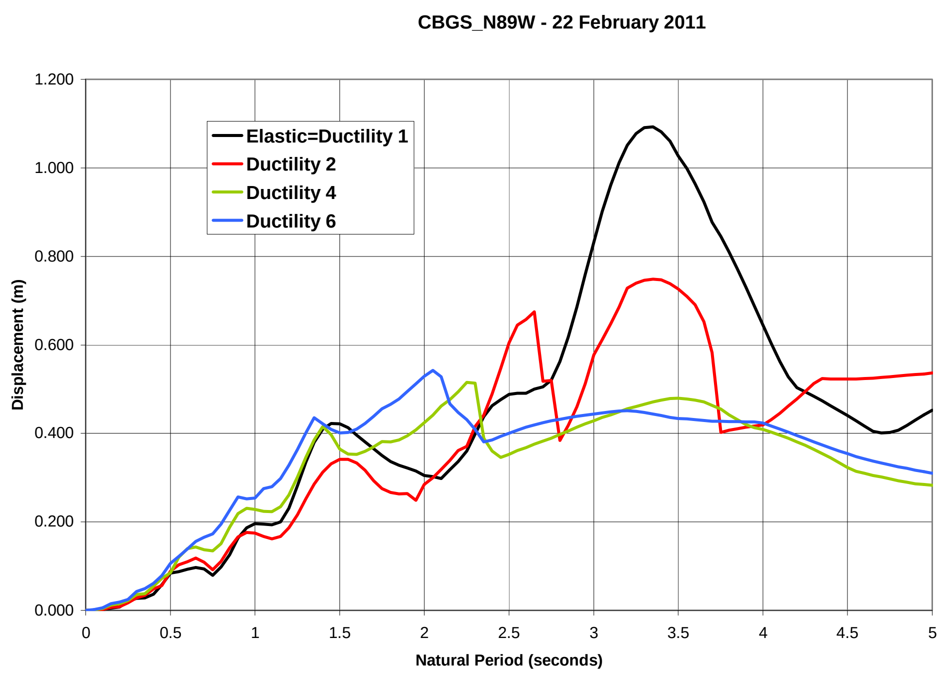


Figure 22. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component

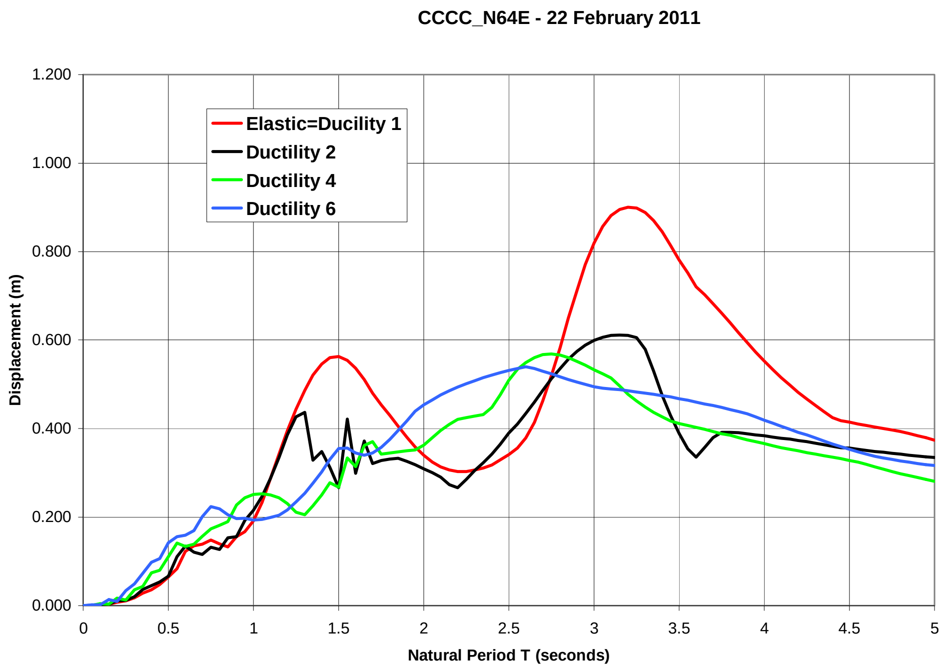


Figure 23. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component

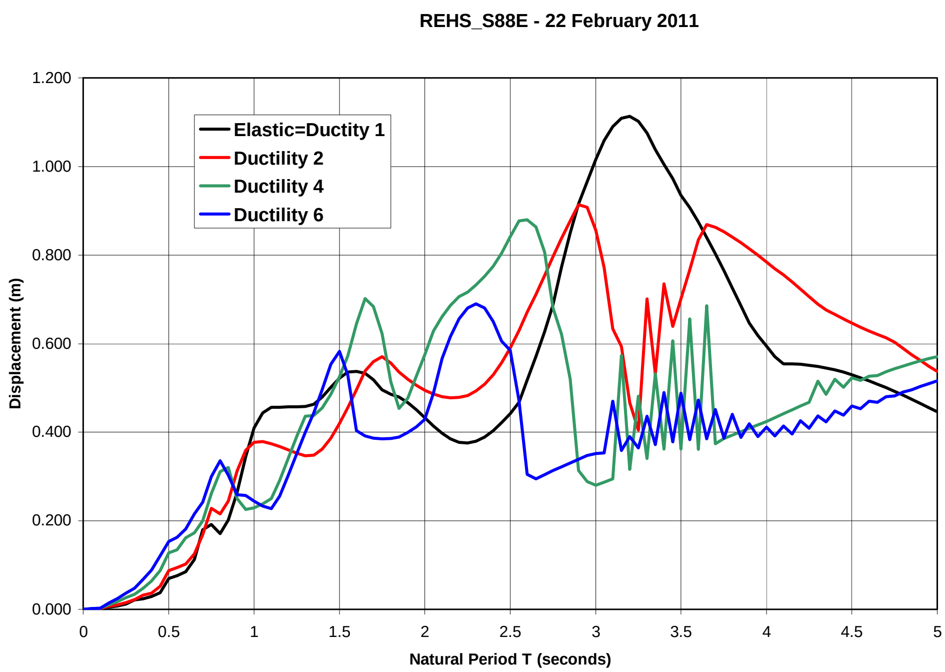


Figure 24. Displacement Response Spectra
Resthaven. 22 February 2011- South 88° East Component

It can be seen that none of these spectra fit the equal displacement concept. The inelastic displacements are greater than the elastic displacements for nearly all natural periods less than 3 seconds and for greater periods the inelastic spectra of often less than the elastic spectra. The Christchurch Cathedral College spectra show very large displacements for ductility 2 at the longer natural periods.

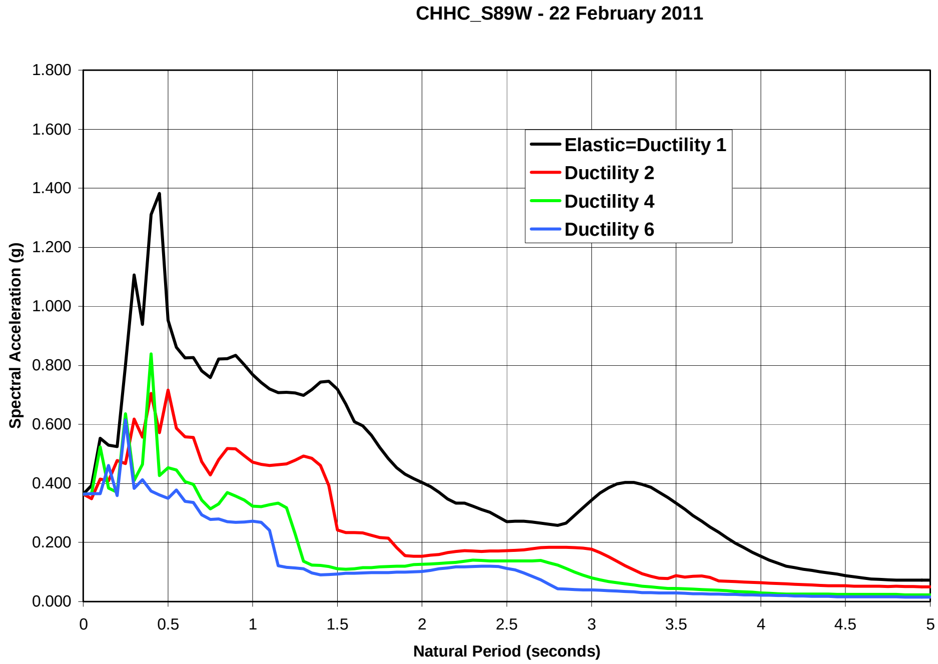


Figure 25. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component

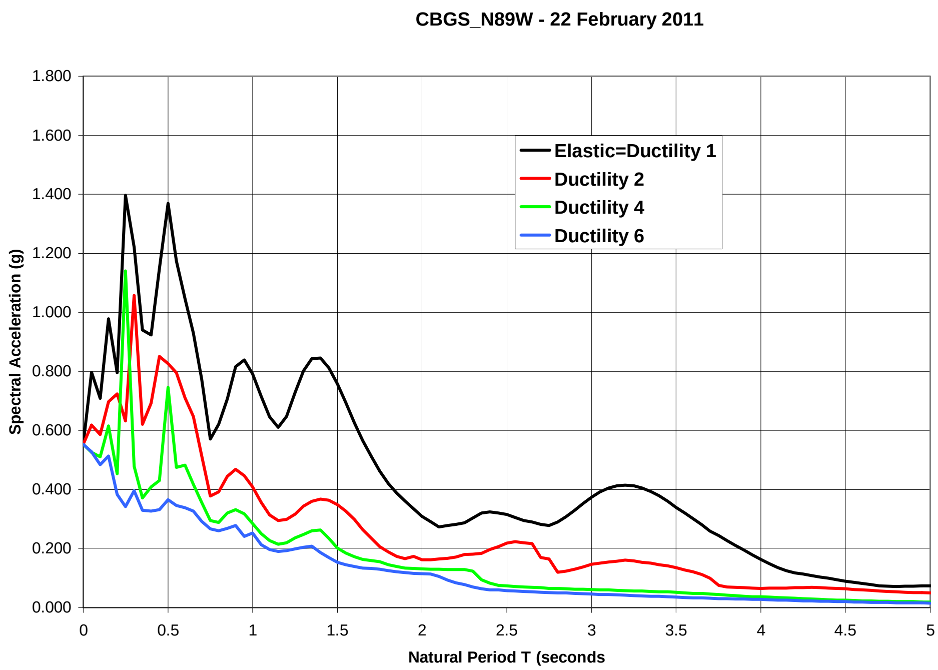


Figure 26. Acceleration Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component

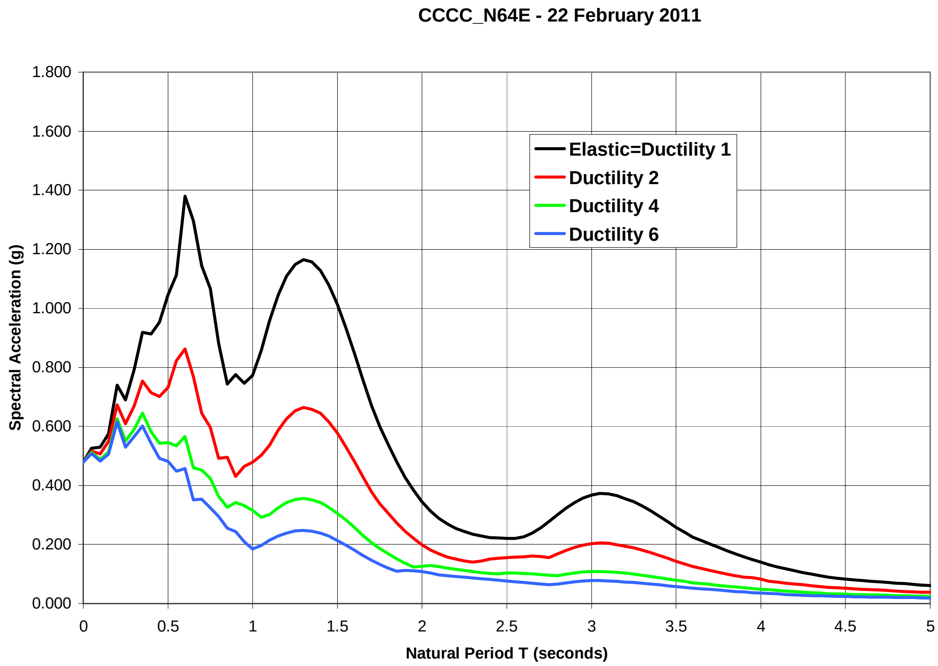


Figure 27. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component

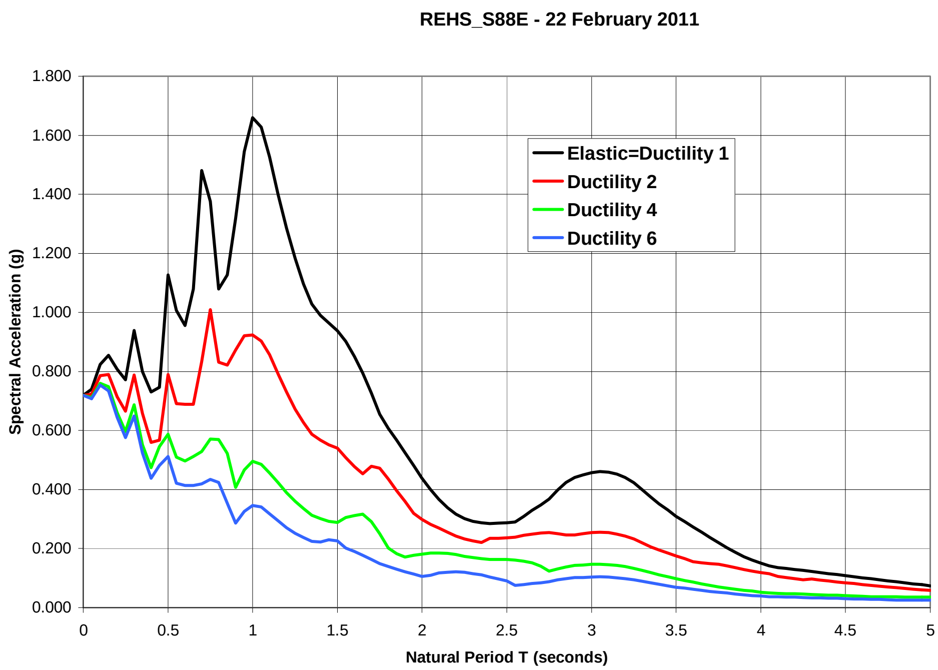


Figure 28. Acceleration Response Spectra
Resthaven. 22 February 2011- South 88° East Component

The acceleration records, in general, show large accelerations for short natural periods of free-vibration. This is probably the effects of the very close earthquake epicentre with near-fault effects, but at about a 3 second period the effects of the soft deep underlying soils is also evident. Again study of the force reduction factors shows that the yield forces, as a function of the elastic forces (accelerations), are not reduced by dividing by the ductility factor as expected from the equal displacement concept, but by numbers greater than the ductility factor, the ration often being up to twice the ductility factor.

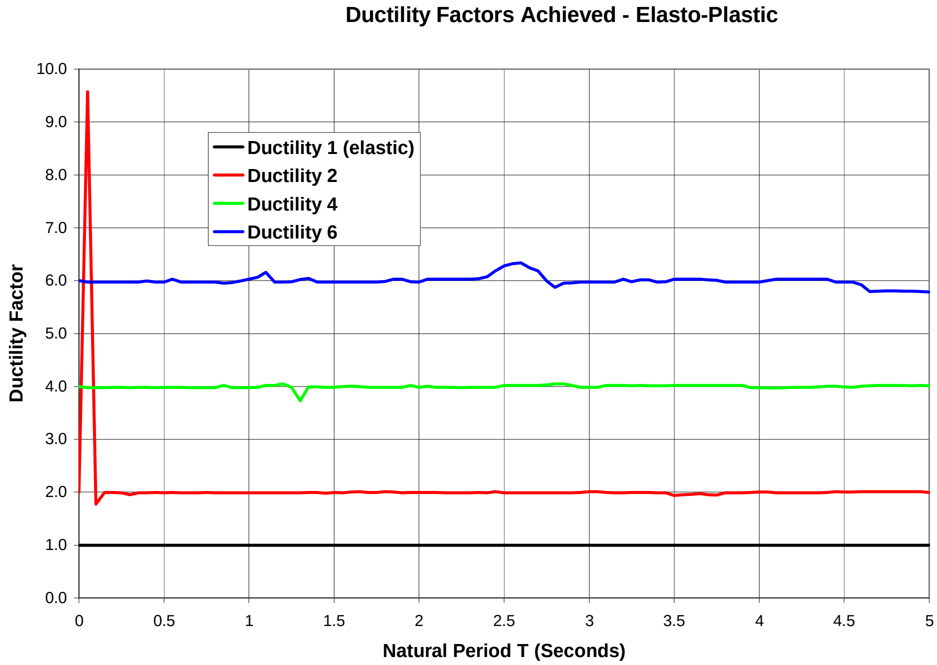


Figure 29. Achieved versus Target Ductility
Christchurch Hospital 22 February 2011- South 89° West Component

Figure 29 show good convergence for most target ductility values except for the first natural period (0.05 seconds) for ductility 2. Most spectra show better convergence over the whole range of the natural periods.

9 13 June 2011 Earthquake Horizontal Components

The records indicate that the predominant shaking in this earthquake is in the North-South direction.

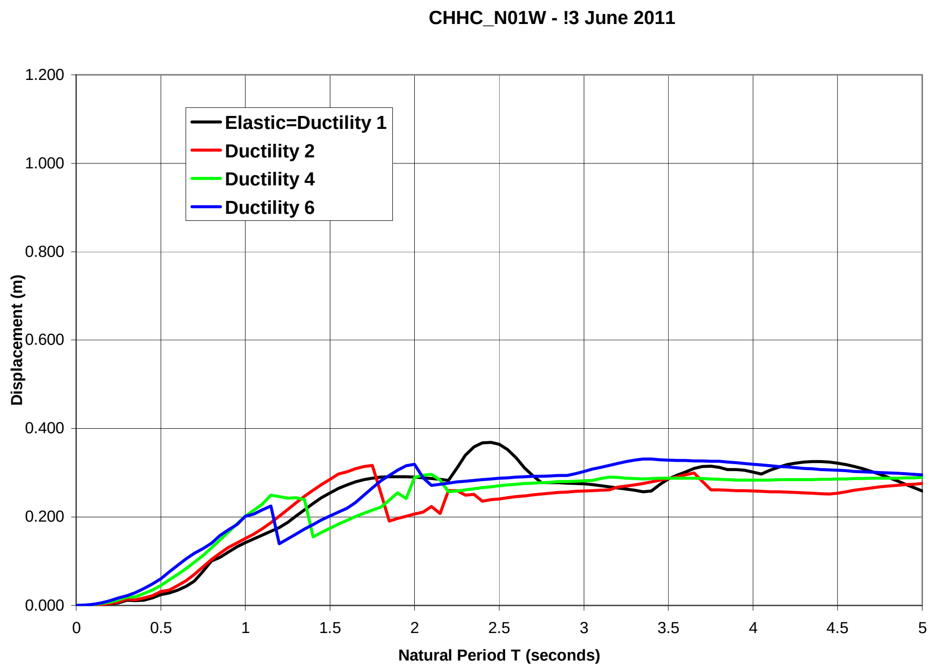


Figure 30. Displacement Response Spectra Christchurch Hospital, 13 June 2011- North 01° West Component

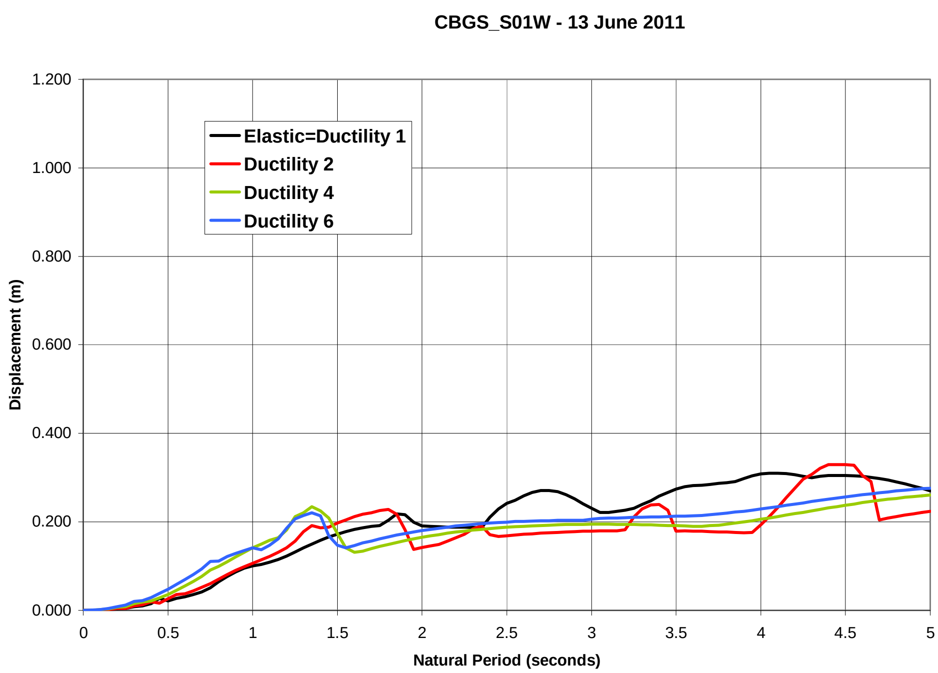


Figure 31. Displacement Response Spectra Christchurch Botanic Gardens. 13 June 2011- South 01° West Component

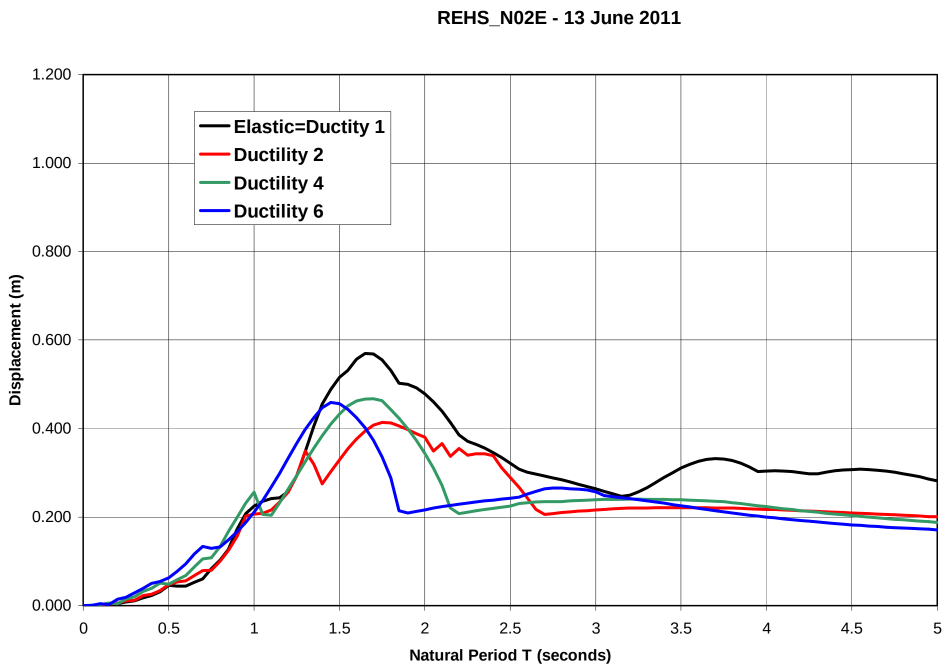


Figure 32. Displacement Response Spectra
Resthaven. 13 June 2011- North 02° East Component

These spectra appear to show a better fit to the equal displacement concept than do the other earthquakes. However, the inelastic displacements are greater than the elastic displacements for nearly all natural periods less than 2 seconds.

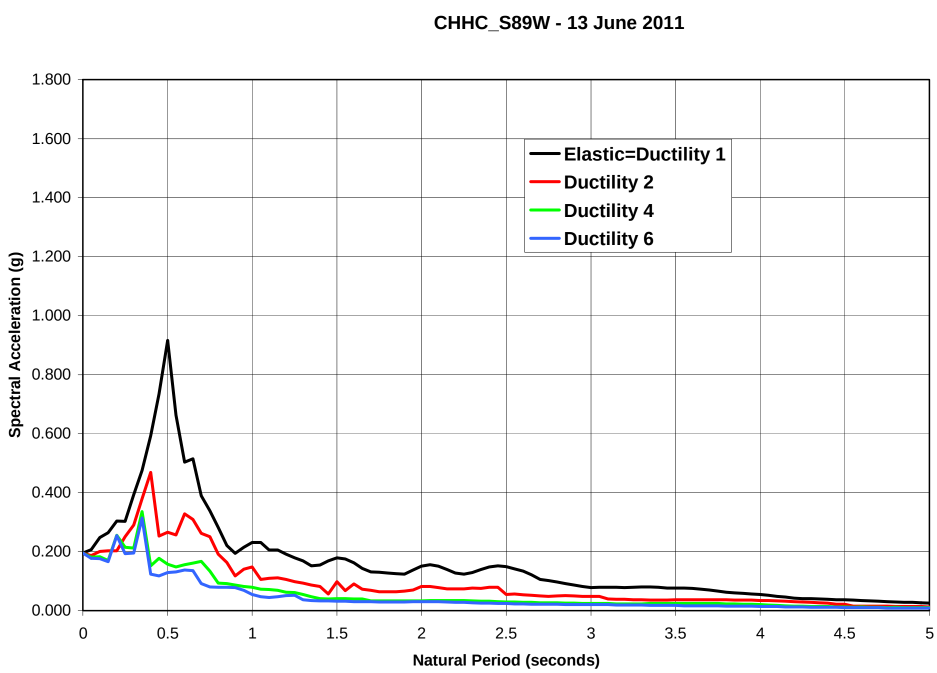


Figure 33. Acceleration Response Spectra
Christchurch Hospital, 13 June 2011- South 89° West Component

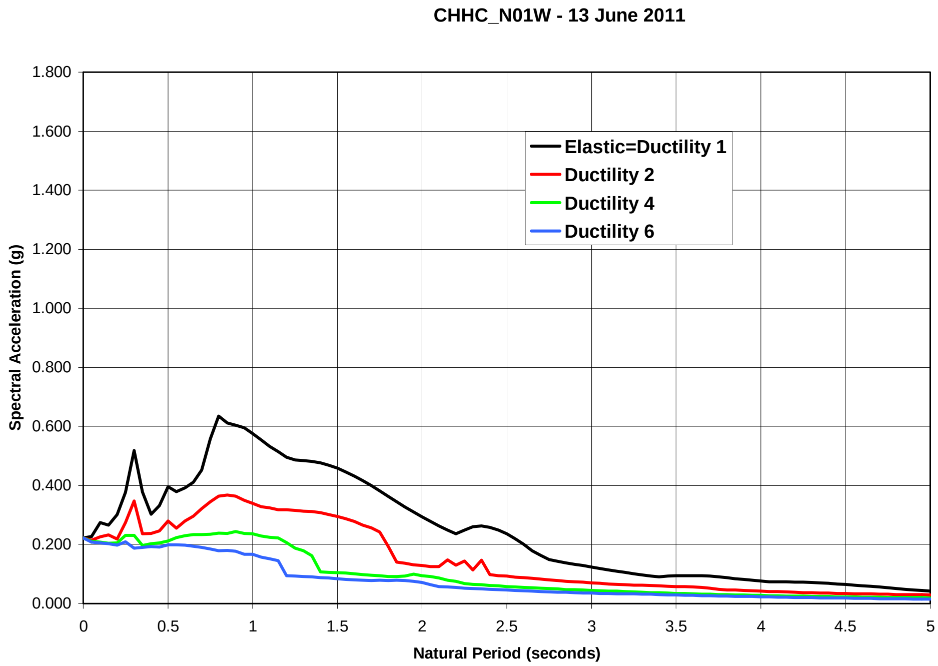


Figure 34. Acceleration Response Spectra
Christchurch Hospital 13 June 2011- North 01° West Component.

Figure 33 shows a dominant response at the short natural periods whereas the North-South component has marked increase in response at natural periods greater than 0,5 seconds.

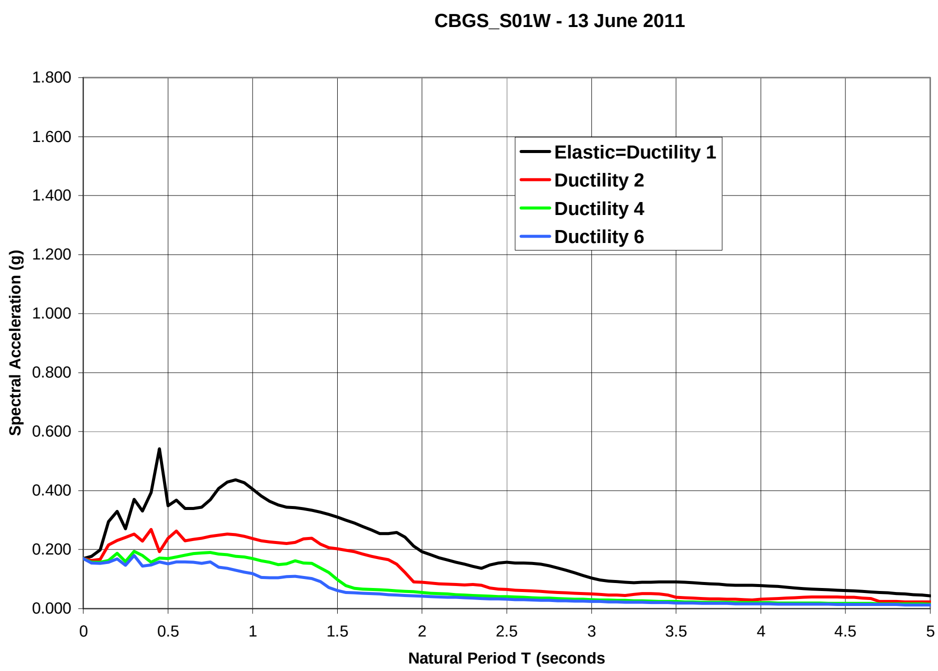


Figure 35 Acceleration Response Spectra
Christchurch Botanic Gardens. 13 June 2011- South 01° West Component

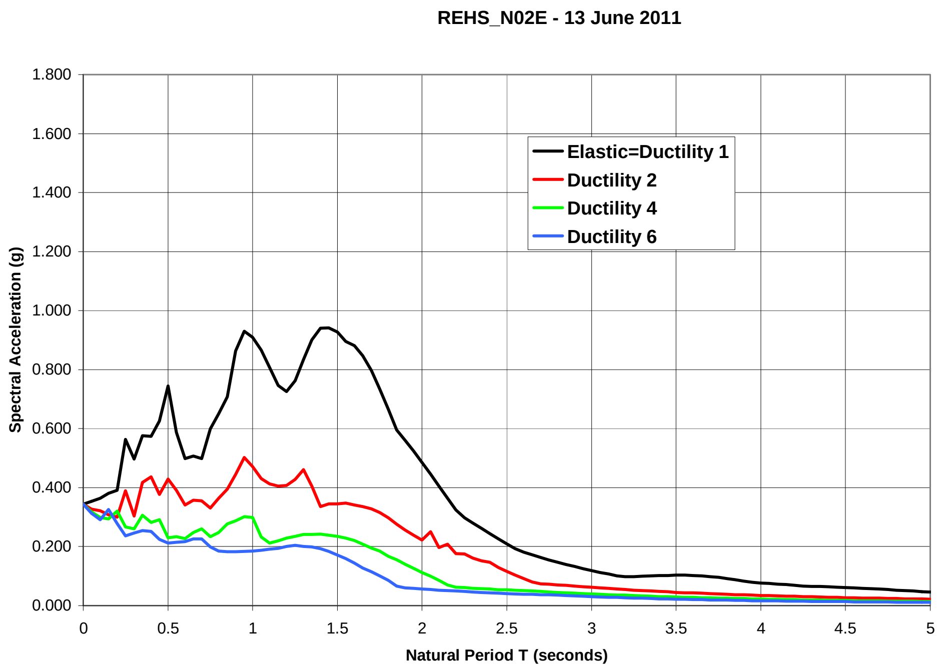


Figure 36. Acceleration Response Spectra
Resthaven. 13 June 2011- North 02° East Component

All of the these acceleration spectra show large accelerations in the 1 second to 2 second natural period range, showing the amplification due to the soft underlying soils. The epicentre was a little further away than for the February earthquake and the orientation of the predominant motion does not appear to be directed towards the city centre as was evident in the February earthquake. The effects of the deep soft underlying soils can be seen in the spectra. As was observed for the earlier earthquakes a study of the force reduction factors shows that the yield forces as a function of the elastic forces (accelerations) are not reduced by dividing by the ductility factor as expected from the equal displacement concept.

It must be noted that although the differences are not so noticeable in the spectra for this earthquake the differences are masked by the fact that when compared with the September and February earthquakes the displacements are much smaller and the plots for all earthquakes are drawn using the same scales.

10 22 February 2011 Earthquake Vertical Components

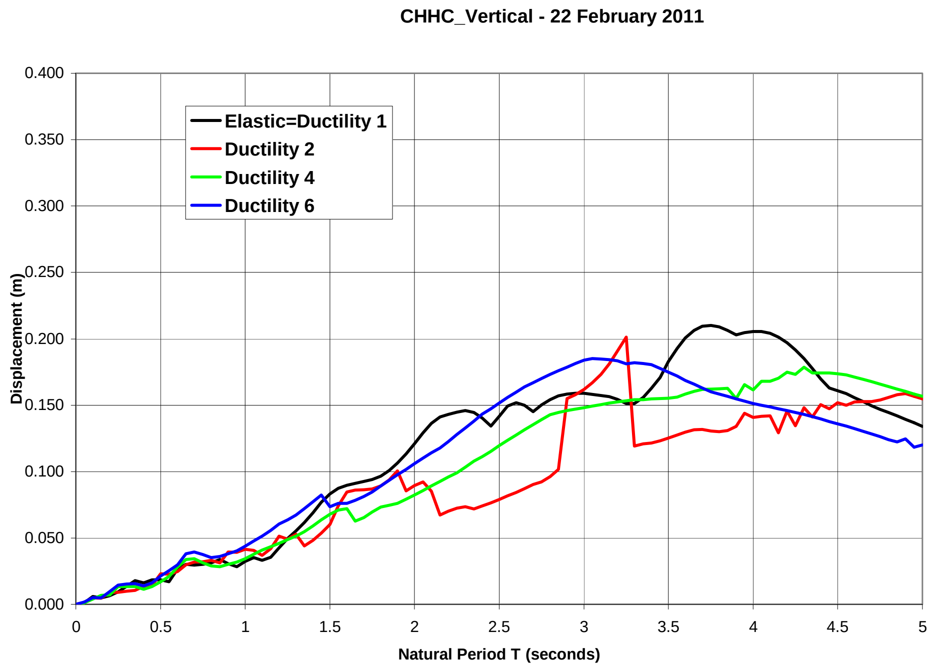


Figure 37. Displacement Response Spectra
Christchurch Hospital 22 February 2011- Vertical Component

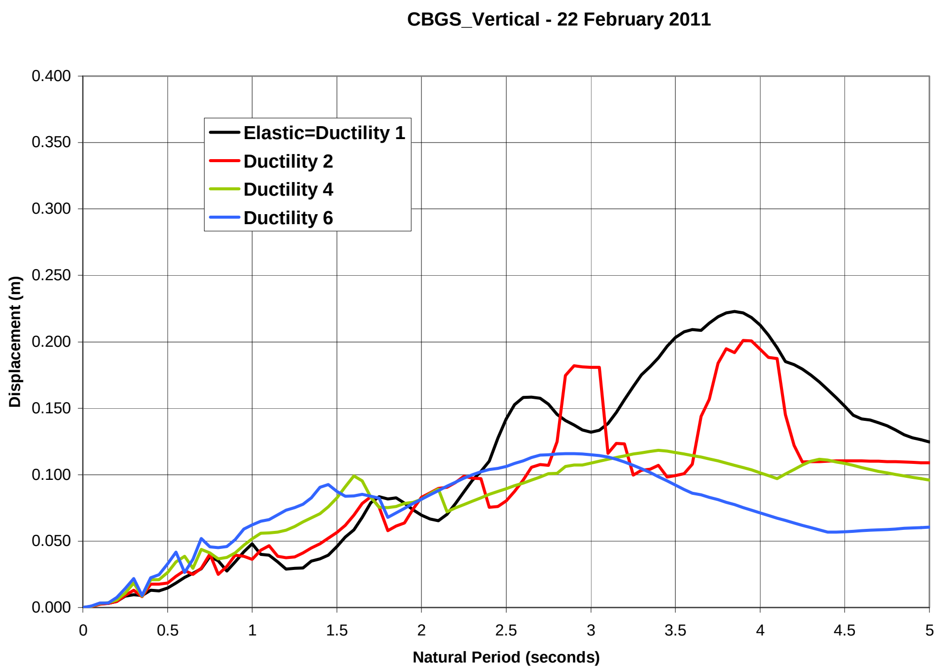


Figure 38. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- Vertical Component

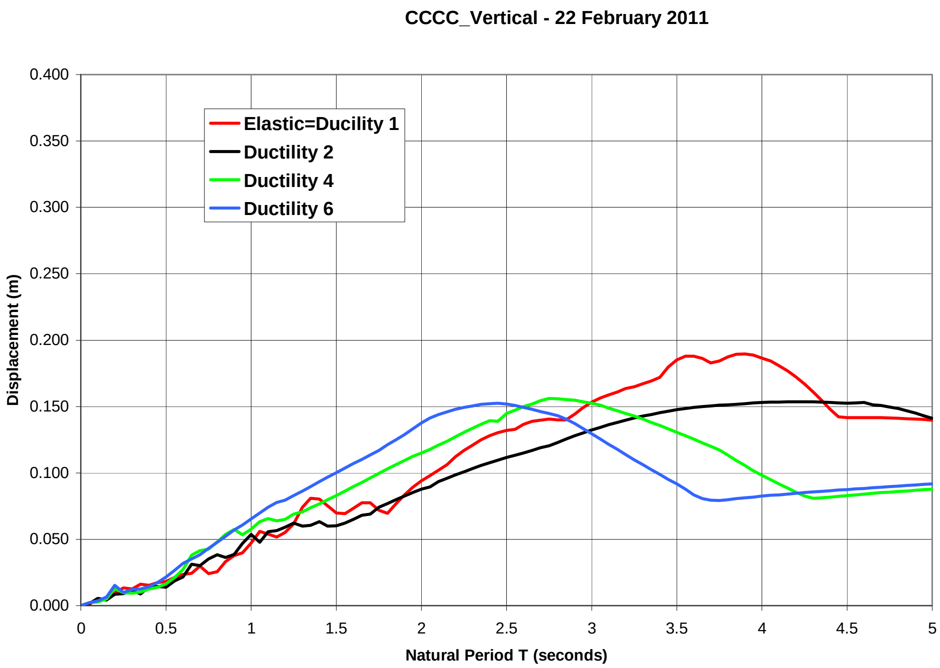


Figure 39. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- Vertical Component

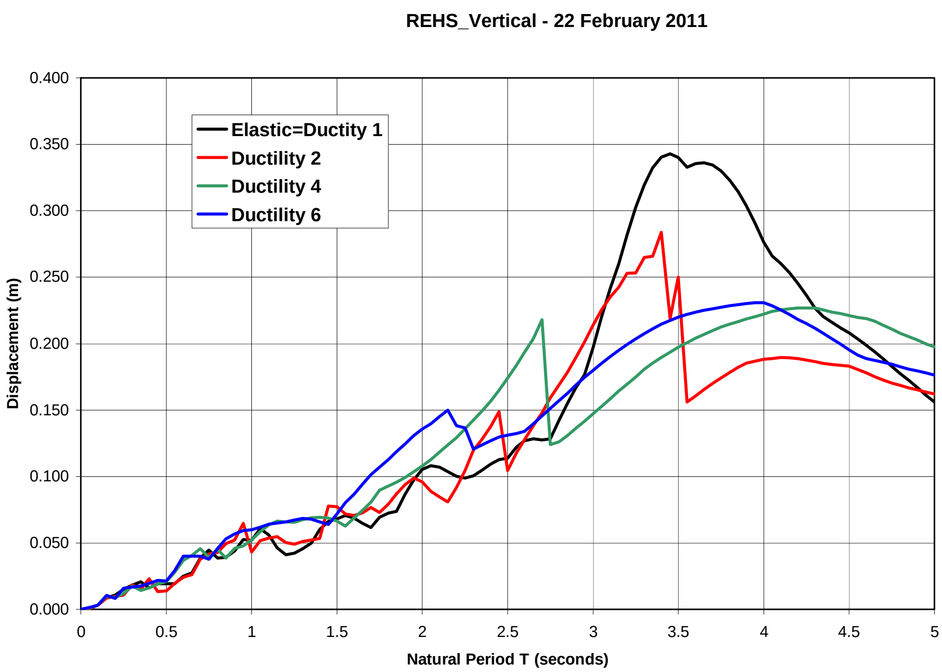


Figure 40. Displacement Response Spectra
Resthaven. 22 February 2011- Vertical Component

These spectra, apart from that at Christchurch Cathedral College spectra, show a marked peak at a natural period of about 3 seconds. This again probably is an effect of the soft soils underlying the city. In general the inelastic displacements are greater than the elastic displacements for natural periods less than 3 seconds.

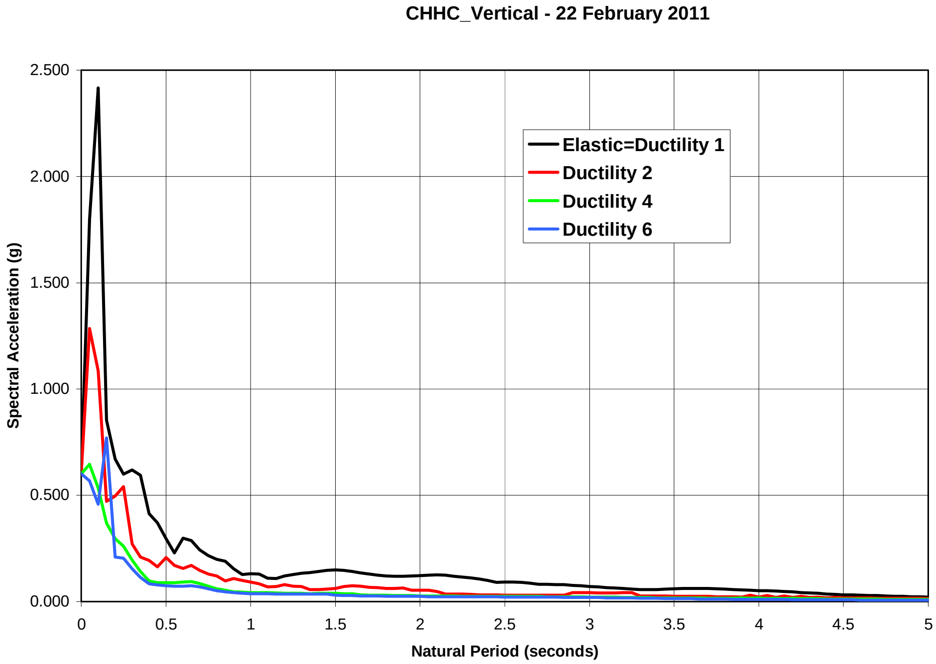


Figure 41. Acceleration Response Spectra Christchurch Hospital 22 February 2011- Vertical Component

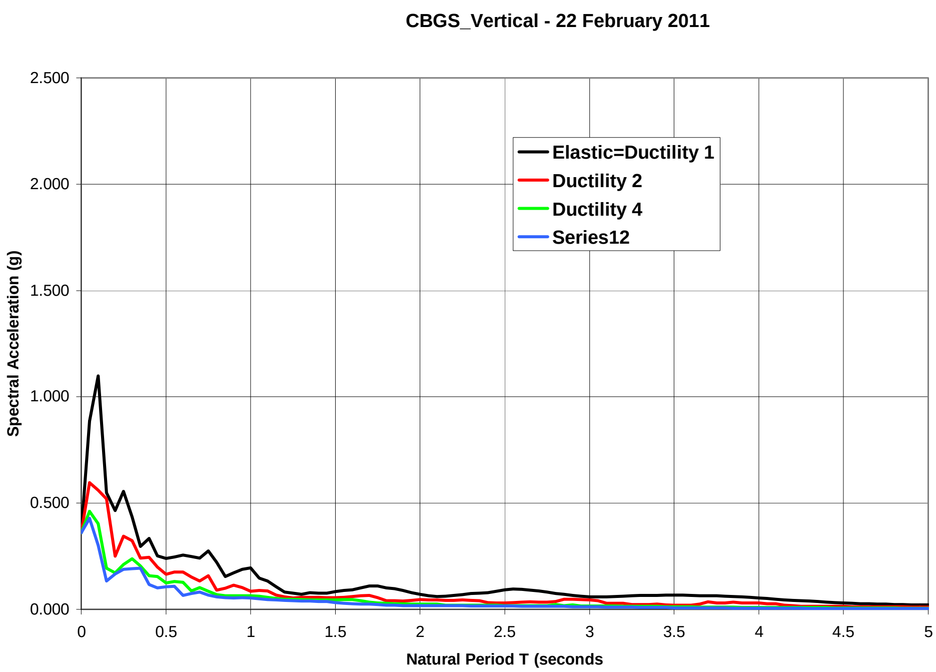


Figure 42. Acceleration Response Spectra Christchurch Botanic Gardens. 22 February 2011- Vertical Component

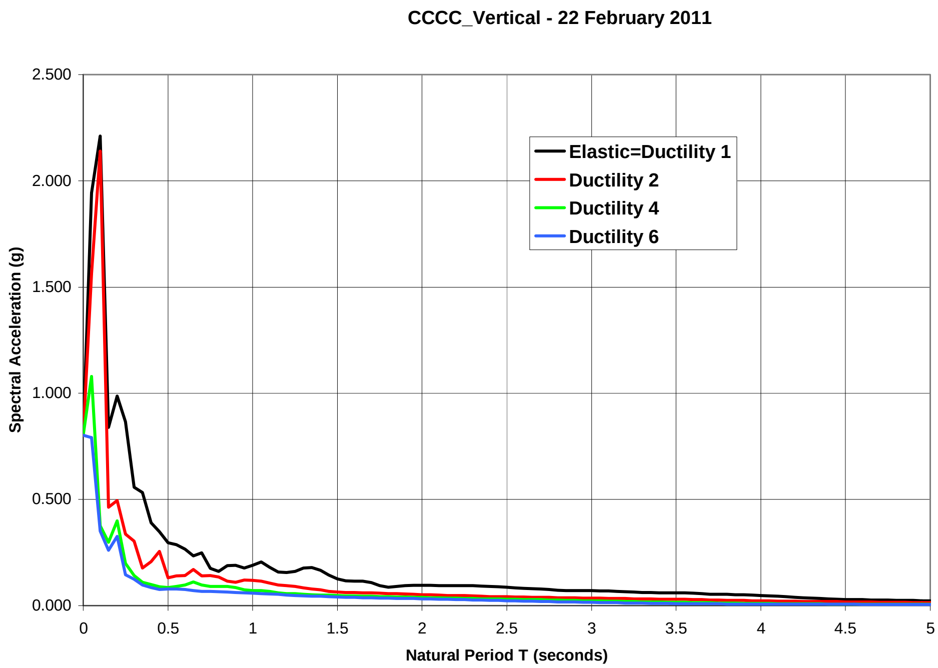


Figure 43. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- Vertical Component

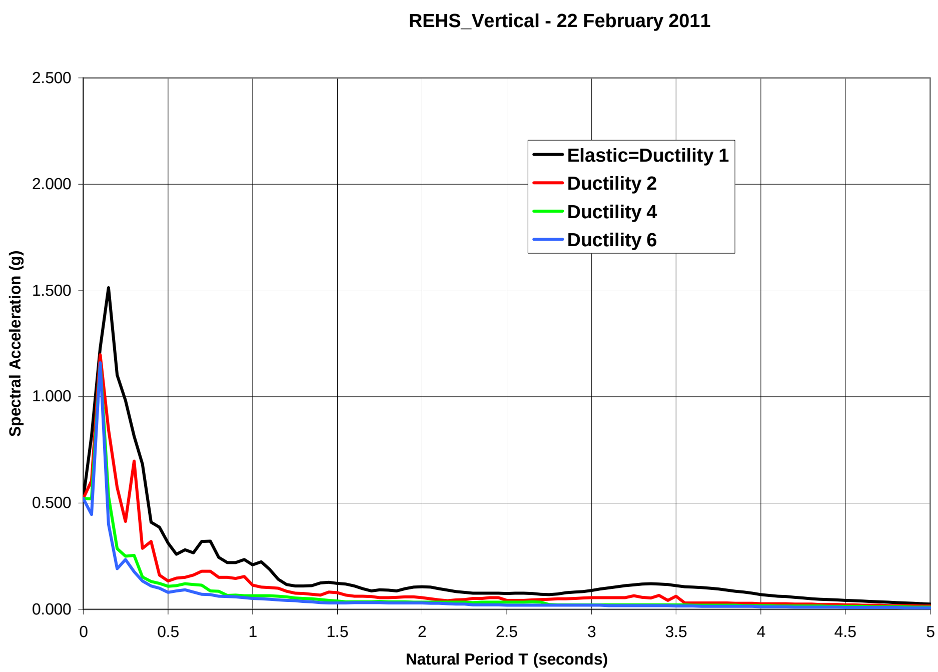


Figure 44. Acceleration Response Spectra
Resthaven. 22 February 2011- Vertical Component

These acceleration spectra show the predominance of the high frequency content of the vertical acceleration records. This is a feature seen in almost all earthquake acceleration records.

11. Relationship between the Observed Response Spectra and the Design Response Spectra (NZS1170.5:2004)

Figures 45 to 56 present the averaged spectra of the records listed with each figure plotted together with the design response spectra given in NZS1170.5.

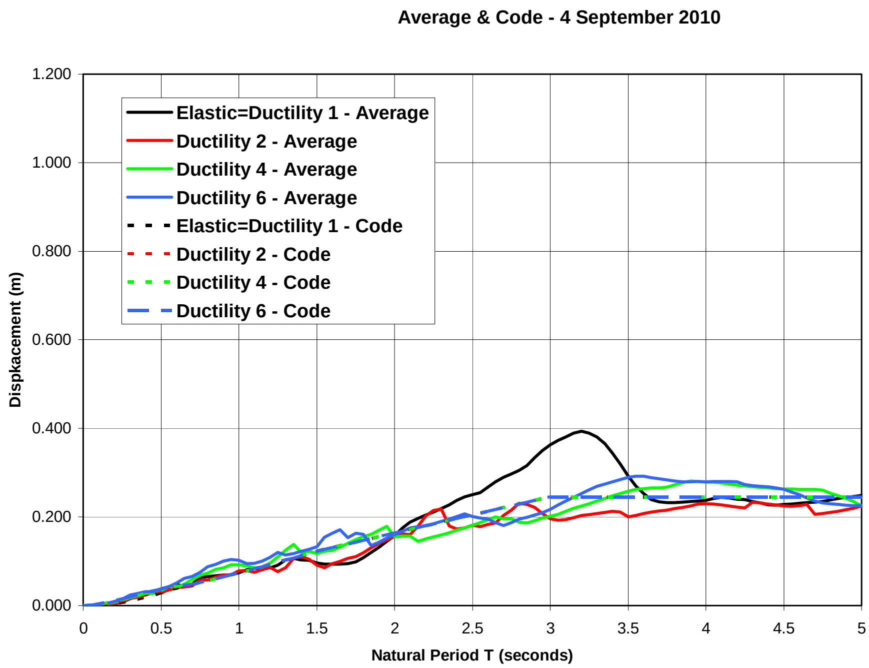


Figure 45. Average Displacement Response Spectra vs NZS1170.5 Christchurch 4 September 2010
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

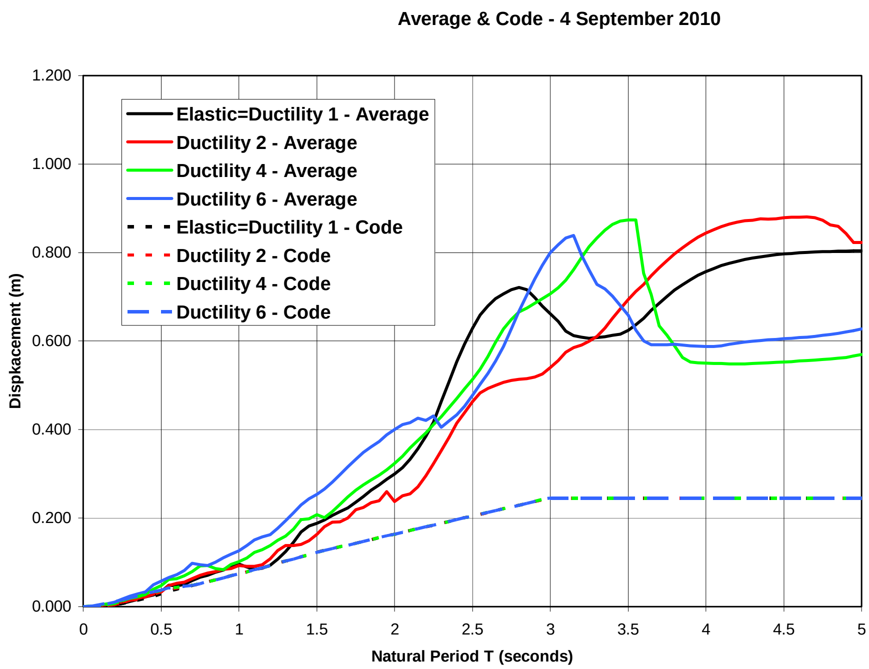


Figure 46. Average Displacement Response Spectra vs NZS1170.5 Christchurch 4 September 2010
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

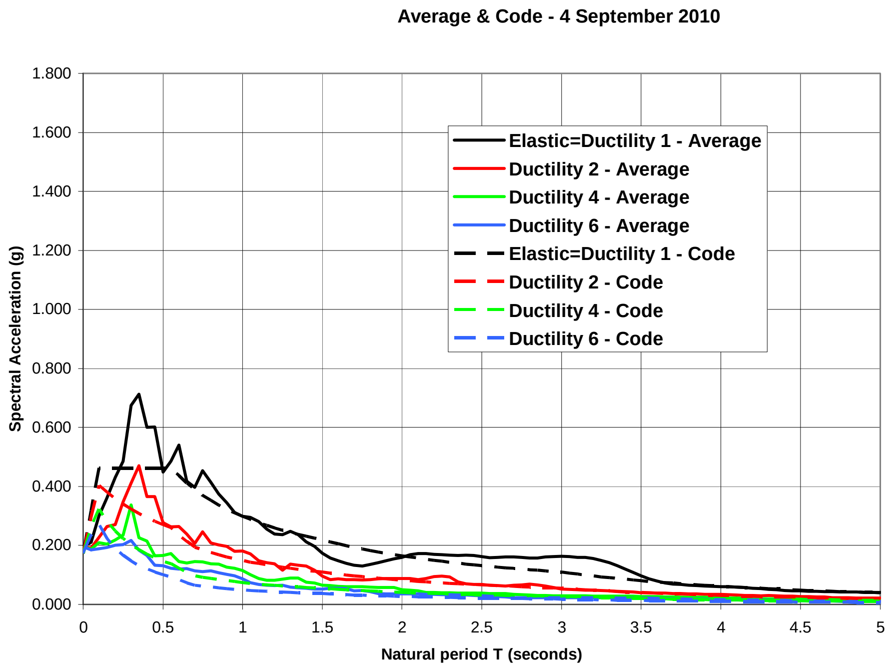


Figure 47. Average Acceleration Response Spectra vs NZS1170.5 Christchurch 4 September 2010
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

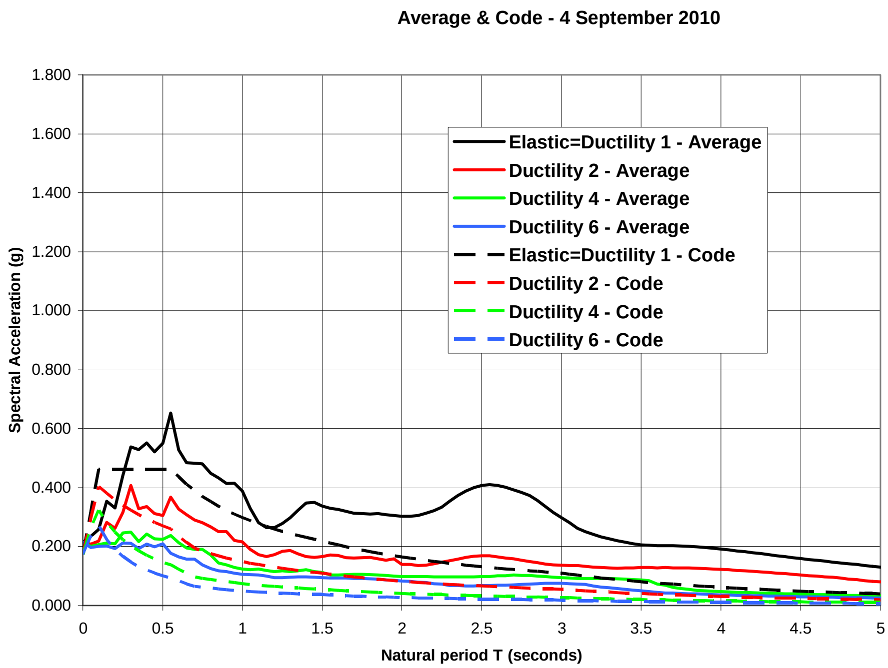


Figure 48. Average Acceleration Response Spectra vs NZS1170.5 Christchurch 4 September 2010
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

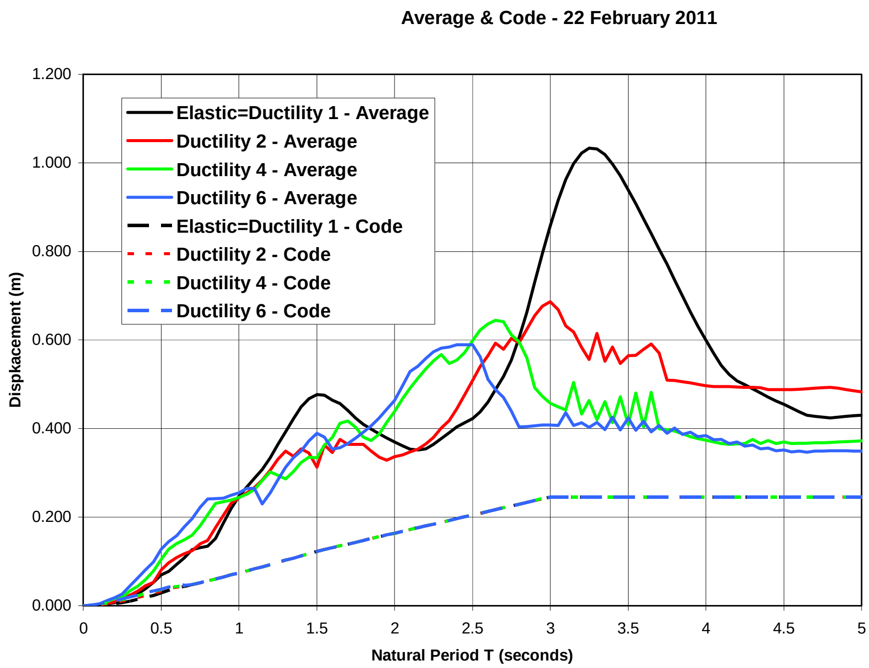


Figure 49. Average Displacement Response Spectra vs NZS1170.5 Christchurch 22 February 2011
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

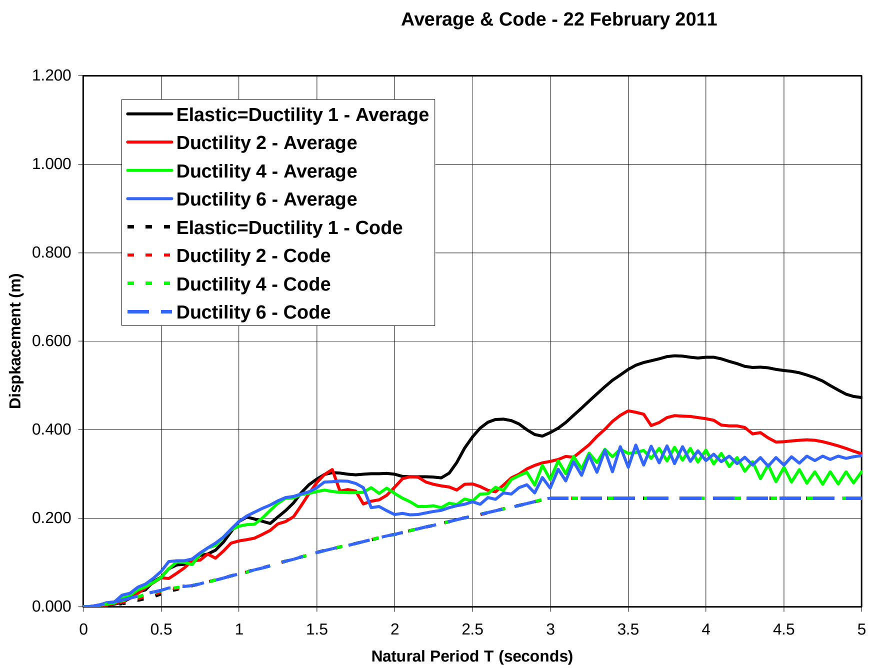


Figure 50. Average Displacement Response Spectra vs NZS1170.5 Christchurch 22 February 2011
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

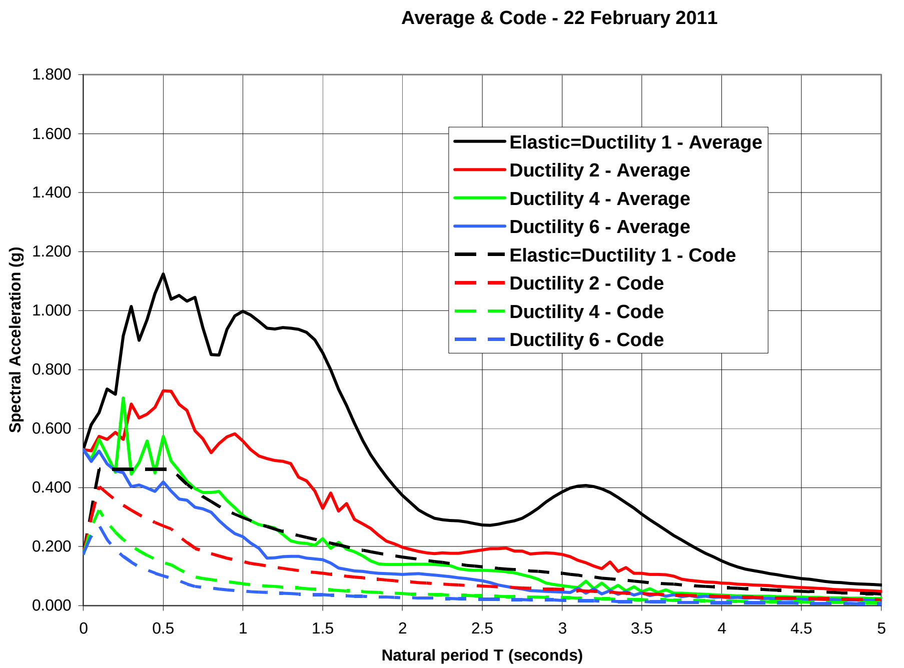


Figure 51. Average Acceleration Response Spectra vs NZS1170.5 Christchurch 22 February 2011
CHHC_S89W, CBGS_N89W, CCCC_N64E, REHS_N88W

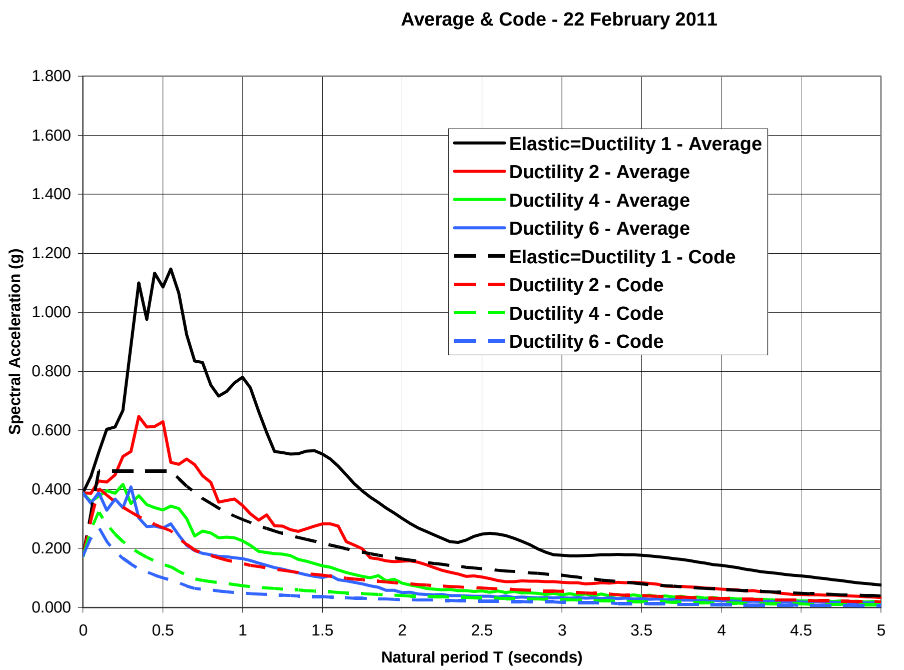


Figure 52. Average Acceleration Response Spectra vs NZS1170.5 Christchurch 22 February 2011
CHHC_N01W, CBGS_S01W, CCCC_N26W, REHS_N02E

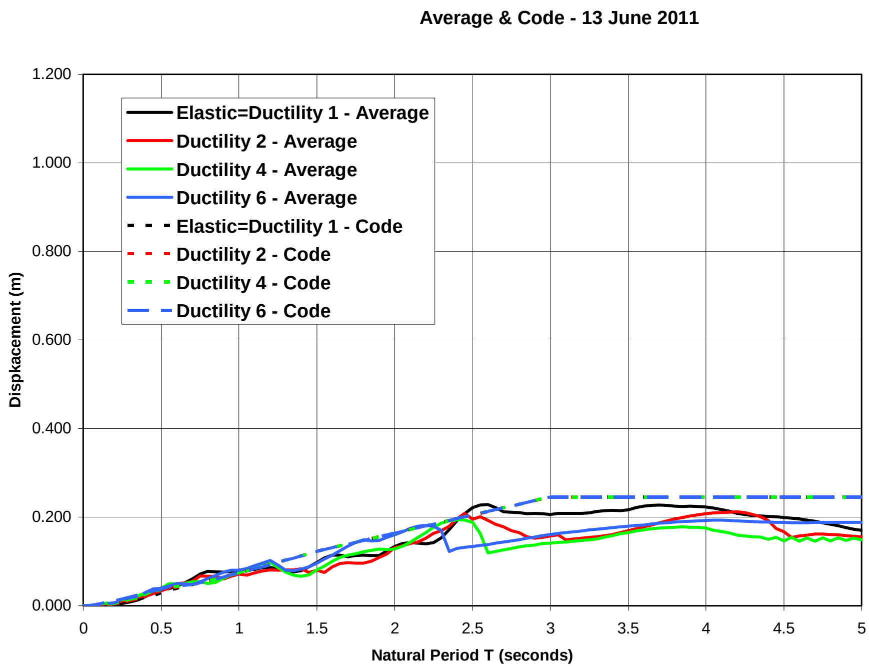


Figure 53. Average Displacement Response Spectra vs NZS1170.5
Christchurch 13 June 2011
CHHC_S89W, CBGS_N89W, REHS_N88W

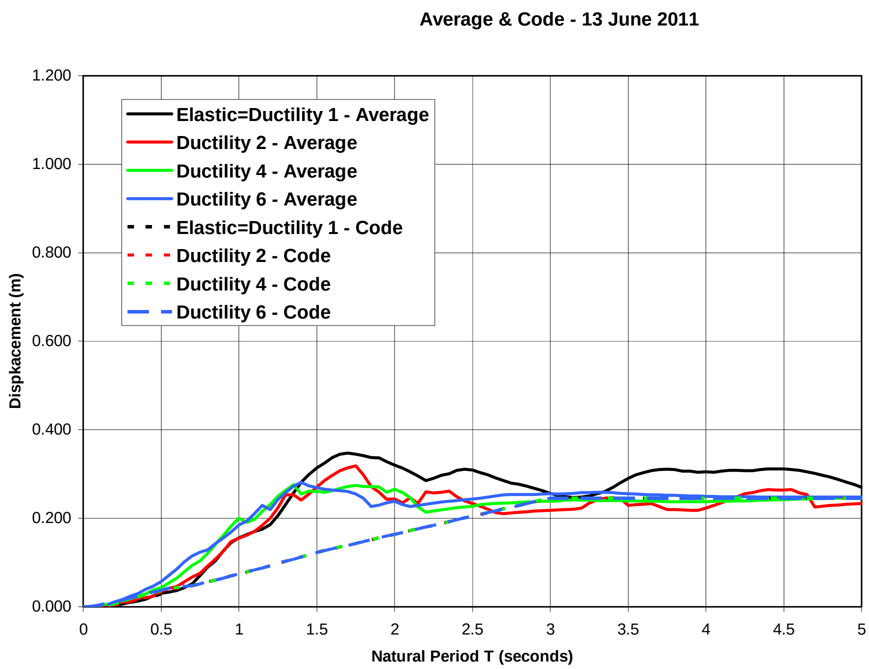


Figure 54 Average Displacement Response Spectra vs NZS1170.5
Christchurch 13 June 2011
CHHC_N01W, CBGS_S01W, REHS_N02E

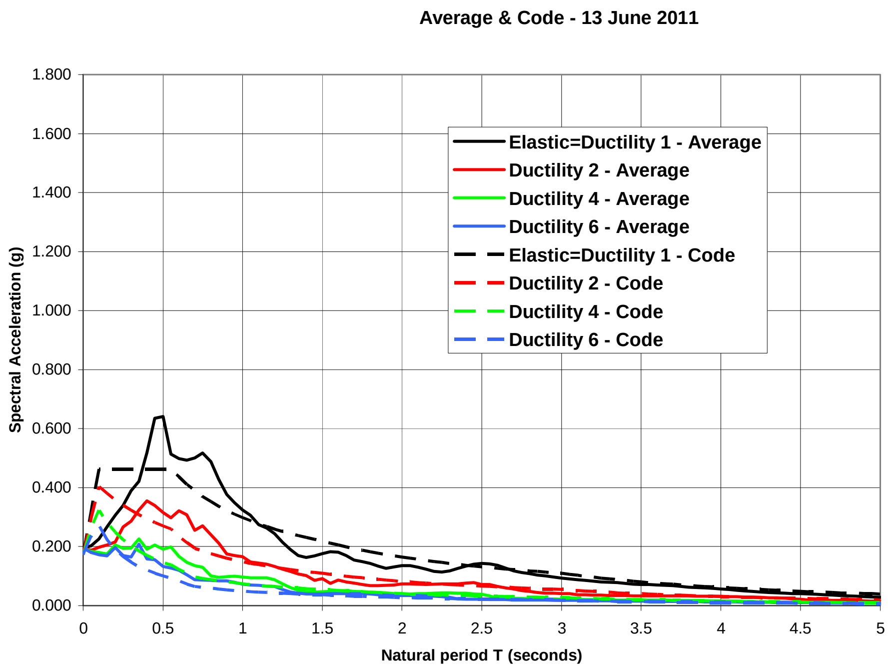


Figure 55. Average Acceleration Response Spectra vs NZS1170.5
Christchurch 13 June 2011
CHHC_S89W, CBGS_N89W, REHS_N88W

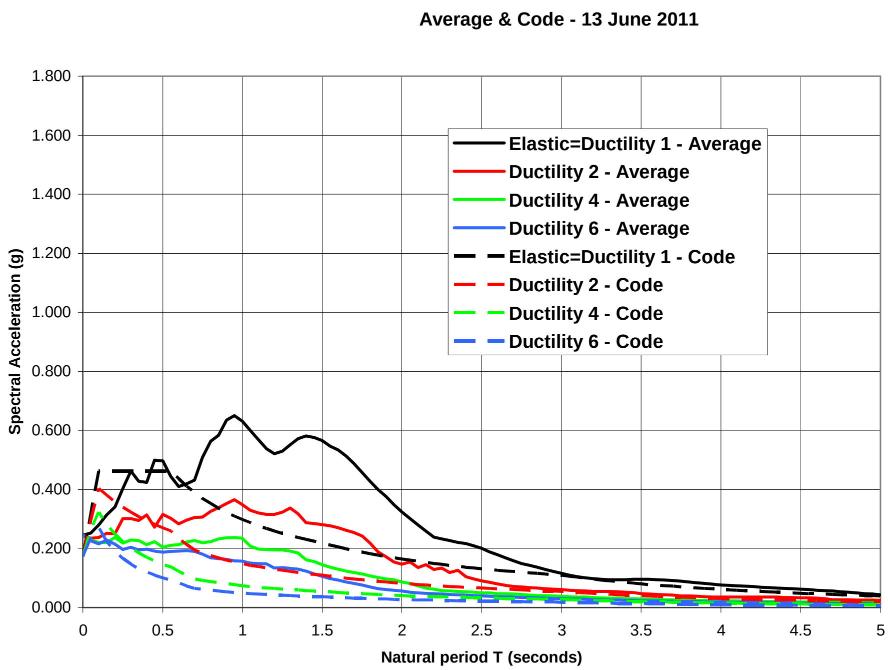


Figure 56. Average Acceleration Response Spectra vs NZS1170.5
Christchurch 13 June 2011
CHHC_N01W, CBGS_S01W, REHS_N02E

From the plots it can be observed that the shaking the Christchurch CBD in the September earthquake and the June earthquake was predominantly in the North-South direction while the shaking in the February earthquake had a predominant shaking in the East-West direction.

In the September earthquake the acceleration spectra were above the design values for nearly all natural periods and of the order of twice the elastic design spectra in the longer 2.5 second period range. The displacement spectra also show very large displacements with the displacements being of the order of two to three times the design displacements for almost all natural periods of free-vibration.

In the February earthquake the accelerations are up to three times the elastic design acceleration which accounts for the very large amount of damage. If one considers that the 2250 years return **R** factor is 1.8 then this represents a very rare event. The only questions is whether the 475 year risk for the Christchurch region was correct or was this between a once in 2500 year and a once in 10,000 year event. The displacement spectra also show very large displacements with the displacements again being of the order of two to three times the design displacements for almost all natural periods of free-vibration. It can also be observed that equal displacement does not fit the displacement spectra very well.

In the June earthquake the acceleration spectra show values up to twice the design values in the 0.7 to 2.0 second period range, and these are not as large as for the February earthquake the epicenter was a little further away. However, the effects on buildings already severely compromised by the February earthquake were very damaging. In the North-South direction the displacements were up to twice the design values in the 0.5 to 2.5 second period range and close to the design values for other natural periods. The deviation from equal displacement does not seem to be so marked in this earthquake spectra but this is masked because the smaller values were plotted using the same scales as for the September and February earthquakes.

In both the September and February earthquakes the inelastic displacements were greater than the elastic displacements to natural periods of the order of 2.5 seconds rather than the 0.7 seconds used by the current design standard. At longer natural periods the variation was more unpredictable with large ductility values in some cases showing greater displacements than those for linearly elastic buildings whilst in other cases showing significantly smaller displacements. If the February earthquake is considered a near-fault event for Christchurch then this may account for some of the very strong variations. Another effect that may lead to poor comparisons with behaviour in other earthquakes could be the effects of the underlying deep soft foundation material and subsequent liquefaction.

Vertical Components in the February earthquake.

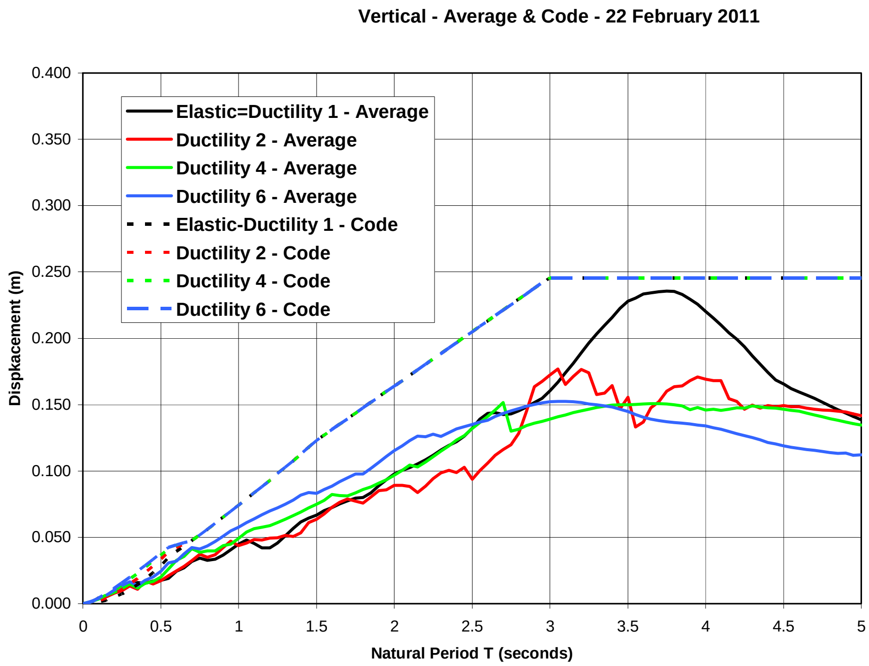


Figure 58. Average Vertical Displacement Response Spectra vs NZS1170.5 Christchurch 22 February 2011

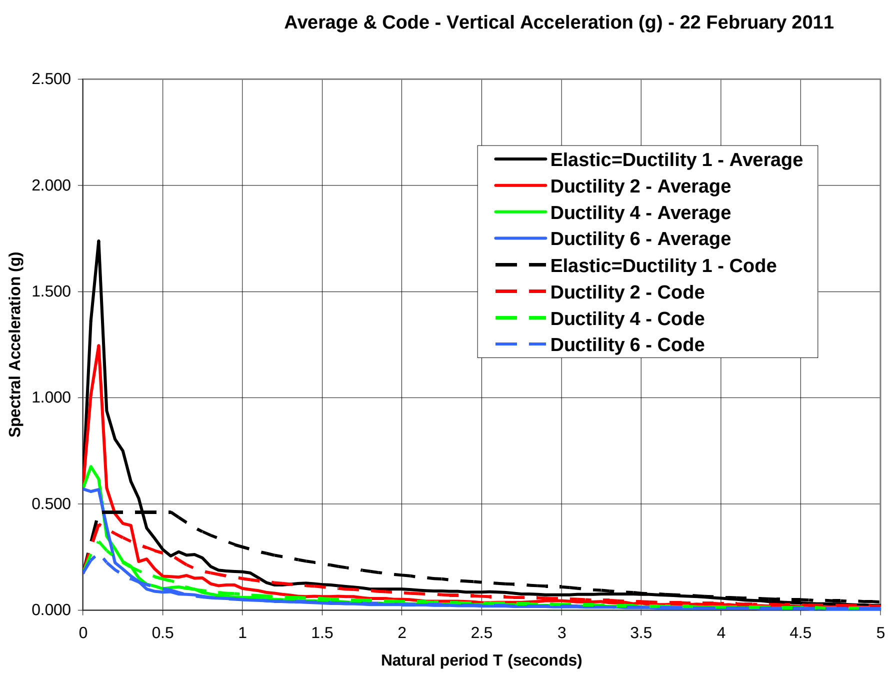


Figure 59. Average Vertical Acceleration Response Spectra vs NZS1170.5 Christchurch 22 February 2011

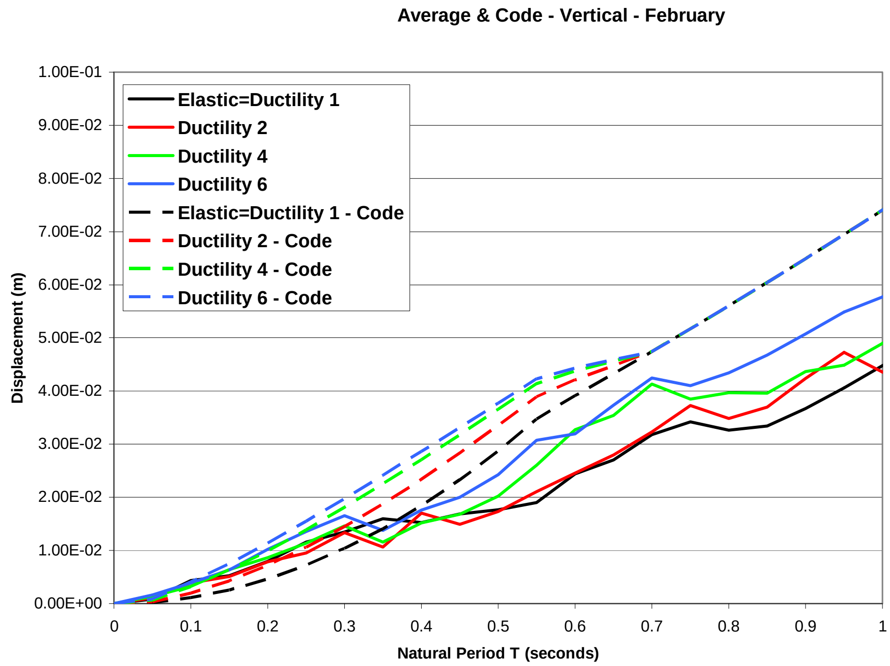


Figure 60. Average Vertical Displacement Response Spectra vs NZS1170.5 Christchurch 22 February 2011 – 0.0 to 1.0 Seconds

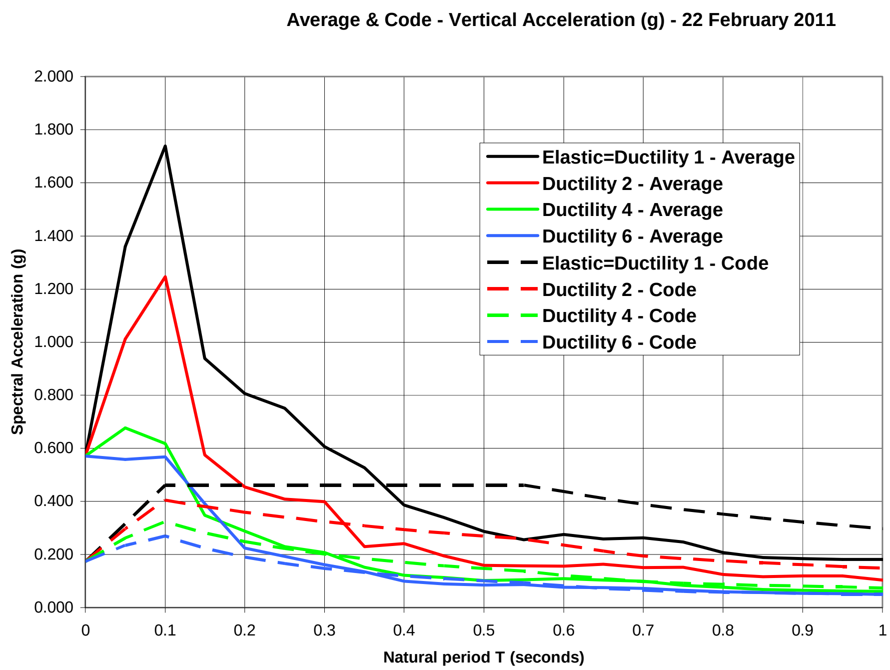


Figure 61. Average Vertical Acceleration Response Spectra vs NZS1170.5 Christchurch 22 February 2011 – 0.0 to 1.0 Seconds

In general the displacements are less than the design values obtained from the design standard NZS1170.5. However, the accelerations observed are much greater than the design values in the 0.0 to 0.4 second natural period range. The natural period of free-vibration for most building in the vertical direction will be very short, when compared with their natural periods of free-vibration in the horizontal directions and these high accelerations may be of major concern, in particular the effects on the flexural strengths of reinforced concrete column and wall members.

The vertical acceleration spectra for the February earthquake do not support the usual notion that they are of the order of two-thirds of the horizontal acceleration spectra. The recorded accelerations near the epicenter of the February were exceptionally large, possible the largest ever recorded. The accelerations in the CBD were also very large and whether this is to change design spectra or not will be resolved when it is decided that the Christchurch earthquakes were similar to other earthquakes or a very rare event to be treated differently. It must be noted that the very high accelerations really only apply for very low natural periods of free-vibration.

12. Variations in Inelastic Spectra due to Inelastic Modelling Assumptions

In all the analyses reported in the above parts of this report, the building is assumed to have an elastic-perfectly plastic hysteresis model such that after the yield point is reach the stiffness of the system is zero. This simple hysteresis model was a very common analysis model in the 1960s and 1970s when there was a belief that the shape of the hysteresis model did not have a great effect on the maximum displacements [Otani,1978,1980,1981]. In the past 30 years many better and more realistic models have been developed to model structural member behaviour, particularly for reinforced concrete structures.
The elasto-plastic model is shown in Figure 62 where the structure has an elastic stiffness of $\mathbf{k_0}$ until the yield action $\mathbf{F_y}$ is reached and then the stiffness becomes zero, i.e. perfectly plastic.

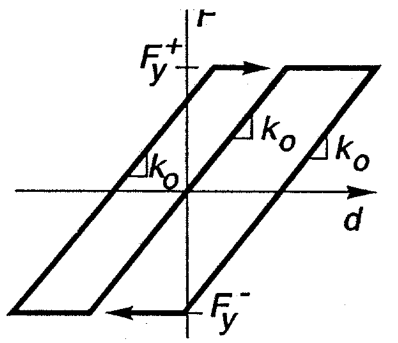


Figure 62. Elasto-plastic hystersis

The most popular model for reinforced concrete systems is the Modified Takeda hysteresis [Carr, 2010] and this is shown in Figure 63. This loop allows for degradation of stiffness with increasing ductility and better represents the behaviour observed in laboratory tests of reinforced concrete systems.

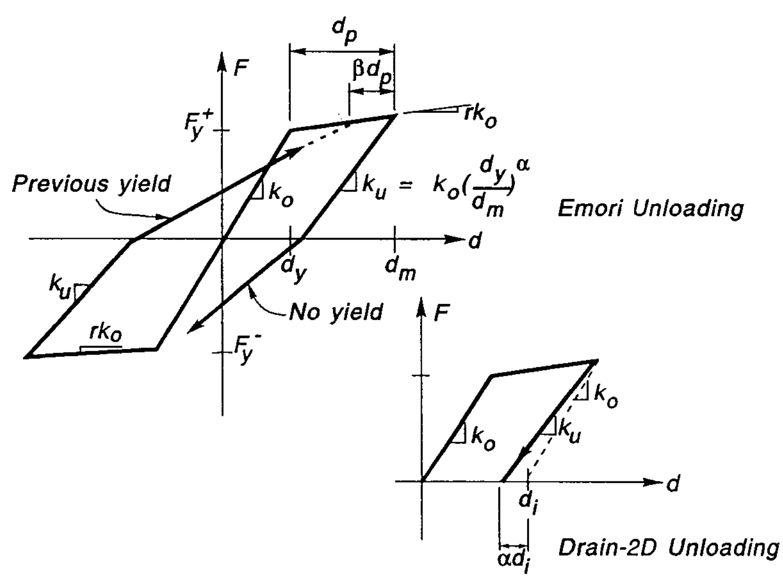


Figure 63. Modified Takeda hysteresis

There are more complicated hysteresis models now available but they are more appropriate in modeling individual members of a building than in representing the behaviour of a whole building. Most of these better models require many more structural parameters which are often difficult to define for building components but are almost impossible to define for a general building model. The hysteresis used in the comparative analysis uses the Emori Unloading model and the unloading parameter **α** is set to 0.4 and the reloading parameter **β** is taken as 0.6 which are reasonable values for reinforced concrete. The post-yield stiffness factor, or strain-hardening factor, **r** is taken as 0.03.

Figure 64 shows the hysteresis loop observed in a building with a natural period of free-vibration of 1.0 seconds with a target ductility of 4.0. Figure 65 shows the same building using the Modified Takeda hysteresis loop**.** They show that for this short duration earthquake that the building only yields a small number of times. Further, Figure 65 shows that the yield force for a building that is not perfectly plastic after yield will be less than the maximum force acting on the building as the force increases post-yield as the building still has some stiffness after yielding.

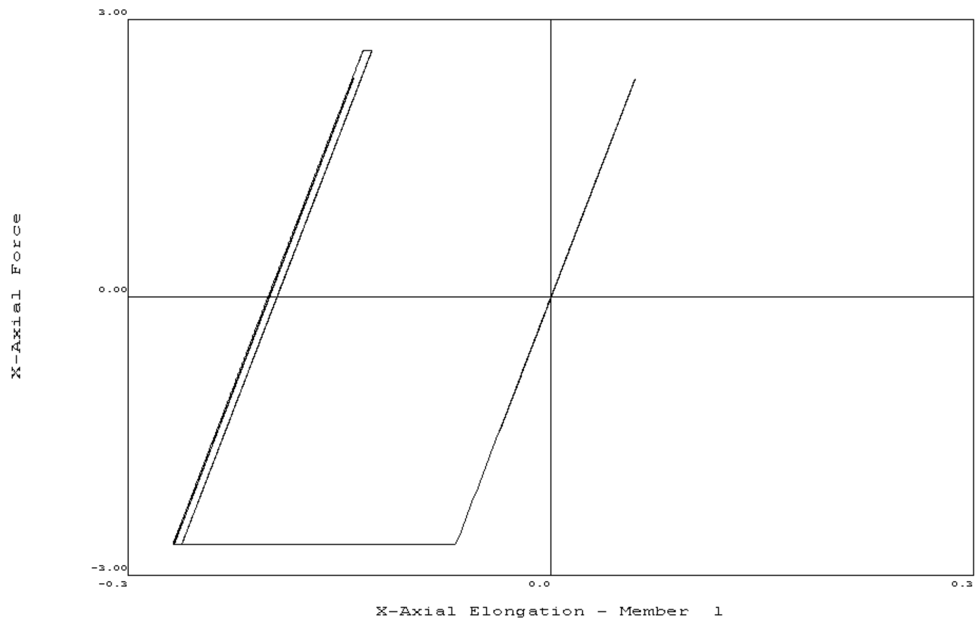


Figure 64. Elasto-plastic - T=1.0, μ=4.0

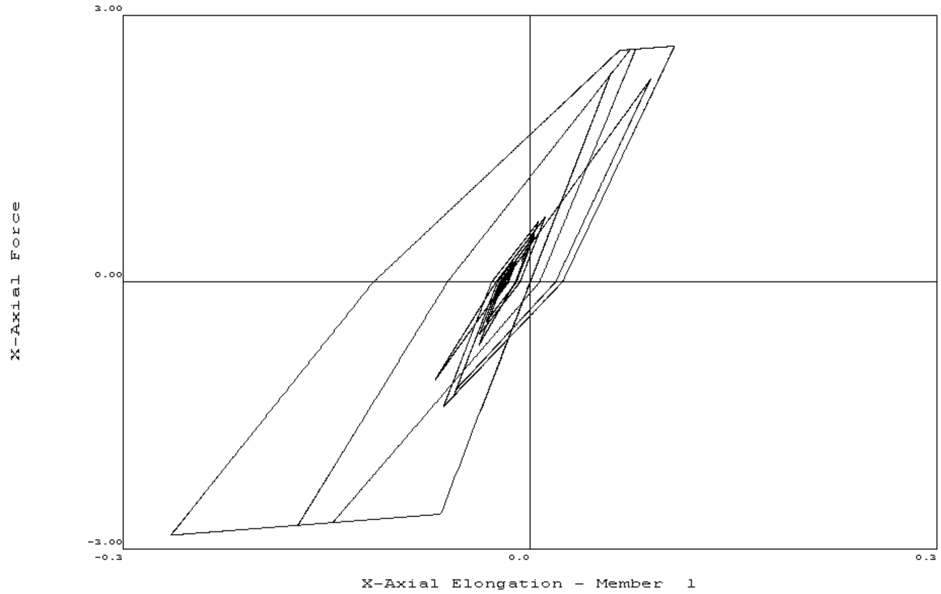


Figure 65. Modified Takeda - T=1.0, μ=4.0

Even where all the beam and column members in a building are assumed to have an elasto-plastic hysteretic behaviour the building itself does not exhibit the same behavior. Not all the members in the building reach the yield point at the same time but instead the yielding progressively occurs in more and more members until the fully plastic mechanism assumed in design is reached. An example is shown in Figure 66 for a 12 storey reinforced concrete frame where an earthquake loading pattern from NZS1170.5 is slowly increased until yield starts to occur in the frame members.

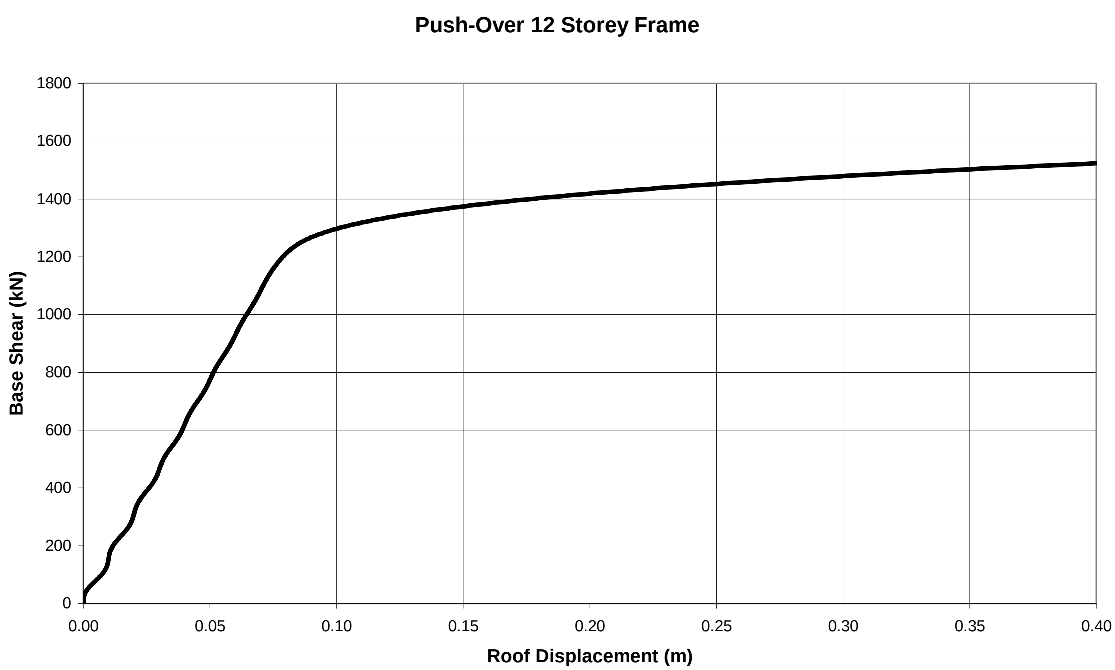


Figure 66. Push-Over Analysis of a 12 Storey Reinforced Concrete Frame.

The resulting spectra obtained using the Modified Takeda hysteresis are similar to those obtained using the elasto-plastic hysteresis with the variation between the spectra for different ductility levels being reduced and the iterations required for the yield force level to obtain the target ductility being fewer.

Only some of the comparisons will be reported here but the results for all the results are shown in Appendix E. The greatest variation is seen in the displacement spectra where it has been seen that the inelastic displacements may be greater or less than the elastic displacements whereas in the acceleration spectra the elastic accelerations are always greater than the inelastic spectra.

Equal displacement does not hold though the differences are not as marked for the Modified Takeda hysteresis model. As was noted for the earlier results the reduction in inelastic acceleration spectra from the elastic acceleration spectra does not have the simple relationship with the ductility factor as is assumed in the equal displacement concept. In many cases the reduction factor is much greater than the ductility factor.

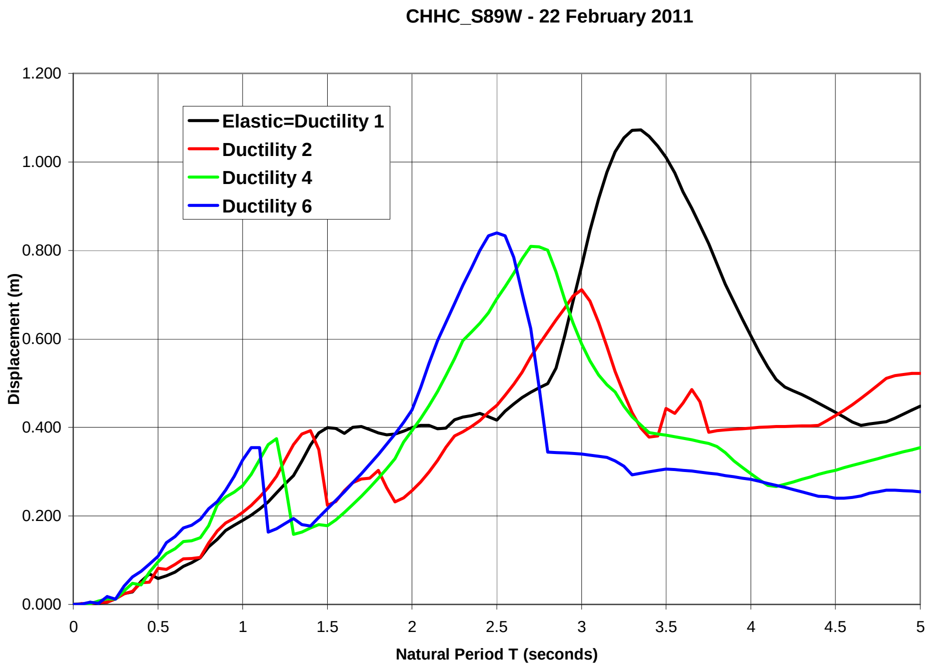


Figure 67. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Elasto-plastic hysteresis.

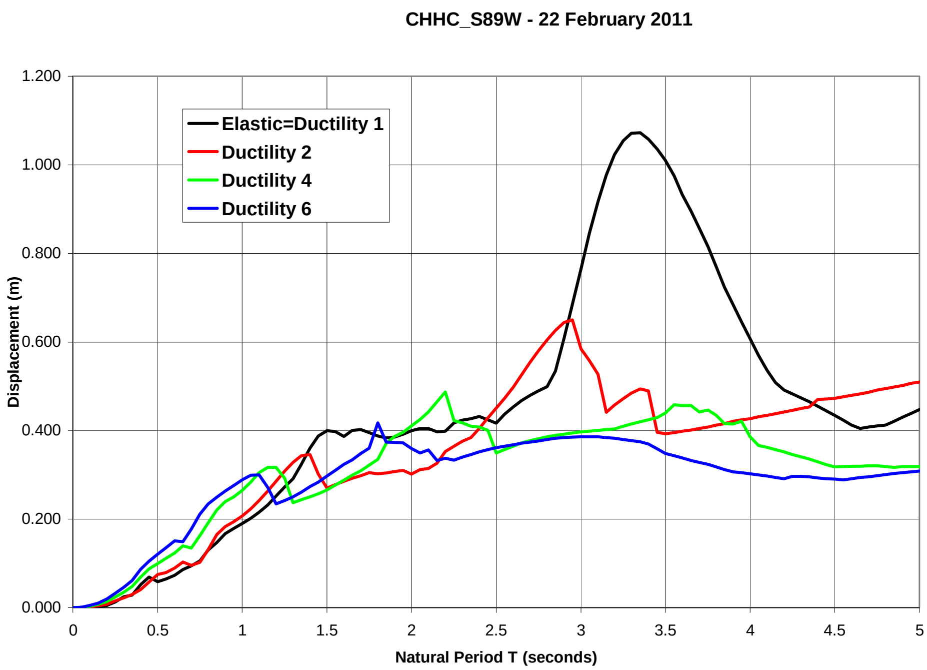


Figure 68. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Modified Takeda hysteresis.

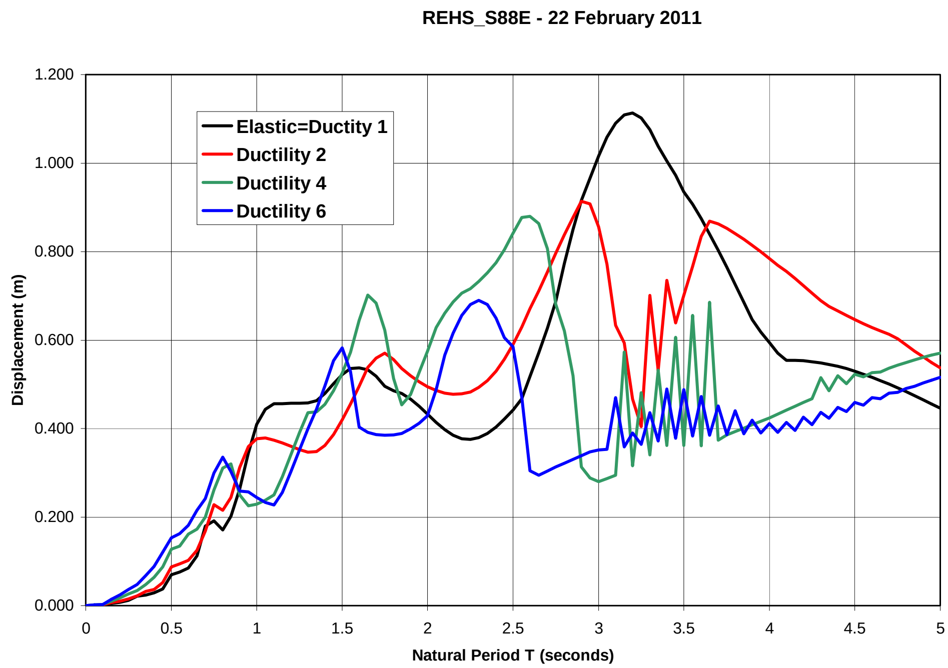


Figure 69. Displacement Response Spectra Resthaven. 22 February 2011- South 88° East Component Elasto-plastic hysteresis.

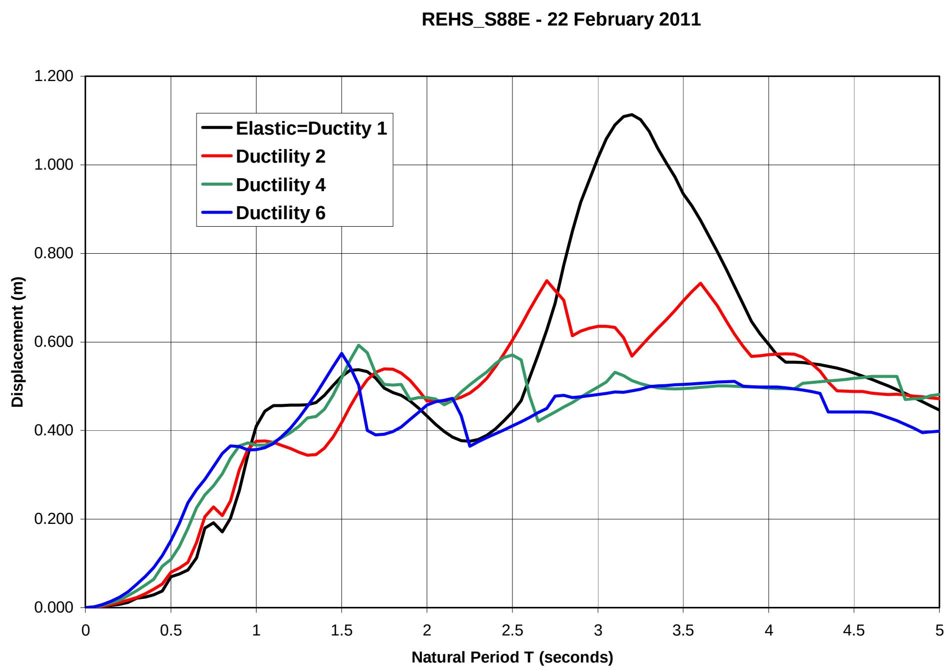


Figure 70. Displacement Response Spectra Resthaven. 22 February 2011- South 88° East Component Modified Takeda hysteresis.

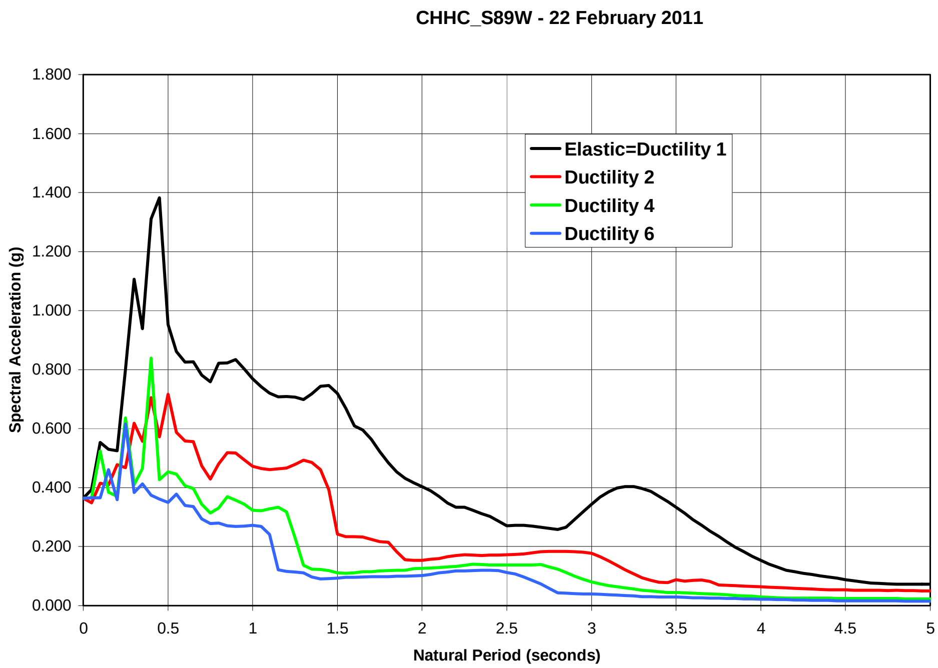


Figure 71. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Elasto-plastic hysteresis

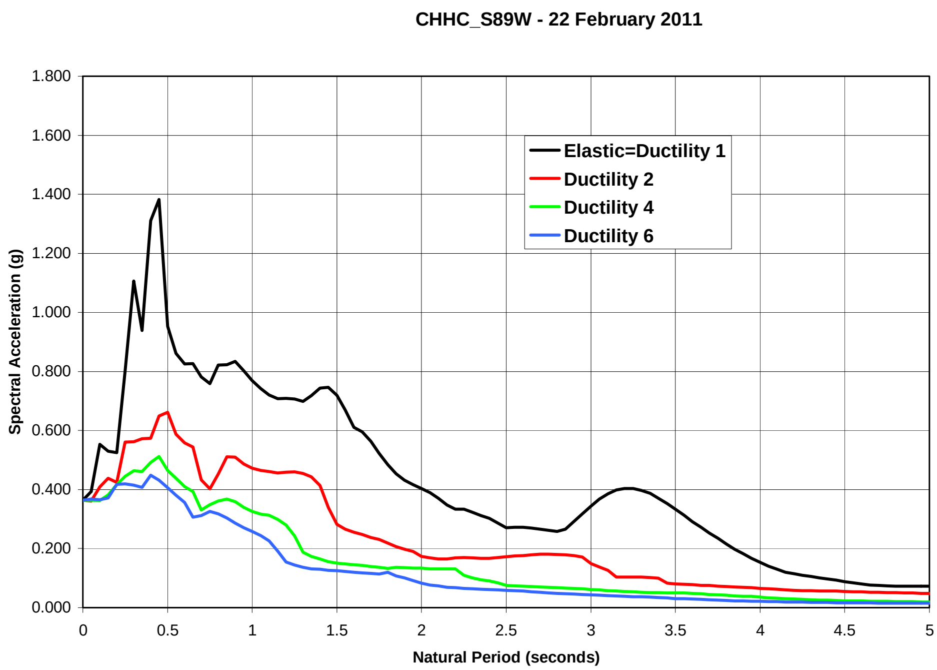


Figure 72. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Modified Takeda hysteresis

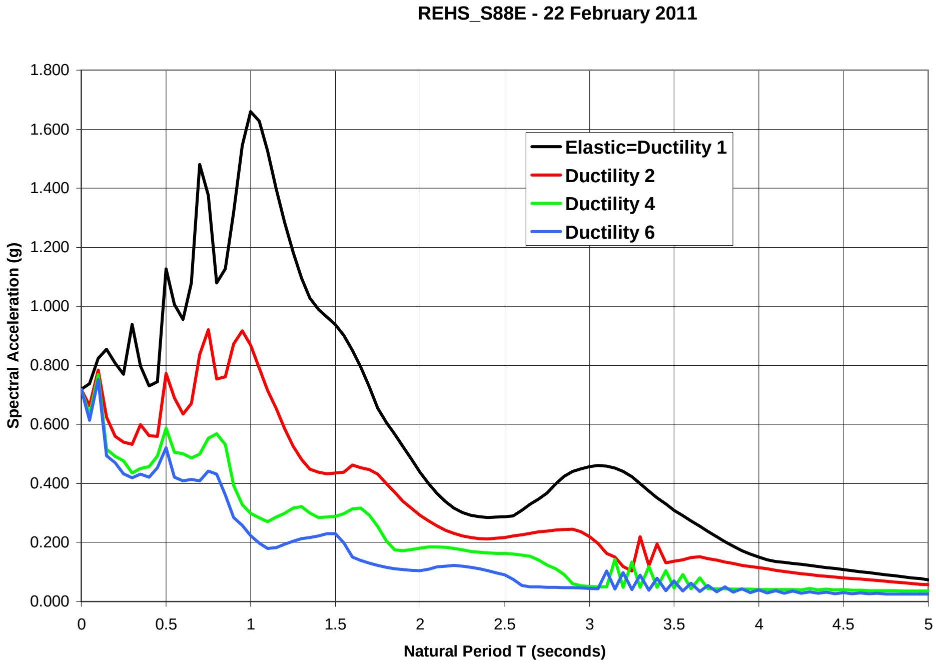


Figure 73. Acceleration Response Spectra
Resthaven. 22 February 2011- South 88° East Component
Elasto-plastic hysteresis

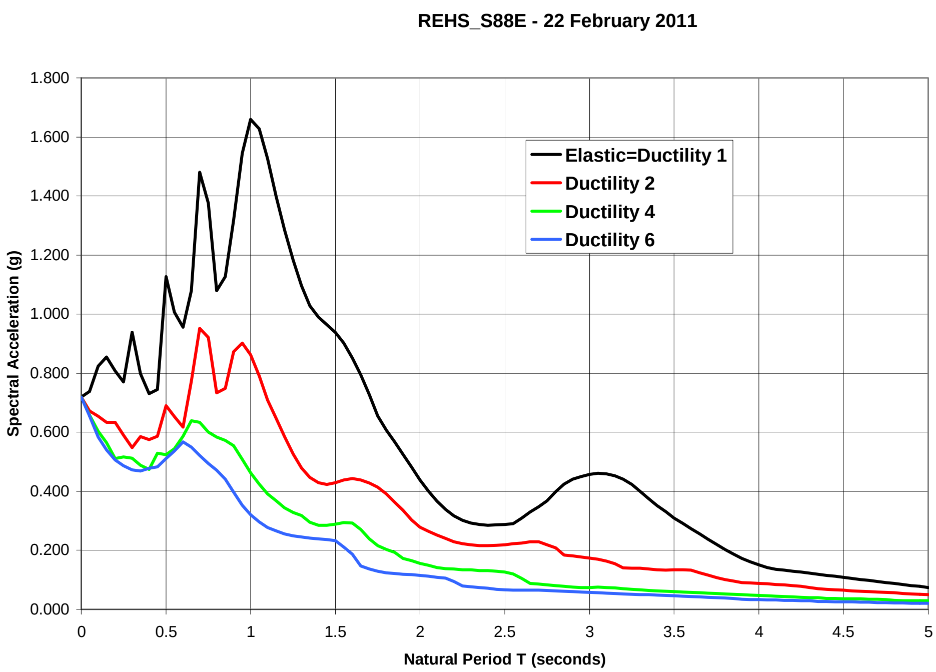


Figure 74. Acceleration Response Spectra
Resthaven. 22 February 2011- South 88° East Component
Modified Takeda hysteresis

13. The Significance of the Christchurch Earthquakes in Relation to Design Values

As can be seen in the above sections the Canterbury earthquakes have imposed much greater excitation upon buildings in Christchurch than they were ever designed to withstand.

The accelerations in all three events were greater than the design values, in some cases by very large margins and in virtually all cases the imposed displacements were very severe.

There are several questions that may be raised with respect to the design earthquakes relative to what was observed in the Canterbury earthquakes and some of these these fall outside the scope of this report. They are:

1. Does the level of design, the 475 year return period, meet the expectations of the citizens of New Zealand?

2. Is the level of damage resulting from the use of high ductility factors in design acceptable to building owners and users?

3. Are the hazard levels assigned to the Christchurch correct after what has happened in the Canterbury region? The fact that there were no known faults in the area may have led to a reduction of the expected hazard.

4. In particular, near-field effects were not considered for design in Christchurch as there were no known active faults near the CBD. This may not be of great relevance for the majority of buildings in Christchurch as the near fault effects were not part of any seismic design code before the mid 1990s. However, it may be an issue for new design or assessing existing buildings in the Christchurch area. For other seismic design in New Zealand how many cities and towns have faults near by that are not known at present?

5. The use of the equal displacement concept in the design standards needs to be given greater investigation. The technique, in general, has given smaller reduction factors than shown with these inelastic spectra for the design for most period ranges. This will be somewhat conservative in a force-based design. However, the computed displacements are not the elastic displacements multiplied by the ductility factor as is current practice. This may lead to an underestimation of the inter-storey drifts, thus an under-estimate of the structural damage, damage to both building fit-out and building claddings. This may also under-estimate the P-Delta effects in tall buildings which could lead to collapse as the columns tilt further than anticipated. Beam elongations, largely ignored in current design, may be larger and may compromise pre-cast floor support. There are probably some concerns about displacement-based design as the spectral displacements are the starting point and if the current inelastic spectra are inappropriate then further research is warranted.
6. There is a question as to whether these records are affected by the presence of the buildings in which the instruments are located. These buildings have their own

natural periods of free-vibration and this may affect the recorded motion of the ground near the building. The presence of a building is known to change the ground motion in its immediate vicinity. Free-field instruments are more useful in the sense that they are away from the local effects of buildings but getting power and internet connections to such free-field sites is more difficult and expensive to arrange. One or more recording sites in the centre of the CBD would also have been useful.

14. Factors that Influence the Lateral Displacement of Buildings due to Seismic Actions

The following factors influence the lateral displacement of buildings due to seismic actions:

1. Magnitude of the earthquake excitation. This is determined by estimating the seismic risk and the importance of the building in determining the return period used for design. These are set out in the design standard but also should reflect the expectations of society in terms of the acceptable damage following an event which is hoped to be of low chance of occurring.

2. The natural period of free-vibration. As can be seen in the design displacement spectra and from those computed from the Canterbury earthquakes, the longer the natural period of free-vibration (larger **T**) the greater the displacement.

3. The lateral stiffness of the building. Structural walls are stiffer than framed buildings and some countries, Chile for example, have much more severe limitations on allowable deformation than New Zealand and as a result structural walls in buildings are more common. However, the result of using stiffer buildings will be larger accelerations (seen from the acceleration design spectra, where, if the building is stiffer when the natural period of free-vibration is smaller and the accelerations are higher). Walls, though stiffer, require stiff foundations if the wall is not to rock, and in Christchurch such foundations may be difficult and expensive to achieve. However, rocking does provide a degree of base-isolation and would still reduce the damage to members, fit-out and claddings (provided that the building does not end up leaning following the earthquake).

4. The degree of inelastic behaviour does have an effect on the lateral displacements of the building, usually increasing the inter-storey drift where large inelasticity occurs but, as shown for larger ductility in these spectra, may reduce the overall displacements of the building.

5. Foundation flexibility will have a major effect on the building displacements. This is particularly the case where foundation rocking can take place, and in Christchurch would be very difficult to prevent unless the structure is on piles driven to firm support. Traditionally this has not been properly considered by most structural designers but good computational models are now becoming available and one hopes that designers have learnt from the past year in Canterbury.

6. P-Delta effects. Most analysis assumes that the building displacements are small when compared to the geometry of the building. When the building displaces in a horizontal direction the columns are no longer exactly vertical but have an inclination. This means that the vertical loads on the columns are now applied to an off-set column top and this off-set introduces a further overturning effect on the building. These effects are only of importance if the displacements are large (NZS1170.5 allows a drift of up to approximately 1.5 degrees) and the applied axial loads on the columns are large, i.e. generally for buildings taller than 4 to 6

stories. The loadings standard NZS1170.5 clause 6.5.4, has a simplified method of including the effects in the design calculations.

15. Force Reduction Factors

If the equal displacement concept applied then the strengths required for the inelastic design would be scaled down from the elastic strengths by the ductility factor, i.e. the design force would be the elastic spectral acceleration divided by the ductility factor **μ.** In the discussion of the results for the Canterbury earthquakes it was stated that the force reduction factors were often much greater than the ductility factor. To illustrate the discrepancy the force reduction factors found for the February earthquake at Christchurch Hospital are shown. As the elasto-plastic results are more sensitive the results for the more realistic Modified Takeda hystersis are also shown.

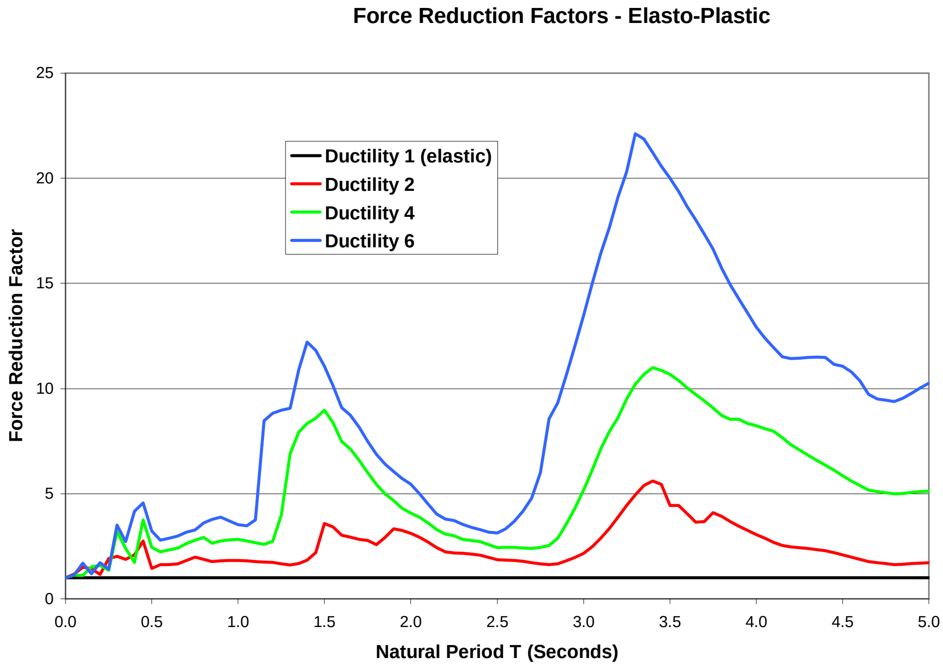


Figure75. Force Reduction Factors – Elasto-Plastic Hysteresis. Christchurch Hospital – February 22 – South 89º East Component.

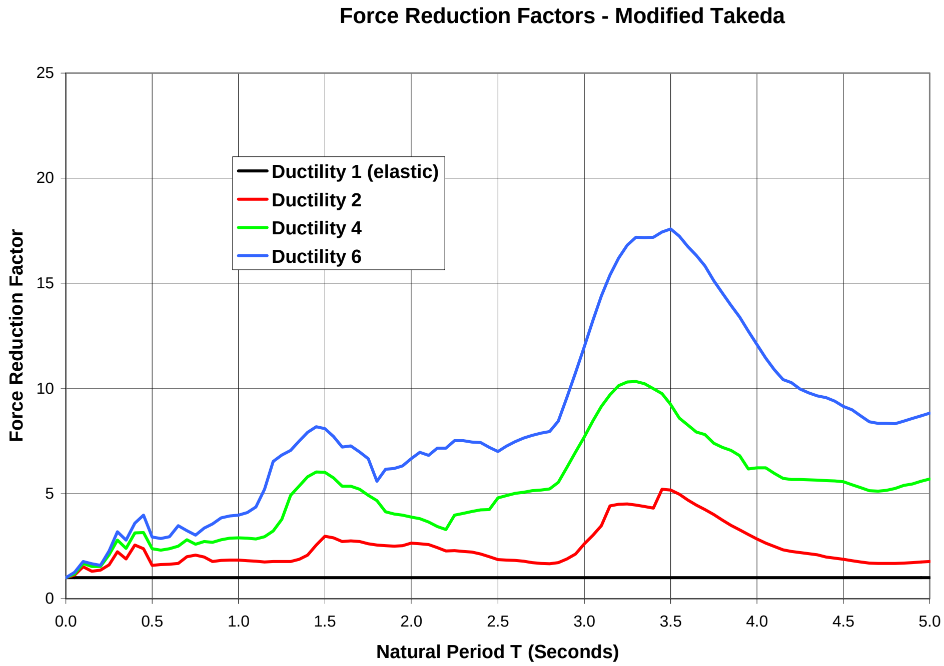


Figure76. Force Reduction Factors – Modified Takeda Hysteresis. Christchurch Hospital – February 22 – South 89º East Component.

It is evident that in most cases the Force Reduction Factor is considerably greater than the ductility factor. The further illustrates the differences seen with what would be expected from the equal displacement concept.

16. Conclusions

The results presented in this report do not support the use of the equal-displacement concept that has been in use in seismic design codes since about 1960. Given the uncertainty in any seismic design in that the designer has no idea what the next earthquake will bring, then the large variations in the displacement spectra are acceptable. The design profession does not seem to take this level of uncertainty in the design displacements into consideration when comparing their design inter-story drifts with the allowable limits in the standards. The differences with the equal displacement concept have been observed from the early 1970s, but these differences were often regarded as a peculiarity of a particular ground motion or a particular hysteresis model that was being used in the analysis where the difference were marked.

The marked differences shown here may be an effect of the Christchurch ground motions, and maybe they are peculiar or atypical of earthquake ground motions in general. They are short duration ground motions, recorded on top of soft soils and the ground behaviour and building responses may have been affected by the unusual very high vertical accelerations seen in the February event. There should be a study of other recent earthquake records, both those recorded in New Zealand and overseas to see if the continued use of the equal displacement concept is acceptable. If the equal displacement concept is to be retained by the design standards then the standards should reflect the imprecision associated with the concept.

It is suggested that some free-field instruments should be established to give records that do not have the risk of being modified by the presence of the building in which they are installed. It is also suggested that with the large number of buildings in the CBD that some recordings from the centre of the CBD should be part of future planned instrumentation. There also seems to be a complete lack of well instrumented buildings where there are instruments located over the height and plan of the building in Christchurch. A large number of buildings erected in other seismically active parts of the world are required to have such instrumentation systems.

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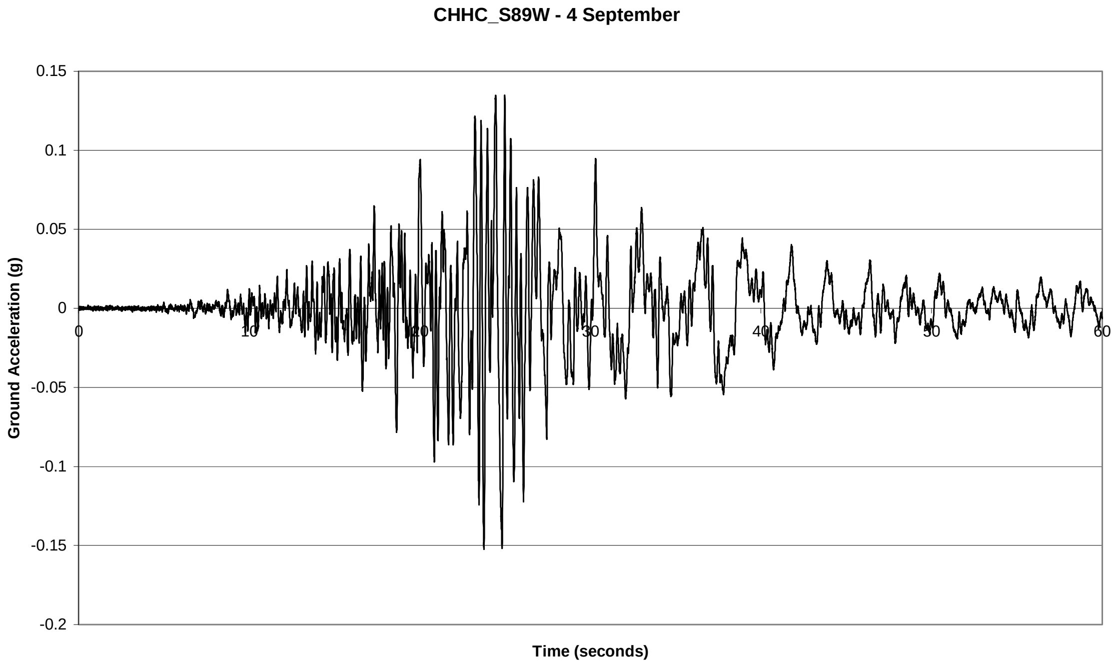
Appendix A
Ground Acceleration Records (accelerograms)

4 September 2010 Acceleration records.



.**Figure A-1. Ground Acceleration History.**
Christchurch Hospital 4 September 2010- South 89° West Component

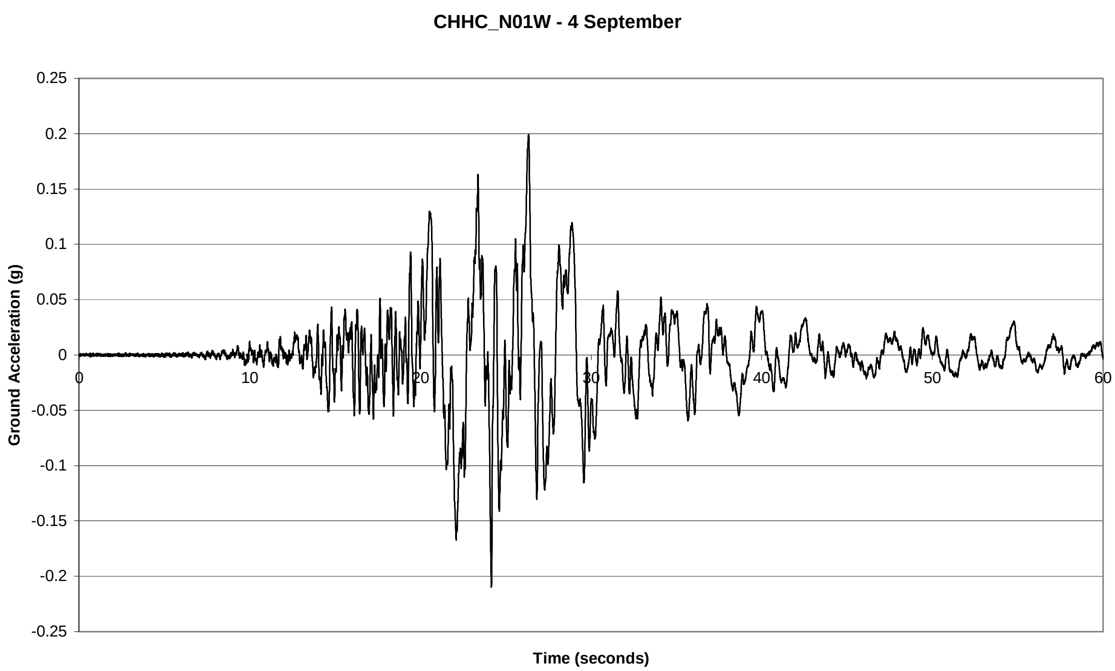


Figure A-2. Ground Acceleration History.
Christchurch Hospital 4 September 2010- North 01° West Component

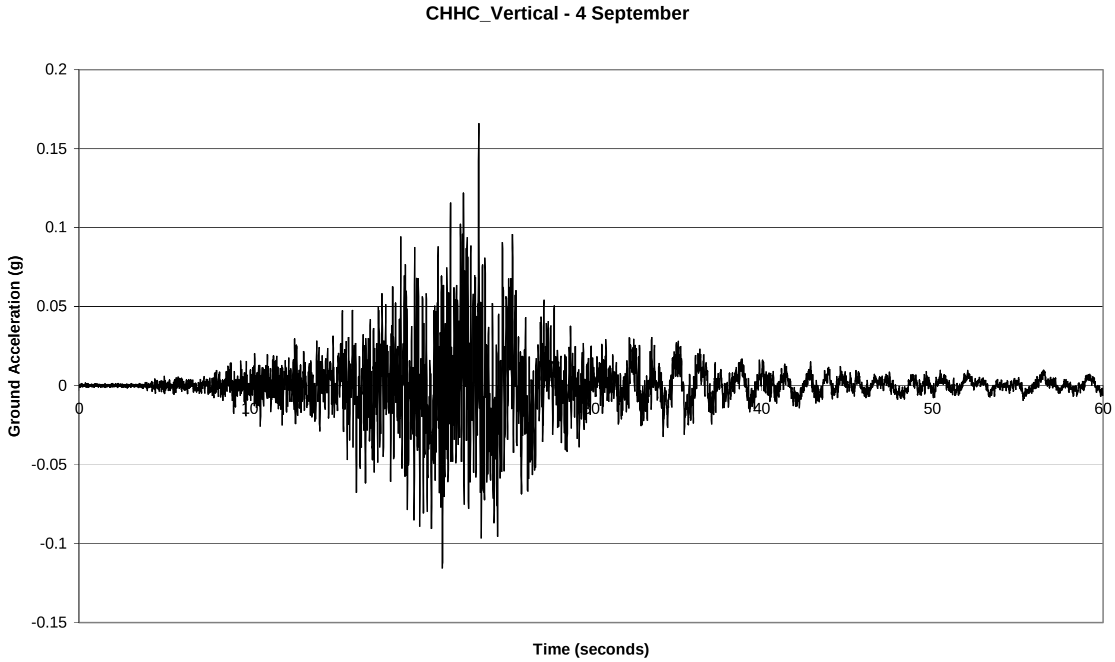


Figure A-3. Ground Acceleration History.
Christchurch Hospital 4 September 2010- Vertical Component

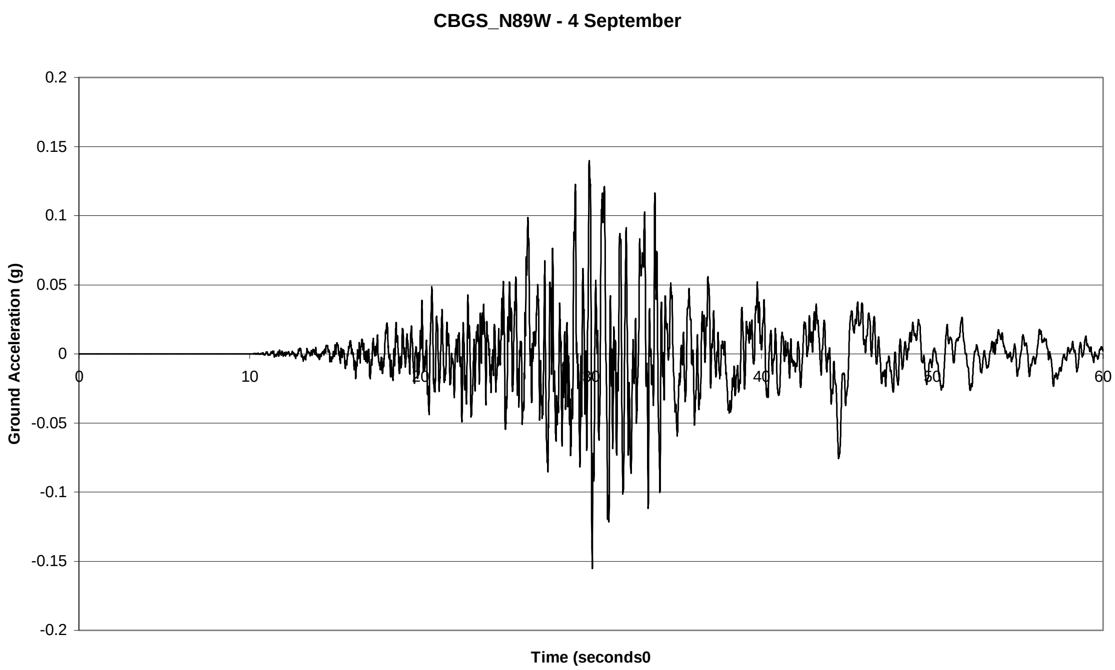


Figure A-4. Ground Acceleration History.
Christchurch Botanic Gardens. 4 September 2010- North 89° West Component

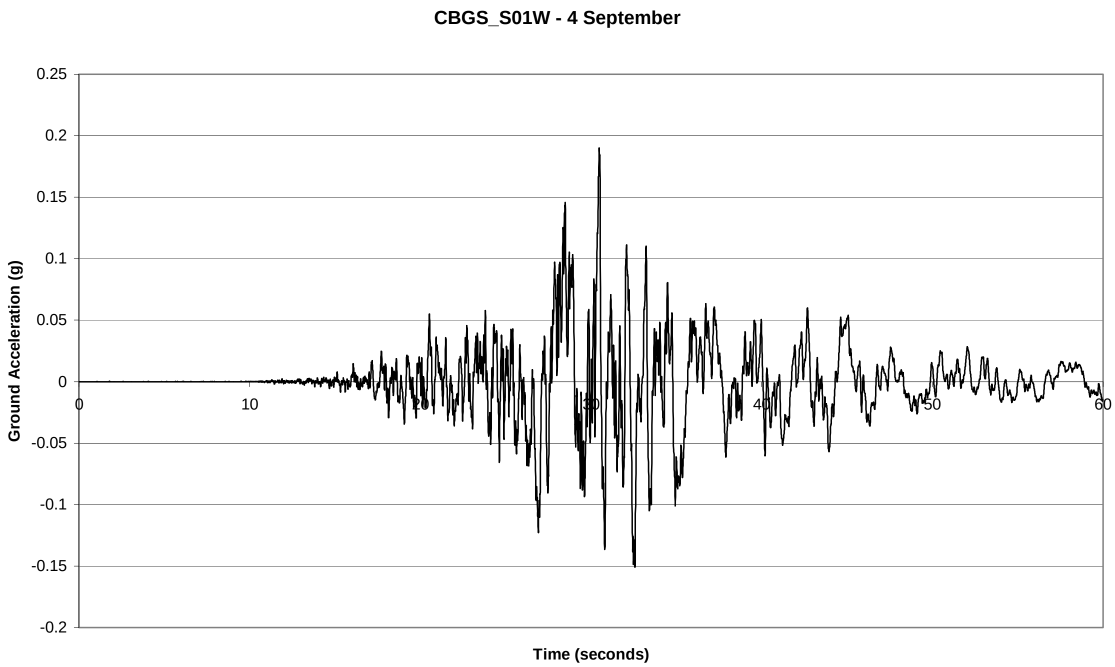


Figure A-5. Ground Acceleration History.
Christchurch Botanic Gardens. 4 September 2010- South 01° West Component

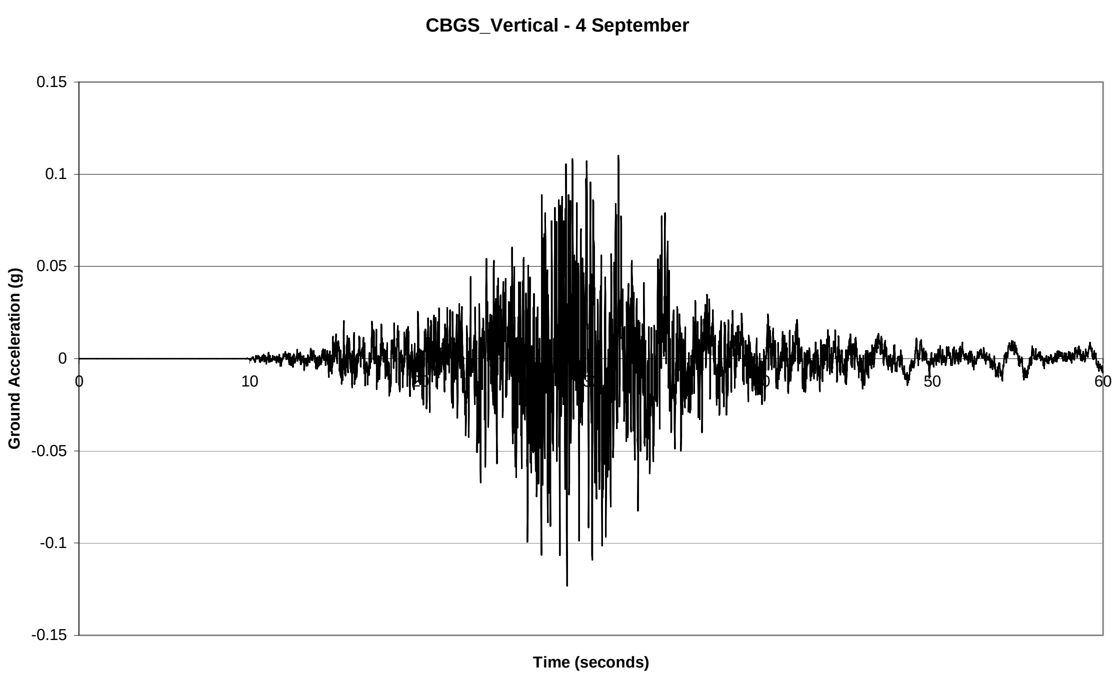


Figure A-6. Ground Acceleration History.
Christchurch Botanic Gardens. 4 September 2010- North 89° West Component

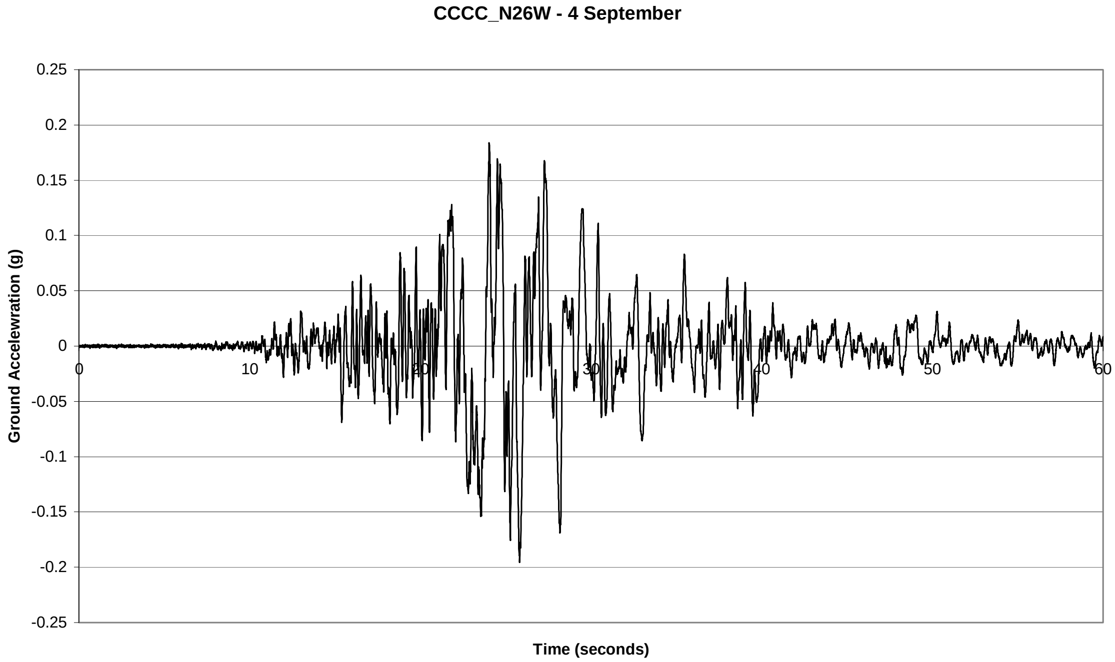


Figure A-7. Ground Acceleration History.
Christchurch Cathedral College. 4 September 2010- North 26° West Component

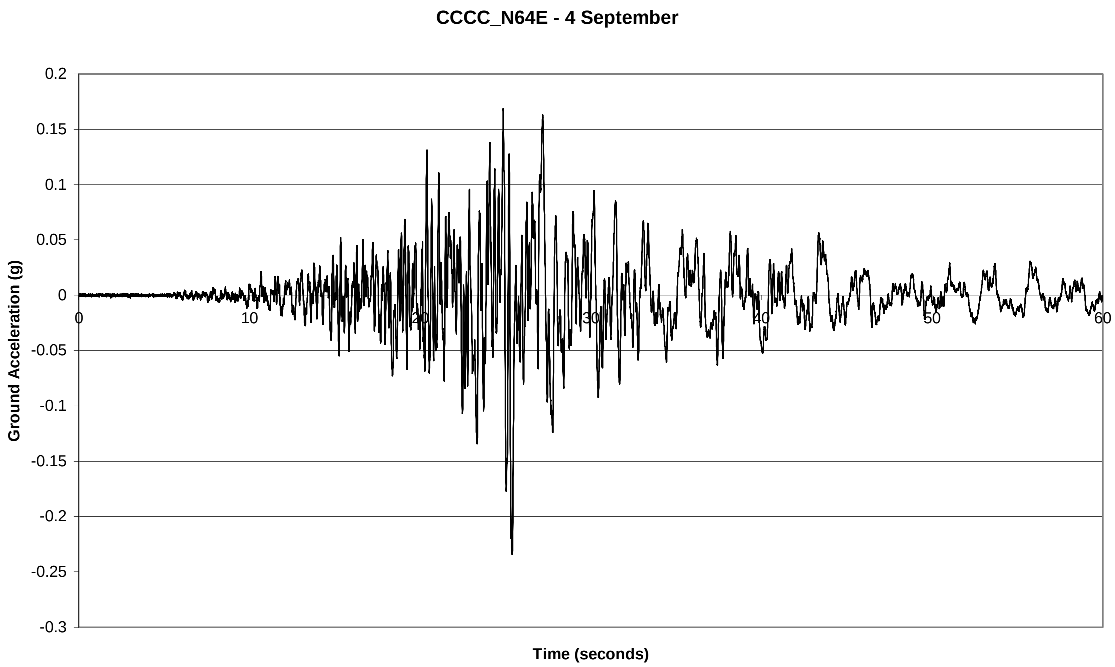


Figure A-8. Ground Acceleration History.
Christchurch Cathedral College. 4 September 2010- North 64° East Component

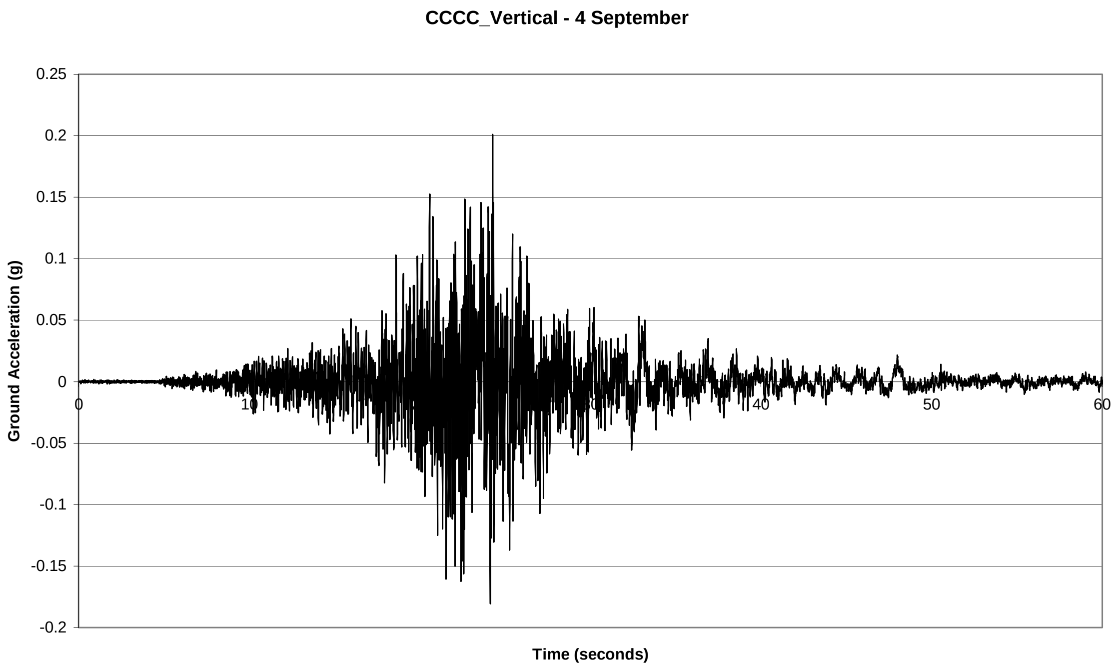


Figure A-9. Ground Acceleration History.
Christchurch Cathedral College. 4 September 2010- Vertical Component

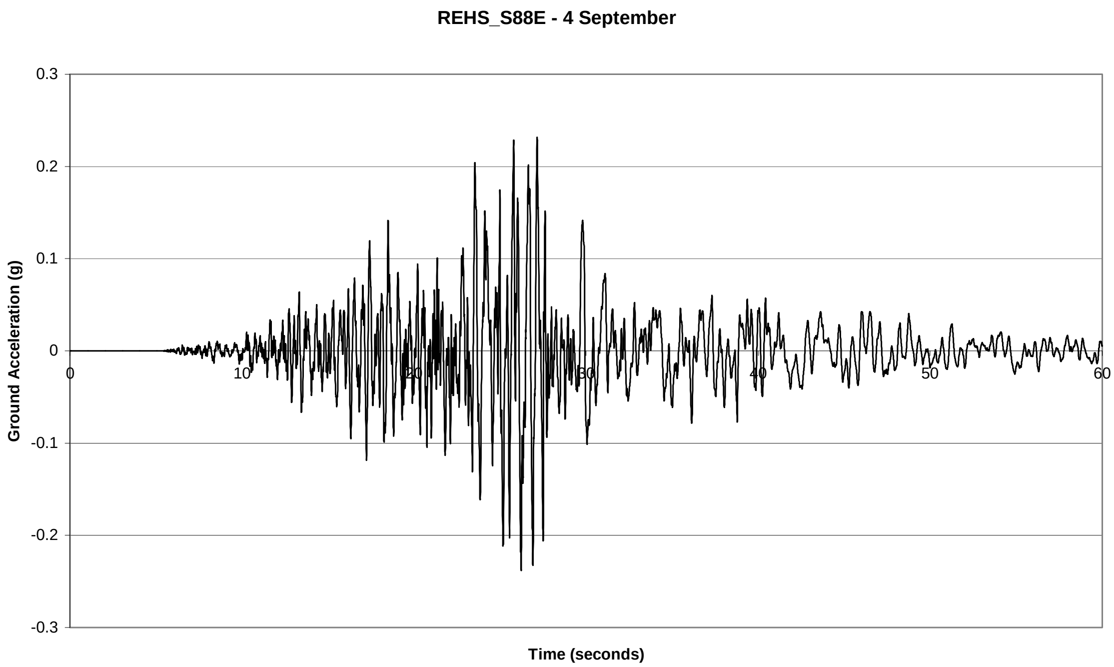


Figure A-10. Ground Acceleration History.
Resthaven. 4 September 2010- South 88° East Component

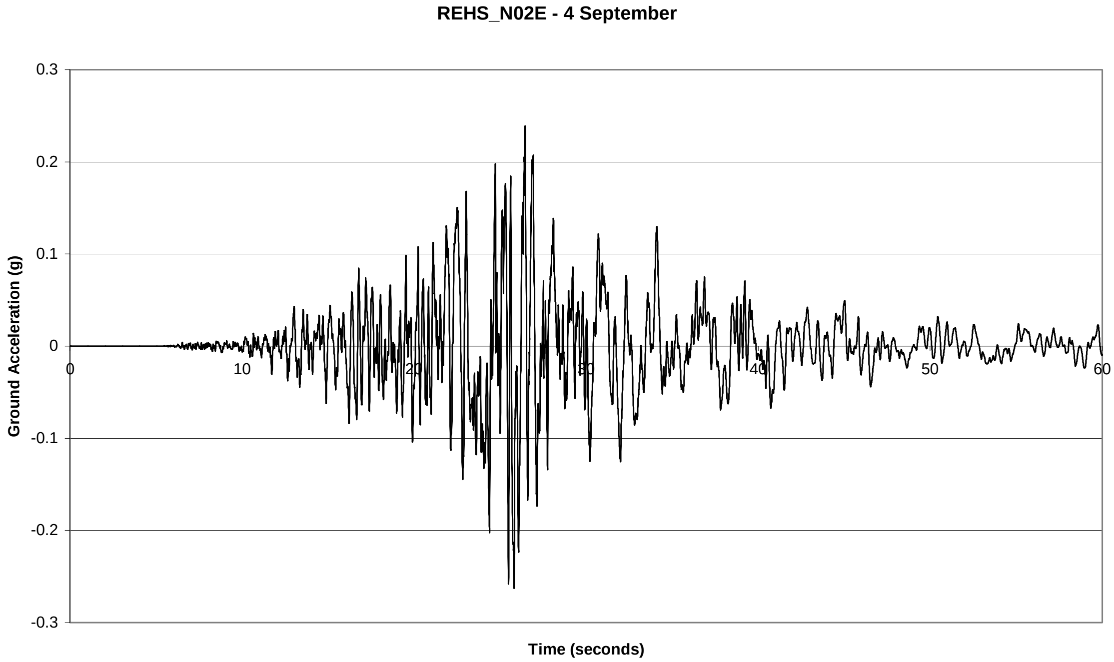


Figure A-11. Ground Acceleration History.
Resthaven. 4 September 2010- North 02° East Component

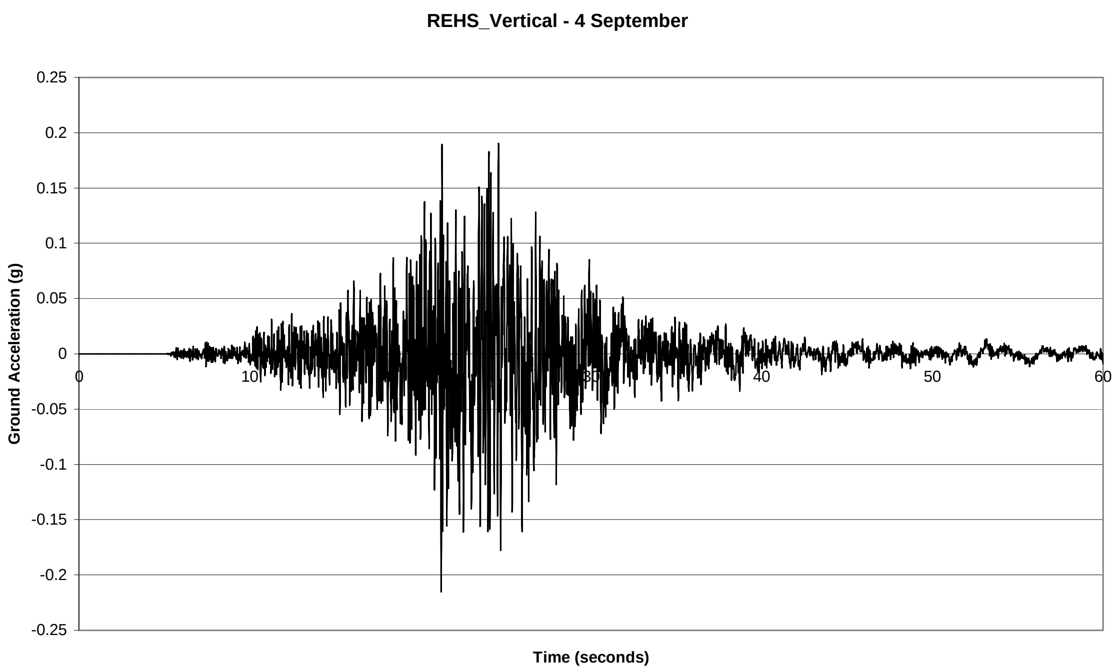
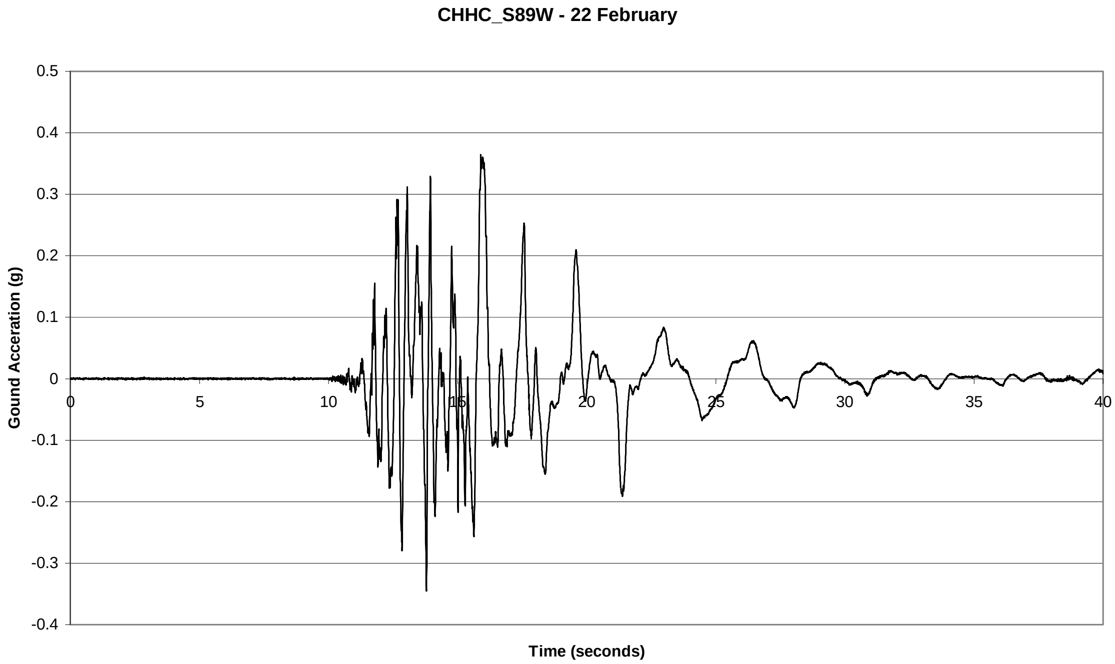


Figure A-12. Ground Acceleration History.
Resthaven. 4 September 2010- Vertical Component

22 February 2011 Earthquake Ground Acceleration Records



.Figure A-13. Ground Acceleration History.
Christchurch Hospital 22 February 2011- South 89° West Component

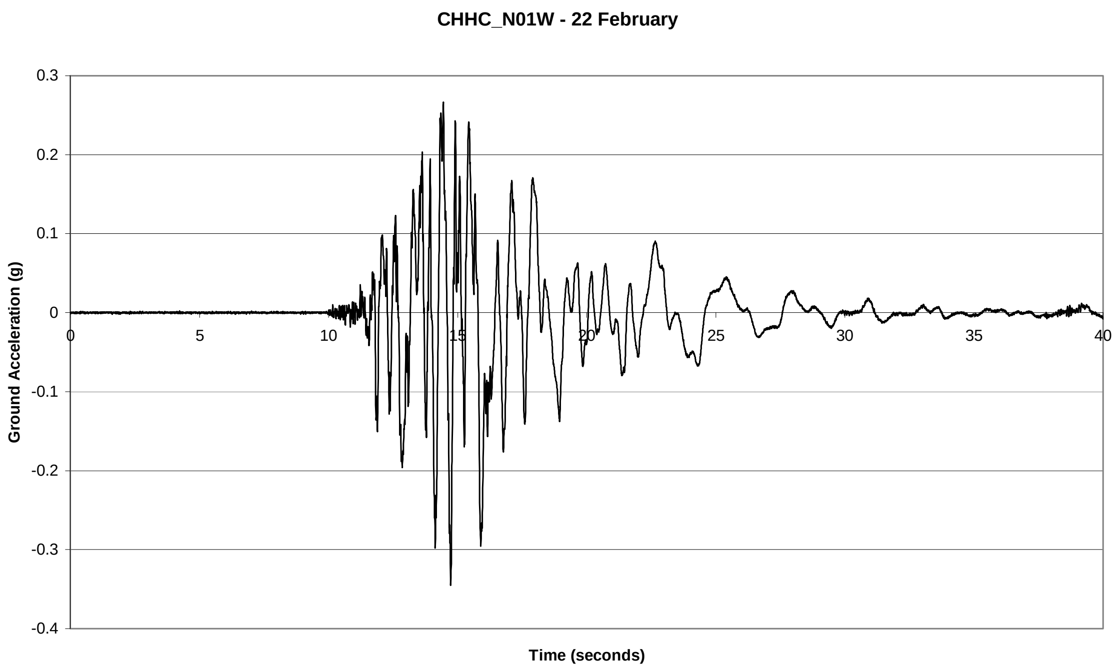


Figure A-14. Ground Acceleration History.
Christchurch Hospital 22 February 2011- North 01° West Component

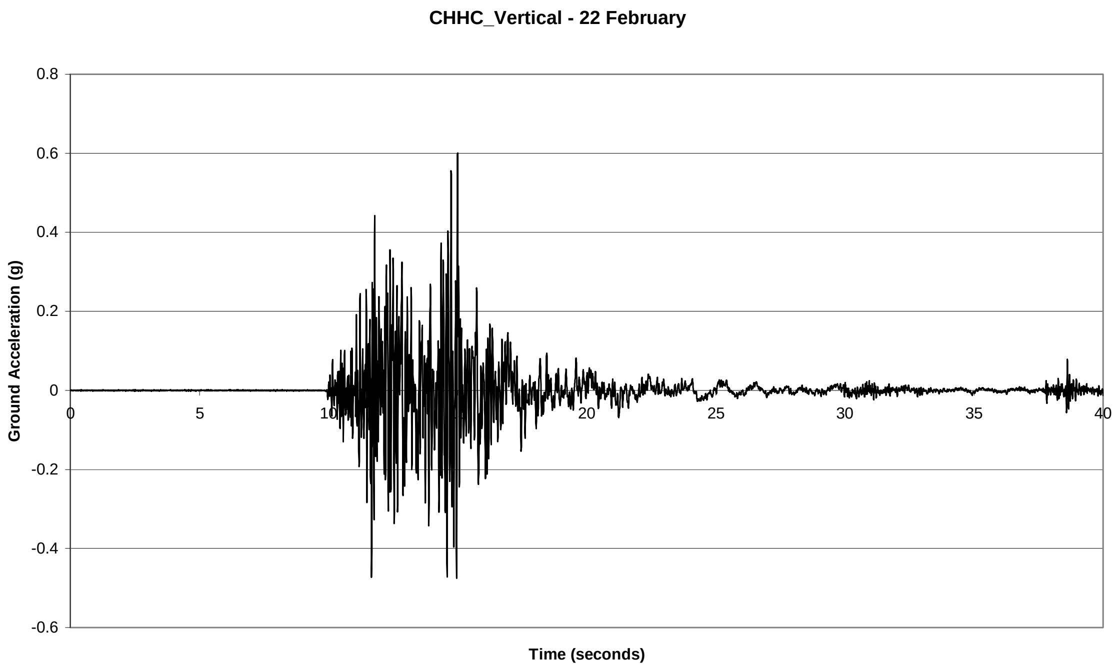


Figure A-15. Ground Acceleration History.
Christchurch Hospital 22 February 2011- Vertical Component

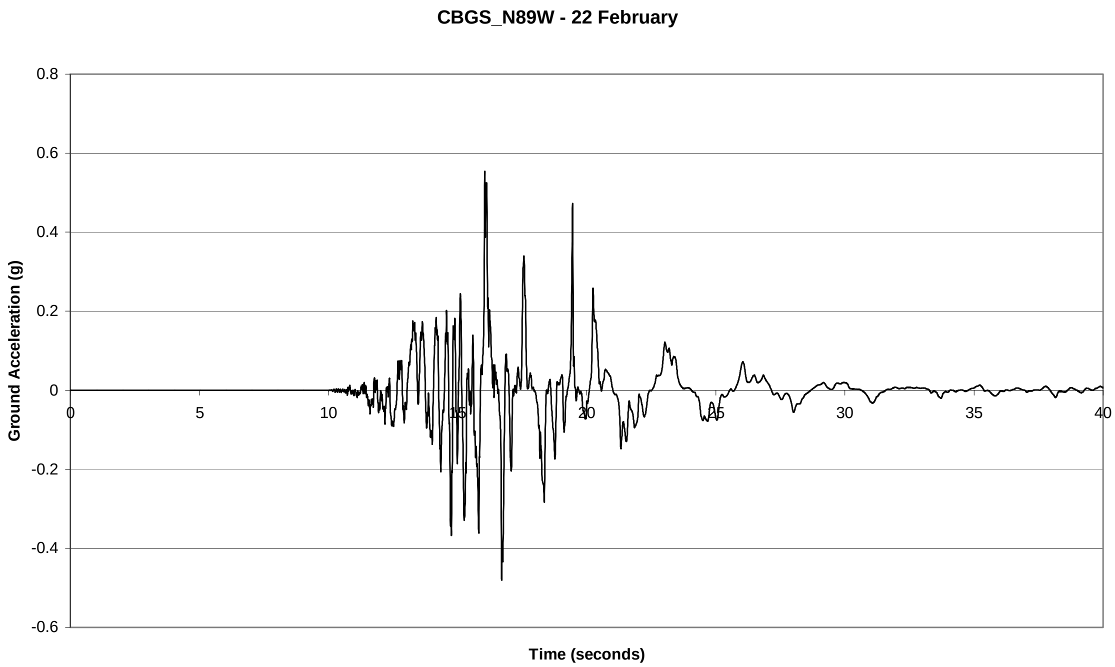


Figure A-16. Ground Acceleration History.
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component

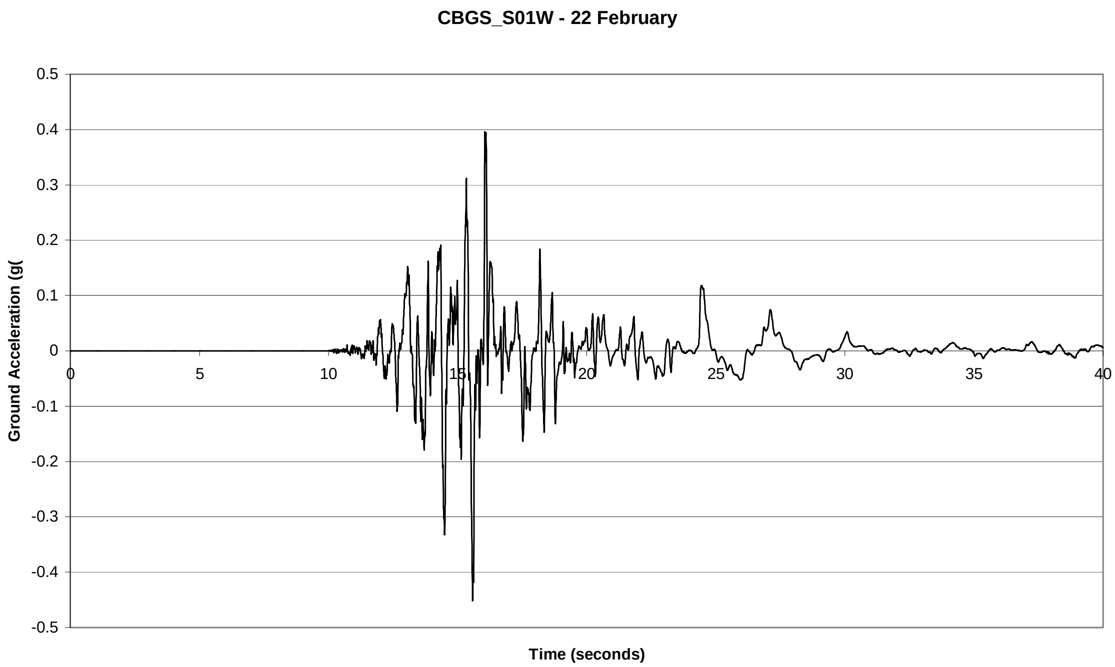


Figure A-17. Ground Acceleration History.
Christchurch Botanic Gardens. 22 February 2011- South 01° West Component

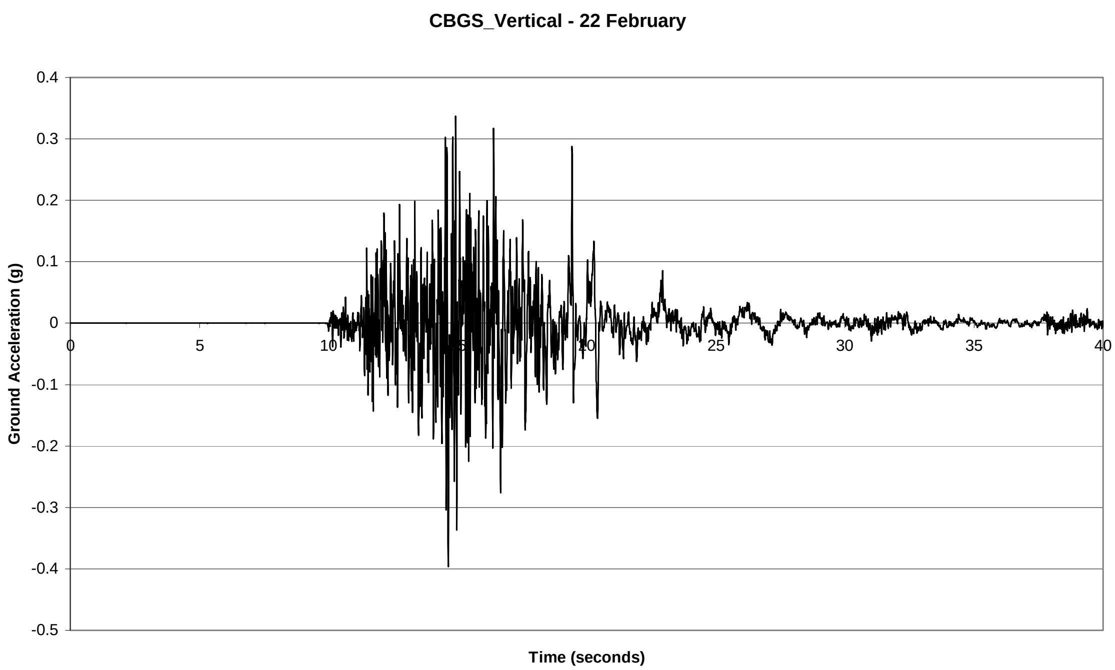


Figure A-18. Ground Acceleration History.
Christchurch Botanic Gardens. 22 February 2011- Vertical Component

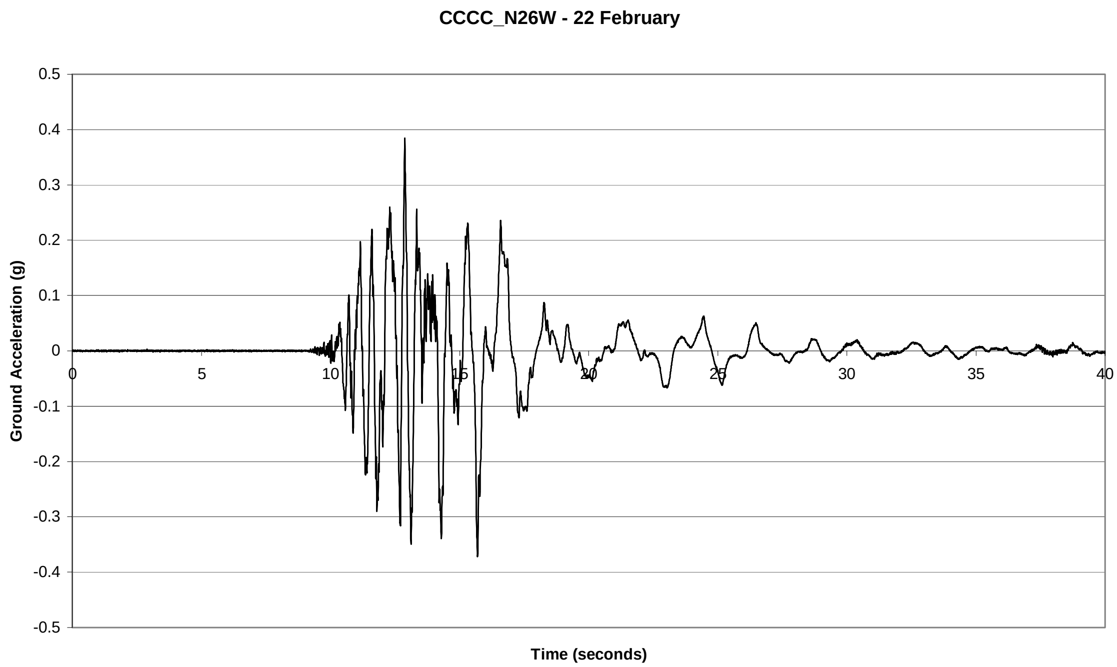


**Figure A-19. Ground Acceleration History.
Christchurch Cathedral College. 22 February 2011- North 26° West Component**

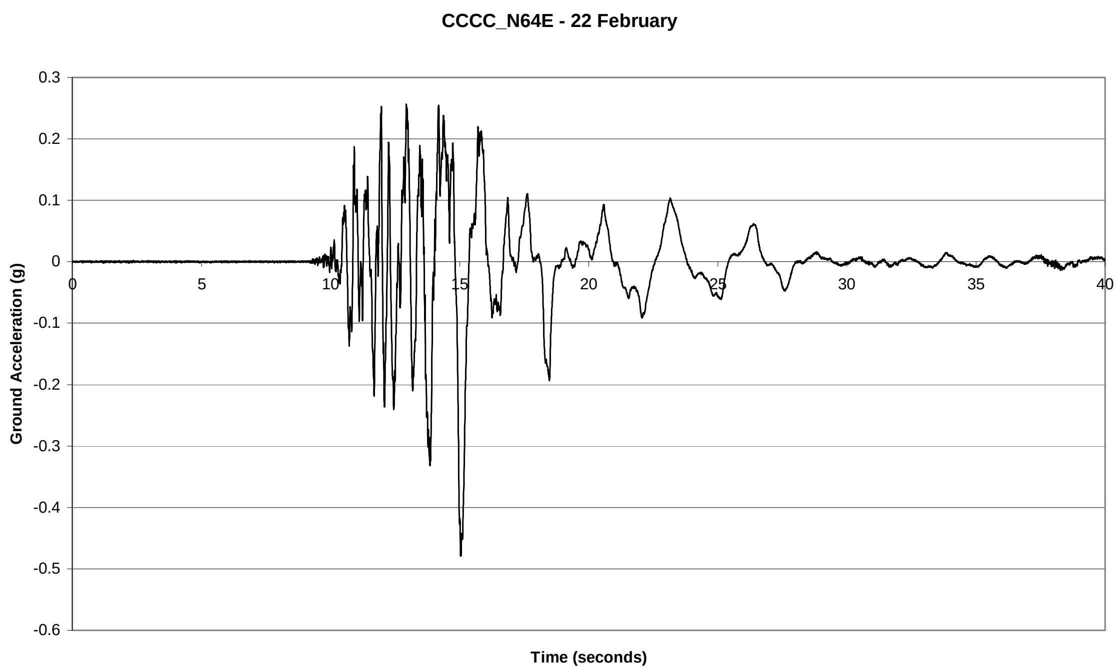


**Figure A-20. Ground Acceleration History.
Christchurch Cathedral College. 22 February 2011- North 64° East Component**

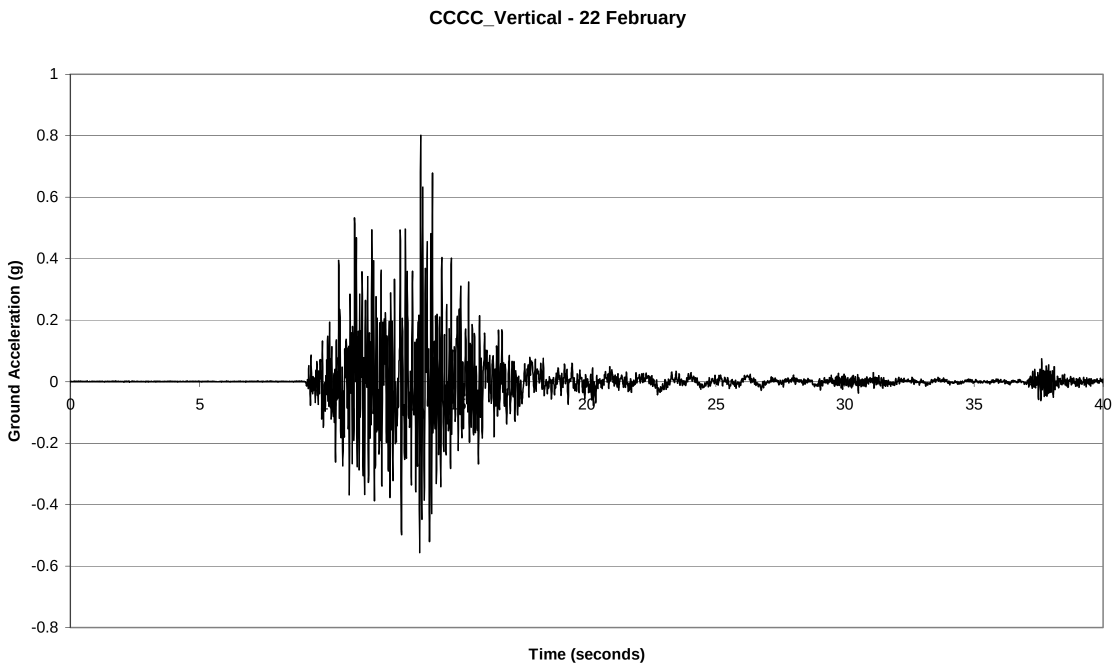


Figure A-21. Ground Acceleration History.
Christchurch Cathedral College. 22 February 2011-Vertical Component

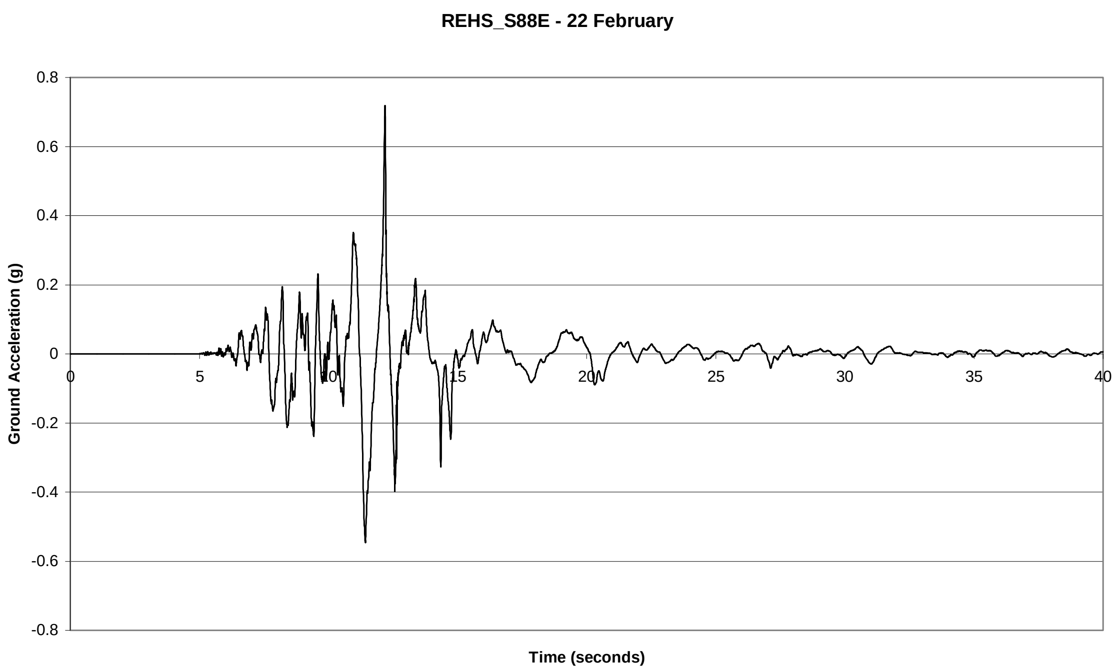


Figure A-22. Ground Acceleration History.
Resthaven. 22 February 2011- South 88° East Component

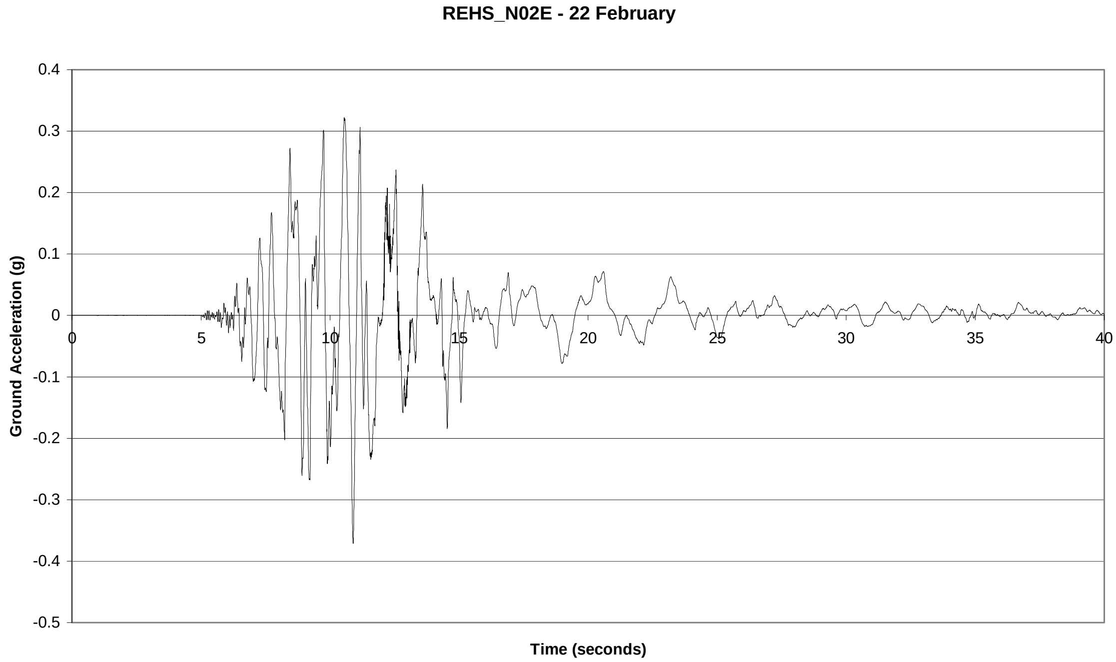


**Figure A-23. Ground Acceleration History.
Resthaven. 22 February 2011- North 02° East Component**

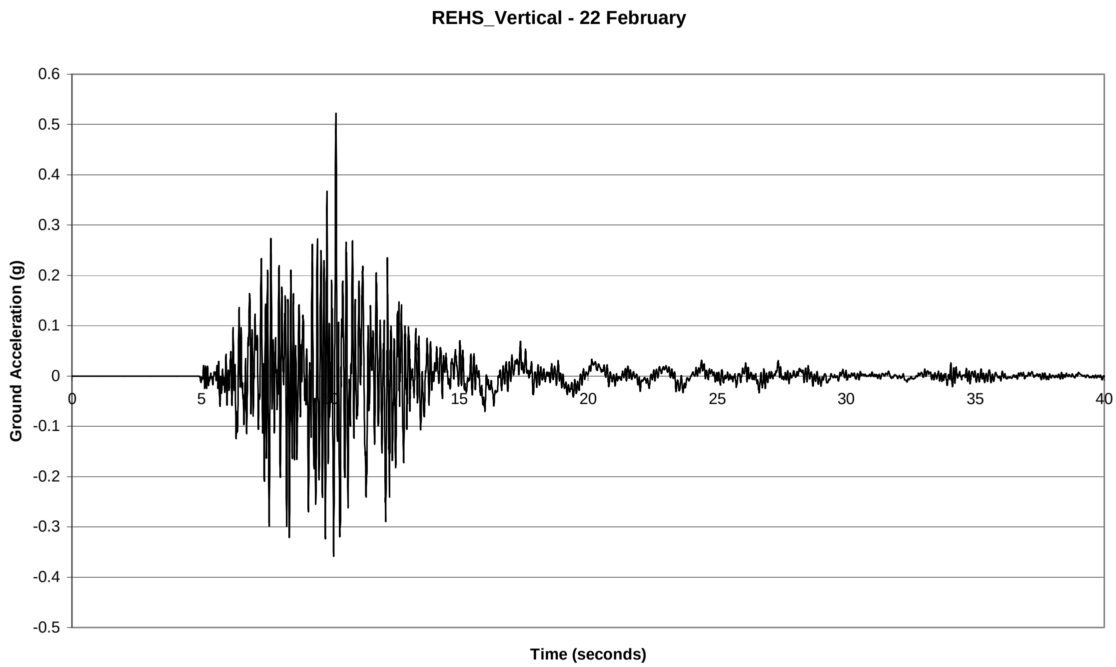


**Figure A-24. Ground Acceleration History.
Resthaven. 22 February 2011- Vertical Component**

13 June 2011 Acceleration records.

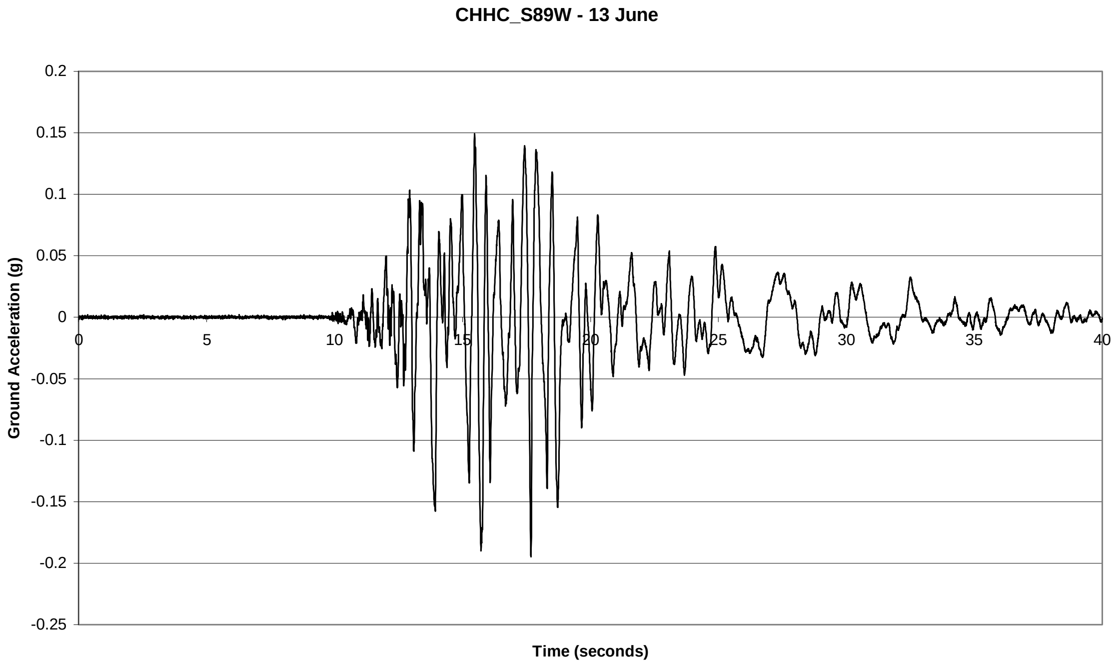


**Figure A-25. Ground Acceleration History.
Christchurch Hospital 13 June 2011- South 89° West Component**

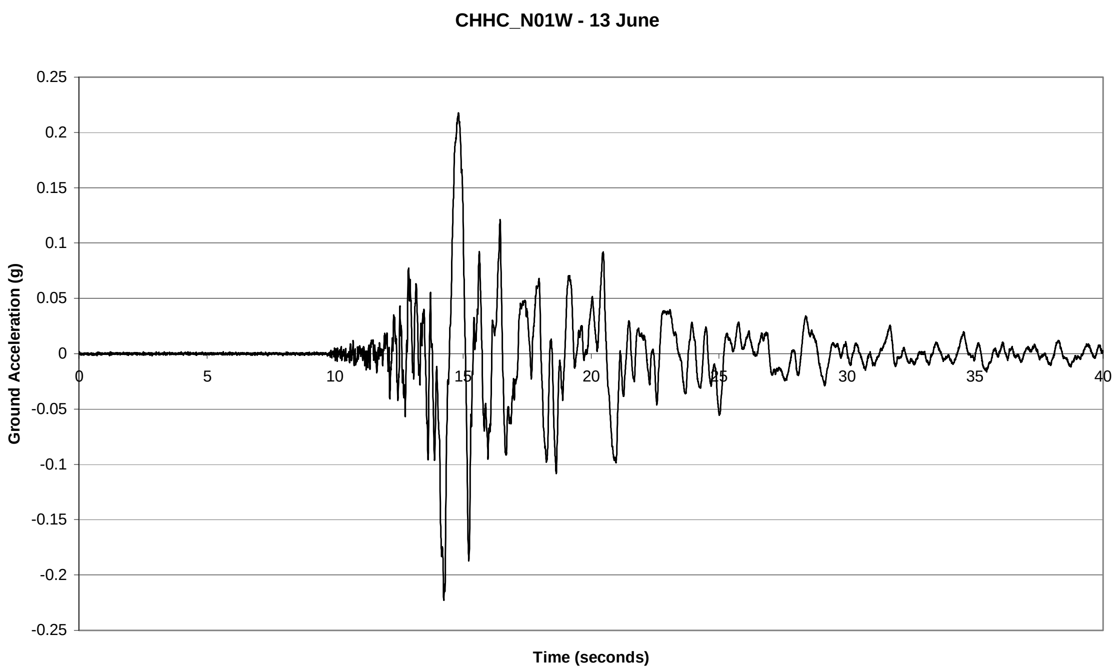


**Figure A-26. Ground Acceleration History.
Christchurch Hospital 13 June 2011- North 01° West Component**

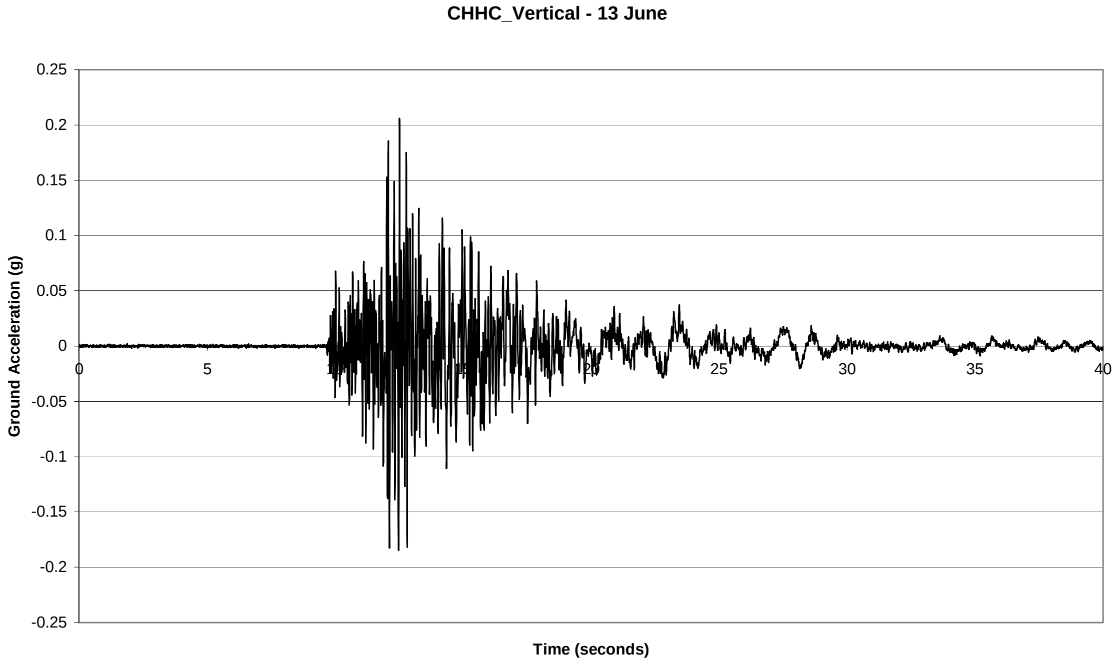


Figure A-27. Ground Acceleration History.
Christchurch Hospital 13 June 2011- Vertical Component

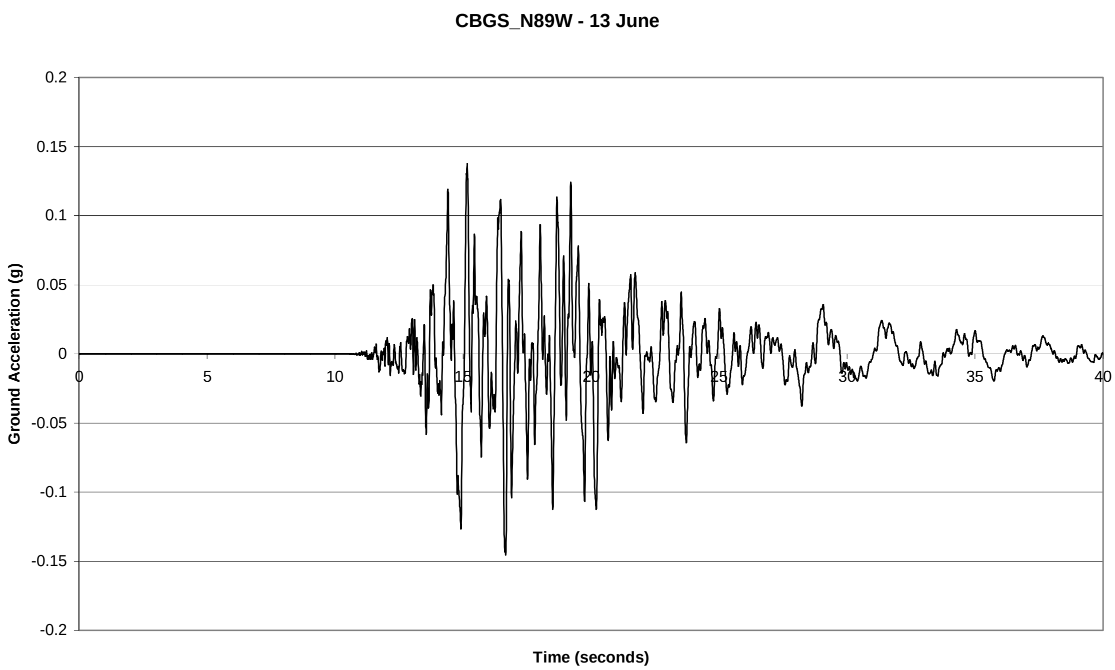


Figure A-28. Ground Acceleration History.
Christchurch Botanic Gardens. 13 June 2011- North 89° West Component

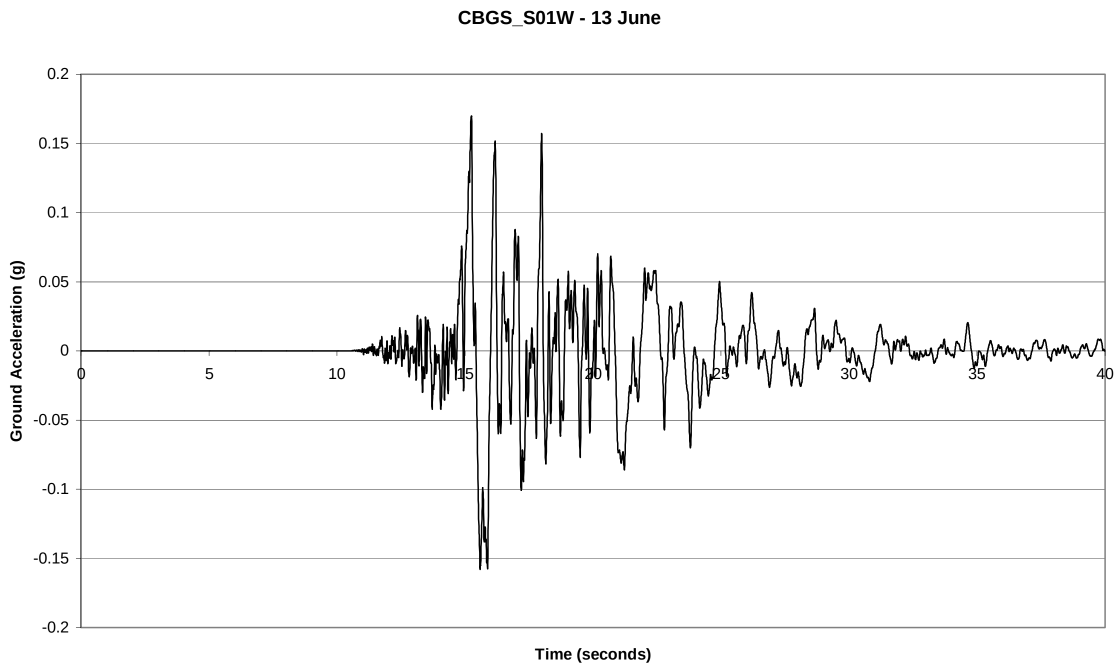


Figure A-29. Ground Acceleration History.
Christchurch Botanic Gardens. 13 June 2011- South 01° West Component

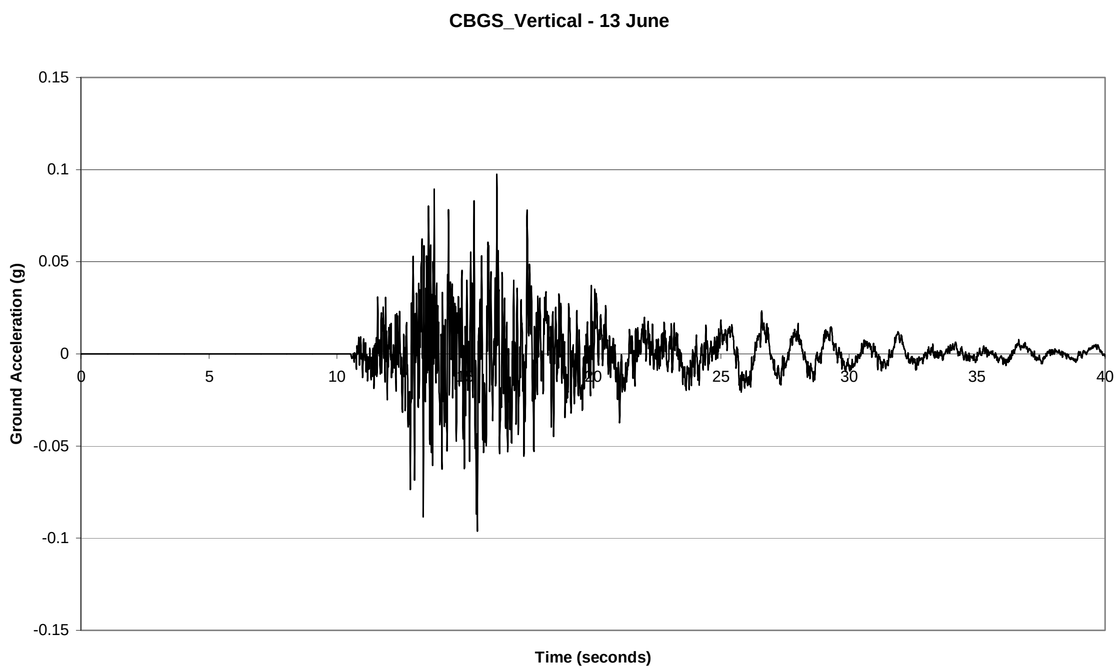


Figure A-30. Ground Acceleration History.
Christchurch Botanic Gardens. 13 June 2011- Vertical Component

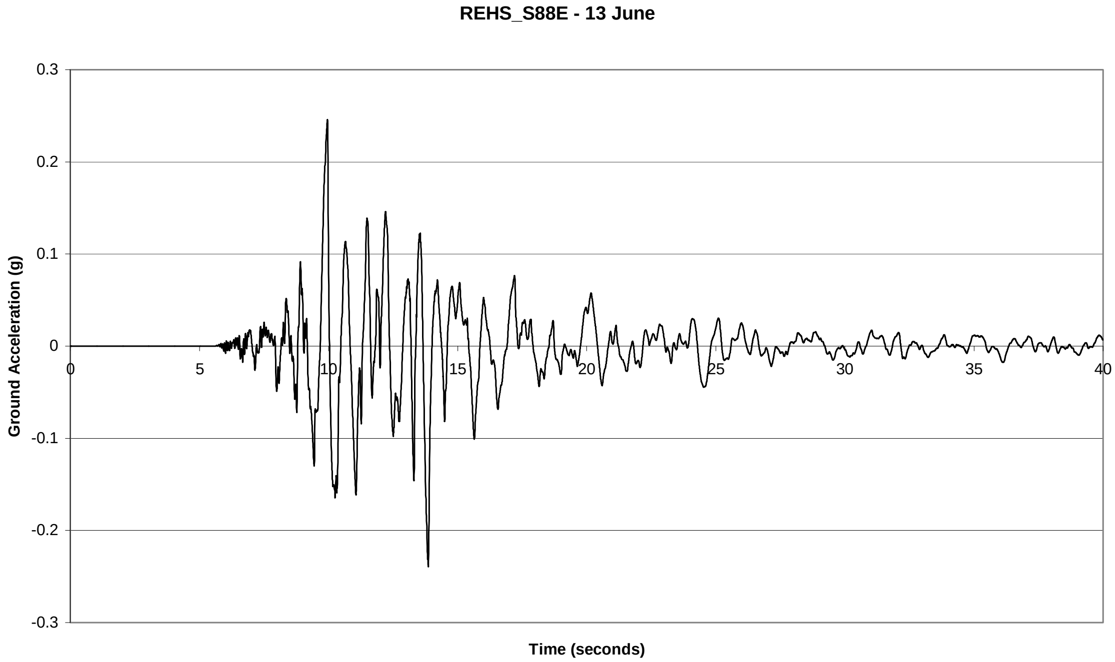


Figure A-31. Ground Acceleration History. Resthaven. 13 June 2011- South 88° East Component

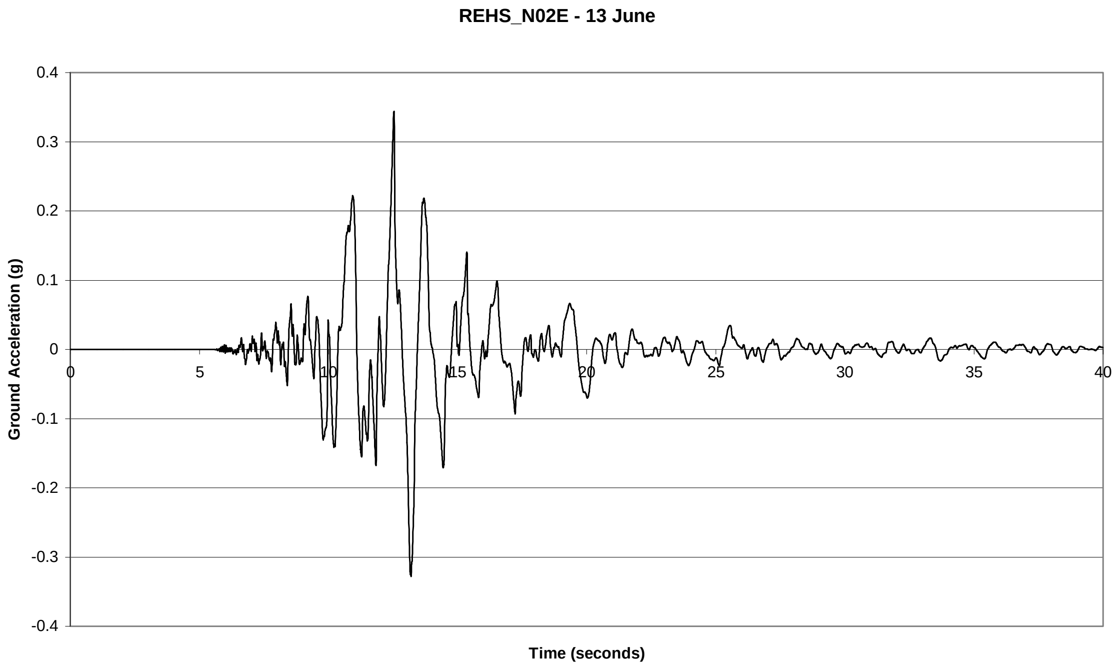


Figure A-32. Ground Acceleration History. Resthaven. 13 June 2011- North 02° East Component

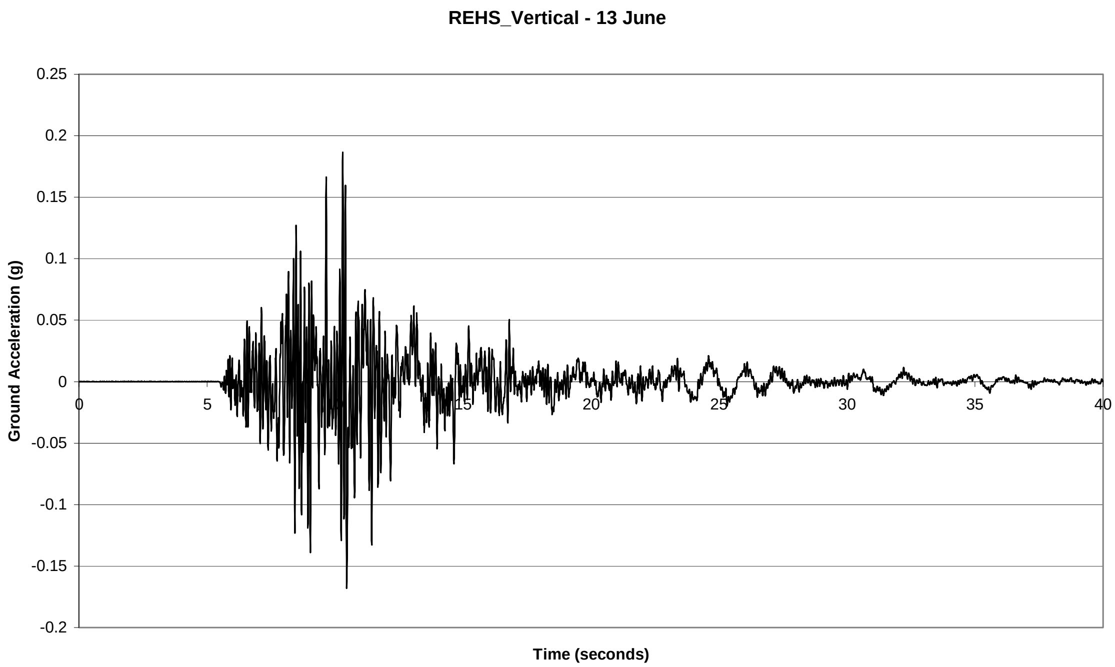


Figure A-33. Ground Acceleration History. Resthaven. 13 June 2011- Vertical Component

Appendix B
4 September 2010 Earthquake Horizontal Components

The inelastic displacement and acceleration spectra for the 4September 2010 earthquake components are shown here.

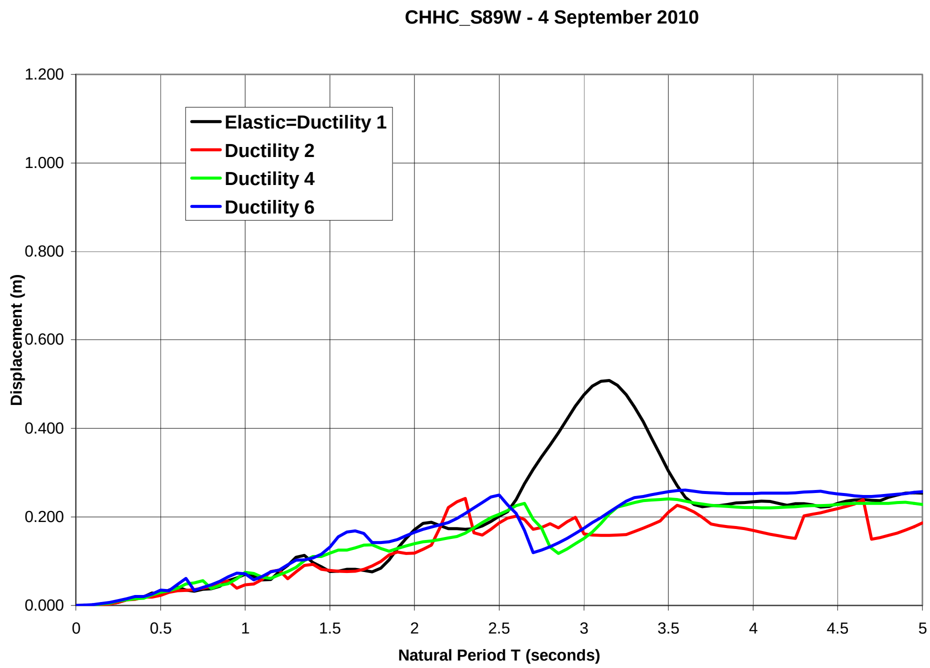


Figure B-1. Displacement Response Spectra
Christchurch Hospital, 4 September 2010- South 89° West Component

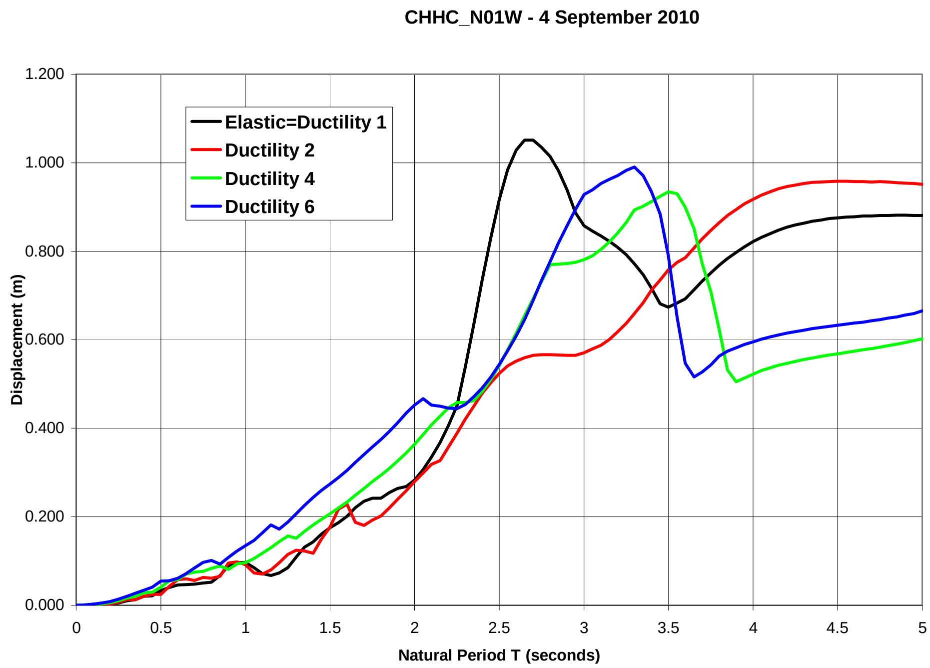


Figure B-2. Displacement Response Spectra
Christchurch Hospital, 4 September 2010- North 01° West Component

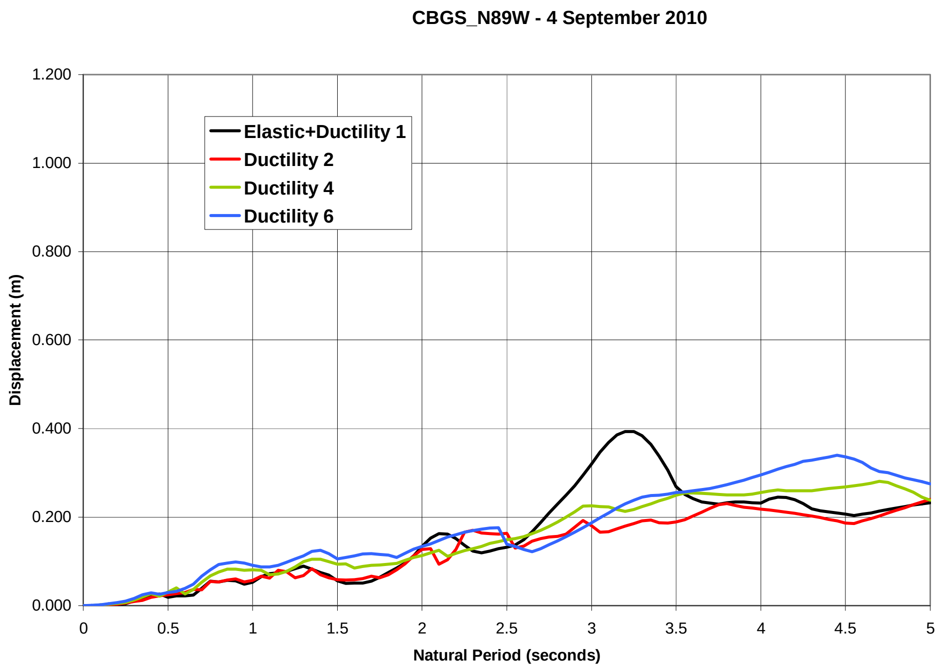


Figure B-3. Displacement Response Spectra
Christchurch Botanic Gardens. 4 September 2010- North 89° West Component

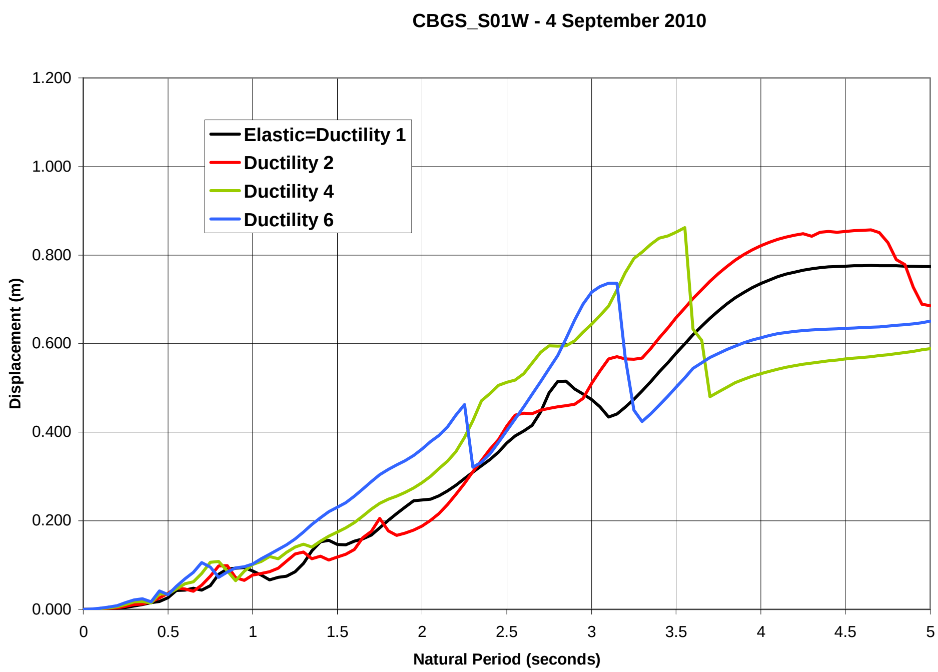


Figure B-4. Displacement Response Spectra
Christchurch Botanic Gardens. 4 September 2010- South 01° West Component

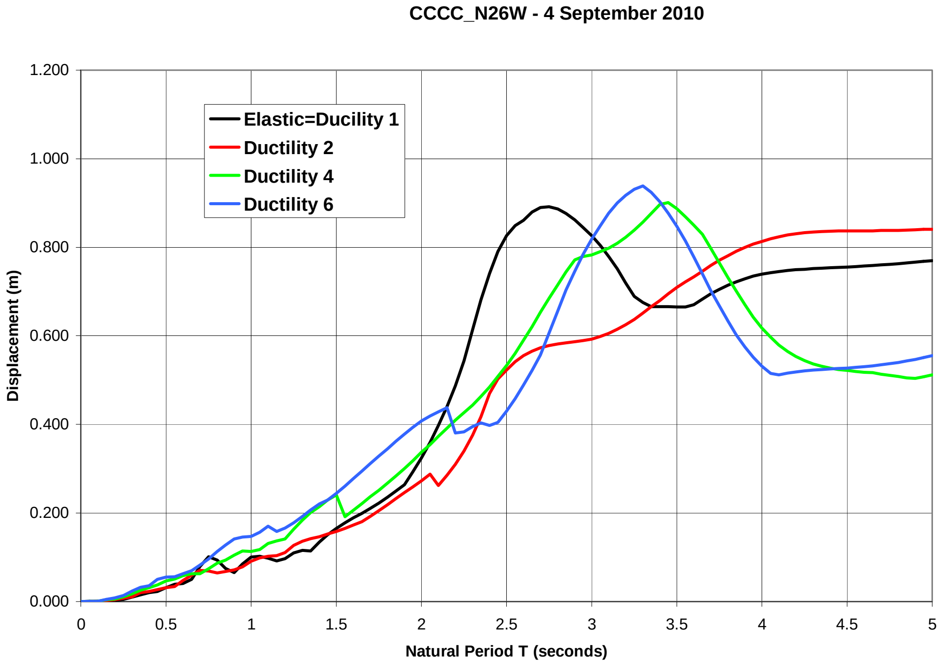


Figure B-5. Displacement Response Spectra
Christchurch Cathedral College. 4 September 2010- North 26° West Component

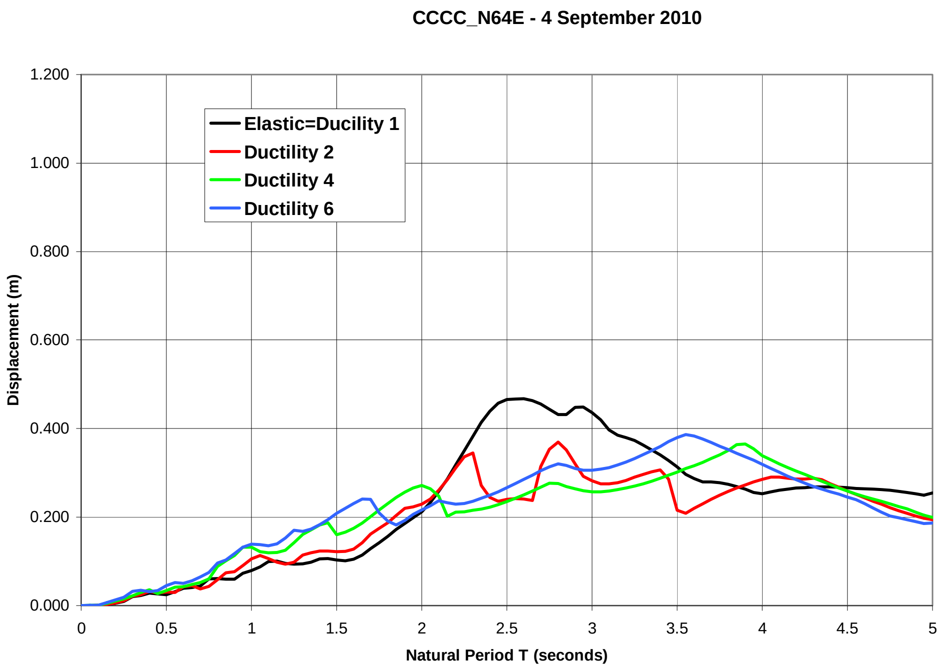


Figure B-3. Displacement Response Spectra
Christchurch Cathedral College. 4 September 2010- North 64° East Component

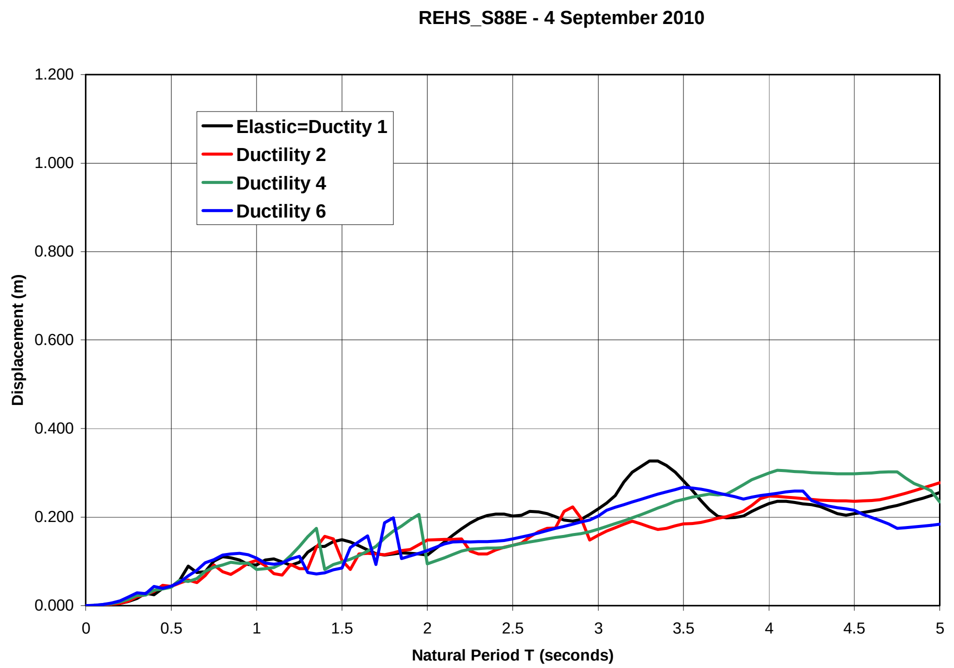


Figure B-7. Displacement Response Spectra
Resthaven. 4 September 2010- South 88° East Component

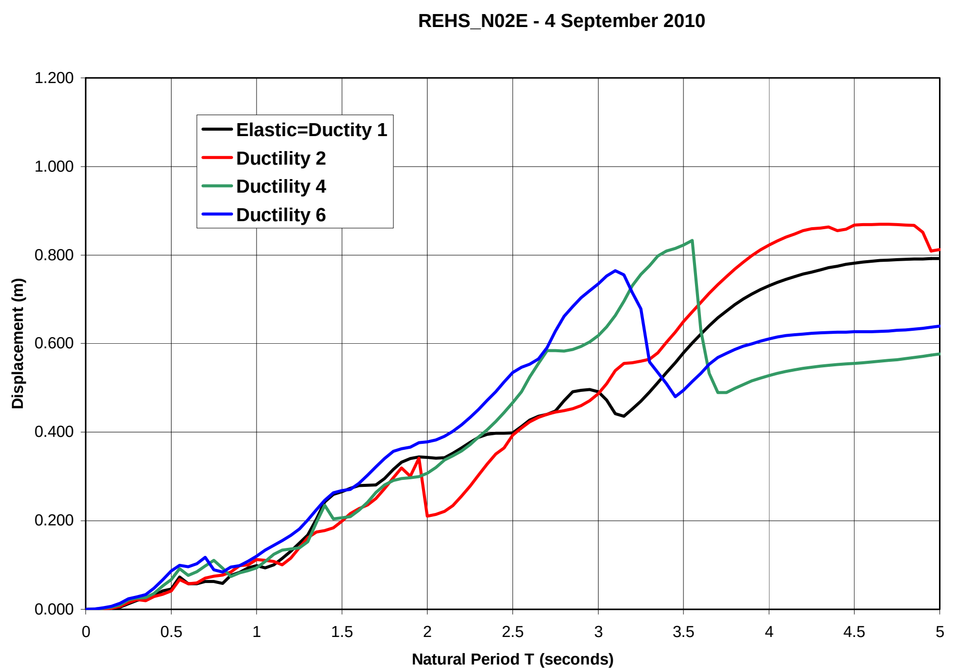


Figure B-8. Displacement Response Spectra
Resthaven. 4 September 2010- North 02° East Component

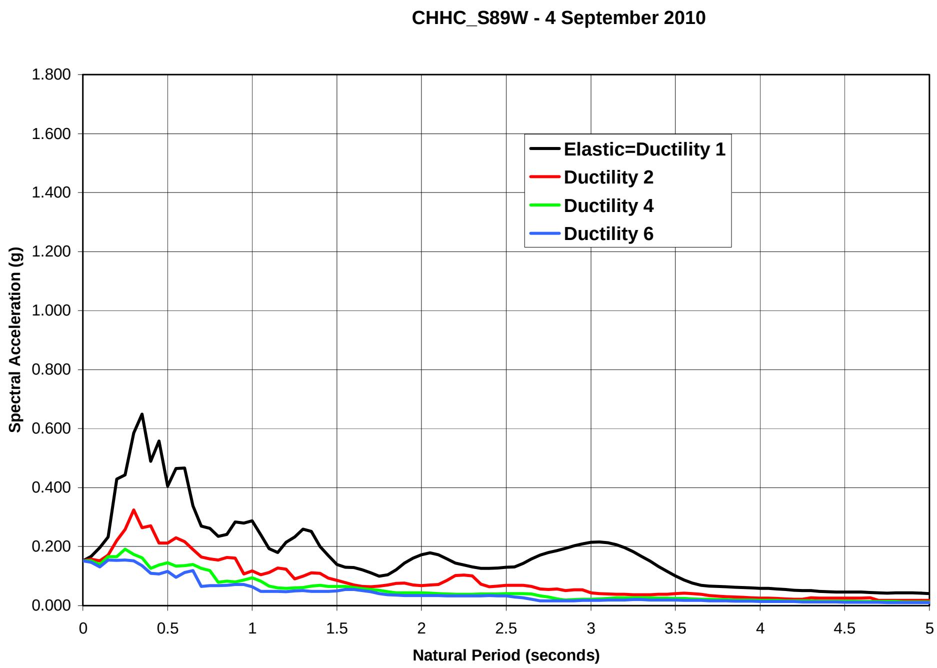


Figure B-9. Acceleration Response Spectra
Christchurch Hospital, 4 September 2010- South 89° West Component

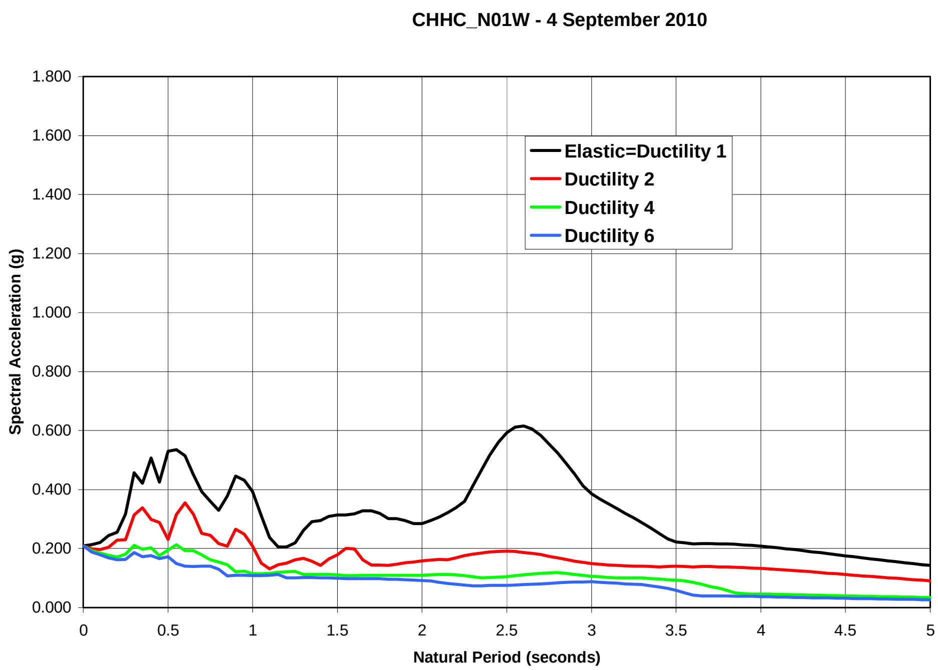


Figure B-10. Acceleration Response Spectra
Christchurch Hospital 4 September 2010- North 01° West Component.

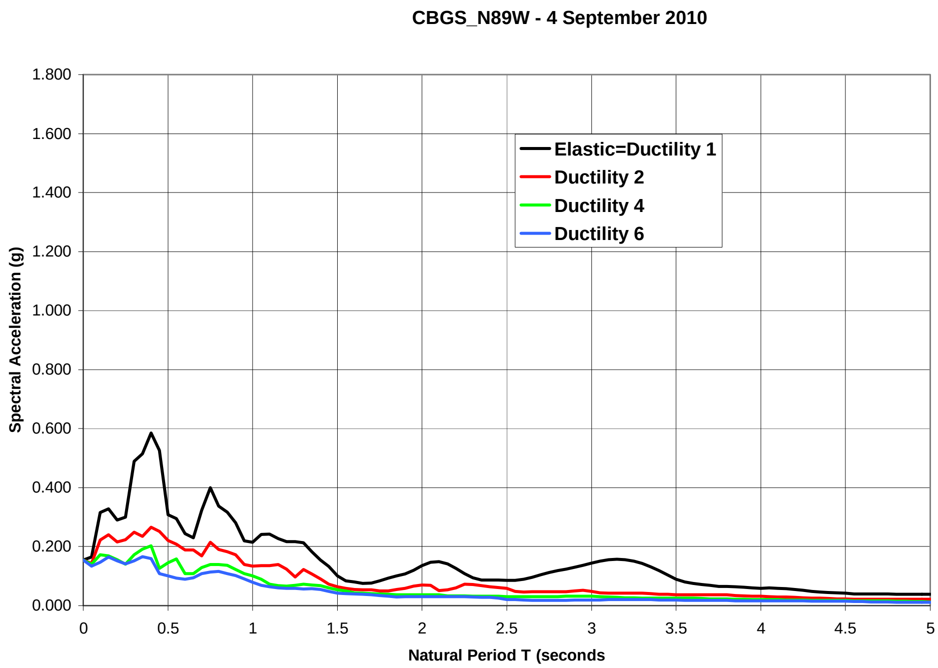


Figure B-11. Acceleration Response Spectra
Christchurch Botanic Gardens. 4 September 2010- North 89° West Component

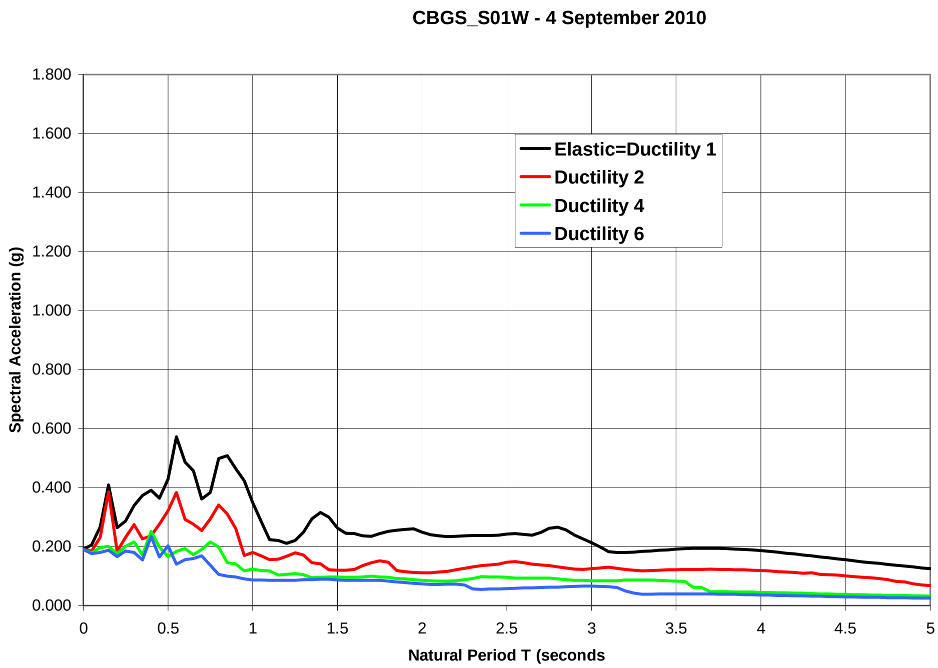


Figure B-12. Acceleration Response Spectra
Christchurch Botanic Gardens. 4 September 2010- South 01° West Component

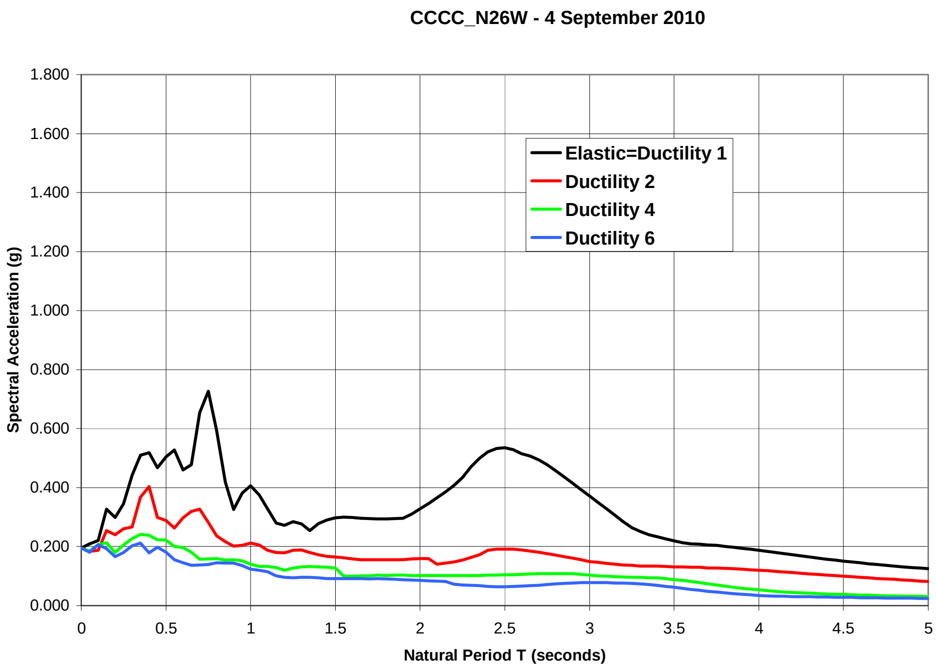


Figure B-13. Acceleration Response Spectra
Christchurch Cathedral College. 4 September 2010- North 26° West Component

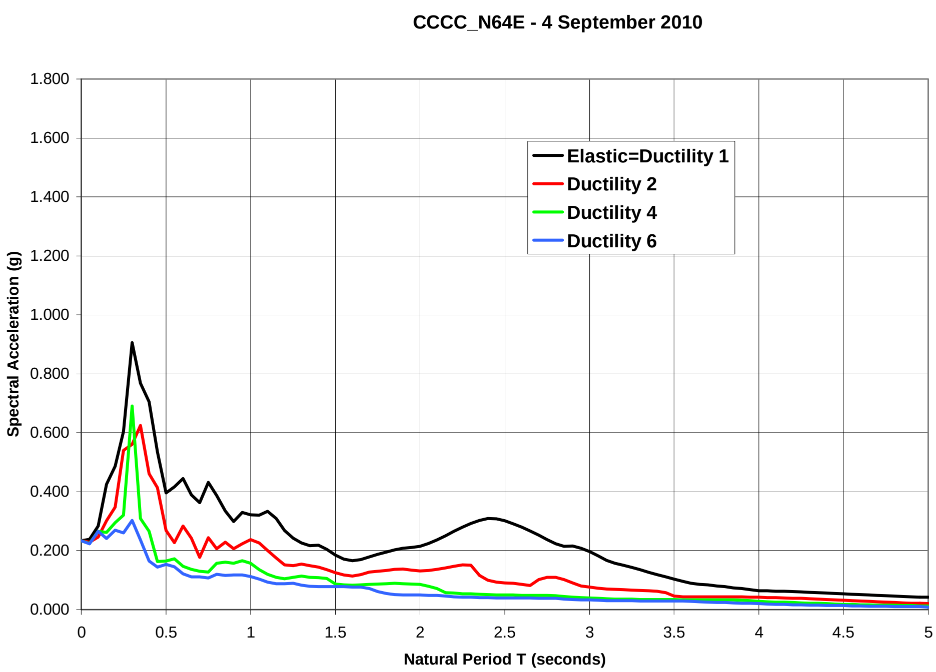


Figure B-14. Acceleration Response Spectra
Christchurch Cathedral College. 4 September 2010- North 64° East Component

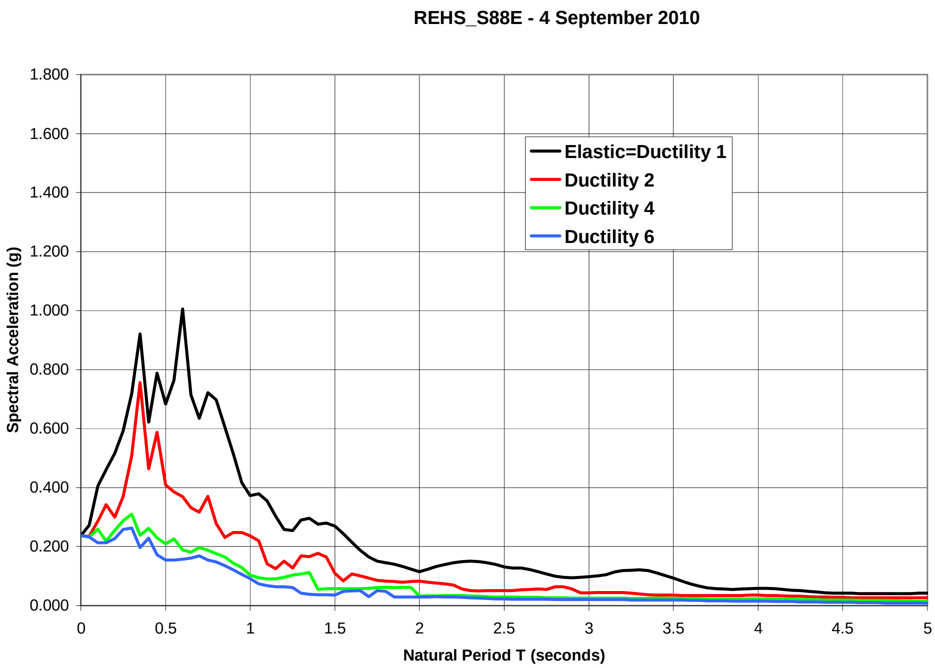


Figure B-15. Acceleration Response Spectra Resthaven. 4 September 2010- South 88° East Component

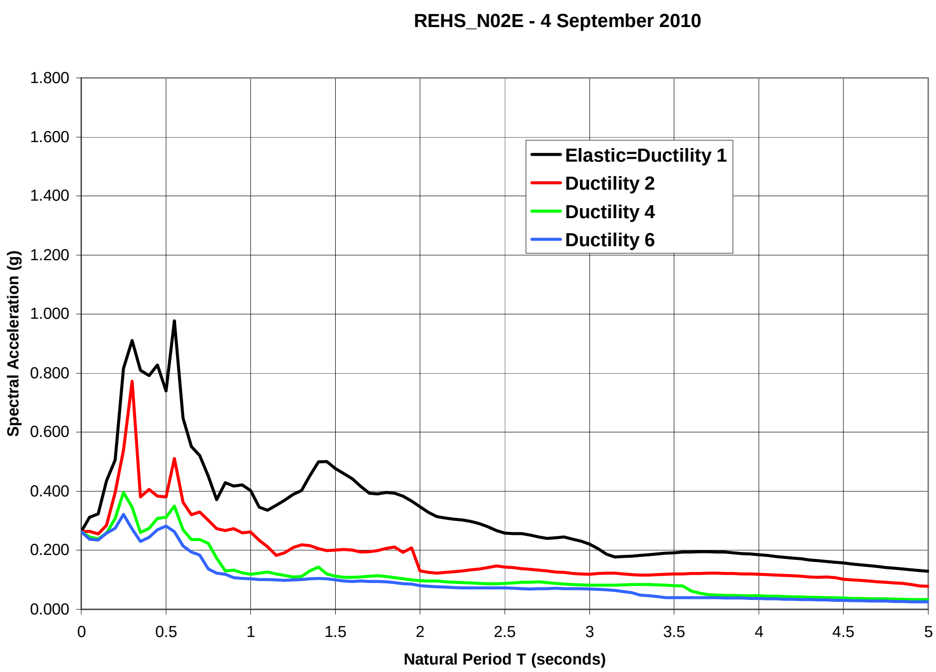


Figure B-16. Acceleration Response Spectra Resthaven. 4 September 2010- North 02° East Component

Appendix C
22 February 2011 Earthquake Horizontal Components

The inelastic displacement and acceleration spectra for the 22nd February 2011 earthquake components are shown here.

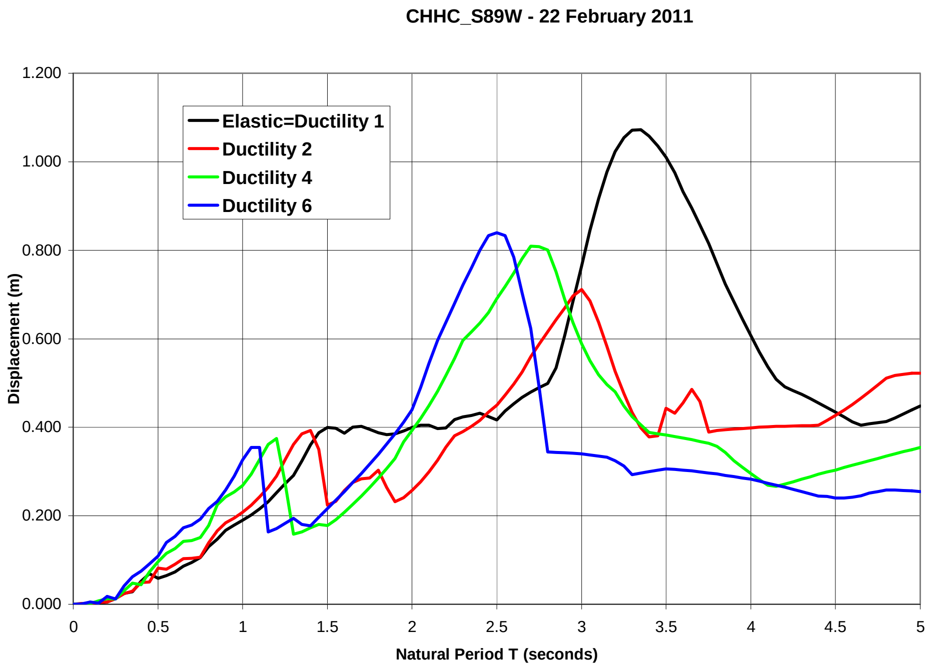


Figure C-1. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component

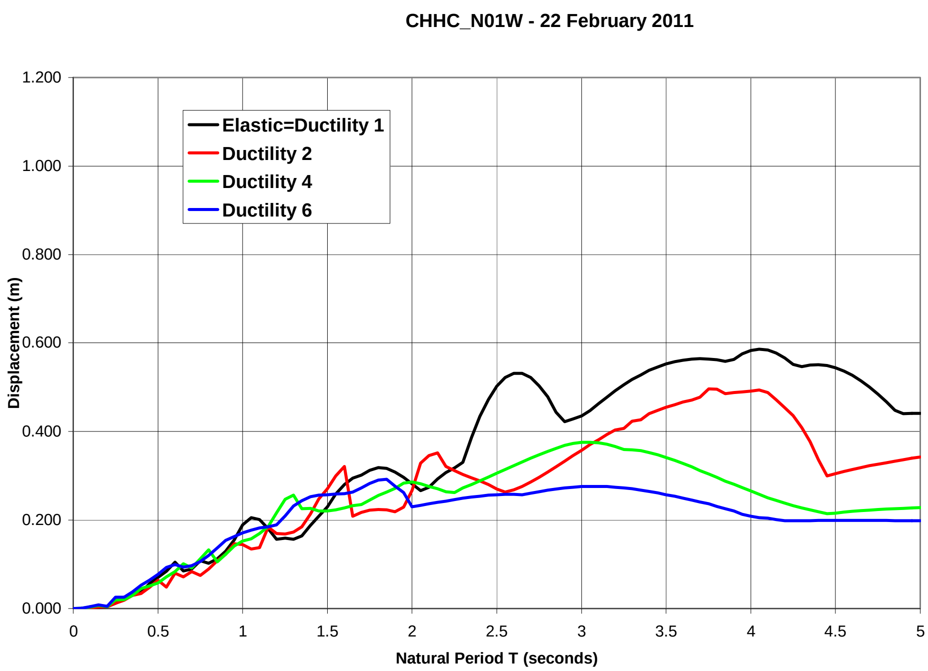


Figure C-2. Displacement Response Spectra
Christchurch Hospital 22 February 2011- North 01° West Component

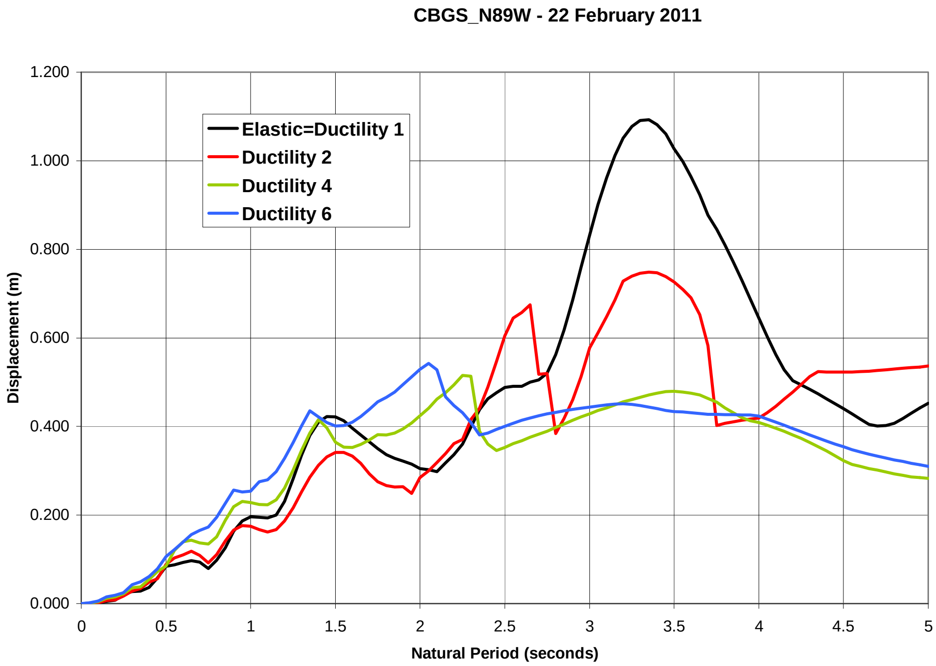


Figure C-3. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component

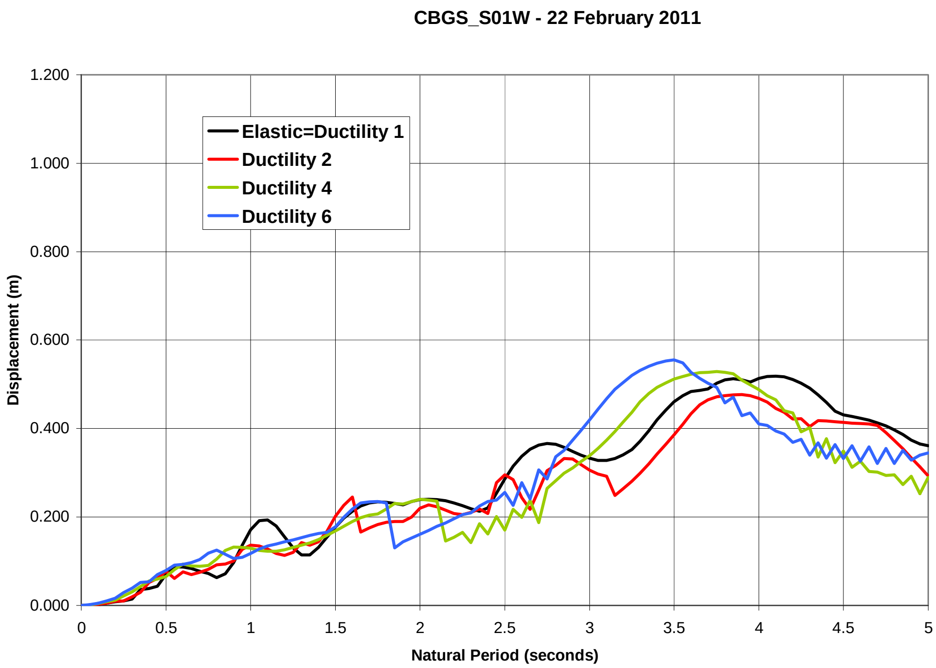


Figure C-4. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- South 01° West Component

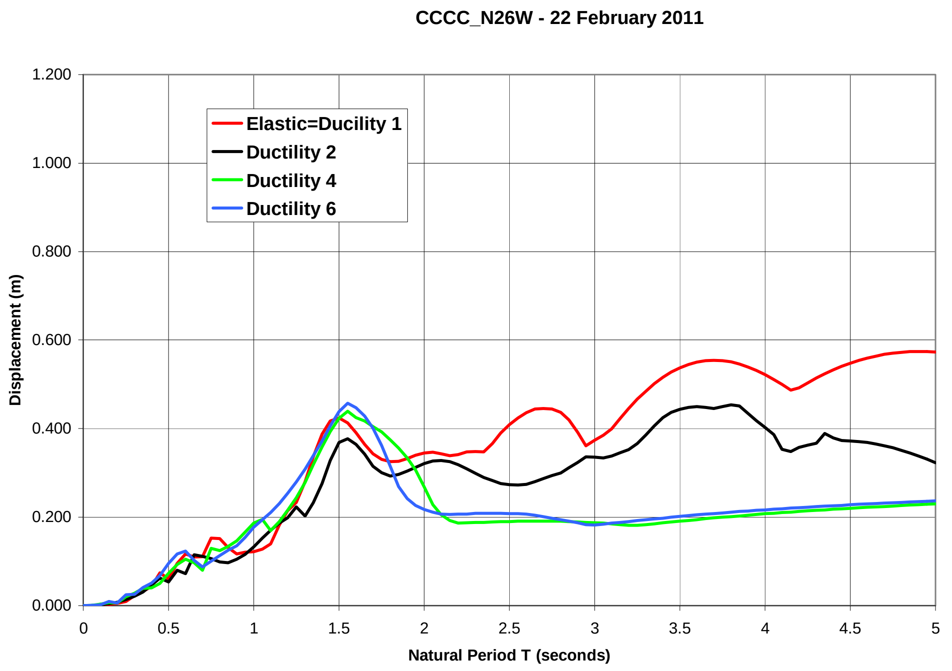


Figure C-5. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- North 26° West Component

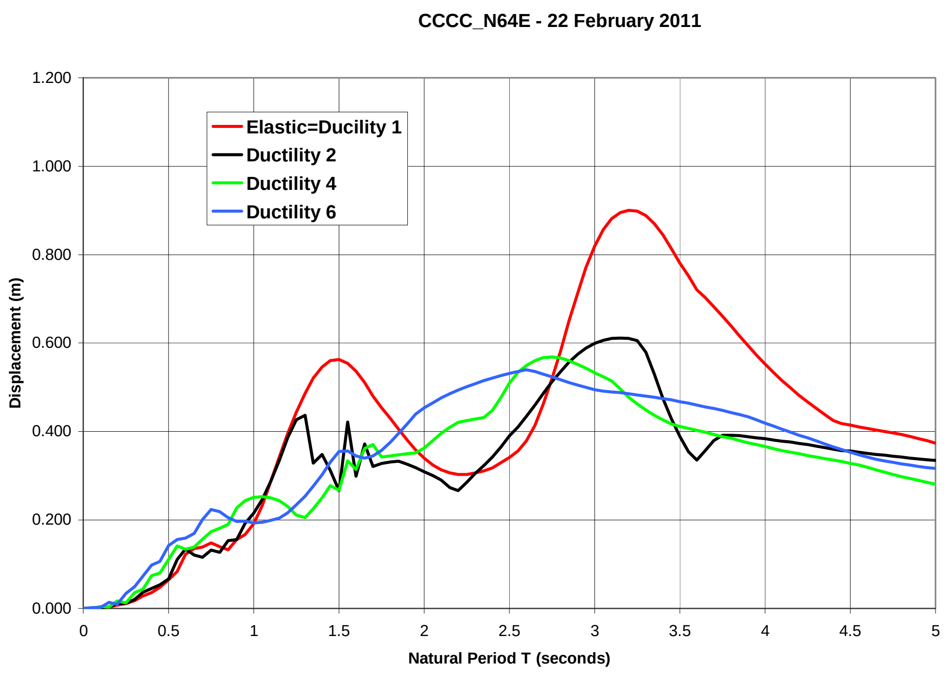


Figure C-6. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component

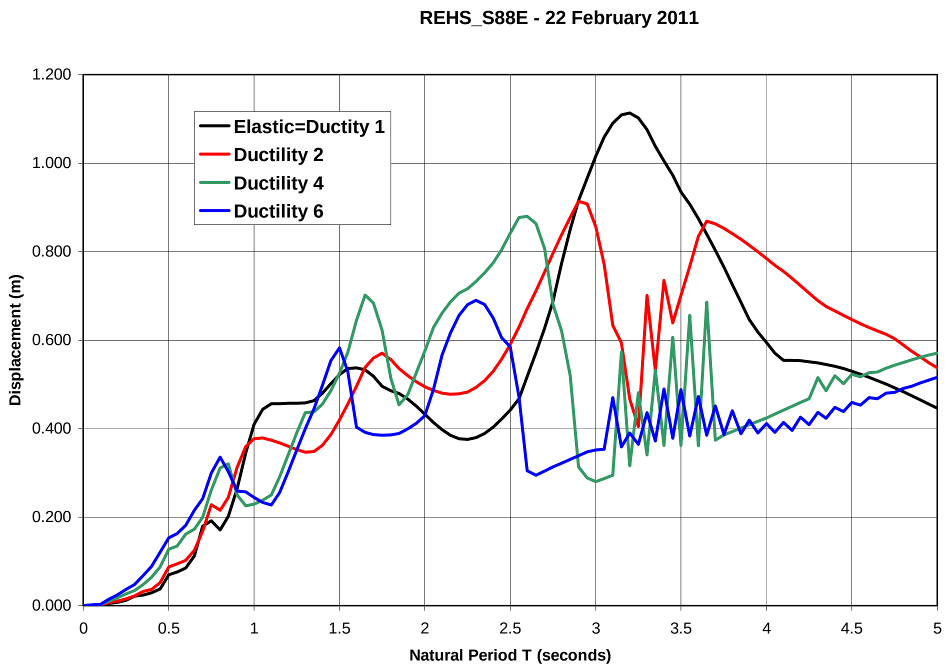


Figure C-7. Displacement Response Spectra
Resthaven. 22 February 2011- South 88° East Component

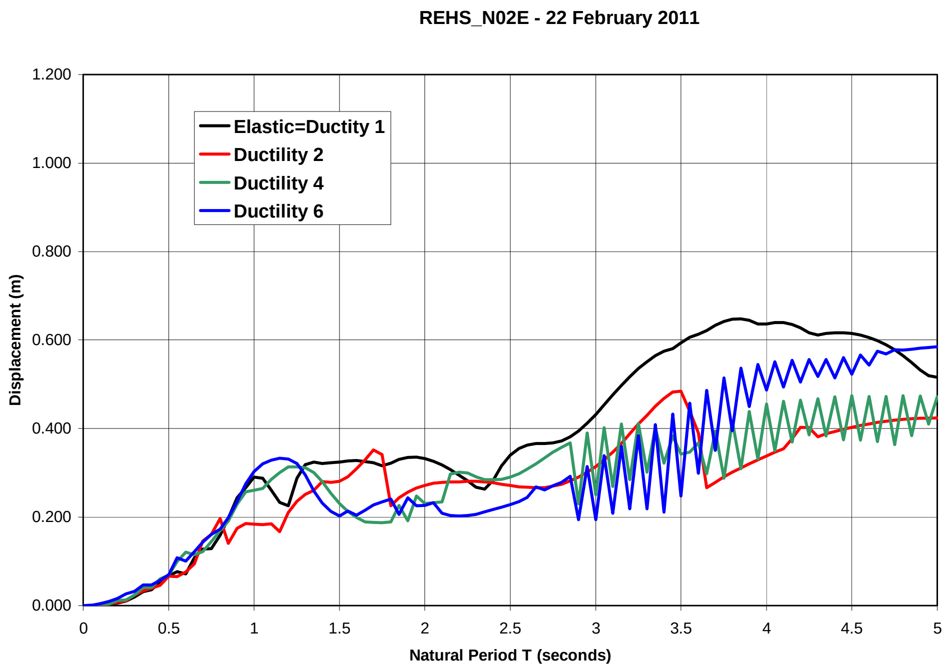


Figure C-8. Displacement Response Spectra
Resthaven. 22 February 2011- North 02° East Component
Note: Very sensitive displacement responses for high ductility for the longer natural periods

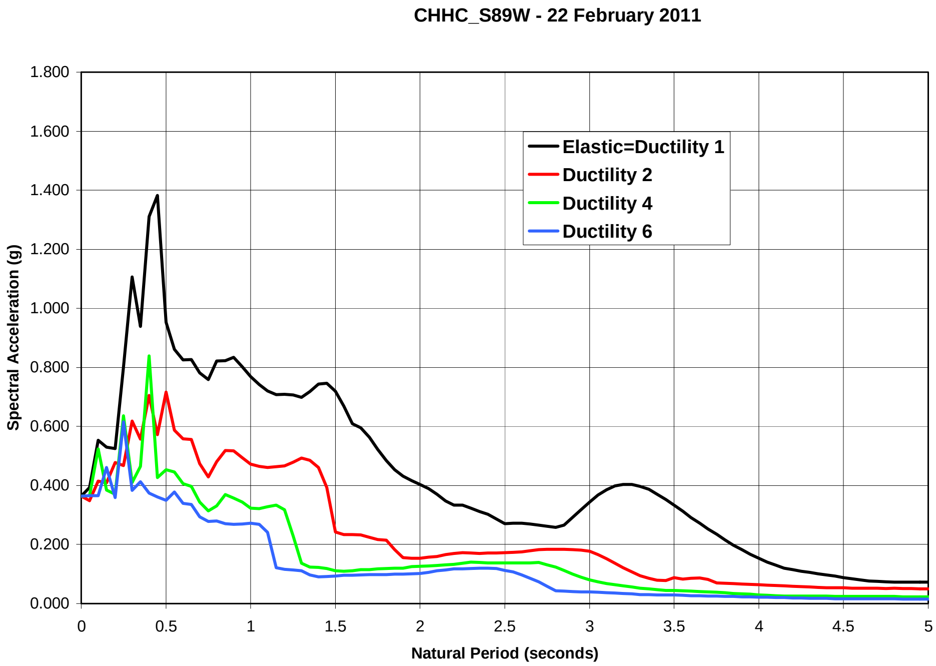


Figure C-9. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component

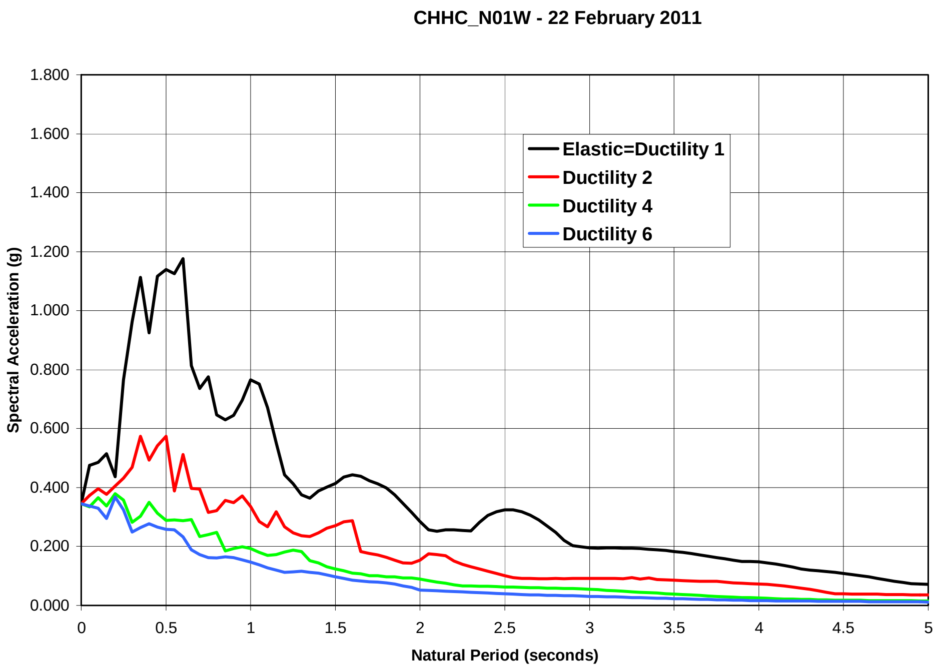


Figure C-10. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- North 01° West Component.

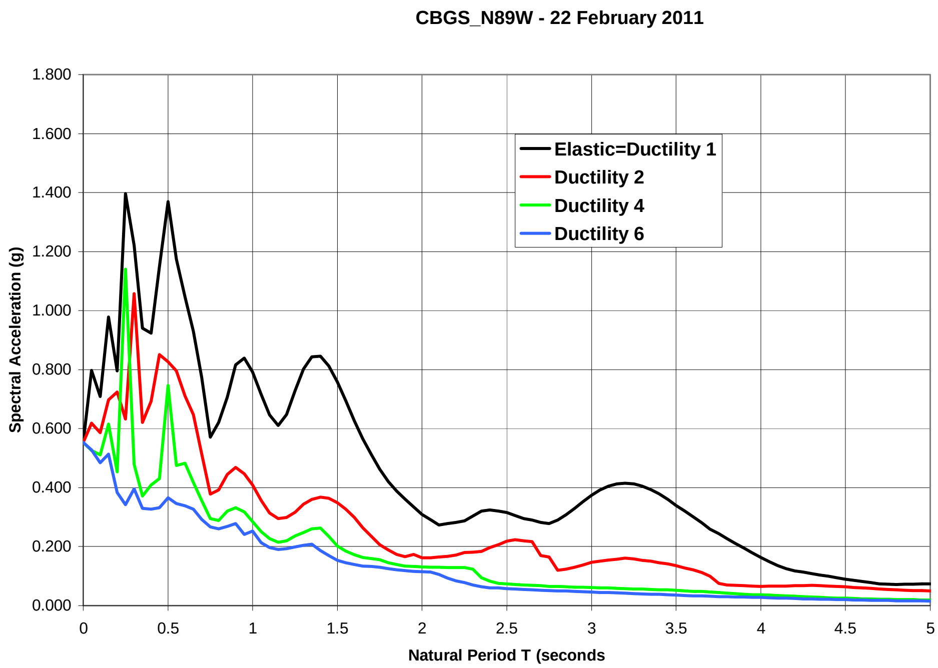


Figure C-11. Acceleration Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component

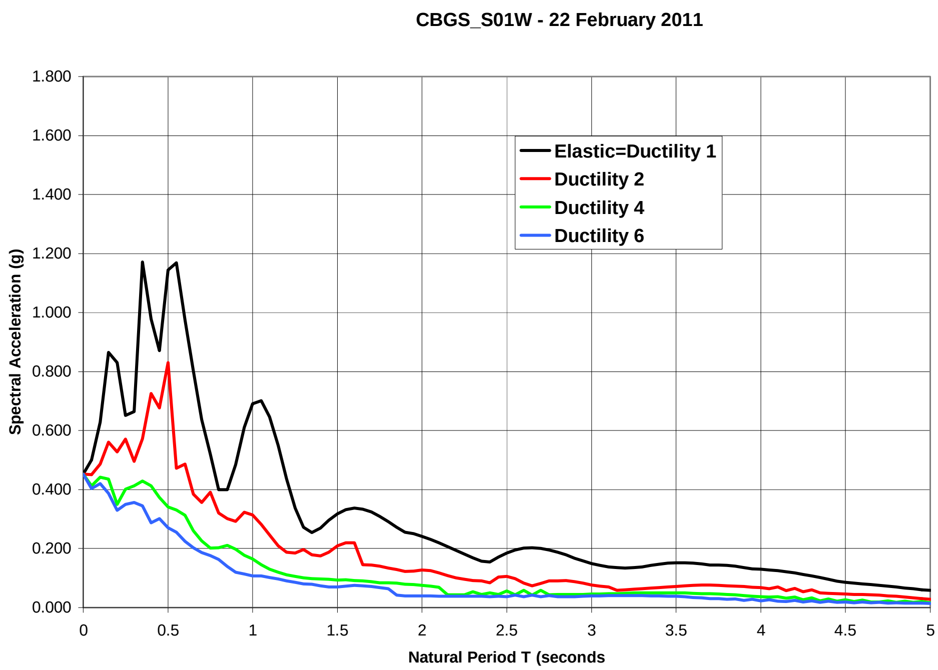


Figure C-12. Acceleration Response Spectra
Christchurch Botanic Gardens. 22 February 2011- South 01° West Component

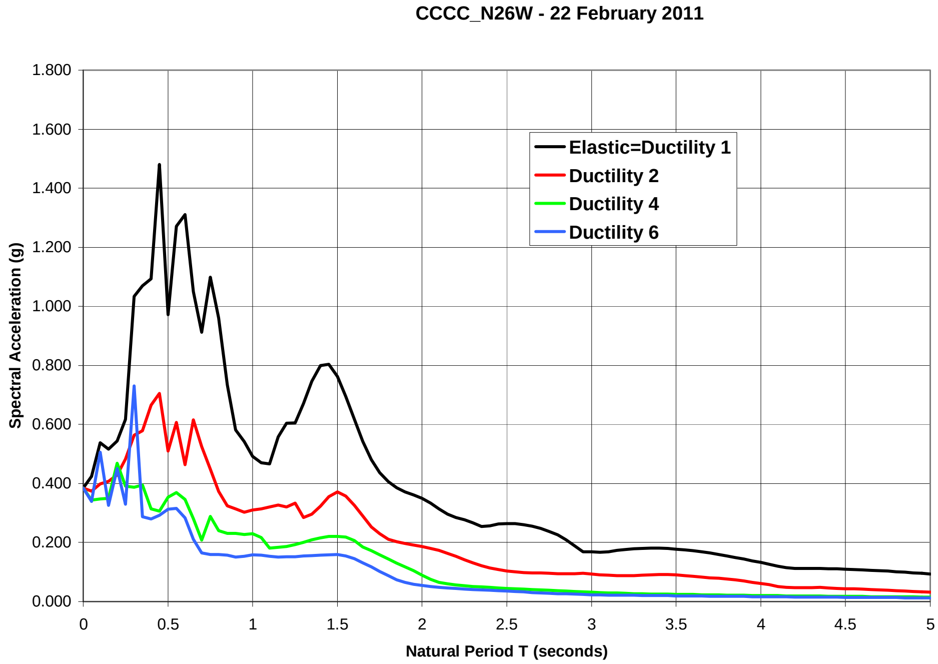


Figure C-13. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- North 26° West Component

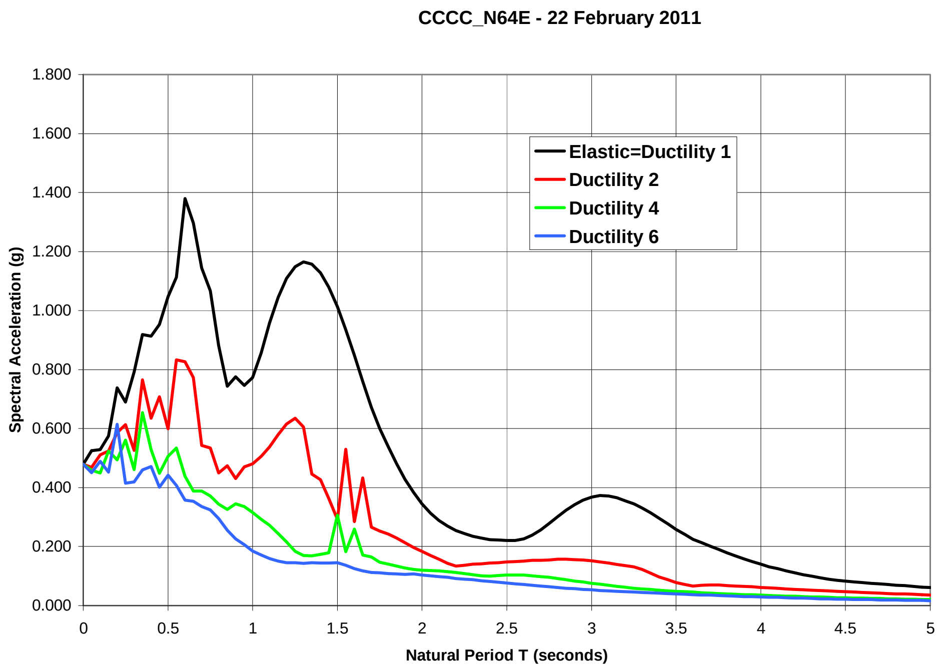


Figure C-14. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component

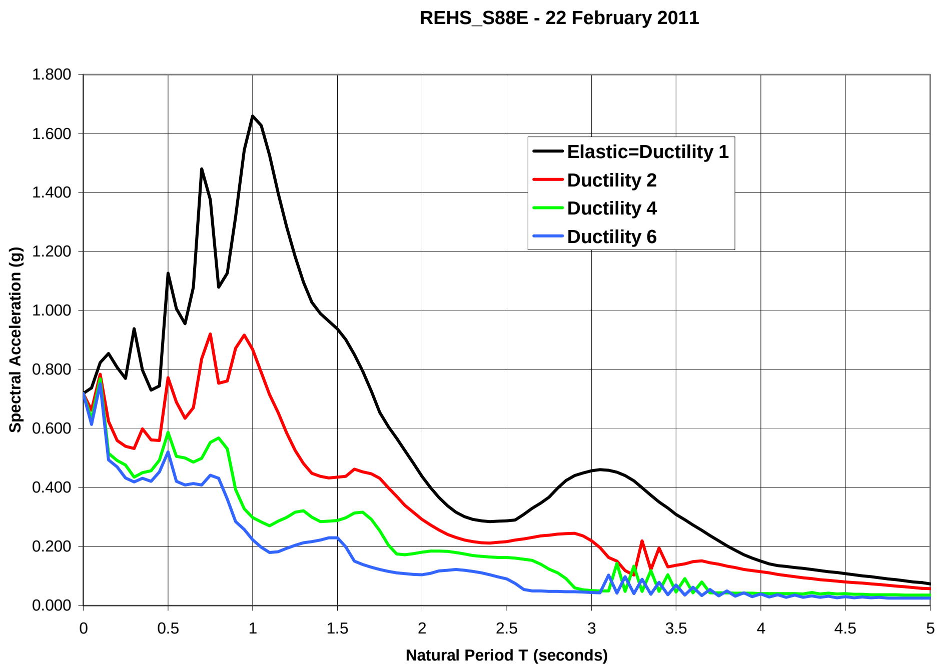


Figure C-15. Acceleration Response Spectra
Resthaven. 22 February 2011- South 88° East Component

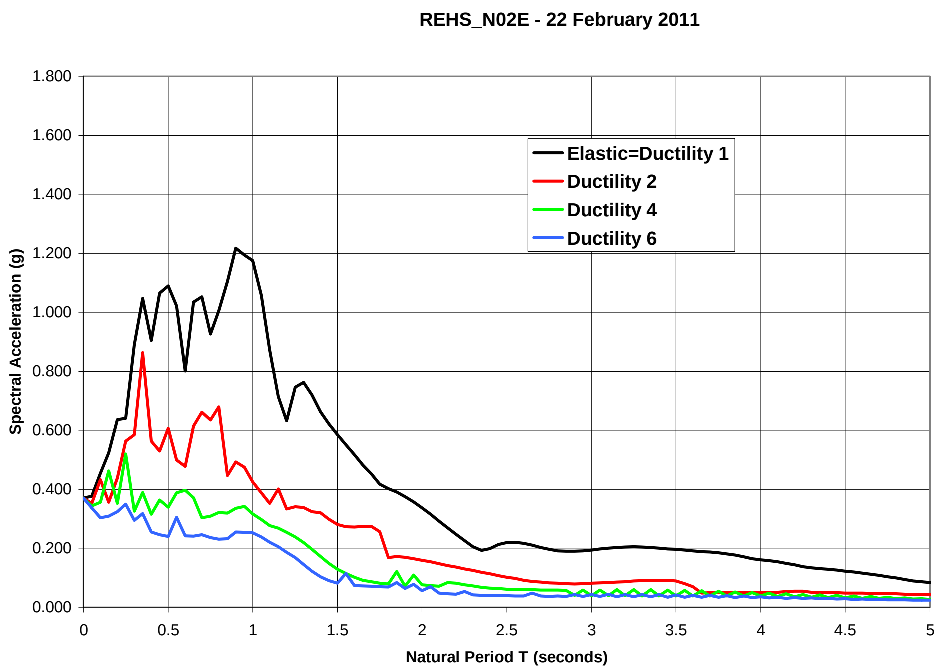


Figure C-16. Acceleration Response Spectra
Resthaven. 22 February 2011- North 02° East Component
Note: Sensitive acceleration responses for high ductility for longer natural periods

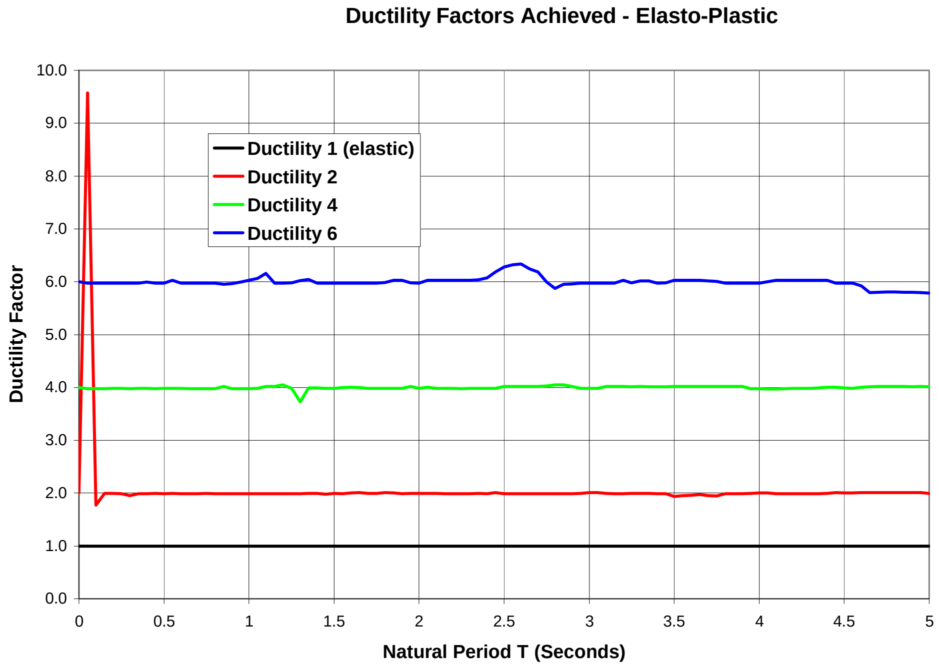


Figure C-17. Achieved versus Target Ductility
Christchurch Hospital 22 February 2011- South 89° West Component

Figure C-17 shows good convergence for most target ductility values except for the first natural period (0.05 seconds) for ductility 2.

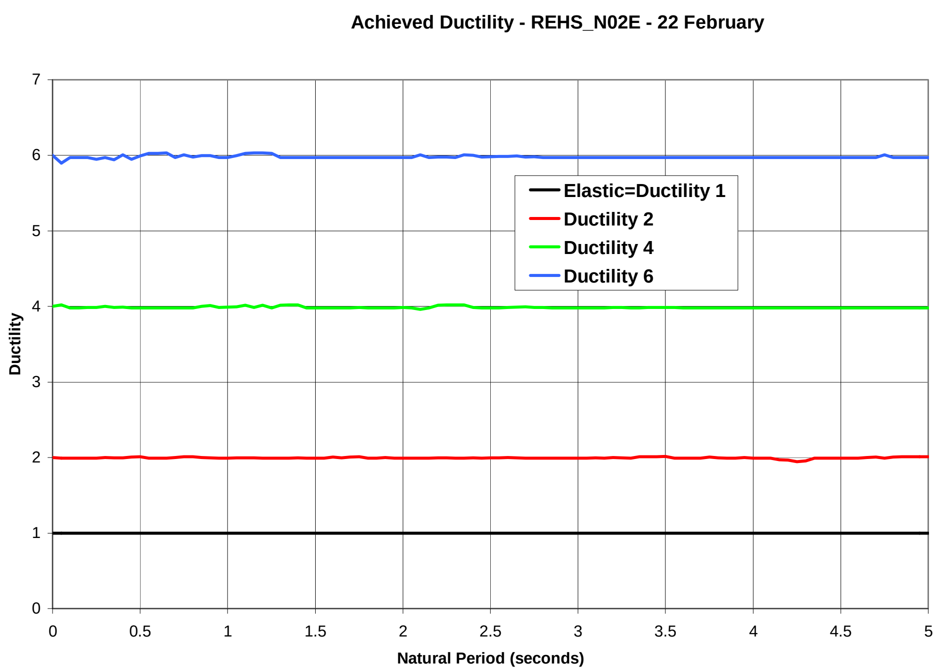


Figure C-18. Achieved versus Target Ductility Ratios
Resthaven. 22 February 2011- North 02° East Component

Figure C-16 shows that although the displacement spectra, and to a lesser extent the acceleration spectra show sensitivity with the Resthaven accelerogram for the larger ductility values the match to the target ductility is very good.

Appendix D
13 June 2011 Earthquake Horizontal Components

The inelastic displacement and acceleration spectra for the 13th June 2011 earthquake components are shown here.

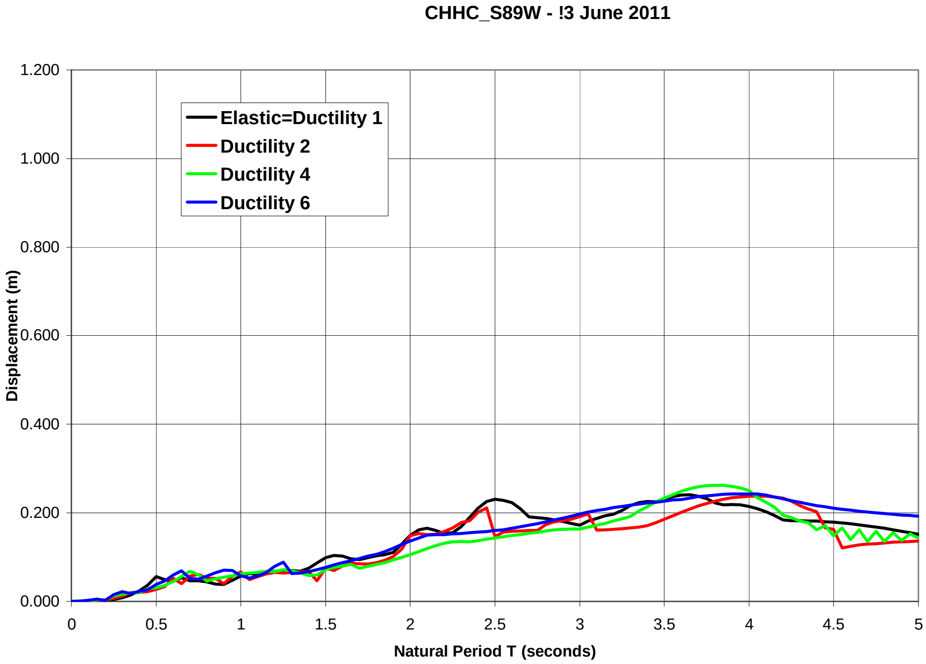


Figure D-1. Displacement Response Spectra
Christchurch Hospital 13 June 2011- South 89° West Component

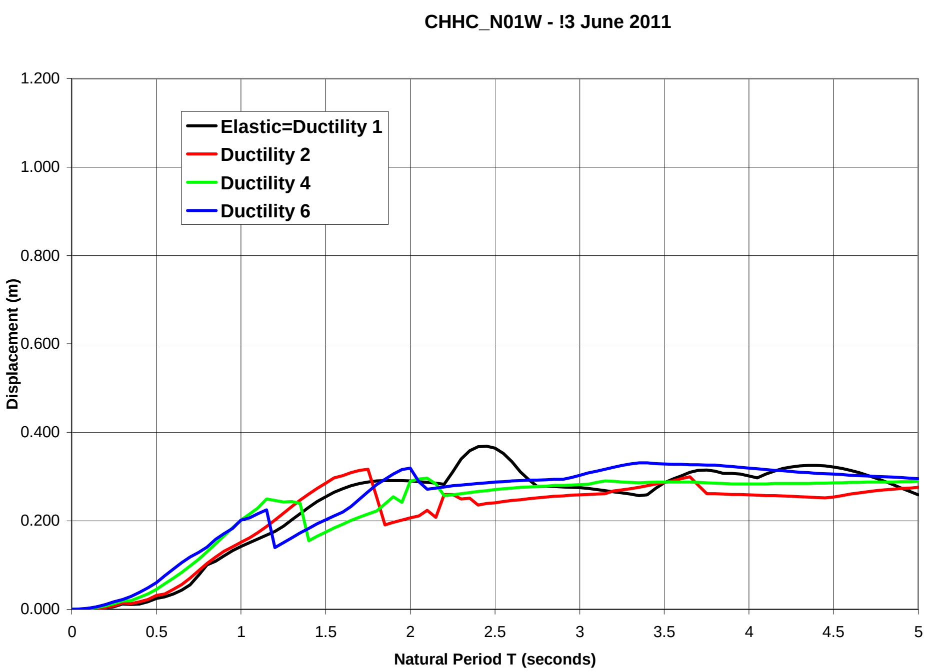


Figure D-2. Displacement Response Spectra
Christchurch Hospital, 13 June 2011- North 01° West Component

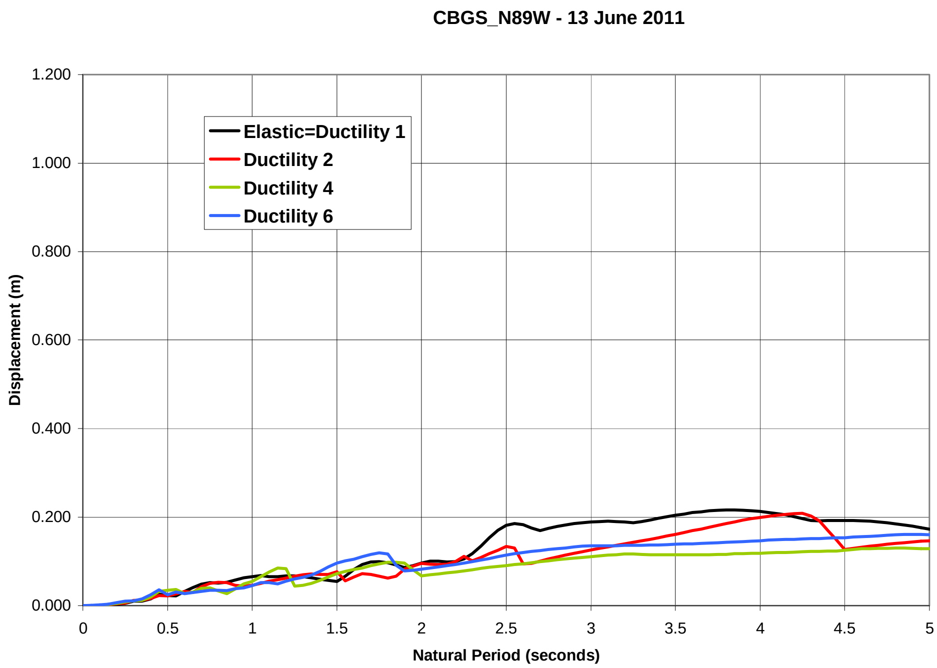


Figure D-3. Displacement Response Spectra
Christchurch Botanic Gardens. 13 June 2011- North 89° West Component

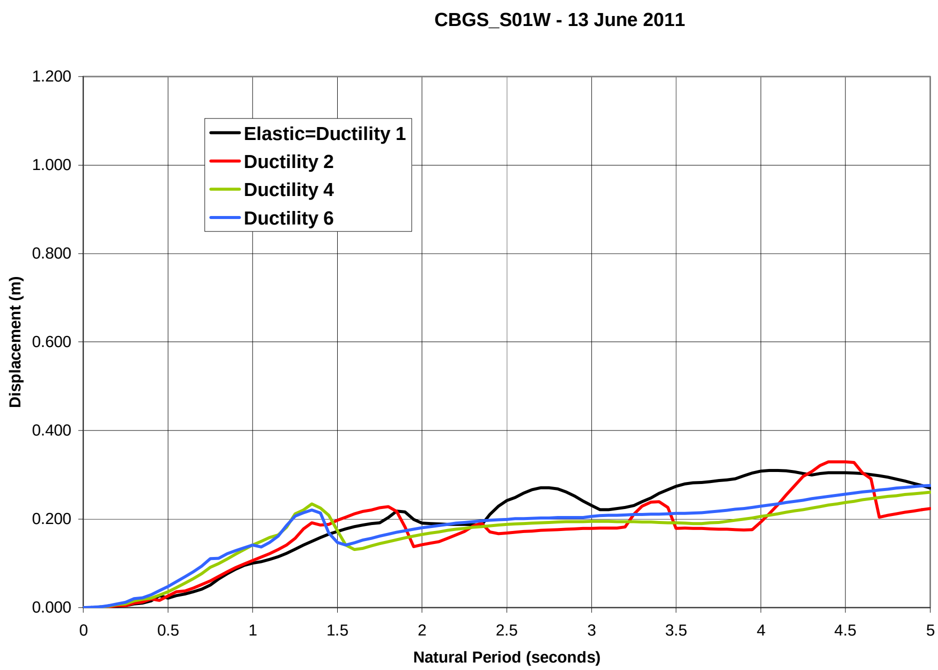


Figure D-4. Displacement Response Spectra
Christchurch Botanic Gardens. 13 June 2011- South 01° West Component

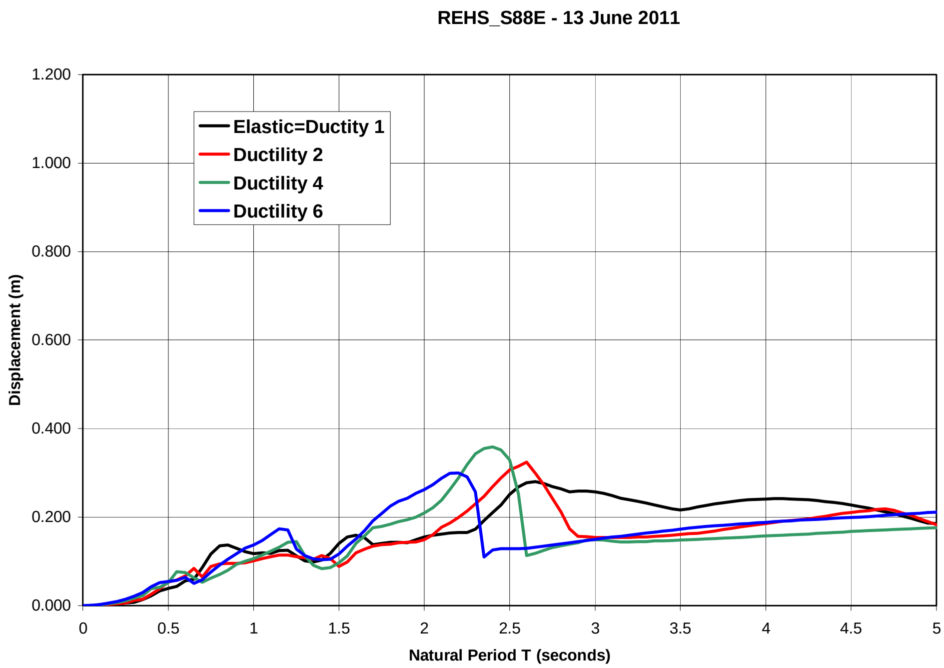


Figure D-5. Displacement Response Spectra
Resthaven. 13 June 2011- South 88° East Component

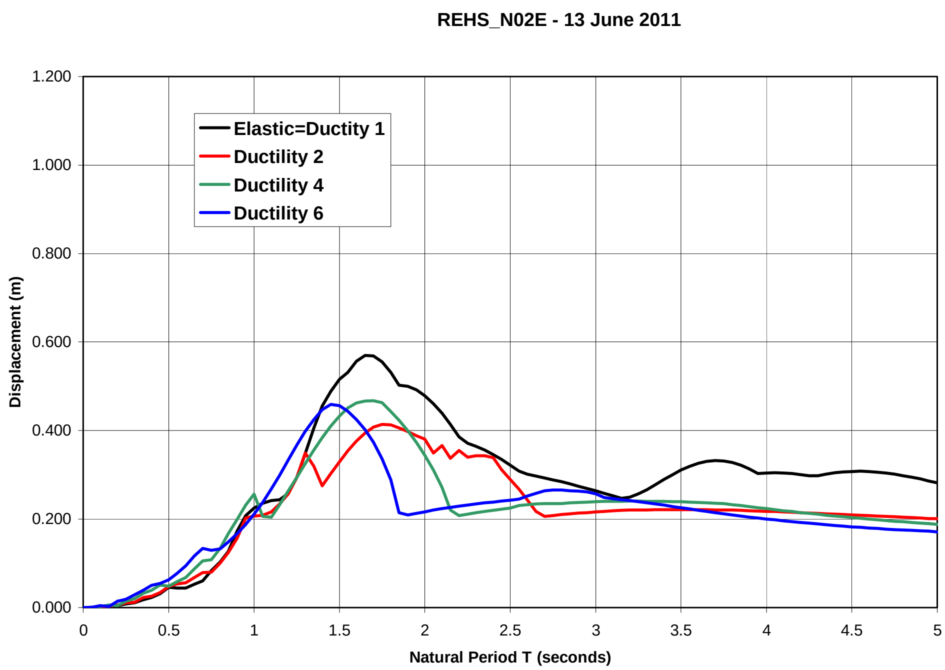


Figure D-6. Displacement Response Spectra
Resthaven. 13 June 2011- North 02° East Component

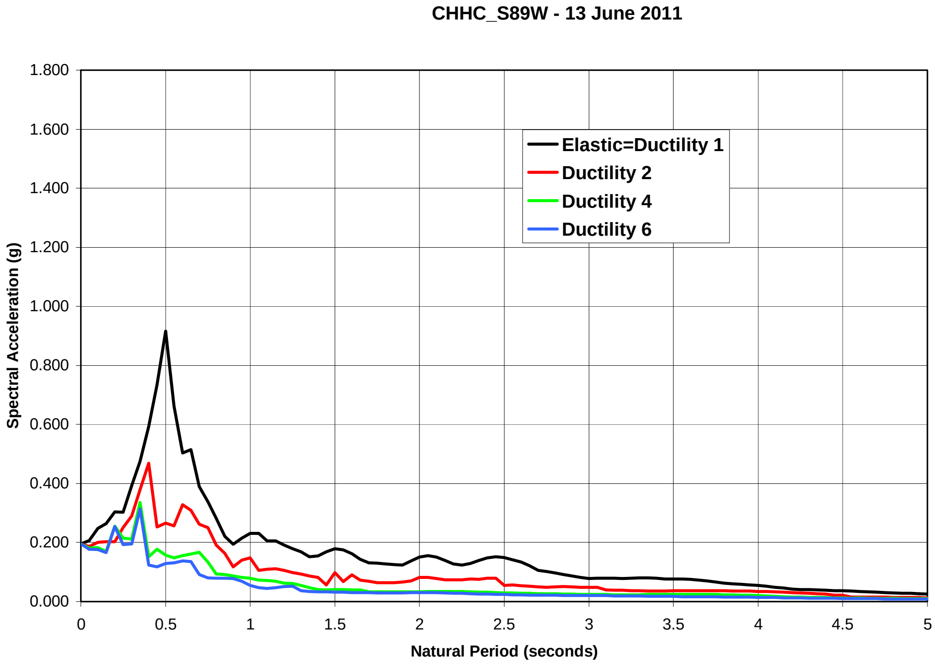


Figure D-7. Acceleration Response Spectra
Christchurch Hospital, 13 June 2011- South 89° West Component

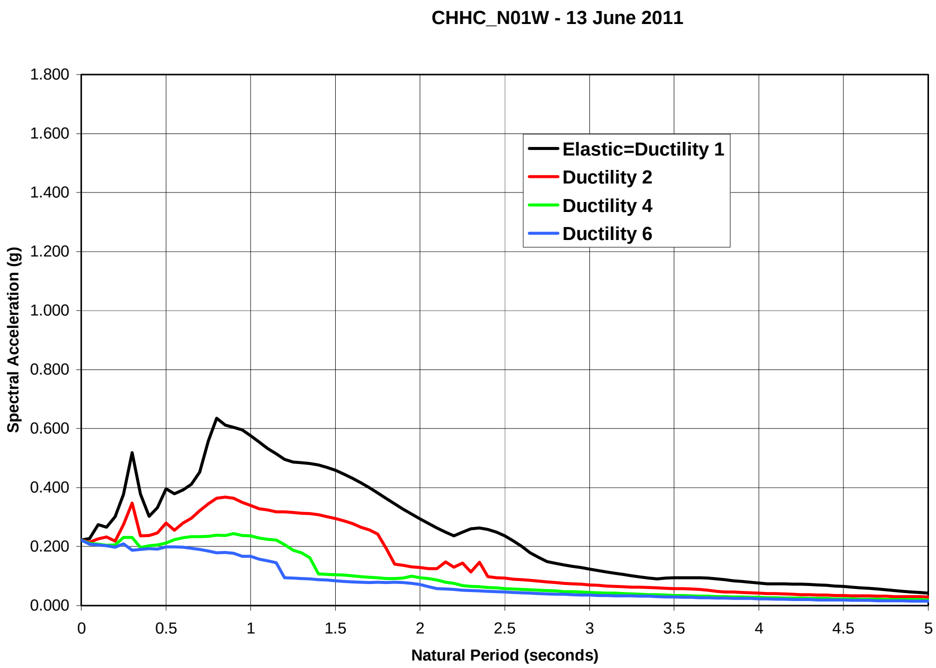


Figure D-8. Acceleration Response Spectra
Christchurch Hospital 13 June 2011- North 01° West Component.

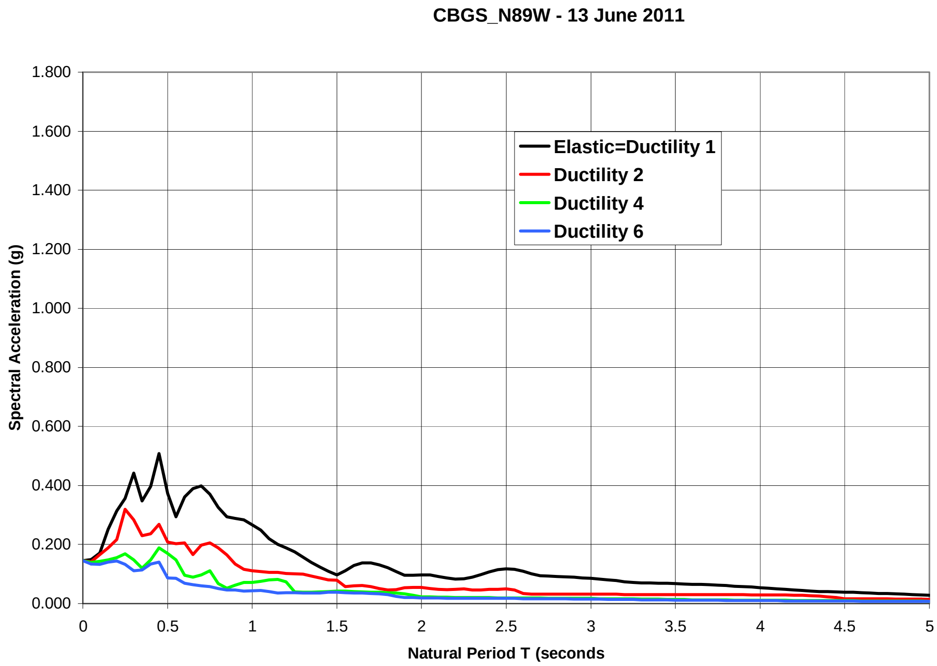


Figure D-9. Acceleration Response Spectra
Christchurch Botanic Gardens. 13 June 2011- North 89° West Component

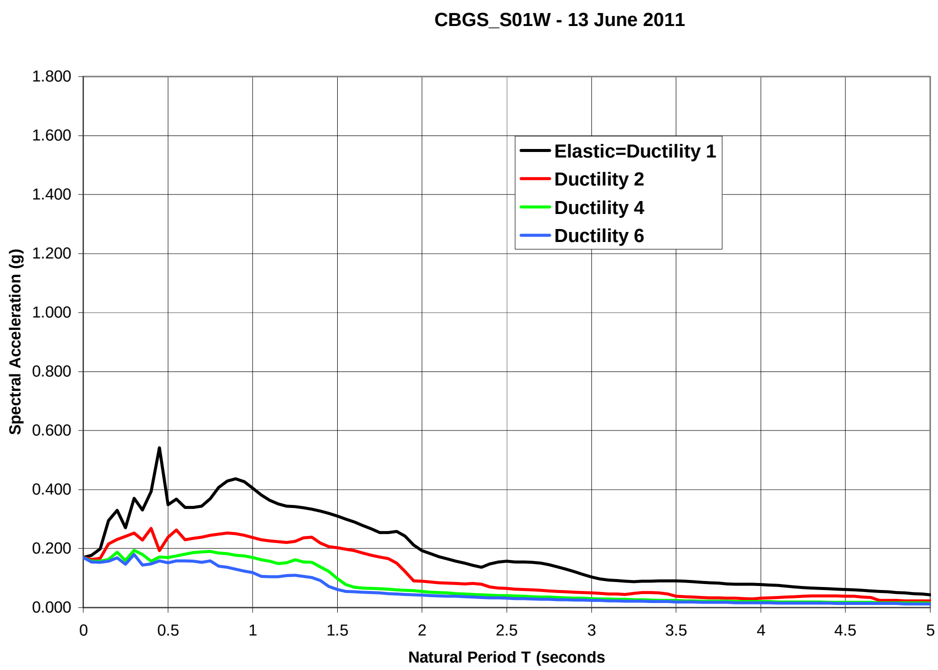


Figure D-10. Acceleration Response Spectra
Christchurch Botanic Gardens. 13 June 2011- South 01° West Component

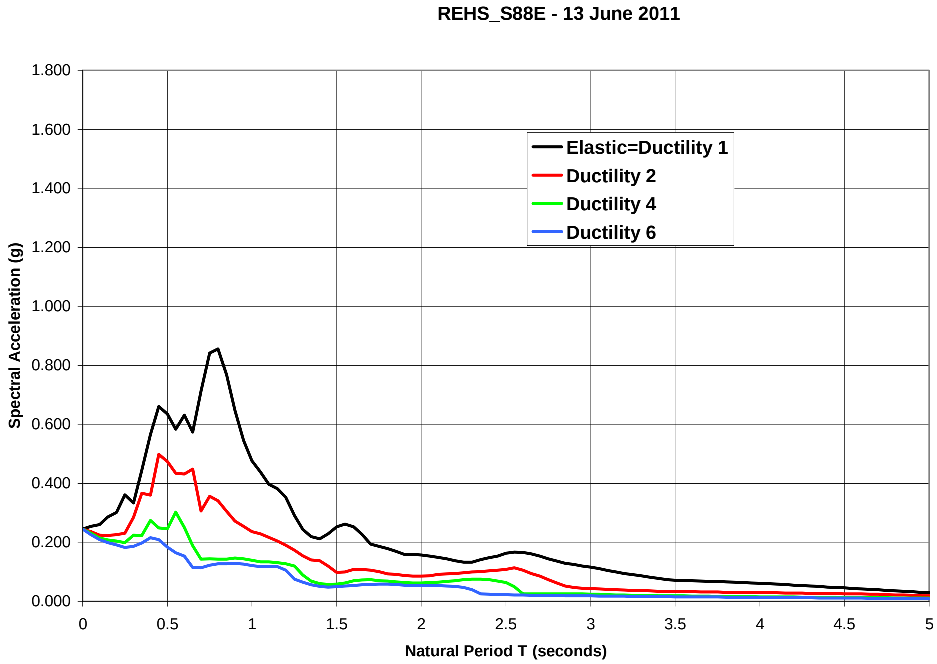


Figure D-11. Acceleration Response Spectra Resthaven. 13 June 2011- South 88° East Component

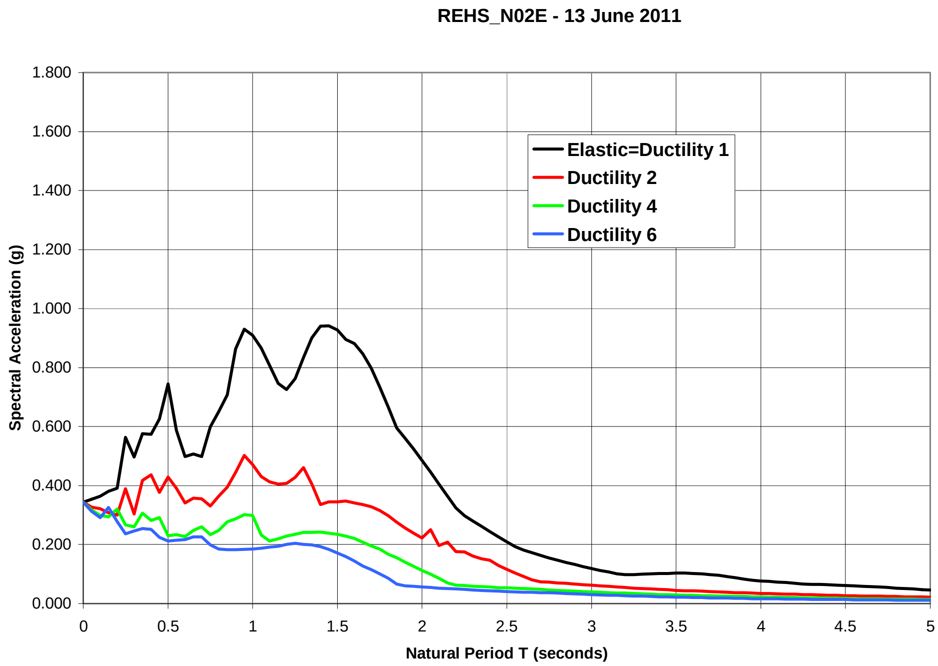


Figure D-12. Acceleration Response Spectra Resthaven. 13 June 2011- North 02° East Component

Appendix E
22 February 2011 Earthquake Horizontal Components – Modified Takeda Hysteresis

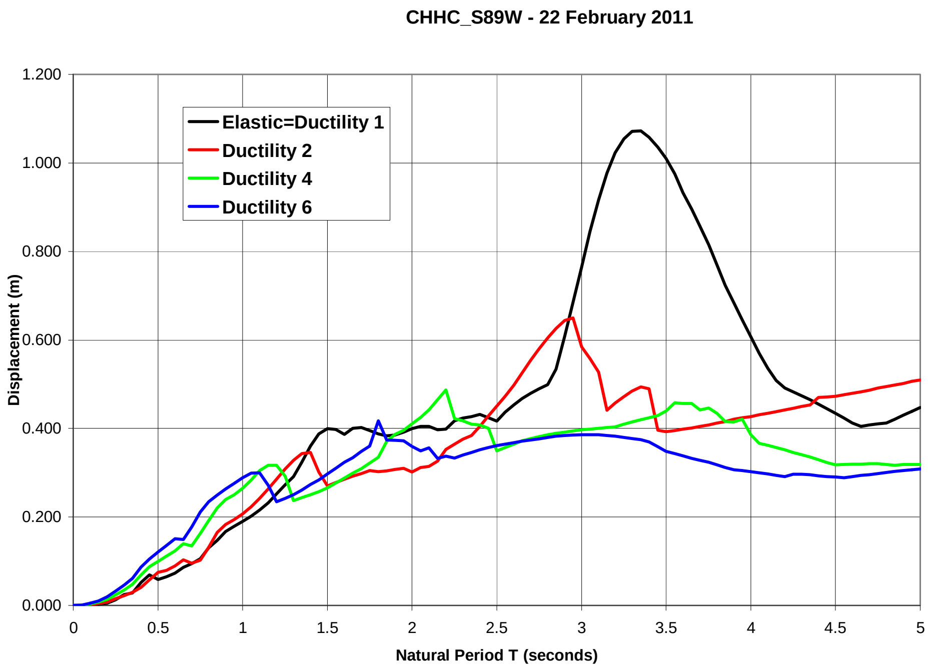


Figure E-1. Displacement Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Modified Takeda Hysteresis

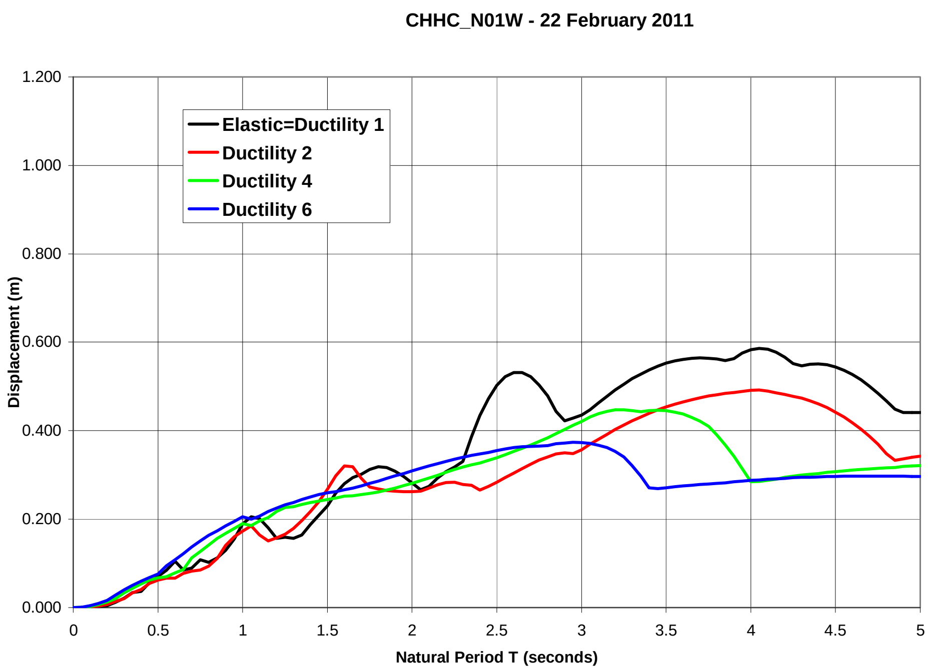


Figure E-2. Displacement Response Spectra
Christchurch Hospital 22 February 2011- North 01° West Component
Modified Takeda Hysteresis

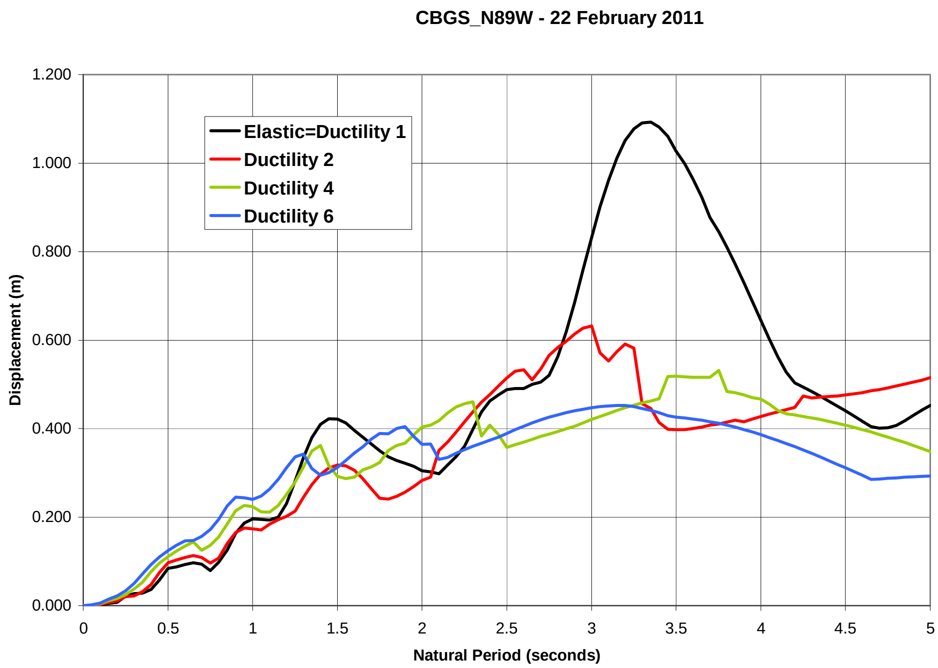


Figure E-3. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component
Modified Takeda Hysteresis

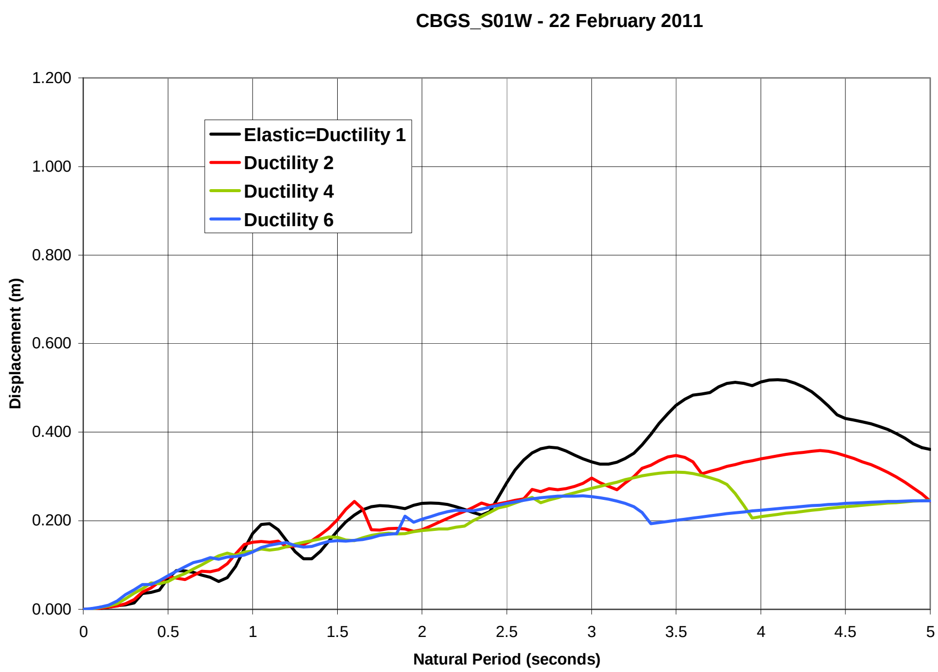


Figure E-4. Displacement Response Spectra
Christchurch Botanic Gardens. 22 February 2011- South 01° West Component
Modified Takeda Hysteresis

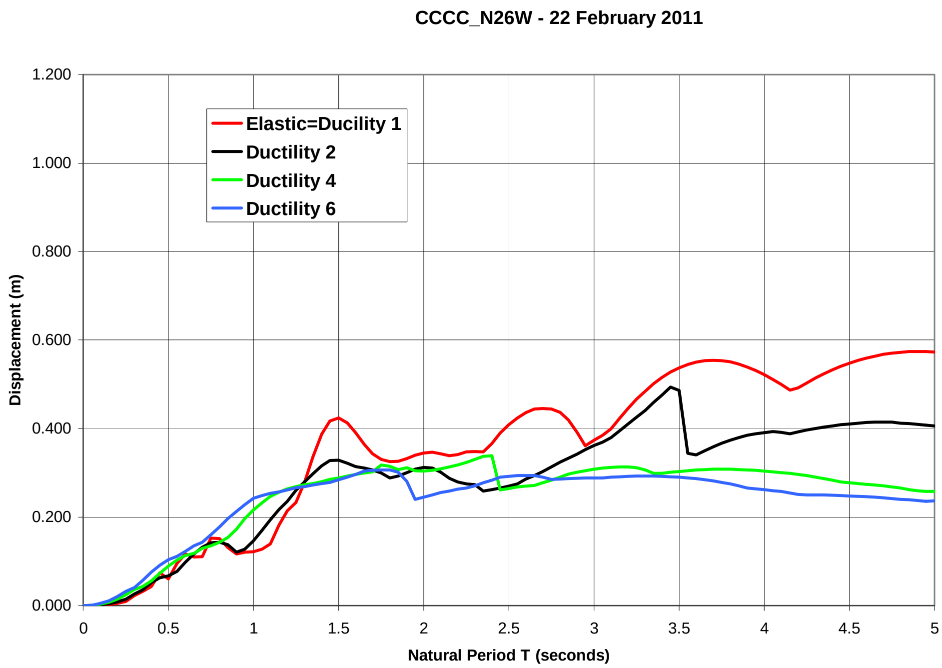


Figure E-5. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- North 26° West Component
Modified Takeda Hysteresis

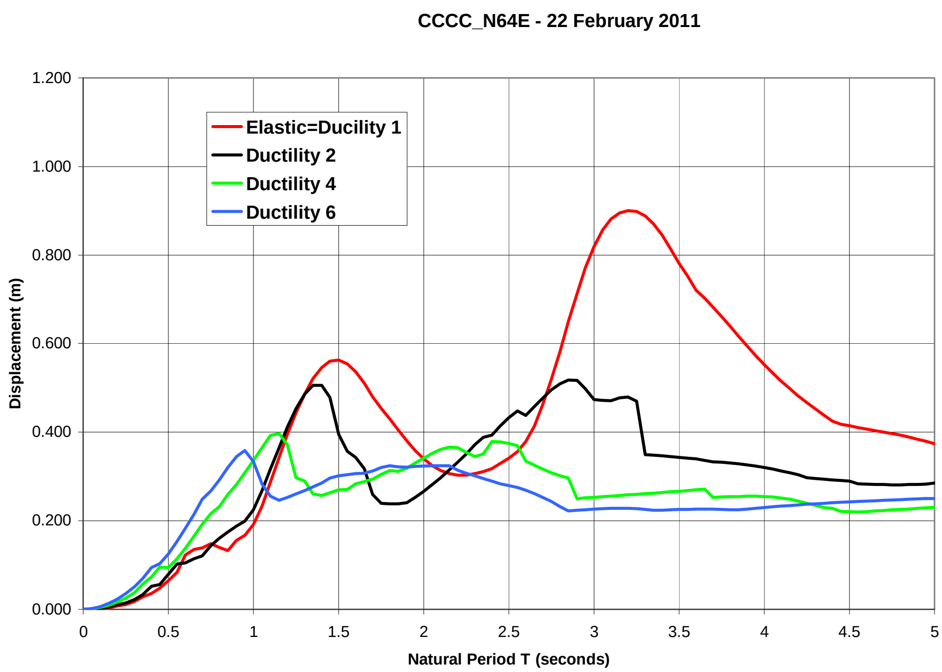


Figure E-6. Displacement Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component
Modified Takeda Hysteresis

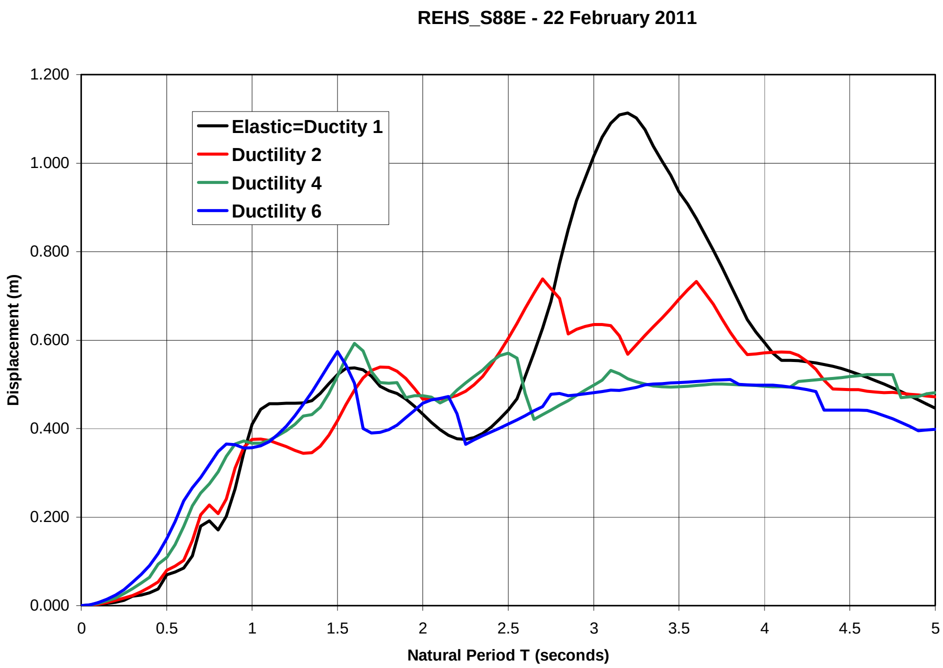


Figure E-7. Displacement Response Spectra
Resthaven. 22 February 2011- South 88° East Component
Modified Takeda Hysteresis

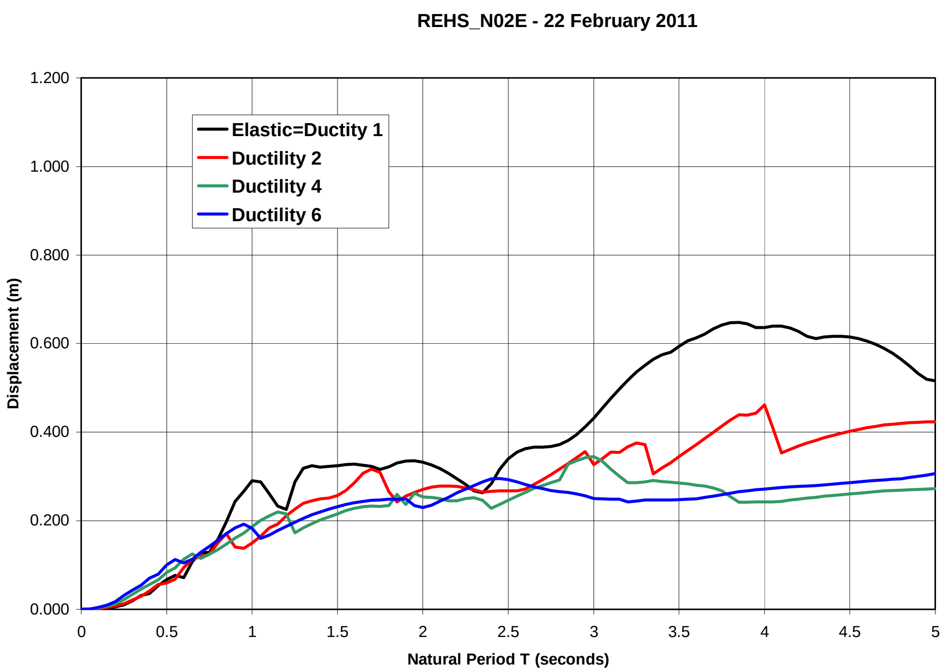


Figure E-8. Displacement Response Spectra
Resthaven. 22 February 2011- North 02° East Component
Modified Takeda Hysteresis

Note: The very sensitive responses for high ductility values for the longer natural periods no longer occur.

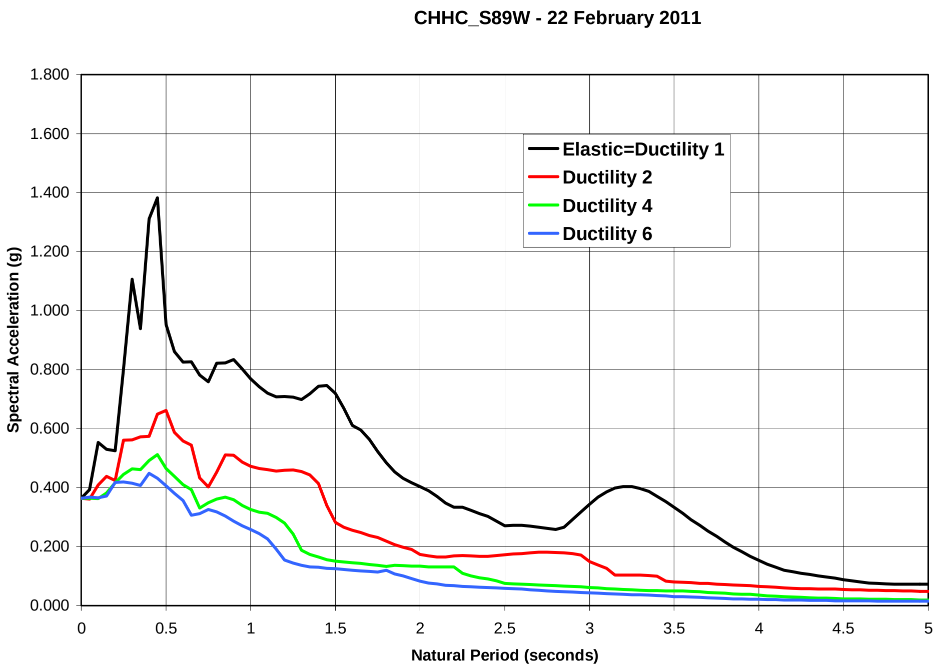


Figure E-9. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- South 89° West Component
Modified Takeda Hysteresis

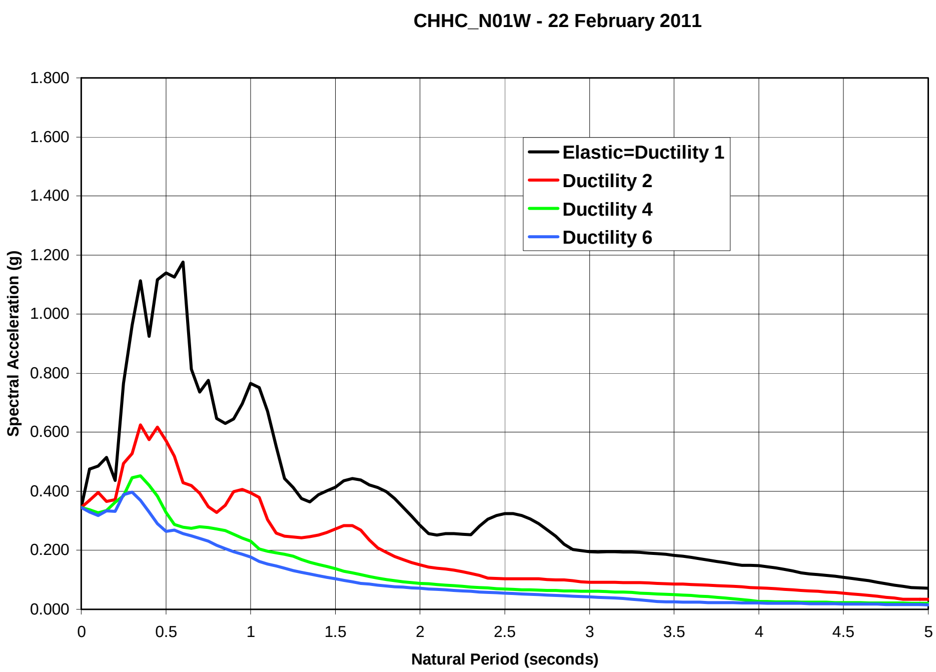


Figure E-10. Acceleration Response Spectra
Christchurch Hospital 22 February 2011- North 01° West Component.
Modified Takeda Hysteresis

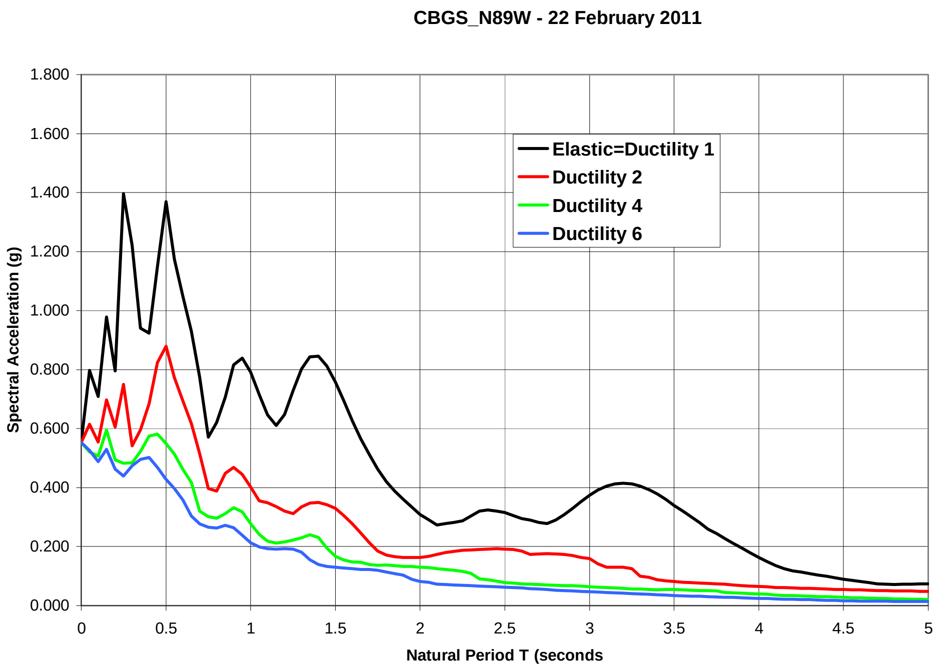


Figure E-11. Acceleration Response Spectra
Christchurch Botanic Gardens. 22 February 2011- North 89° West Component
Modified Takeda Hysteresis

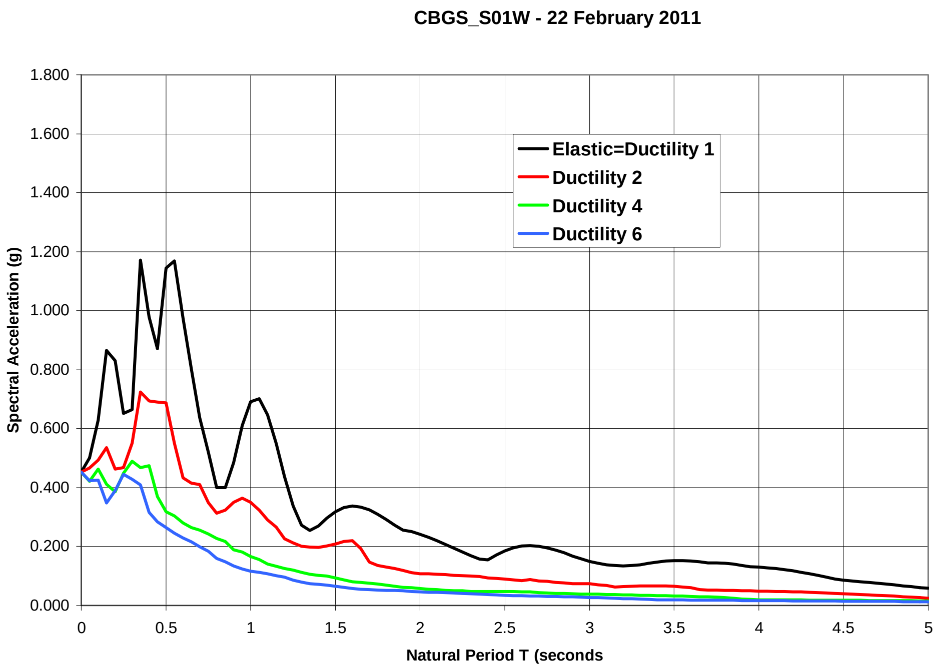


Figure E-12. Acceleration Response Spectra
Christchurch Botanic Gardens. 22 February 2011- South 01° West Component
Modified Takeda Hysteresis

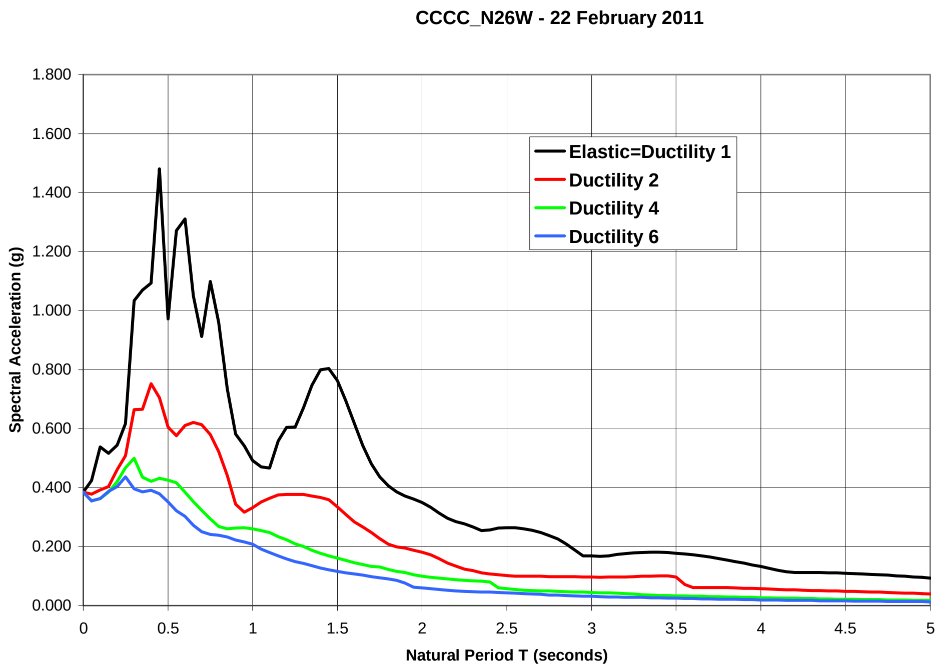


Figure E-13. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- North 26° West Component
Modified Takeda Hysteresis

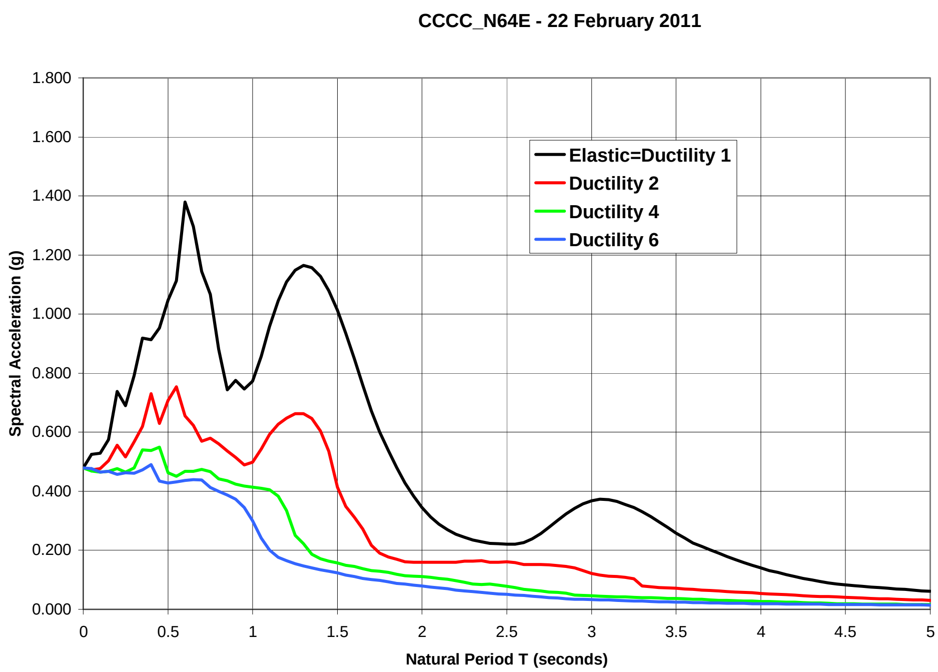


Figure E-14. Acceleration Response Spectra
Christchurch Cathedral College. 22 February 2011- North 64° East Component
Modified Takeda Hysteresis

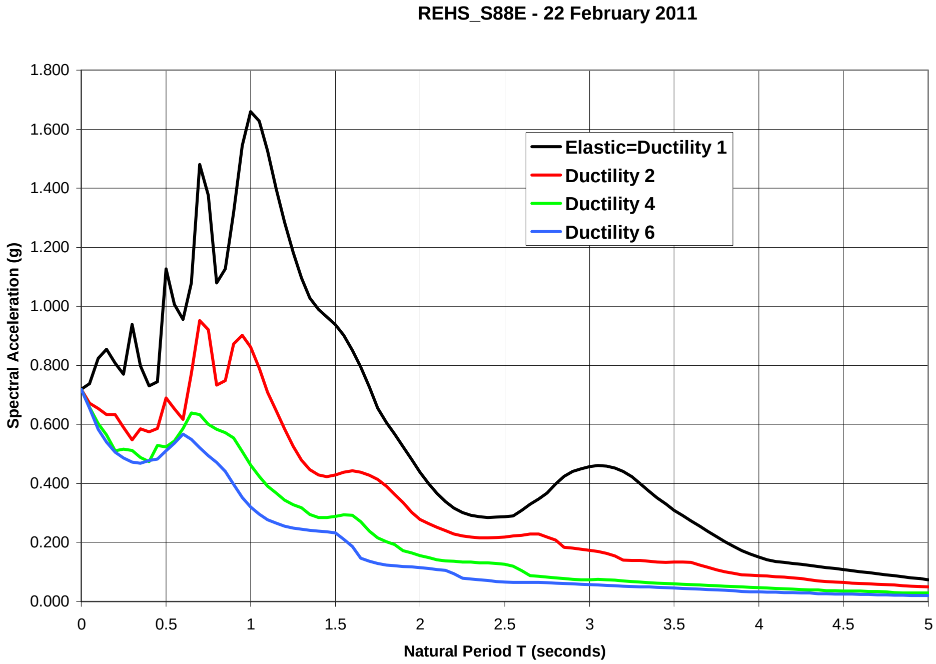


Figure E-15. Acceleration Response Spectra
Resthaven. 22 February 2011- South 88° East Component
Modified Takeda Hysteresis

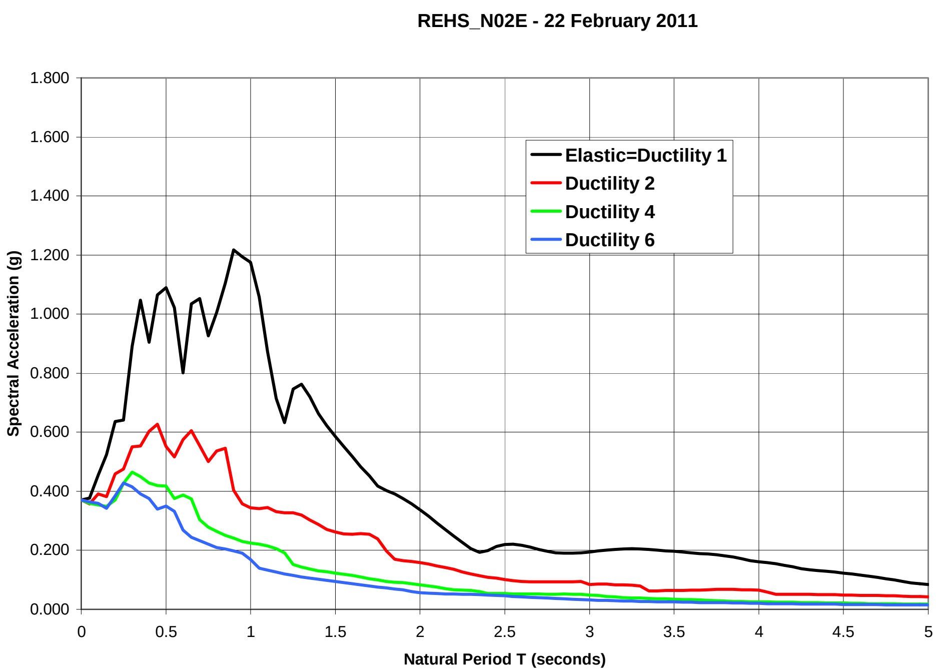


Figure E-16. Acceleration Response Spectra
Resthaven. 22 February 2011- North 02° East Component
Modified Takeda Hysteresis

Note: The very sensitive responses for high ductility values for the longer natural periods no longer occur.

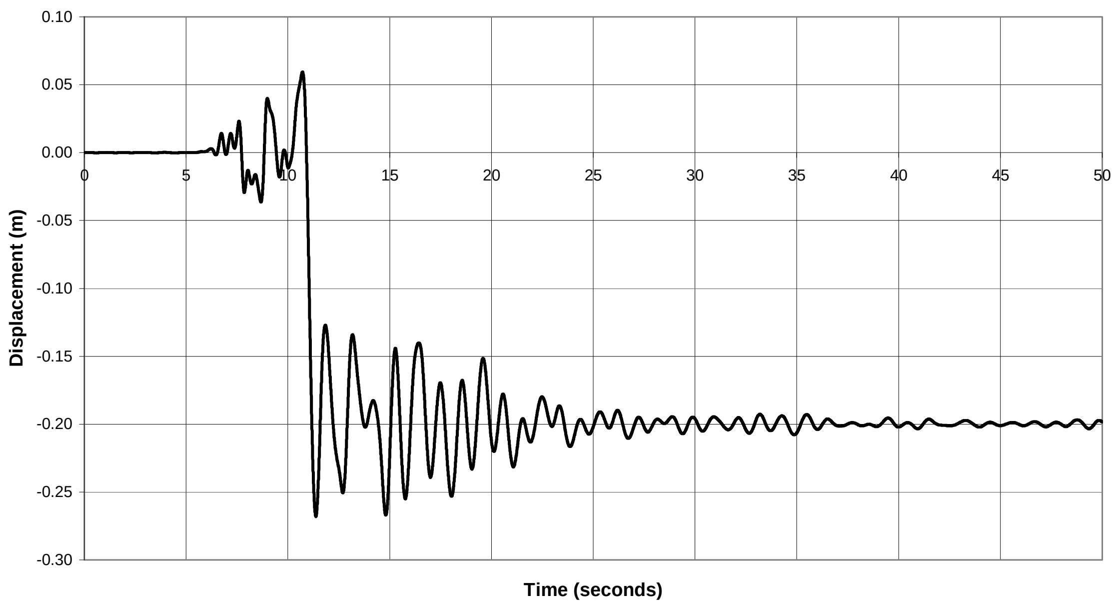
Appendix F. Verification of results – Comparison with Ruaumoko.

To verify that the results obtained with Inspect are correct comparisons were run using the program Ruaumoko2D to compare the computed displacements and accelerations.

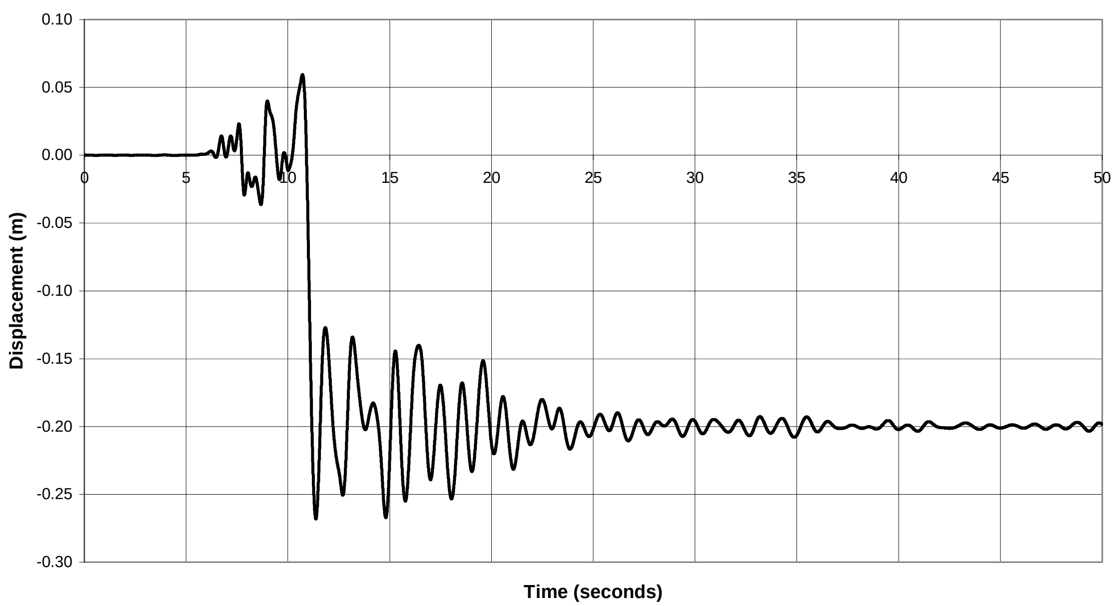
The comparisons reported here are for a Natural Period of Free Vibration of 1.0 seconds and for a target Ductility of 4. The yield force obtained from **Inspect** was used as the input yield force for the spring member in **Ruaumoko2D**. Both programs used the same excitation and the same time-step of 0.0025 seconds.

Displacements

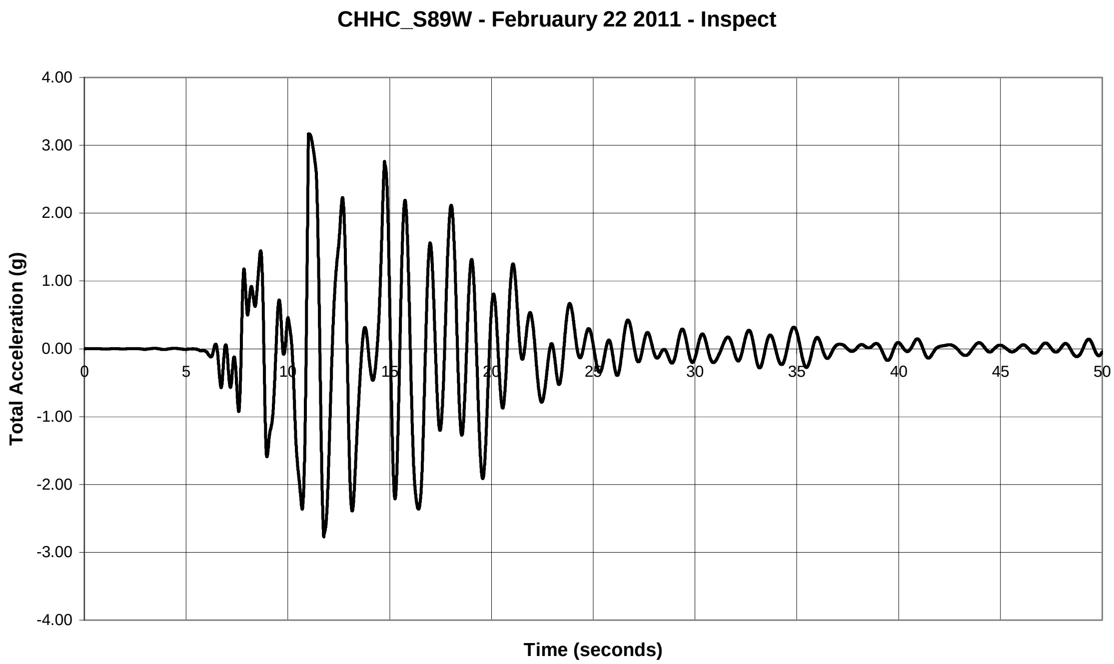
CHHC_S89W - Februaury 22 2011 - Inspect

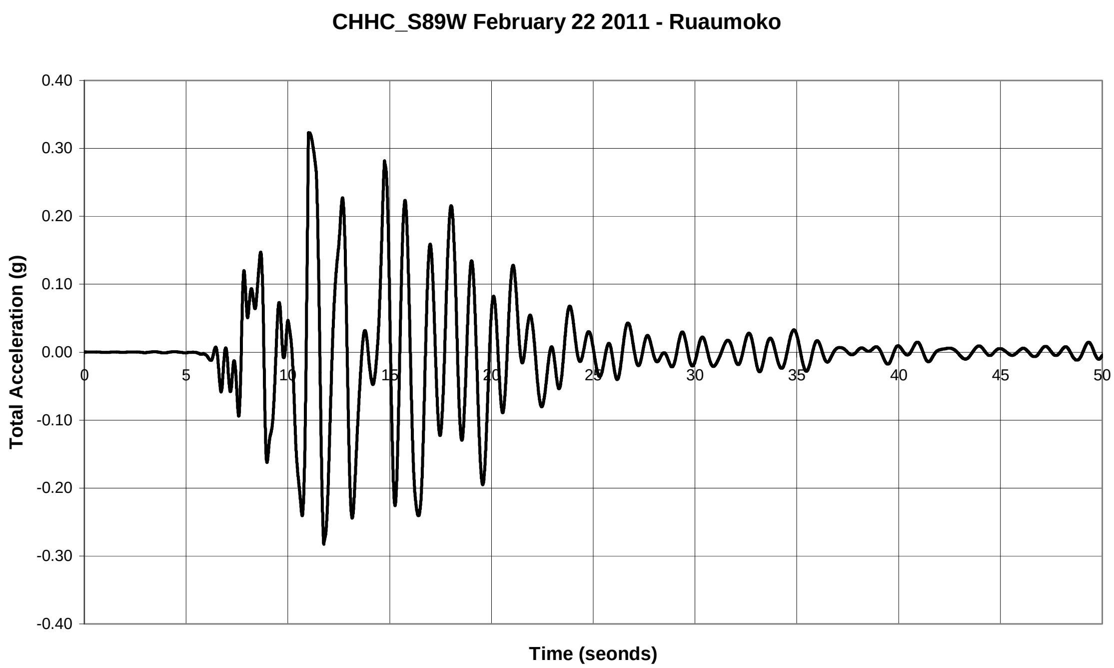


CHHC_S89W February 22 2011 - Ruaumoko

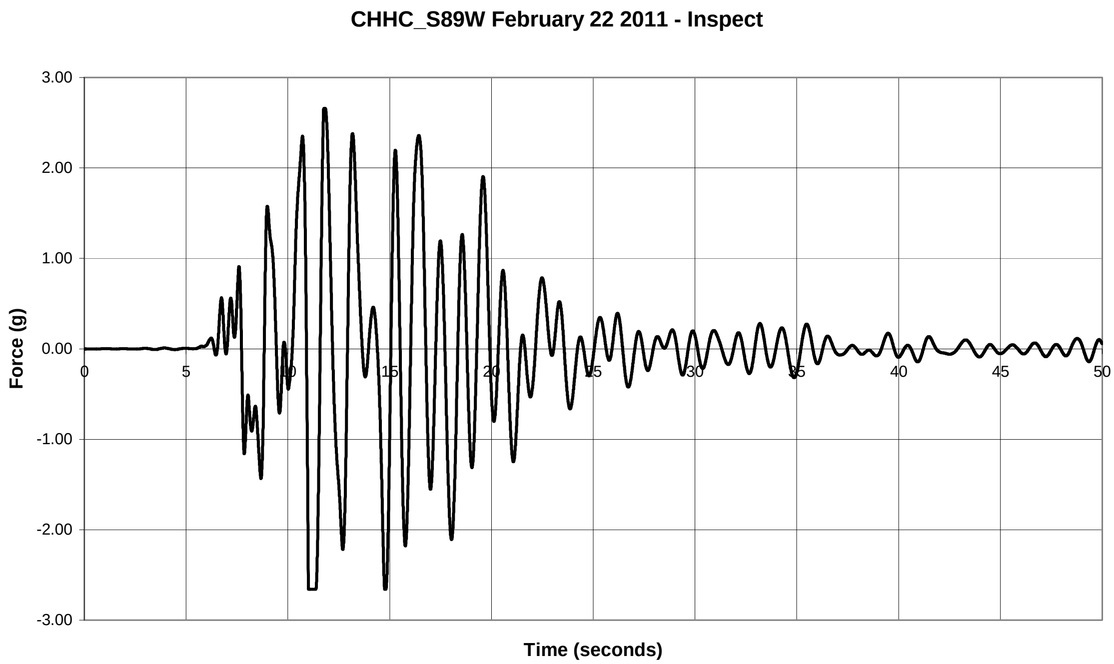


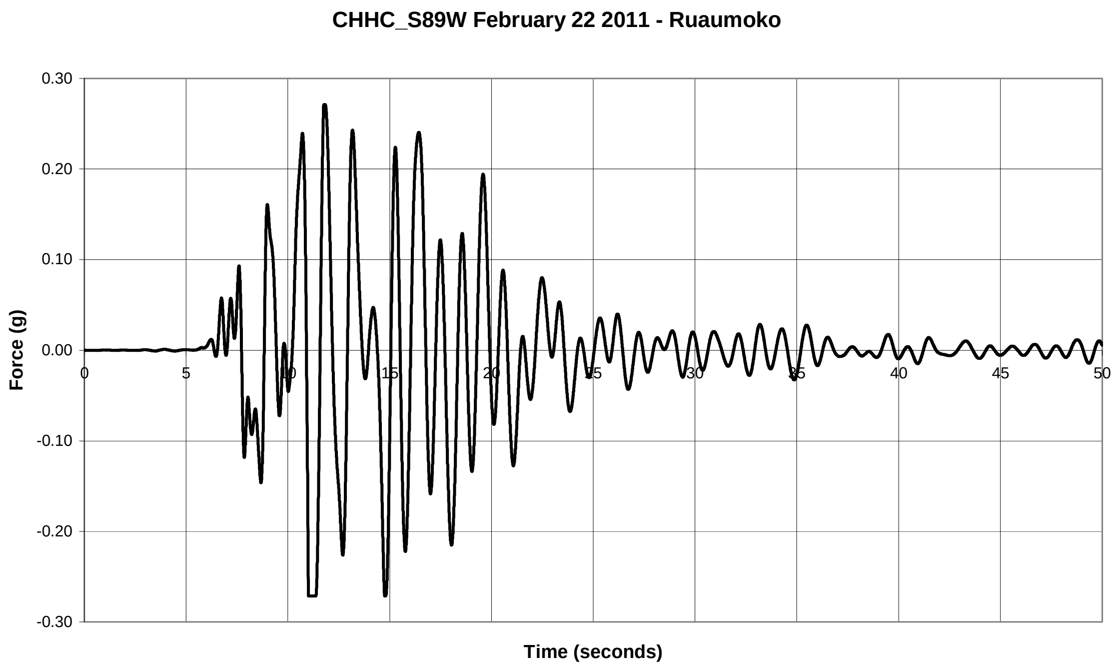
Total Accelerations.



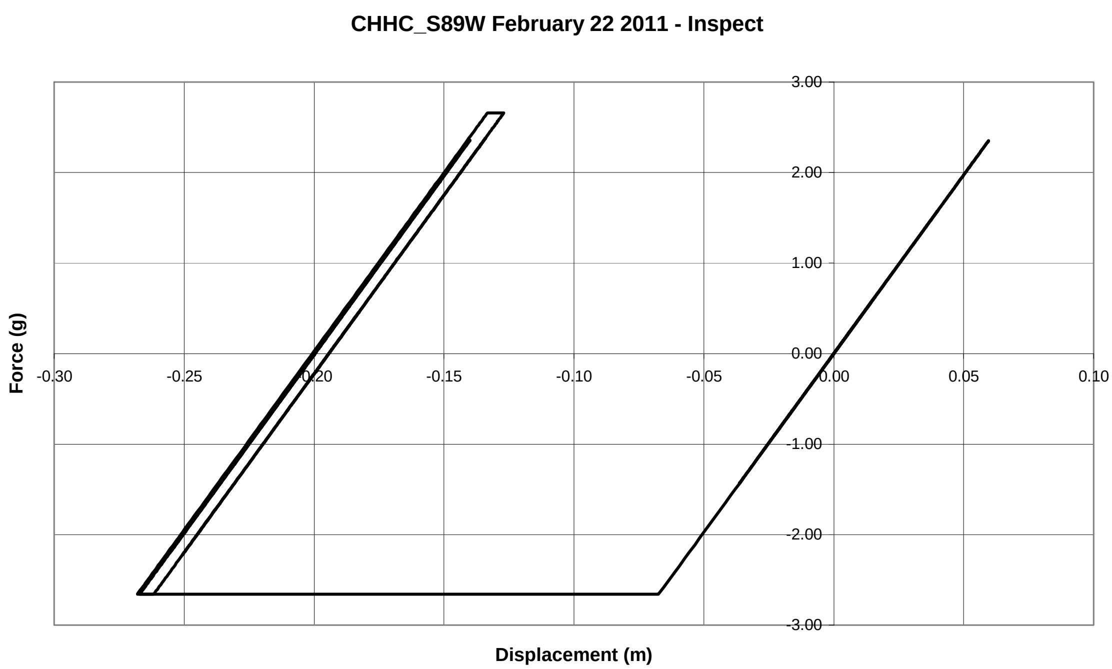


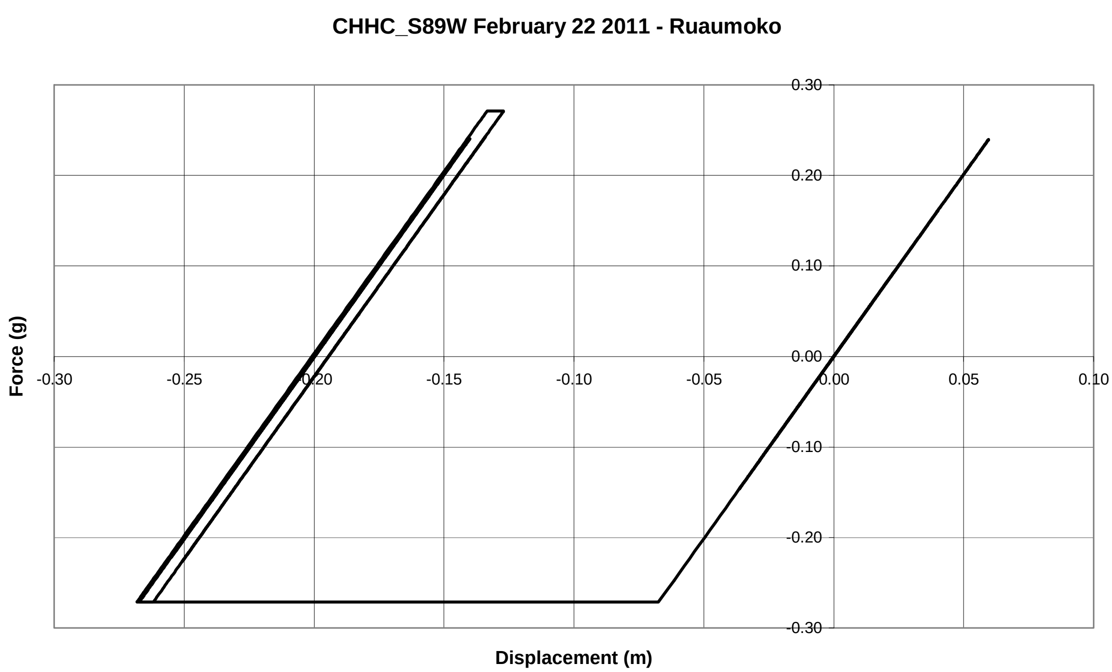
System Member Force.





Hysteretic behaviour.





	Inspect	Ruaumoko
Spectral Displacement	**0.2680**	**0.2681**
Spectral Acceleration (g)	**0.3229**	**0.3229**
Yield Force (g)	**0.2711**	**0.2710**
Ductility Achieved	**3.98**	**3.98**

Comparison of results for the two programs.