Insights into the seismic performance of eccentrically braced framed buildings

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ABSTRACT

The 2010 Christchurch earthquake series allowed insights to be gained into the performance of medium rise eccentrically braced frame ("EBF") buildings up to around 20 storeys high. An advanced elastic-plastic finite element analysis method using ABAQUS software was successfully developed to better assess the amount of cyclic plastic strain accumulated ("PEQ") in the active links of the EBF. The analysis involved sequential imposed displacement analyses using displacement increments derived from an elastic time history analysis of the structure using near-by ground motion records. The ground motion records were selected on the basis of similar ground subsurface conditions to those at the building and were filtered to remove low and high frequency noise. The peak displacements calculated from the elastic time history analyses were compared to the corresponding peak displacement observations in the building at movement joints post-earthquake and the analyses were moderated where needed. Observations made of gypsum board lining cracking and door jamming were also useful in providing additional confirmation to the analytical results. The moderating exercise allowed adjustment to be made for secondary structural elements, local ground conditions and the building's foundation configuration not directly allowed for in the analytical modelling. Broad visual assessments of PEQ in the active links based upon Luders cracking of mill scale was also found to be consistent with the analytical results. The amount of PEQ expected to develop the designed structural ductility and resulting from the application of the design rules in the Steel Structures Standard NZS 3404:1997 was assessed by subjecting the full building model to the same quasi-static cyclic imposed displacement series used for laboratory testing the ductility of structural steel assemblies in New Zealand. The upper limit of PEQ able to be reliably developed before steel rupture by ductile fracture was set using the local strain limits in ASME BPVC VIII.2:2021. Leeb hardness testing results within the active links and

in the adjacent collector beam portions of the members were used to identify comparative pre-earthquake and post-earthquake yield stress in the active links as starting points for the elastic-plastic analyses. Care was required to ensure that hardness testing results were compared at locations of maximum strain hardening. The elastic-plastic ABAQUS analyses showed that those locations vary with active link length, depth and web stiffener configuration. Difficulties were found in using the Leeb hardness testing results to infer PEQ due to this and also to the effect of strain-aging. The PEQ in the active links assessed analytically was also used to estimate the changes in Charpy V-Notch impact energy (“CVN”) and the steel’s changed susceptibility to brittle modes of fracture. The changes indicated were consistent with a fracture that appeared to have occurred in an after-shock some months after the main events.

1 INTRODUCTION

This paper describes the methods developed to assess residual performance of EBF designed in accordance with the New Zealand Steel Structures Standard NZS3404:1997 and to loadings from the New Zealand Loadings Standard NZS 1170.5:2004 after significant seismic events. The method uses accumulated cyclic plastic strain or equivalent plastic strain PEQ in the active links as the primary indicator of residual performance as also used in ASME BPVC VIII.2:2021.

2 EXPECTED PERFORMANCE OF EBF IN AN EARTHQUAKE?

The design of EBF in NZS3404:1997 is based on a capacity or failure hierarchy being developed in which the EBF is configured so that the active link or shear link (Figure 1) becomes the weakest but ductile link. Therefore cyclic strain is focussed into the active links and the other members are protected, particularly the gravity load bearing columns and remain largely elastic in a large earthquake.

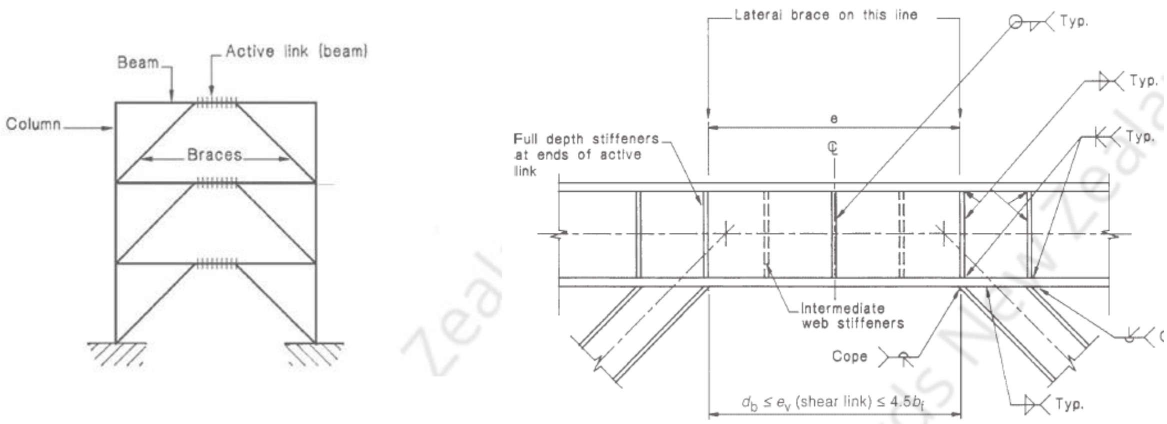

Figure 1 EBF member terminology from NZS3404:1997. The Beam is also referred to as a “Collector Beam”

Response Spectrum Analysis (“RSA”) is commonly used to assess the design level demands of the active links so that they may be sized to yield at design level actions set by the New Zealand Loadings Standard NZS 1170.5:2004. These are loadings which have been factored down from the expected elastic response of the structure to give benefit for the specified ductility that the EBF is designed for and expected to be able to accommodate after the active links yield.

The collector beams, braces connected alongside the active links (Figure 1) and the adjacent columns of the EBF are then sized using the actions resulting from the RSA, factored upwards to account for the increase in demands on them due to strain hardening and grade variability in the active links.

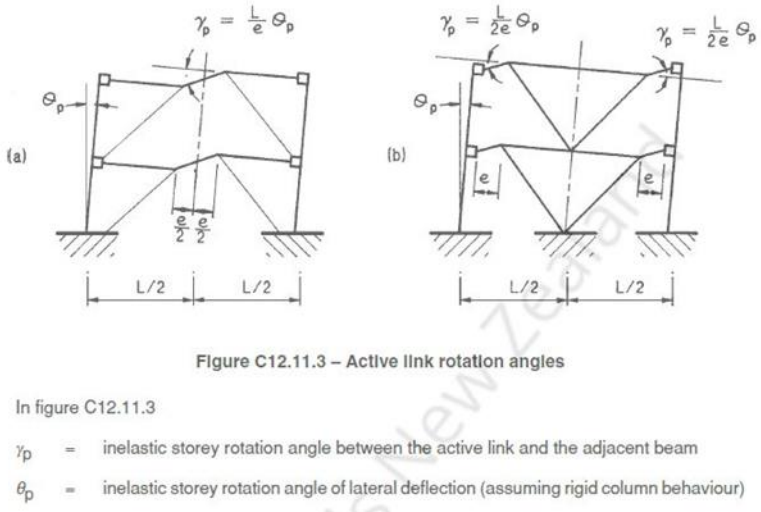

Figure 2 Idealised deformation of EBF from NZS3404:1997 when laterally deformed by earthquake loads. The active links deform and rotate inelastically developing plastic strains while the braces, collector beams and columns remain elastic.

3 HOW IS COMPLIANCE WITH B1 ACHIEVED POST-EARTHQUAKE?

The primary issue therefore in terms of assessing the residual ability of an EBF to perform in compliance with the New Zealand Building Code Clause B1 Structure after an earthquake, is largely whether the level of PEQ that was developed in the active links has caused them to become unacceptably susceptible to rupture by either brittle or ductile fracture under a further set of NZS 1170.5:2004 seismic loading demands.

To assess this the PEQ developed in the active links by the prior earthquakes needs to be assessed along with their ability to accumulate the additional PEQ necessary to satisfy a further set of NZS1170.5:2004 cyclic displacement demands.

The PEQ expected to be developed in EBF designed to comply with NZS3404 is discussed in Section 4.

The upper limit for PEQ is discussed in Section 5.

An assessment of what PEQ has occurred may be made by visual observation methods such as classifying levels of mill scale cracking on unpainted steel within the active links, where those observations are possible. Leeb hardness testing may be used to assist that with care as discussed in Section 6.

An analytical method to assess PEQ after an earthquake is described in Section 7.

4 PEQ EXPECTED IN NZS3404:1997 COMPLIANT EBF

The level of PEQ expected to be developed in active links and associated limits for PEQ are not explicitly stated in the Steel Structures Standard NZS 3404:1997. Determining what that desired PEQ ability is in active links can be reasonably done with reference to how steel structures have been assessed to comply with the specified ductility levels in the Steel Structures Standard. That is generally considered to have been by applying a quasi-static cyclic displacement test sequence described by MacRae (1990) to structural assemblies ("NZ Loading History") (Figure 3).

The NZ Loadings History displacement sequence may reasonably be applied virtually to a full elastic-plastic model of the structure in ABAQUS to simulate the same thing in an equivalent manner.

The displacements are imposed at the centre of mass of the full structure 3D model so that the effects of torsional eccentricity to the centre of stiffness may be incorporated. The expected strain accumulation demands can be assessed by applying the NZ Loading History ductility factors for each loading cycle to the first yield displacement profile of the building.

The first yield displacement profile of the centres of mass of each floor may be determined using RSA displacements at first yield of the first active link and the resulting PEQ determined for the relevant design ductility level at completion of the relevant displacement cycles.

There is no requirement implied in NZS3404:1997 for a structural assembly to have to sustain the cyclic displacement demands of the NZ Loading History sequence associated with its specified ductility more than once. So by extension there is a reasonable expectation for a structure consisting of assemblies tested that way by simulation analysis to be able to sustain that sequence at least once in achieving its specified ductility. Though it may be able to sustain more displacement cycles than that.

Therefore it seems reasonable that if the active links in the structure can sustain the additional amount of PEQ developed from application of a second sequence of NZ Loading History imposed displacements in addition to the PEQ developed during the prior earthquakes then the EBF may be considered to remain compliant with NZS3404:1997, NZS 1170.5:2004 and therefore the Verification Method cited by Clause B1 Structure of NZBC based upon an Alternative Solution approach.

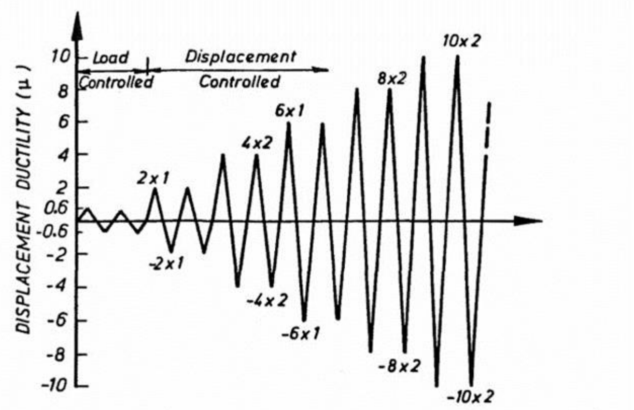

Figure 6.23. Loading Regime for Testing of Specimens

Figure 3 NZ Loading History regime described by MacRae 1990

5 PEQ LIMITS FOR ACTIVE LINKS

The upper limit for PEQ in the links before rupture due to ductile fracture occurring may be set by reference to the limits set in ASME BPVC VIII.2:2021.

The limiting PEQ before rupture due to brittle fracture occurs may be made by reference to BS7910:2013+A1:2015 Annex J. The expected change in the 27J temperature in the Charpy V-Notch impact energy transition curves that occur with strain hardening and aging may be estimated with reference to Hyland (2008).

6 ISSUES WITH ESTIMATING PEQ USING LEEB HARDNESS TESTING

Hardness testing such as by Leeb hardness testers can be useful but must be treated with caution. While hardness testing gives very good indications of the surface yield stress it is not straight forward to develop reliable strain measurements from the readings. This is due to the following factors:

6.1 In Grade variation of yield stress

A wide variation of as-supplied yield stress is allowed within a typical steel grade designation. For example a complying Grade 300 steel may have a yield stress between 280 and 400 MPa and still comply with the material variation factor ϕ_{om} in Table 12.2.8(2) NZS3404:1997. Therefore unless the as-supplied or pre-earthquake yield stress or hardness is known then a hardness reading of 400 MPa may indicate that nothing or a lot has happened to the active link.

6.2 Variable PEQ distribution due to link configuration

The PEQ distribution pattern in active links also varies depending upon whether the link acts in a dominant flexural or shearing mode (NZS3404:1997 Cl 12.11.1), and how many web stiffeners have been installed. Flexural active links tend to have high flange or upper and lower web plastic strains due to flexural effects. Where shorter shear dominated active links tend to have plastic strains focussed at the centre of the web panels (Figure 5).

Therefore Leeb hardness testing needs to be in the locations relevant to the active link configuration. This can best be determined for the particular configuration by FEA assessment.

6.3 Roller straightening effects

Roller straightening effects during the manufacture of the sections can also cause weak axis flexural strain hardening to occur in the upper webs on the surfaces that can result in high localised surface hardness values that don't represent the variation of yield stress through the web.

6.4 Strain-aging effects

Strain-aging also occurs which can confuse the assessment of strain hardening.

There are two causes of strain ageing of steel identified in the literature. These are by the free carbon or the nitrogen atoms within the steel solute that pin dislocations caused by plastic deformation hindering further movement and strengthening the steel. Higher temperature aging is controlled by free carbon and can happen relatively quickly and be a cause of cracking soon after welding or cold working. Carbon is however considered immobile at room temperature. Therefore, aging at ambient temperatures has been blamed upon the presence of nitrogen. Carbide and nitride formers like Vanadium and Niobium are sometimes added to reduce strain ageing but these can also be detrimental to heat affected zone toughness upon welding.

Significant strain aging effects of AS/NZS 3679.1 G300 compliant steel were found after natural aging for 12 months or more after pre-straining and led to significant increases in yield stress and reductions of Charpy V-Notch energy and fracture toughness beyond what were measured immediately after pre-straining (Hyland(2008)).

Traditional testing for aging by short-term raising of temperature may therefore be effective in identifying carbon based ageing effects, but not for the slower nitrogen based ageing.

The elevated yield stress caused by strain-aging in conjunction with initiating cracks or geometrical constraints can lead to brittle or rapid flat fracture at relatively low gross stresses. Reduced CVN energy at a given temperature is an indicator that strain hardening and aging will affect the resilience of the active link to further straining events.

6.5 Benchmarking of hardness required in adjacent unaffected members

EBF configured in accordance with NZS3404:1997 design rules require PEQ to be focussed into the active links, the collector beams either side of the active links should remain largely unaffected by the earthquake induced deformation that occurs in the active links so hardness readings taken in them at the representative locations should allow a reasonable benchmark of the pre-earthquake hardness and yield stress to be assessed. It should also be recognised that some yielding of the active links may have also occurred due to straightening procedures used during fabrication and installation.

7 ANALYTICAL ASSESSMENT OF PEQ CAUSED BY AN EARTHQUAKE

PEQ due to an earthquake may be assessed analytically if appropriate ground motion records are available. Those analytically derived assessments increase in reliability and usefulness where peak inter-storey displacement observations in some locations within the building can be referenced to moderate the analytically derived inter-storey displacements. Analytical findings are also useful in directing more comprehensive in-situ investigations and understanding of how active links were affected.

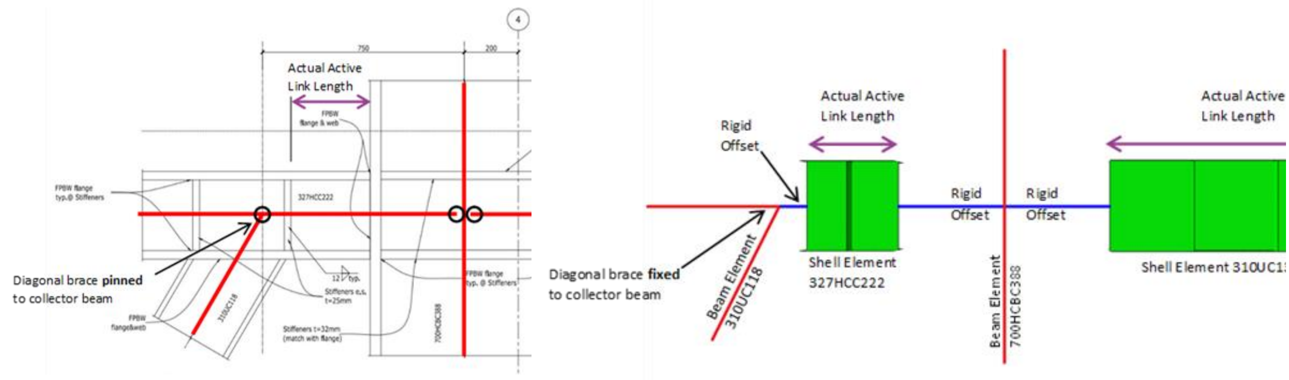

Figure 4 Comparison of frame modelling of active links (left) using traditional rotational hinges compared to ABAQUS combined shell and frame modelling (right)

Structural analysis software based on frame elements with rotational hinge elements at ends (Figure 4), can't calculate PEQ in active links and instead report rotational measures. This is because active links are often dominated by shearing actions in the webs with little if any plastic strain occurring in their flanges due to flexural rotation. The distribution of PEQ in their webs is also affected by the presence of web stiffeners (Figure 5).

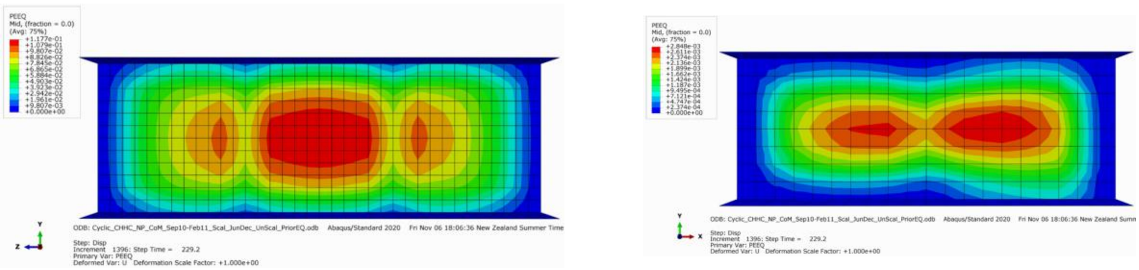

Figure 5 Examples of the effect of two web stiffeners (left) compared to one (right) on the location of maximum PEQ in a shear dominated active link

Finite Element software packages such as ABAQUS with sophisticated shell elements allow PEQ development in the active links to be modelled much more directly (Figure 4) and allow for cyclic plastic strains and strain-hardening behaviour to be calculated directly step by step through an analysis.

Cyclic plastic strain development has been analysed in this way by mechanical engineers designing high pressure vessels subject to loading and unloading cycles. Therefore ASME BPVC VIII.2 :2021 is a useful reference for PEQ compliance criteria and sets rules for modelling using finite element software that are also relevant for structural applications where cyclic displacements due to seismic actions occur.

Benchmarking checks of the limiting PEQ requirements of ASME BPCV VIII.2:2021 have previously been undertaken by the authors against experimental moment end plate testing results published by HERA (Short et al. 2004). This found that the ASME BPCV VIII.2:2021 compliance requirements are and were exceeded in the actual testing indicating that the ASME limits are conservative.

7.1 Quasi-static time history analysis

ABAQUS offers significant power to assesses PEQ. However, this can't be done directly for a large structure using a dynamic time history analysis, due to the changing stiffness of the many mesh elements plastically deforming in the active link webs. Therefore a quasi-static time history analysis ("QSTHA") method was developed that uses a series of incremental sequential static imposed displacement analyses with the plastic strain accumulated from each analysis. The imposed displacements are applied at the centre of mass of each floor level in orthogonal directions simultaneously.

The displacement series may be determined at 0.02 second steps from an elastic time history analysis of the full building model. As an alternative the building displacements may be determined more quickly by approximating the structure to a simplified equivalent cantilever translational model ("ECM") by time history analysis ("ECMTHA") for each orthogonal direction with the same first two translational modes of response and participation factors in each direction as the full building model. The ECM can also be a useful tool to quickly compare relative response to different ground motion records using response spectrum analysis ("ECMRSA") and shorten analysis times.

7.2 Filtering of ground motion records

The use of nearby ground motion records to analyse the displacement sequence of tall buildings offers significant benefit. However caution is required in using the ground motions because of integration drift or error that can occur due to the effects of noise in the records. Integration drift occurs when double integrating accelerometer records to develop displacement time series data. It is a mathematical issue that gets amplified significantly with natural frequencies of ground motion less than 0.25 Hz. Large errors of non-physical displacement can result. These errors mean that the commensurate plastic strain development in the active links will also be in significant error.

Care needs to be taken in selecting filters to remove low frequency noise or else large distortions can occur in the displacements calculated during time history analyses. Ground motion records therefore must be filtered before being used analytically to remove both lower and higher frequency noise from the records. For example the use of GNS records, with extended band pass filter 0.01 to 0.1 and 24.5 - 25 Hz, can significantly over estimate building and inter floor displacement demands compared to those found using 0.10- 0.25 and 24.5 - 25 Hz bandpass filters.

It also means that time history analyses for buildings with natural frequencies less than 0.25 Hz are unlikely to be reliable and should be treated with caution. If the dominant forcing frequencies of the ground motion fall largely at or above the first natural frequency of the building (Figure 6), then filtering out lower frequency data is unlikely to have any significant effect on the building analysis.

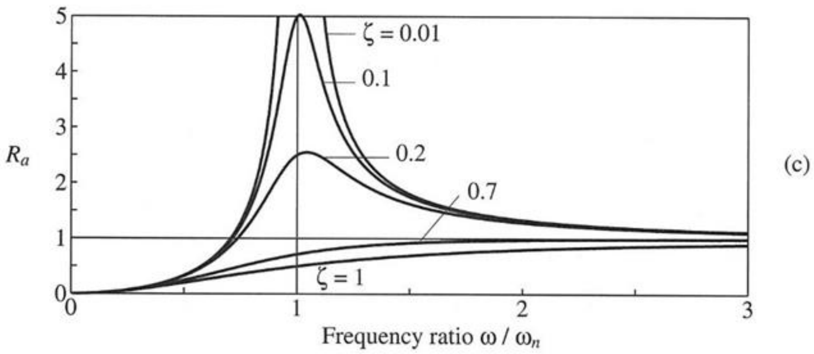

Figure 6 Response factors for acceleration from Fig 3.2.7 of Chopra (2007) showing how the acceleration response factor Ra varies with the ratio of the forcing frequency ω / natural frequency ω_n of the mode.

Low frequency noise in earthquake records can also be simply that and of only nuisance value if left in the records for time history analysis or development of response spectra. The effect of different low-cut filter limits and low frequency noise on analytical displacements can be seen in the differing regularity of displacements in analyses processing sinusoidal motion where noise has been filtered and where it hasn't. (Figure 7).

Where the effect of low frequency ground motion may be significant then the use of direct displacement devices is recommended by seismologists for measurement of fault movements (Boore et al.(2005)). Such a recommendation makes good sense also for tall structures with low first natural frequencies.

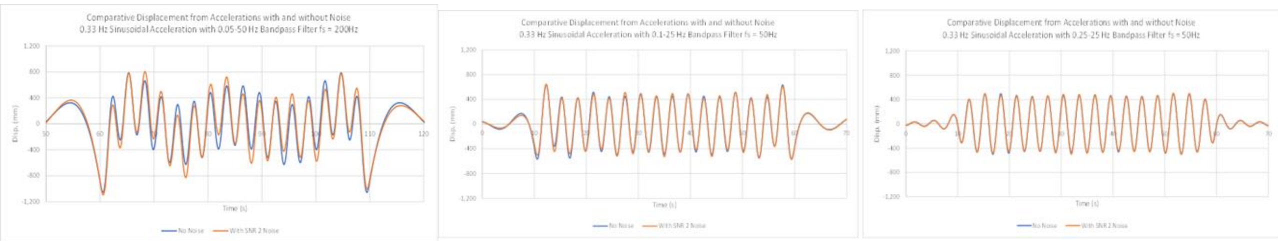

Figure 7 Displacements from sinusoidal acceleration for building with 1st mode 0.33 Hz showing effect of increasing low-cut filter limits. (From left to right): a) 0.05 Hz low-cut limit results in large spikes at beginning and end of record, secondary wave effect and greater peaks of noisy signal compared to no noise signal ; b) 0.1 Hz low-cut limit has reduced initial final spikes, reduced secondary wave effect and close matching of noisy and no-noise signal peaks; c) 0.25Hz low-cut limit has no initial and final displacement spikes, minimal secondary wave and almost identical matching of noisy and no-noise signal peaks. This shows that for the 0.33 Hz 1st mode frequency a low-cut filter limit of 0.25 Hz results in the most accurate calculated displacements whereas at low-cut limit of 0.05Hz results in significant and distorted nonphysical displacements.

7.3 Effect of subsurface conditions on ground motion records

The effect of near-surface ground conditions and liquefaction at a recording site can affect ground motion readings also. So while a ground motion may have been recorded near to a particular building, if the recording site had been affected by liquefaction and the building itself hadn't then the use of the that record would likely result in unreliable analytical displacements. Therefore finding records that have been taken in locations with similar subsurface geology to the site is preferable.

Liquefaction effects are also earthquake energy and frequency dependent and so may occur in one event at a ground motion station but not in a subsequent aftershock. Building foundation systems may also affect response to ground motion. For example if a building has braced diagonal piling much like a wharf structure, then its effective lateral founding level will be lowered compared to one with just vertical piling, and it may be less affected by liquefaction effects that may have occurred above that lower founding level.

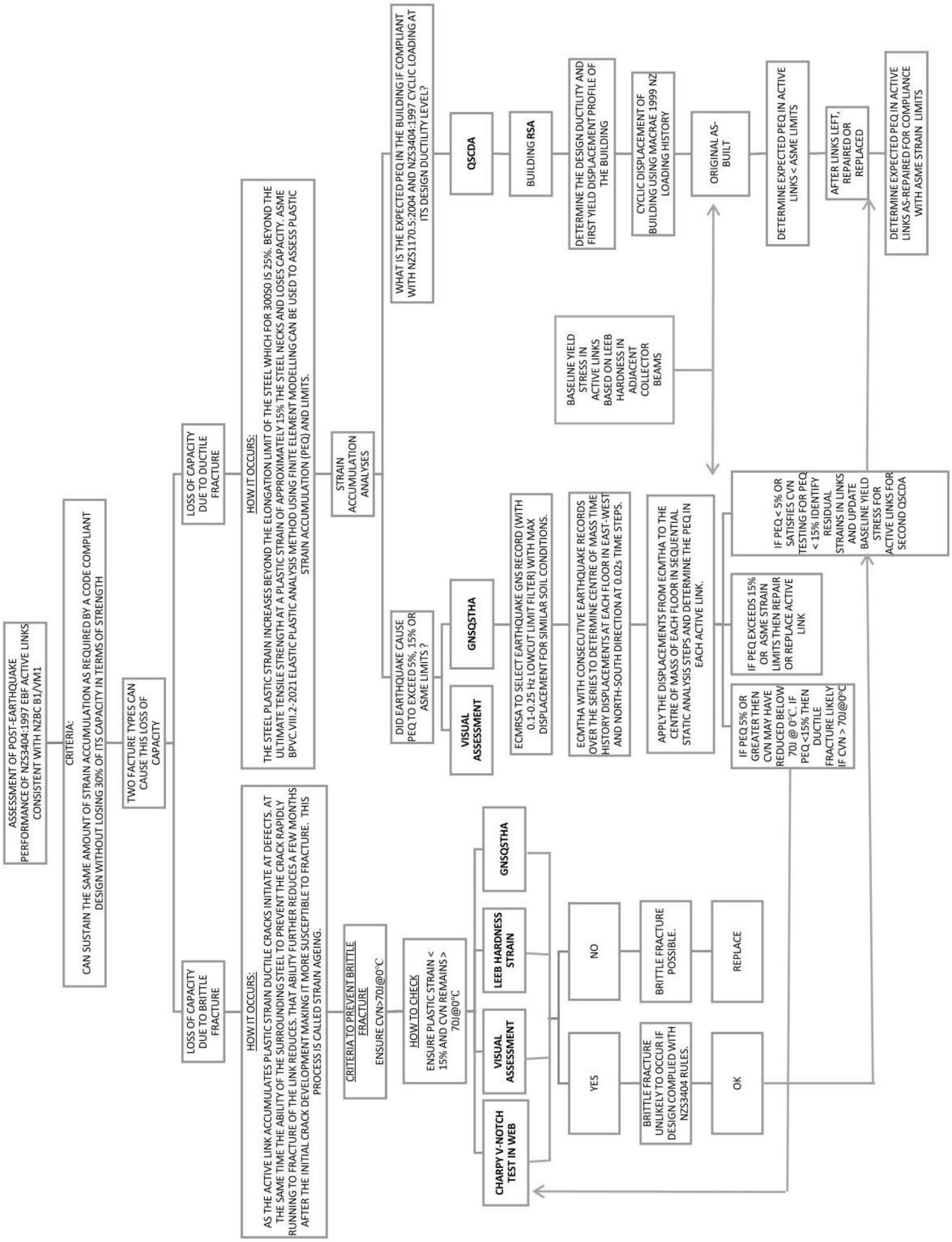

Figure 8 Flowchart of proposed method for assessment method of post-earthquake performance of active links in NZ3404:1997 eccentrically braced frames consistent with NZBC VM/B1

7.4 Moderation of analytical results with field observations of peak displacement

Physical peak displacement observations made post-earthquake at movement joints in places such as stairwells are valuable in allowing the analytical displacements to be moderated to conform with those observations. These allow difficult to model effects of secondary elements and damping on the structural response to be incorporated in the analytical assessment of PEQ in the active links.

Qualitative comparisons of analytical displacements to observations of cracking severity in adjacent gypsum board linings and jamming of doors is also a further broad reality check.

An unloading analysis undertaken of the full building model of a building around 20 storeys high identified that only 1 to 2% of total displacement was due to plastic deformation even though a significant number of active links had developed significant PEQ, justifying the use of elastic time history analyses to determine the translational displacement input steps for such EBF analyses.

The levels of PEQ in the active links determined by the sequential analyses were also found to be consistent with the broad visual assessments of PEQ based upon the level of steel mill-scale and coatings cracking reported.

7.5 Flowchart of Assessment Method

A flowchart summarising the steps in the assessment method is shown in Figure 8. When GNS ground motion records are used for the analyses, GNS is used as a prefix such as GNSQSTHA.

8 CONCLUSIONS

Advanced elastic-plastic analysis of EBF using ABAQUS shell elements in the active links to assess the development of PEQ in those active links has been found to offer improved insights into the residual ability of earthquake affected EBF buildings to meet compliance requirements with NZBC Clause B1 Structure. Filtering of ground motion records from low frequency noise is important to achieve reliable results from time history analyses using them. Peak displacement observations post-earthquake at movement joints allow analytical modelling to be moderated. It was also found that visual assessment and hardness testing appropriately done of active links can provide results consistent with the analytical results. The expected PEQ demands for compliance with B1 Structure cited Verification Method NZS 3404:1997 can be assessed using simulation testing of the full structure model by imposed displacement analysis using the NZ Loadings History. Upper limits for PEQ set in ASME BPCV VIII.2:2021 are appropriate also for use in assessing active links of EBF subject to cyclic seismic demands.

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