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A preliminary investigation of validity of conventional single-mode pushover analysis in multi-mode systems

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ABSTRACT

Pushover analysis is a simplified inelastic analysis approach where a structure is subjected to increasing increments of monotonic imposed force or displacements to estimate the inelastic deformations in the structural system. Though there are more advanced techniques of pushovers available, the conventional single-mode pushover method is still the most popular method used in the industry. In this paper, a study is initiated to investigate the validity of conventional pushover methods as applied to multi-mode systems. A 12-storey reinforced concrete wall structure is used as an example. The structure is subjected to conventional pushover analysis, and the results are benchmarked against the Incremental Dynamic Analysis (IDA) backbone. The IDA is the dynamic counterpart of static pushover obtained by performing a series of dynamic analyses on a suite of ground motions, each scaled to several levels of seismic intensity. The proposed preliminary outcome of the analysis is that the conventional, single-mode pushover method as applied to wall structures, though might capture the strength aspect may result in an erroneous estimate of the inelastic displacement capacity of the structure. This may result in unintended consequences for the assessment of the structure.

1 INTRODUCTION

In the last decade, with further development of seismology studies in New Zealand and the update in the national seismic hazard model, there is a foreseeable change in the seismic demand that most of the existing buildings within the Wellington region will experience. As a result, an increasing number of those buildings are to be assessed and strengthened to a higher level of resilience.

To improve the understanding of a structure’s performance under an increasing level of lateral demand, a common approach is to perform a non-linear static pushover analysis (NLSPA, also popularly known as the conventional pushover analysis) to a pre-defined displacement value. Other non-linear analysis methods, such as time history analysis can also be used to determine structure performance. However, performing non-linear time history analysis requires a high degree of technical skill and effort to perform accurately. Therefore, there is a benefit in understanding how NLSPA performs against more rigorous procedures which take into account the dynamic characteristics of the structure.

A similar study was done by Mway and Elnashai (2001) by comparing pushover results to "dynamic pushover" idealised envelopes obtained from the incremental dynamic collapse for 12 RC frame buildings using 2D models with the combination of two design ground acceleration (0.15g and 0.3g) with only a single ground motion considered in the analysis. They found pushover results can provide a similar result as the incremental dynamic analysis if lateral load distributions are cautiously selected, and outcomes should be carefully articulated and interpreted.

This preliminary study aims to determine whether the capacity estimated by an incremental dynamic analysis is well approximated by a NLSPA for a case study structure with lateral systems of reinforced concrete walls modelled in 3D and located in a high seismic zone, also its impact on the global structural rating using a capacity spectrum format (CSM/ADRS).

2 METHODOLOGY

2.1 Structure

The structural model used in this paper is based on an existing building in central Wellington. Simplifications have been made to the real structure to reduce complexity during the analysis process.

The structure is a 12-storey reinforced concrete building with a rectangular plan measuring 48m by 16m. It has a storey height of 4 m at level one and 3.5 m at all other levels, as shown in Figure 2.1. The lateral load resisting system in the longitudinal direction (east-west) consists of ductile reinforced concrete walls designed to rock on their foundations. In the transverse (north-south) direction, the lateral resisting system is reinforced concrete moment frames. The masses are concentrated to be at the floor level. This structure is founded on deep reinforced concrete piles with caps. Those are connected to the ground floor slab via ground beams. Only the wall direction is used for the present study.

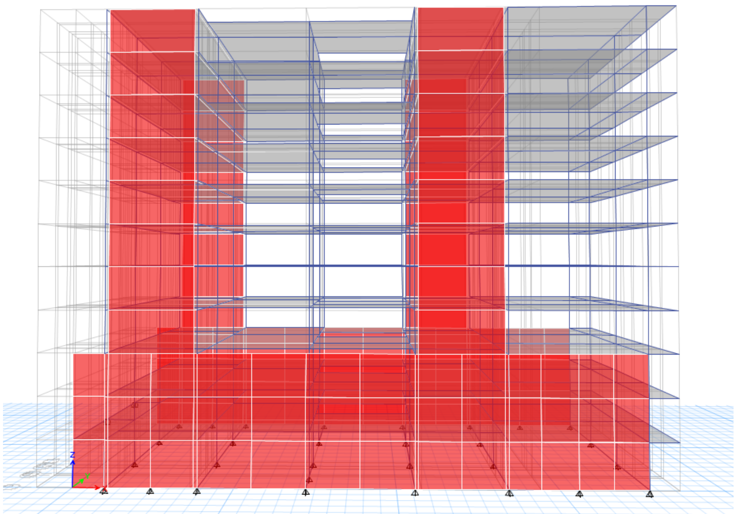

Figure 2.1 Indicative Structure Layout

2.2 Modelling Philosophy

The superstructure has been modelled using an equivalent frame technique for efficient non-linear analysis. It involves defining different structural members by line elements located at actual member centrelines and assigning those elements with the relevant characteristics of the true member.

Fixed base boundary conditions are assumed, and Giberson one component beam elements with lumped plasticity were adopted for the analysis. A Giberson element consists of an elastic beam in series with inelastic flexible springs. The element is formulated in the flexibility domain

Non-linearity is introduced into the equivalent frame model by means of discrete flexural plastic hinges. The plastic hinges are allocated to each structural element where non-linearity is expected. The properties of the hinges have been derived based on the guidance of NZS 3101:2006 (A3) (Standards New Zealand, 2006) and EQ-Assess guidelines C5 Revised (EQ Assess, 2018).

2.3 Non-Linear Static Pushover Analysis

NLSPA is a method that is commonly used to understand the force-deformation behaviour of a structure. The procedure is performed based on the assumption that the inelastic dynamic behaviour of the structure may be approximated by a monotonic non-linear static analysis. A linear displacement pattern is applied to the global structural system. The load is increased incrementally until a pre-defined maximum displacement at the control point in the structure is reached. Conventional pushover assumes that the structure acts predominantly in its first mode of vibration, and other modes have no significant contribution to the response of the building. The plot of the force required as a result of each consecutive displacement increment forms the overall structural backbone of the structure.

2.4 Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is a dynamic counterpart to the static pushover. In a normal static analysis, a pattern of constant or adaptive load is incrementally increased, and at each increment, the system is solved. In a similar manner, in an IDA, the input ground motion is scaled in an incremental manner, and for each increment, the system is solved dynamically. Figure 2.2 shows an illustrative manner in which the IDA is performed if only one ground motion is used.

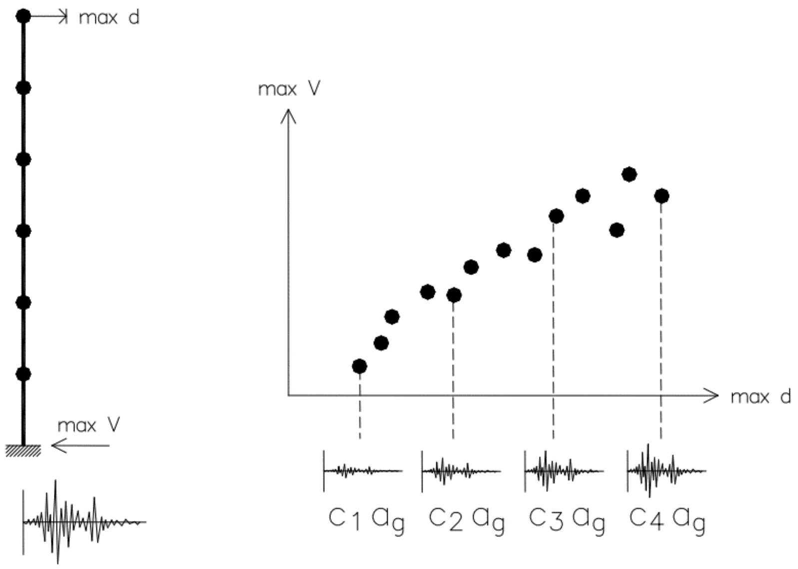

Figure 2.2: IDA for a single ground motion

IDA requires a suite of ground motions to be selected with appropriate seismological signatures to the site. Each has been scaled using the procedure in NZS 1170.5:2004 to match a target spectrum for the site.

2.5 Plastic Hinge Computation

The model's post-yielding behaviour inherits from that of the plastic hinges at beams, columns, and frame elements representing walls. For each of those, plastic hinges were assigned at two ends of a member. Beams in this model have been assigned with moment hinges, walls and columns assigned with P-M moment hinges.

In this study, the hinge backbone is stylised, as shown in Figure 2.3. Point A is the origin with no demand or deformation. Point B sits at the yielding state, beyond which the plastic deformation starts to develop. Point C defines the hinge deformation when the member is loaded to its ultimate capacity. In this study, the force component of Point C is equal to the yield capacity of the section as only the deformation is of interest once it passes the yielding state. Residual strength past the ultimate state is represented by Point D, partnered up with Point E to adequately limit the deformation of the hinge before it reaches the total loss of resistance.

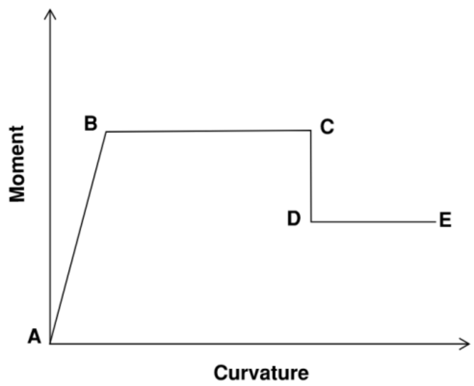

Figure 2.3: The A-E curve for Moment vs Curvature (ASCE, 2007)

To define Point B, the sectional flexural capacity at yield, M_y, is calculated using the Navier-Bernoulli plane-section hypothesis using Equation 1.

$$\frac{M_y}{bd^3} = \varphi_y\{E_C\frac{\xi_y^2}{2}\left(0.5(1+\delta_1)-\frac{\xi_y}{3}\right)+\frac{E_S}{2}\left[(1-\xi_y)\rho_1+(\xi_y-\delta_1)\rho_2+\frac{\rho_v}{6}(1-\delta_1)\right](1-\delta_1)\} \quad (1)$$

Where b is the width of the compression zone; d is the section effective depth; φ_y is the yield curvature; ρ_1 and ρ_2 are the ratios of the tension and the compression reinforcement (normalised to bd); ρ_v is the ratio of "web" reinforcement; E_s is the steel elastic modulus; E_c is the elastic modulus of concrete, and ξ_y is the compression zone depth at yielding (normalised to d).

The corresponding yielding curvature, Φ_y, is given in Equation 2 (Fardis, 2007).

$$\Phi_y = \frac{f_y}{E_S(1-\xi_y)d} \quad (2)$$

To define Point C, the ultimate curvature of the section was calculated as the lesser of Equation 3 and 4 below, following the guidance from Seismic Assessment of Existing Buildings – Concrete Buildings C5 (EQ Assess, 2018).

$$\Phi_{ult} = \frac{\varepsilon_{s,max}}{c_{prob}} \quad (3)$$

and:

$$\Phi_{ult} = \frac{\varepsilon_{c,max}}{d-c_{prob}} \quad (4)$$

Where c_{prob} is the neutral axis depth at probable (ultimate) capacity; $\varepsilon_{c,max}$ is the accepted maximum concrete compressive strain at the extreme fibre of the section. $E_{s,max}$ is the maximum accepted strain of the reinforcing steel in tension, and d is the effective depth of longitudinal tension reinforcement.

This study assumes that residual strength (refers to Point D and E) of the member equals 20% of the yielding capacity. Point E of all backbones terminated at 120% of the ultimate curvature calculated above.

3 IMPLEMENTATION

3.1 Conventional Pushover Analysis

To perform a pushover curve in SAP2000, a structure was modelled as shown in Figure 3.1. In this model, material non-linearity is considered by assigning plastic hinges to concrete beams, columns, and walls. Structure seismic weight was re-assembled to column nodes as point loads. P-Delta was applied. A non-linear gravity load case of [1.0G+0.3Q] is defined and added to the pushover load case.

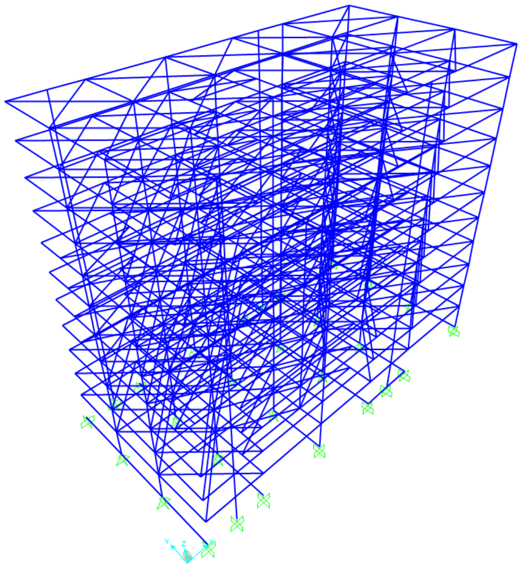

Figure 3.1: Existing structure modelled in the software

Figure 3.2 shows the result of the NLSPA.

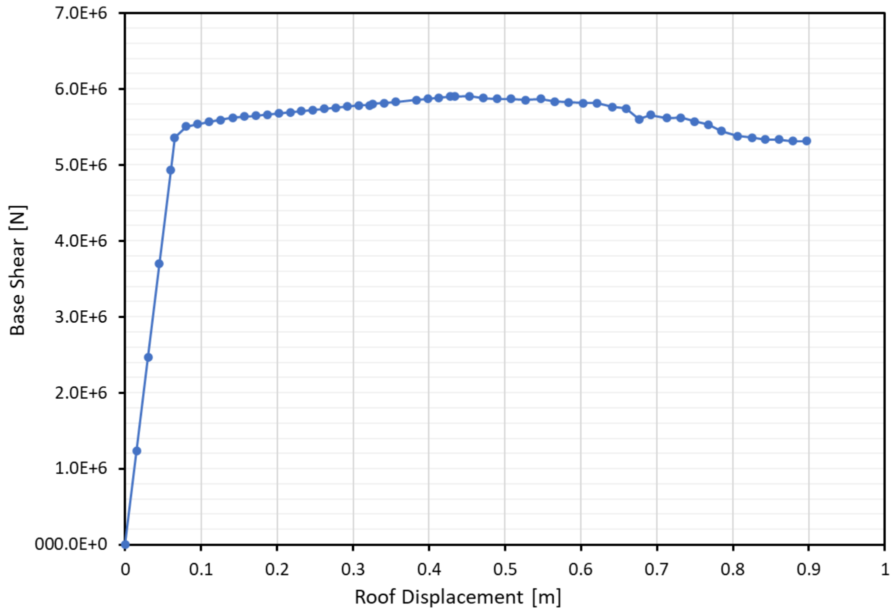

Figure 3.2: Results NLSPA for the longitudinal direction of the structure (reinforced concrete wall system)

3.2 Ground Motion Selection and Scaling for Incremental Dynamic Analysis

A target spectrum was developed for a site in central Wellington with importance level 2, Soil Class C, and a design life of 50 years.

The ground motions used in this study were selected in Oyarzo-Vera et al. (2012) which are appropriate for a structure on Soil Class C in Wellington. Ground motion scaling was performed between a range of 0.4 and 1.3 times the fundamental period of vibration of the structure ($T_1 = 1.5$ s).

These scaled motions are:

- El Centro, Imperial Valley, USA, 1940, k1k2=1.44
- Tabas, Iran, 1978, k1k2=0.55
- La Union, Mexico, 1985, k1k2=2.18
- Lucerne, Landers, USA, 1992, k1k2=0.85
- Arcelik, Kocaeli, Turkey, 1999, k1k2=3.45
- Duzce, Duzce, Turkey, 1999, k1k2=0.636
- HKD085, Hokkaido, Japan, 2003, k1k2=0.73

One approximate test of the efficacy of the scaling is a check of the average dispersion of spectral accelerations in the range of period for which scaling was performed for both the unscaled and scaled ground motion suites. The coefficient of variation of the unscaled motions is around 0.3, and after scaling this reduces to 0.12, suggesting that the scaling has better aligned the intensities of the ground motions.

Scaling the ground motions to meet the target spectrum permits the generation of a suite which have a relatively consistent intensity. To run the IDA, the scaling factors of the motions in the suite were then further modified to range from low intensity to high intensity. The time history results from analyses performed on the accumulated dataset were then used to generate the IDA curve.

3.3 Incremental Dynamic Analysis

The results of the time history analyses on the structure are shown in Figure 3.3.

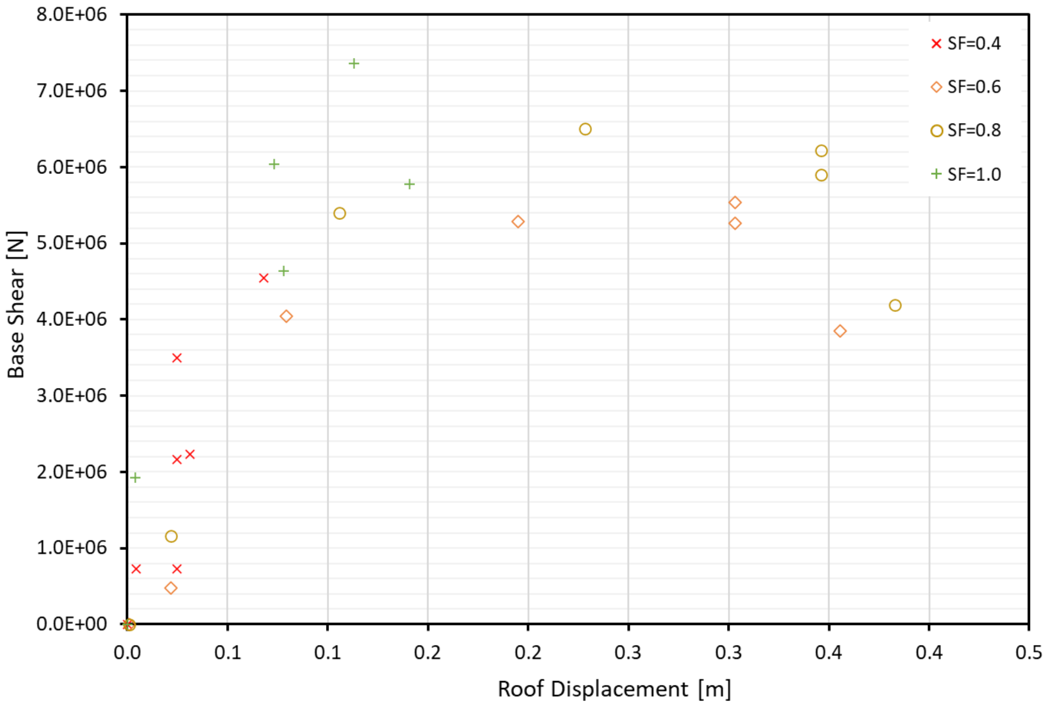

Figure 3.3: Results of time-history analyses at varying levels of intensity in the longitudinal direction of the structure as part of IDA.

The IDA curve was then developed by taking the average base shear results at different bands of displacement. In Figure 3.4, the IDA curve is shown with the time history result for each point coloured

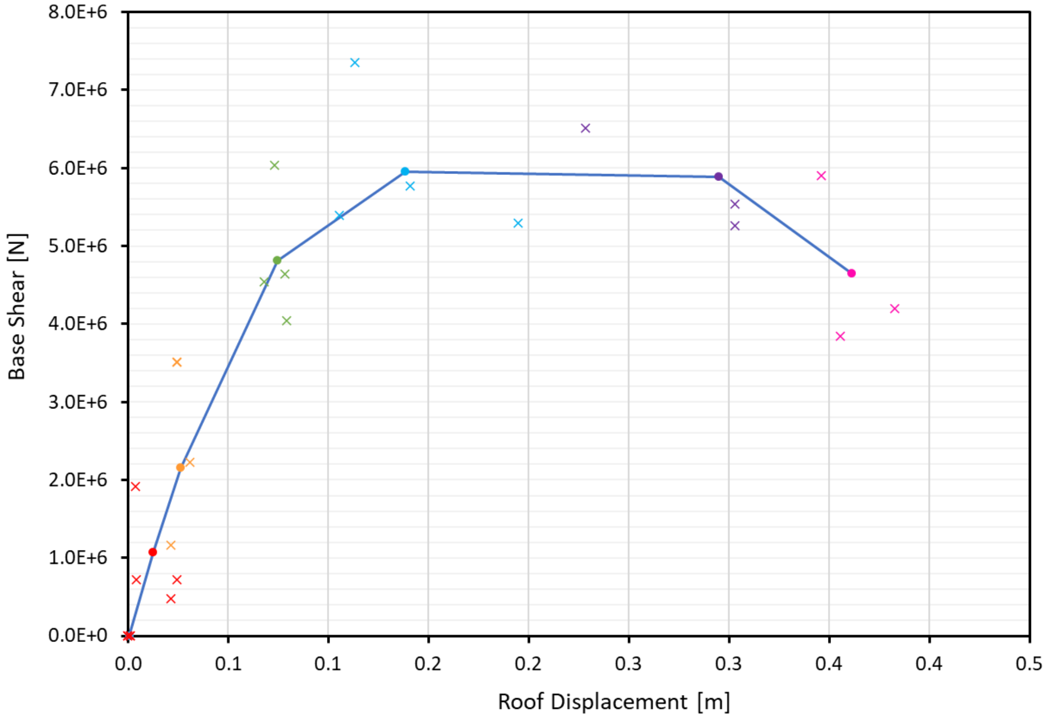

Figure 3.4: Development of IDA curve from time history analysis results

3.4 Comparisons

The two figures below show the comparisons between the NLSPA and IDA.

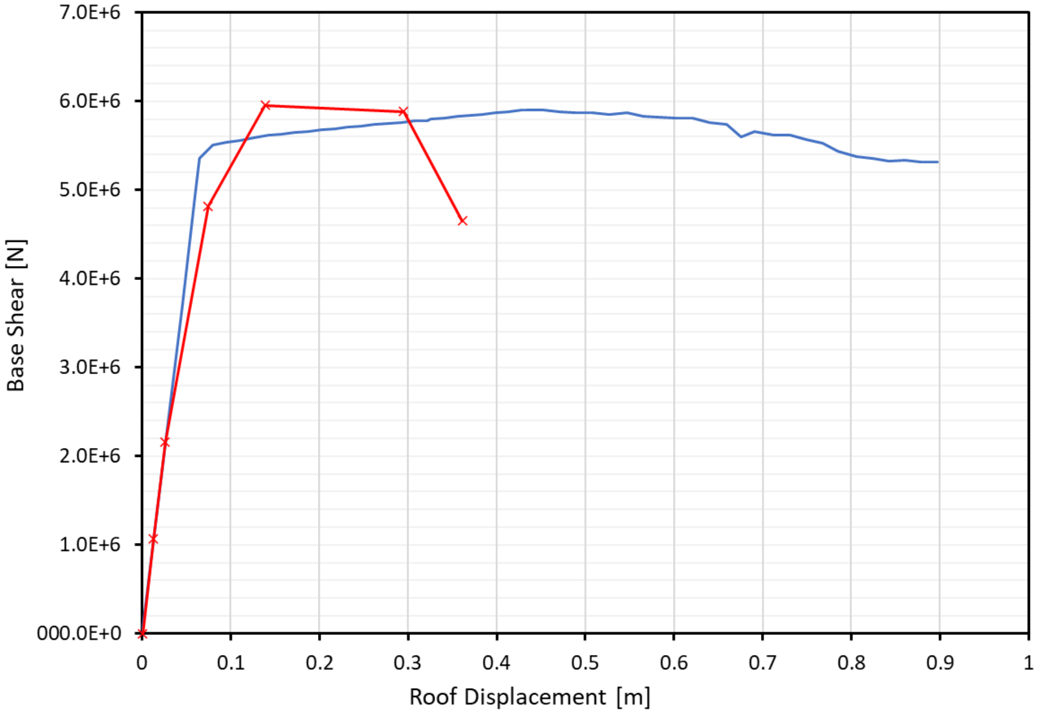

Figure 3.5: Comparison of NLSPA (blue) and IDA (red) for the longitudinal direction of the structure (reinforced concrete wall system)

4 DISCUSSION

From the selected ground motion set, a sufficient number of motions provided stable results to generate an IDA backbone. The IDA backbone is compared against the NLSPA backbone, as shown in Figure 3.4. The force capacity of the structure as assessed by both methods was found to be consistent. The notable difference between the two was the displacement capacity achieved by the structure, with NLSPA predicting a larger ultimate displacement capacity than IDA. This reduction in displacement capacity may be attributed to the change in the migrating inertia coupled with onset of geometric instability captured by IDA, which is overlooked by the NLSPA.

A demonstrative exercise was completed to understand the impact of such a disparity on the percentage of new building standard (%NBS) score when compared with an acceleration-displacement response spectrum comparison.

IDA captures the dynamic response of the structure. Therefore, the first point of degradation in the IDA acceleration displacement capacity spectrum represents the deformation at which a significant number of hinges have reached their residual capacity; in the present case study building as it is only a shear wall, this onset of degradation shows the onset of hinge degradation in the wall. As well understood, when the hinges start to enter the degrading realm of plasticity, the uncertainty associated with computation increases very much. (Vamvatsikos & Cornell, 2002).

So, while estimating the reliable maximum probable deformation capacity point, this degree of uncertainty needs to be accounted and a suggested approach is to take a point away from the degrading point. This is also to ensure that in the case of an earthquake overload, there is a reasonable margin before the hinge degradation starts. As such, the displacement limit of both curves is defined as two thirds of the displacement where significant degradation from the maximum force capacity occurs.

For the NLSPA, the point of the first degradation represents the static point where a significant number of hinges have reached their residual capacity. The EQ Assess guidelines associate this point with being the reliable probable deformation capacity of the structure at an ultimate limit state level. This criterion of the guideline is made under the assumption that there is more reserve strength to ensure resilience without drastic degradation in the case of an earthquake overload.

The most important conceptual point to note here is that when IDA is done, the resilience gets quantified in some manner. However, considerable level of uncertainties exists in the resiliency computation; still, it gives a qualitative estimate of how the structure might start to behave in increasing seismic intensity levels. On the contrary, in the pushover analysis, because the dynamic equilibrium is not followed explicitly, the migrating change in the inertial forces may not be captured, and hence this aspect may not be explicitly addressed. There will also be degradation of stiffness in the hinges after the onset of non-linearity due to reversed cyclic loading in the dynamic analysis.

Figure 4.1 shows the capacity spectra for each analysis method. On each is marked the point where the displacement has been limited, as discussed above. The bilinear idealisations of each curve are also plotted. The bilinear idealisation of the NLSPA has been further limited to an ultimate displacement which causes the wall system to have a ductility no greater than 6 in the ultimate limit state as defined in NZS 3101:2006. (Standards New Zealand, 2006)

Figure 4.2 shows the two curves plotted against demand spectra. Comparing the demand and capacity displacements on this curve to determine %NBS, NLSPA predicts that the structure is rated at 95%NBS. Using the IDA method, the structure achieves a rating of 68%NBS. This is a notable disparity between the two methods, especially since the latter analysis states the building is subjected to a higher risk during an earthquake event.

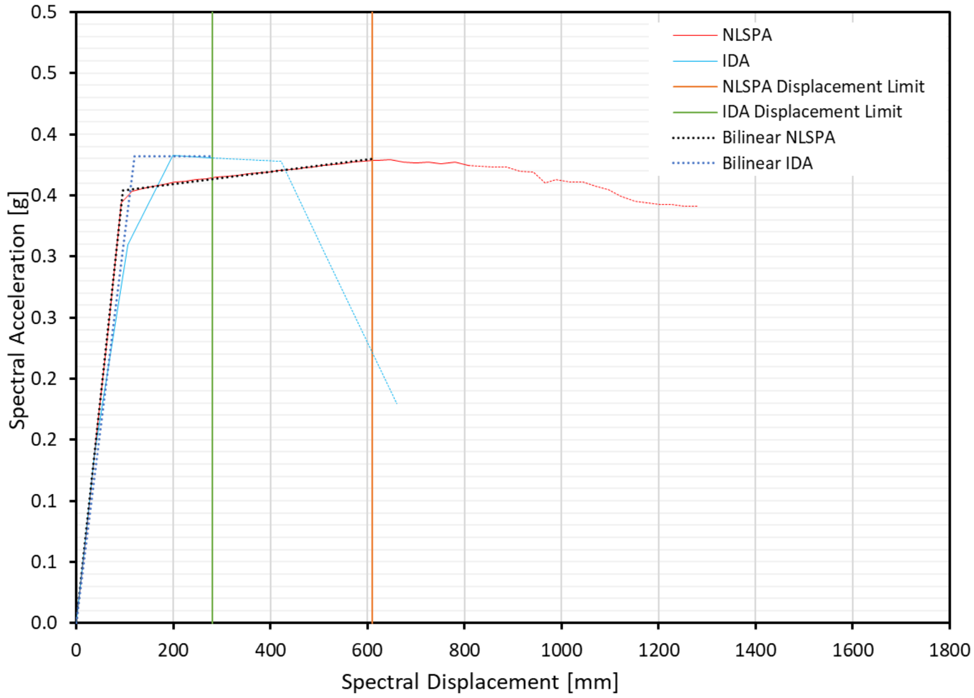

Figure 4.1: Capacity spectra displacement limits for each analysis method.

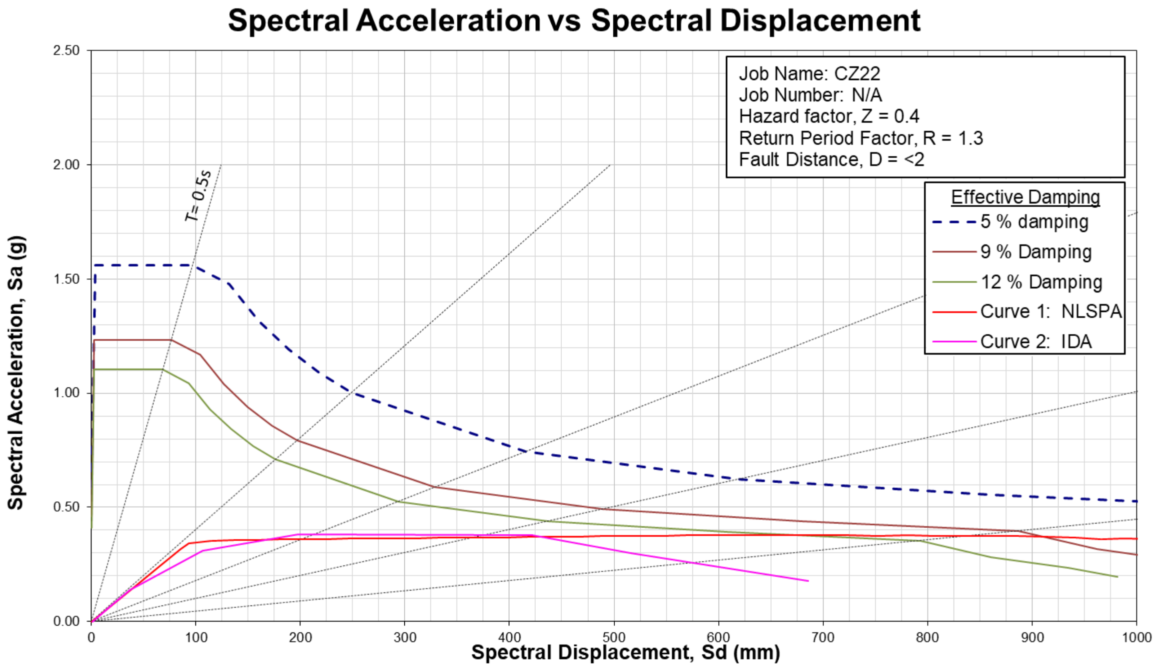

Figure 4.2: Acceleration-displacement response spectrum method comparison.

5 LIMITATIONS AND RECOMMENDATIONS

The design of the current research is a preliminary observation and is subjected to limitations. Recommendations for improving research methods in the future are:

- Currently, only seven ground motions have been selected, which could increase uncertainties in results as each variation between ground motions can significantly impact the result outputs, including a larger sample size meaning that erroneous outputs can be identified more easily.
- This research only looked at a mid-raised reinforced concrete structure using a lumped plasticity approach due to time and resource constraints. Future studies can be carried out by looking at other structure forms, and the difference could be compared.
- The non-linear properties of the hinges used in the model do not capture the stiffness degradation of the concrete elements that they represent but rather assume a bilinear hinge. A more appropriate hysteresis including the effects of cracking could be used to improve the reliability of the model.
- Replacing a lumped placidity model with a more detailed finite element approach to explicitly capture the non-linearity of the wall as a whole would further determine the realistic displacement behaviour under dynamic analysis.

6 CONCLUSION

Preliminary observations based on the present study suggest that NLSPA and IDA exhibit similar force capacity but different displacement capacity, with IDA being lower than NLSPA leading to differences in the %NBS score of the building. The results presented to signify the fact that diligence needs to be adopted when doing seismic assessments of existing buildings with NLSPA. Further research to address the limitations discussed in Section 5 is recommended.

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