DETERMINATION OF STRUCTURAL IRREGULARITY LIMITS – MASS IRREGULARITY EXAMPLE

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SUMMARY

Structures may be irregular due to non-uniform distributions of mass, stiffness, strength or due to their structural form. For regular structures, simple analysis techniques such as the Equivalent Static Method, have been calibrated against advanced analysis methods, such as the Inelastic Dynamic Time-History Analysis. Most worldwide codes allow simple analysis techniques to be used only for structures which satisfy regularity limits. Currently, such limits are based on engineering judgement and lack proper calibration. This paper describes a simple and efficient method for quantifying irregularity limits. The method is illustrated on 3, 5, 9 and 15 storey models of shear-type structures, assumed to be located in Wellington, Christchurch and Auckland. They were designed in accordance with the Equivalent Static Method of NZS 1170.5. Regular structures were defined to have constant mass at every floor level and were either designed to produce constant interstorey drift ratio at all the floors simultaneously or to have a uniform stiffness distribution over their height. Design structural ductility factors of 1, 2, 4 and 6, and target (design) interstorey drift ratios ranging between 0.5% and 3% were used in this study. Inelastic dynamic time-history analysis was carried out by subjecting these structures to a suite of code design level earthquake records. Irregular structures were created with floor masses of magnitude 1.5, 2.5, 3.5 and 5 times the regular floor mass. These increased masses were considered separately at the first floor level, mid-height and at the roof. The irregular structures were designed for the same drifts as the regular structures.

The effect of increased mass at the top or bottom of the structure tended to increase the median peak drift demands compared to regular structures for the record suite considered. When the increased mass was present at the mid-height, the structures generally tended to produce lesser drift demands than the corresponding regular structures. A simple equation was developed to estimate the increase in interstorey drift due to mass irregularity. This can be used to set irregularity limits.

INTRODUCTION

Current earthquake codes define structural configuration as either regular or irregular in terms of size and shape of the building, arrangement of the structural and non-structural elements within the structure, distribution of mass in the building etc. A regular structure can be envisaged to have uniformly distributed mass, stiffness, strength and structural form. When one or more of these properties is non-uniformly distributed, either individually or in combination with other properties in any direction, the structure is referred to as being irregular.

Structural irregularity may occur for many reasons. Some irregularities are *architecturally planned* (**AP**). Examples of these are:

- A factory with heavy machinery, or an educational institution with a library at one floor level that leads to irregular distribution of mass as shown in Figure 1(a);
- A residential building having a car park in the basement producing a flexible first storey as shown in Figure 1(b);
- A shopping complex with setbacks to accommodate boundary offset requirements as shown in the plan of Figure 1(c);
- Buildings with flexible, rigid or no diaphragms at a floor level, or structural plan having different lateral load resisting systems (resulting in torsion) as shown in the plan of Figure 1(d).

A structure can also be irregular because of unplanned effects, which are referred to as *aleatoric uncertainties* (**AU**). These include rearrangement of loadings, as well as material strength and stiffness variations.

For the above reasons, structures are never perfectly regular and hence the designers routinely need to evaluate the likely degree of irregularity and the effect of this irregularity on a structure during an earthquake.

Structural demand estimates are dependent on the analysis method. For example, the most costly 3-D *Inelastic Dynamic Time-History Analysis* (**IDTHA**) method with appropriate modelling can consider most irregularities, but it takes significant time to build, verify and analyse the model for a suite of ground motion records. Aleatory uncertainties may be

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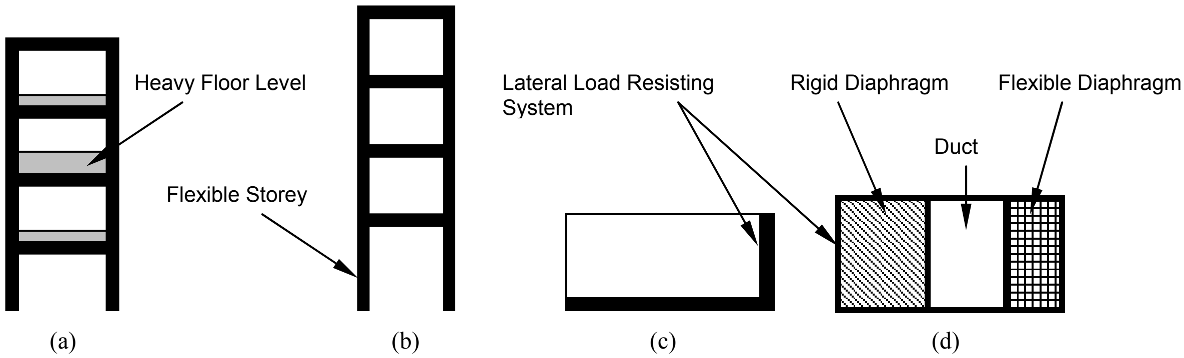


Figure 1: Examples of Vertical and Horizontal Structural Irregularities.

considered by modelling the statistical variation in structural properties as well as that of the ground motions. Also, analysis methods such as the *Equivalent Static* (**ES**) Method, defined in the New Zealand seismic design standard NZS 1170.5 (SNZ 2004); the Modal Response Spectrum Method; and the Pushover Method are simplified methods which have been calibrated against the IDTHA for regular structures. However, such calibrations have not always been carried out for structures with significant irregularity. Appropriate calibration is required for each analysis method to ensure that they estimate a realistic value of likely demand.

The ability to estimate structural demands is also dependent on the structural model. For example, 2-D analysis is generally not able to adequately represent the response of significantly irregular 3-D structures. Similarly, floor diaphragms may need to be modelled to adequately represent the behaviour.

Due to the above reasons, designers need simple methods that are effective in quantifying the irregularities in structures. Having this as the focal objective, this paper aims to answer the following questions:

1. What are the current NZS 1170.5 recommendations for structural irregularities, and where do they come from?
2. Does the past research on mass irregularity justify the current NZS 1170.5 mass irregularity requirements?
3. Can a simple and effective method be developed to quantify the effects of irregularities for structures designed for New Zealand?
4. Can a suite of shear-type structures having a range of structural irregularity be developed?
5. Is it possible to model these structures which can represent realistic behaviour well?
6. Do damping assumptions significantly affect irregularity effects on shear-type structures?
7. Does the NZS 1170.5 ES method predict the displacement response of structures well?
8. What are the effects of magnitude and floor level of mass irregularity on the drift demands?
9. What degree of irregularity corresponds to what change in response?

NZS 1170.5 CURRENT CONSIDERATION FOR IRREGULARITY

The simple ES method (including structural actions and displacement amplification due to *P*-Delta effects) has been used to design many New Zealand structures. NZS 1170.5 permits the ES method to be used to design:

- any structure less than 10 m high;
- any structure having a fundamental translational period of less than 0.4s;
- any structure with a fundamental translational period of up to 2.0s if certain regularity requirements are satisfied.

If the structure does not meet the above requirements, then a more sophisticated, and therefore expensive analysis method needs to be employed.

NZS 1170.5, similar to many other worldwide codes, defines limits for different types of irregularities, and these limits form the basis for applying the ES method. For example, structures are considered to have mass irregularity when the seismic weight, W_i, in any storey is more than 150% of the seismic weight of either adjacent storey. Such a limit of 1.5 for mass irregularity and other limits for other types of irregularities have been specified from engineering judgment rather than from rigorous quantitative analysis. For example, the SEAOC blue book (1999), with recommendations similar to that in NZS 1170.5, states that:

> *".. irregularities create great uncertainties in the ability of the structure to meet the design objectives of [the code] ... These Requirements are intended only to make designers aware of the existence and potential detrimental effects of irregularities, and to provide minimum requirements for their accommodation….",* (*C104.5.3*),

and

> *"Extensive engineering experience and judgment are required to quantify irregularities and provide guidance for special analysis. As yet, there is no complete prescription for ... irregularities"* (*C104.5.1*).

From the above quotes, it is evident that there is a need to quantify regularity limits so that the structures can be designed to have a consistent level of reliability for each type of irregularity and for each analysis or modelling method. This aim is consistent with probabilistic multi-objective *performance based earthquake engineering* (PBEE).

PREVIOUS RESEARCH ON MASS IRREGULARITY

Researchers evaluating the effects of irregularities have mainly focussed on plan irregularities due to non-uniform distribution of mass, strength and stiffness in the horizontal direction (e.g., Chopra and Goel (2004), Aziminejad and Moghadam (2005), Fajfar *et al.* (2006)). Studies that have investigated vertical irregularity effects have given insight into the behaviour of structures with vertical irregularities, but they have not developed general methods for quantifying acceptable irregularity limits. A brief summary of works related to vertical mass irregularity is presented below.

Valmundsson and Nau (1997) investigated the appropriateness of provisions for considering different irregularities as laid out in the 1994 *Uniform Building Code* (UBC). They considered 2-D building frames with heights of 5, 10 and 20 storeys, assuming the beams to be stiffer than the columns. For each structure height, uniform structures were defined to have a constant mass of 35 Mg at all the floor levels, and the storey stiffnesses were calculated to give a set of 6 desired fundamental periods. The maximum calculated drifts from the lateral design forces for the regular structures with the target period were found to lie within the UBC limit. Mass irregularities at three floor levels in the elevation of structures were then applied by means of mass ratios (ratio of modified mass of the irregular structure to the mass of uniform structure at a floor level) ranging between 0.1 and 5, and the responses were calculated for design ductility's of 1, 2, 6 and 10 considering four earthquake records. The increase in ductility demand was found to be not greater than 20% for a mass ratio of 1.5 and mass discontinuity was most critical when located on lower floors. Mass irregularity was found to be the least important of the irregularity effects considered.

Al-Ali and Krawinkler (1998) assessed the effects of vertical irregularities by evaluating the roof drift demands and the distribution of storey demands over the height of the structure. This was obtained by conducting elastic and inelastic dynamic analyses on 2-D single-bay 10-storey generic structures, assuming a column hinge model. A base structure was defined to have a uniform distribution of mass over the height. The stiffness distribution that resulted in a straight-line first mode shape was tuned to produce a first mode period of 3s when designed according to the Modal Superposition technique. Structures with mass irregularities were created by changing the mass distribution of the base model and keeping the same stiffness distribution as the base model. Mass ratios between 0.25 and 4 were chosen and applied either at one floor or in a series of floors, and the stiffness distribution was tuned until the structures had a fundamental period of 3.0s. Dynamic analyses were then conducted on each structure by subjecting them to a suite of 15 ground motion records. *P*-Delta effects were not considered and Rayleigh damping was used to obtain a damping ratio of 5% for the first and fourth modes. It was found that mass irregularities had a relatively small effect on elastic and inelastic storey shear and storey drift demands. It was also shown that mass increase at the top had a larger effect on roof and storey drifts than when increased mass was applied at the mid-height or at the lower floors. Again it was concluded that mass irregularity effects were less than other types of vertical irregularities.

Michalis *et al.* (2006) carried out incremental dynamic analyses on a realistic nine storey steel frame to evaluate the effect of irregularities for each performance level, from serviceability to global collapse. A mass ratio of 2 was applied at a series of floors over the selected frame and the effects of mass irregularity were evaluated. It was found that the influence of mass irregularity on interstorey drifts was comparable to the influence of stiffness irregularity.

Although the above researchers and a few others (e.g Chintanapakdee and Chopra (2004), Tremblay and Poncet (2005)) have given useful insights into the topic of vertical irregularities and their effects on structural response, these studies are not carried out with a design perspective. It may be seen that in many of the cases described above an "apples-to-apples" comparison was not conducted. That is, regular and irregular structures were not generally designed for the same *engineering demand parameter* (**EDP**). For example, irregular structures were sometimes designed to have the same period as the regular structures. This is problematic because it is possible that the design drifts of the irregular structure with the same period may be different from that of the regular structure. They may even violate the code drift requirements. Furthermore, the structures with greater design drifts would be expected to have greater drift demands in the dynamic analysis. A more meaningful comparison is obtained if both regular and irregular structures are designed to the same EDP. Earlier studies are also limited to specific structural type/height and there is a lack of information on the appropriateness of the limit of 150% imposed on mass irregularity. Also, the above studies may not be relevant for structures designed according to NZ code analysis procedures which have some differences from overseas methods.

SIMPLE METHODOLOGY FOR EVALUATING VERTICAL IRREGULARITY EFFECTS

Recognising the need to provide rational basis for structural irregularity limits and to have a consistent meaningful comparison between regular and irregular structures designed according to the NZ code, the following simple methodology is proposed:

1. Define an EDP that characterises structural damage. Peak interstorey drift ratio over all the storeys has been used as the EDP to assess the vertical irregularity effects in this paper.
2. Choose a set of target (design) interstorey drifts that span the range of values that could be used by the designers (e.g. from 0.5% to 3%). Then for each target interstorey drift:
   a. Design a regular structure using the ES method to the target interstorey drift.
   b. Introduce an irregularity into the structure and use the ES method to design this new structure to the same target interstorey drift.
   c. Conduct IDTHA on each structure with each earthquake record and obtain the peak interstorey drift ratio over all the floor levels. The median peak interstorey drift ratio is then obtained for all the records in the suite.
   d. Evaluate the performance for all of the ground motion records as either a) the difference between the median peak interstorey drifts of the two structures or b) the probability that the demand for the irregular building is greater than the demand for the regular building. Only the median peak interstorey drift ratio was considered in this paper.
3. The performance distributions for the chosen degrees of irregularity and target interstorey drift ratio may then be used to characterise the effect of both of these variables and select appropriate limits.

STRUCTURAL FORMS CONSIDERED & DEFINITION OF REGULAR STRUCTURES

Simple one-dimensional models of 3, 5, 9 and 15 storey shear-type structures were considered. These models are adequate for determination of overall structural response (Cruz and Chopra, 1986) and reduce the computational effort.

There is no specification for the distributions of stiffness and strength within structures designed using the NZS 1170.5 ES method. Therefore, two classes of building having constant mass at every floor level were chosen to represent the two extremes of design choice for stiffness that defined the base (regular) structures. It is expected that the configuration of realistic frames would fall between these two extreme design models and hence the results serve as bounds. One class of building was designed for all the storeys to have a *constant interstorey drift ratio* (**CISDR**) and the other class was designed for all the storeys to have a *constant stiffness* (**CS**) with the target interstorey drift ratio at the first storey. These two models and their deflection profiles are shown in Figure 2. In the design, storey stiffnesses were iterated until the critical storey/storeys had the *design* (target) *interstorey drift ratio* (**DISDR**). For the CISDR model, at the end of iteration a

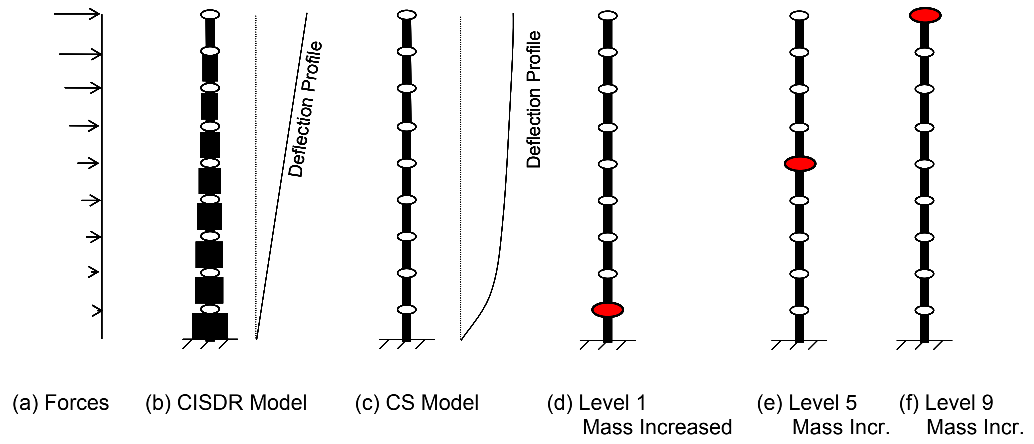


Figure 2: Deformed Shape for Different Structural Configurations and Mass Irregularity.

constant strength-to-stiffness ratio was established at each floor level, so the shear strength provided at each level was the minimum required to resist the equivalent static design forces. But for the CS design model all the storeys had a *constant stiffness*, and the strength distribution over the height was either:

(a) The minimum required to resist the design forces from the ES method, thus having a *varying strength* distribution (**CS-VSTG**) or

(b) Every floor level was provided with a *constant strength* as required to resist the design force at the first floor (**CS-CSTG**). This stiffness-strength configuration ensured that every floor level had a constant strength-to-stiffness ratio.

Likely storey strength-to-stiffness ratios for realistic structures were determined based on approximate empirical relations (Priestley *et al.* 2007) giving yield drift ratios for different types of lateral force resisting systems. Based on this, lower and upper limits of storey strength-to-stiffness ratios of 0.3% and 3% were set (Sadashiva 2010), and structures with storey strength-to-stiffness ratios outside this range were eliminated from this study.

Design Approach: NZS 1170.5 Equivalent Static Method

All the structures were designed according to the NZS 1170.5 ES method. The design parameters and assumptions were:

- Site sub-class chosen - A;
- Shortest distance from the site to the nearest fault, $D = 0$;
- Return period factor, $R_u = 1$;
- Zone hazard factor, $Z = 0.4$ (Wellington), 0.22 (Christchurch) and 0.13 (Auckland);
- Structural ductility factor, $\mu = 1, 2, 4$ and 6;
- Structural performance factor, S_p, for the appropriate μ value;
- Period calculation – Eigenvalue analysis ignoring P-Delta effects.

After calculation of the design base shear without consideration of P-Delta effects, V, P-Delta effects were considered according to *Section 6.5.4.2-Method B* to obtain the additional structural actions and displacements. The horizontal design action coefficient, C_d, used for determining the design base shear is given by Equation 3 (*Cl. 5.2.1.1, NZS 1170.5*).

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu} \quad (1)$$

$$C_{d,\min} = \max\begin{cases}(Z/20+0.02)R_u \\ 0.03R_u\end{cases} \quad (2)$$

$$C_d = \max\{C_d(T_1), C_{d,\min}\} \quad (3)$$

As an example, for a structure in a zone with hazard factor of 0.4 (e.g. Wellington), Figure 3 shows coefficient C_d (calculated according to Equation 1) plotted against fundamental period for ductility factors of 1, 2, 4 and 6. The solid line shows the minimum value of this coefficient, $C_{d,\min}$, calculated according to Equation 2.

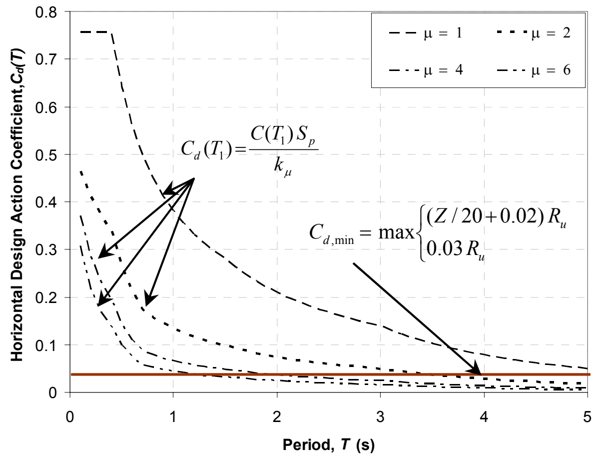


Figure 3: Variation of Horizontal Design Action Coefficient with Fundamental Period for a Zone with Hazard Factor of 0.4.

According to Figure 3, when long period structures are designed with a design ductility factor, the base shear may be governed by Equation 2 which is the horizontal line in Figure 3. These structures have an effective ductility factor lower than the design ductility factor. For example, in Figure 3, for structures designed to have ductility factor of 2 and with periods more than 3.4s, the coefficient C_d is governed by Equation 2, and the effective ductility factor is less than 2. For this reason, structures having base shear governed by Equation 2 were eliminated from this study. Many structures designed for Auckland and Christchurch were controlled by this criterion. So, only results obtained for Wellington are shown in this paper.

Incorporation of Mass Irregularity

A structure with mass irregularity was obtained in the following way. The mass of one floor of a regular structure was modified at a time by means of a mass ratio. Here, mass ratio is defined as the ratio of the modified mass in the new irregular structure, to that of the regular structure. Four mass ratios of 1.5, 2.5, 3.5 and 5 were used. These were applied separately at the first floor level, mid-height and at the roof. This is illustrated in Figures 2(d) to 2(f) for the 9 storey structures. Each time the mass was increased, the structures were redesigned according to the method explained in the earlier section until the critical floor/floors had the target interstorey drift ratio. It should however be noted that the natural periods and base shears of the irregular structures were slightly different from those of the regular structures because their stiffness distribution was adjusted to produce the same specified design interstorey drift as the corresponding CISDR and CS regular structures.

STRUCTURAL MODELLING AND ANALYSIS

To investigate the effects of structural modelling on interstorey drift demands, each frame was modelled in two ways. Frames were initially modelled as a vertical *shear beam* (**SB**), assuming that the columns develop a point of contraflexure at the mid-height of each storey under the earthquake loading (Tagawa *et al.* 2004). Secondly, each frame was modelled as a combination of vertical *shear* beam and a vertical *flexural beam* (representing all of the continuous columns in the structure) that is pinned at the base. This model is labelled as **SFB**, and is shown in Figure 4. The shear beams had a bilinear hysteresis loop with a bilinear factor of 1 %.

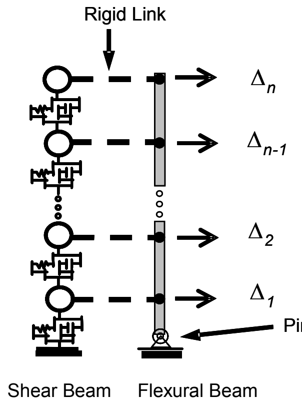


Figure 4: Combined Vertical Shear and Flexural Beam (SFB model).

Previous studies (MacRae *et al.* 2004, Tagawa *et al.* 2006) have shown that the SFB model can represent frame behaviour well. A parameter defined as continuous column stiffness ratio α_{cci} (MacRae *et al.* 2004), representing the stiffness of flexural beam relative to the shear beam at the i^{th} floor level is computed using Equation 4.

$$\alpha_{cci} = \frac{E\,I_i}{H_i^{\,3}\,K_{oi}} \tag{4}$$

where α_{cci} = continuous column stiffness ratio at floor level, i;

E = elastic modulus;

I_i = moment of inertia at the i^{th} floor level;

H_i = storey height of the i^{th} floor level; and

K_{oi} = initial lateral stiffness of the i^{th} floor level.

It is shown (Tagawa 2005) that when parameter $\alpha_{cci} = 0$, the structure behaves like a shear structure in which each storey behaves as an independent *single-degree-of-freedom* (**SDOF**) system. For structures with low post-elastic stiffness, this can result in large interstorey drift concentrations due to soft storey mechanisms. Tagawa has also shown that for real frames, α_{cci} varies between 0.13 and 1.58, and the variation in response between these values was small. Hence, for this study a continuous column stiffness ratio of 0.5 at any floor level was assumed and the additional moment of inertia at each floor level was calculated according to Equation 4.

Choice of Damping Model for Time-History Analysis

A common damping model available in most of the time-history programs (e.g., SAP 2000, ETABS, RUAUMOKO etc.) for linear and non-linear analysis of *multi-degree-of-freedom* (**MDOF**) systems is the Rayleigh damping model, shown in Figure 5. This type of damping model has structural damping matrix, [*C*], given as the summation of mass and stiffness proportional damping models, where the mass damping matrix is given as the product of the proportionality constant, a_o and the mass matrix, [*M*], and the stiffness damping matrix is given as the product of proportionality constant, b_o and the stiffness matrix, [*K*].

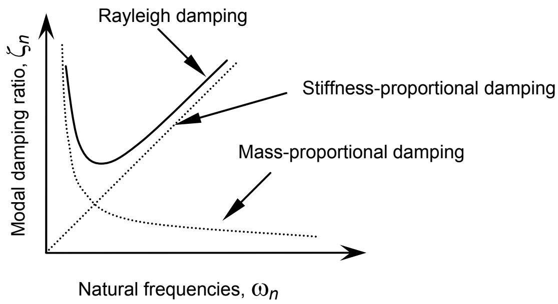


Figure 5: Variation of Modal Damping Ratio with Natural Frequency.

To investigate the effects of different damping models on the responses of regular and irregular structures, three of the Rayleigh damping models available in RUAUMOKO (Carr 2004) time-history program were used. A brief explanation of these damping models is given below.

(a) Initial Stiffness Rayleigh Damping (ISRD): This type of Rayleigh damping is commonly used by the researchers conducting linear or non-linear dynamic time-history analysis. The structural damping matrix is formed considering the elastic stiffness matrix, [K_0], the mass matrix, [*M*], and the proportionality constants based on the elastic frequency, a_o and b_o. The computed damping matrix remains constant throughout the analysis, and the damping forces, F_{damp} , at time *t,* are obtained as a product of the damping matrix and the current velocity, $\dot{u}$, as given by Equation 5.

$$\{F_{damp}(t)\} = [a_0[M] + b_0[K_0]]\{\dot{u}\} \tag{5}$$

During non-linear analysis the structure softens by yielding. This results in a decrease in the tangent structural stiffness and instantaneous natural frequencies. Therefore the damping matrix, which is based on the initial stiffness and initial elastic frequencies, causes a higher damping ratio in the yielding structure which may be unrealistic (Otani 1981).

(b) Tangent Stiffness Rayleigh Damping with Incremental Equation of Motion (TSRD): This is a modified version of initial stiffness Rayleigh damping, and it considers the nonlinearity effects. It uses Newmark's formation of the *Equation of Motion* (**EOM**) in terms of incremental equilibrium. In this case, as the structure yields and stiffness reduces, the tangent stiffness matrix, $[K_t]$ is utilised to compute the damping matrix at each time-step. The damping forces are adjusted in each time-step with the increment of damping forces being product of the tangent damping matrix and the incremental velocities in the structure, $\Delta\dot{u}$. The incremental damping forces are then added to the damping forces existing in the structure at the beginning of time-step to give damping forces at the end of time-step, as in Equation 6.

$$\{F_{damp}(t+\Delta t)\} = \{F_{damp}(t)\} + [a_0[M] + b_0[K_t]]\{\Delta\dot{u}\} \qquad (6)$$

Since the tangent damping matrix is obtained from the tangent stiffness matrix, using the incremental solution to the EOM, at the end of earthquake when the velocity of the structure goes to zero, the damping forces do not necessarily go to zero, thus resulting in unrealistic residual damping forces and displacements.

(c) Tangent Stiffness Rayleigh Damping with Total Equation of Motion (TASRD): This is a modified version of tangent stiffness Rayleigh damping using the absolute form of the EOM. The damping forces at a time-step, given by Equation 7, are computed as the product of tangent damping matrix (obtained from tangent stiffness matrix) and the instantaneous velocities of the structure.

$$\{F_{damp}(t)\} = [a_0[M] + b_0[K_t]]\{\dot{u}\} \qquad (7)$$

This damping model has the properties that: (a) damping forces go to zero at the end of excitation; and (b) damping is appropriate while the structure is elastic.

In addition to careful choice of appropriate damping model, the two modes chosen to apply the user specified damping ratio should be carefully chosen in the analysis. Crisp (1980) has shown that improper selection of modes for applying the damping ratios could lead to high levels of viscous damping in higher modes of free vibration of a structure. Thus for all the analyses, the first mode and the mode corresponding to the number of storeys in the structure were nominated as the two modes with 5% of critical damping (Carr 2004).

Selection and Scaling of Earthquake Ground Motions for Time-History Analysis

Recent work by Baker (2007) has shown that random ground motion record selection can produce unrealistic scaling and increase the scatter of absolute responses. He also suggests that the records matching the shape of the uniform hazard spectrum may incorrectly evaluate the response at different periods. It is expected that the record selection and scaling will have less influence on the relative responses used in this study than on the absolute responses. Hence, the 20 SAC (SEAOC-ATC-CUREE) earthquake ground motion records (tabulated in Table 1) for Los Angeles, with probabilities of exceedance of 10% in 50 years were used for the ground motion suite. Response spectra were developed for each of the selected records. The records were scaled to the NZS 1170.5 elastic design level spectral acceleration values for the period calculated in the city chosen. Here, both the structural ductility factor and the structural performance factor were unity.

Table 1: Ground Motion Suite for Time-History Analysis.

SAC Name	Earthquake Record	Moment Magnitude	PGA (g)
LA01	Imperial Valley, 1940, El Centro	6.9	0.461
LA02	Imperial Valley, 1940, El Centro	6.9	0.676
LA03	Imperial Valley, 1979, Array #05	6.5	0.393
LA04	Imperial Valley, 1979, Array #05	6.5	0.488
LA05	Imperial Valley, 1979, Array #06	6.5	0.301
LA06	Imperial Valley, 1979, Array #06	6.5	0.234
LA07	Landers, 1992, Barstow	7.3	0.421
LA08	Landers, 1992, Barstow	7.3	0.426
LA09	Landers, 1992, Yermo	7.3	0.52
LA10	Landers, 1992, Yermo	7.3	0.360
LA11	Loma Prieta, 1989, Gilroy	7	0.665
LA12	Loma Prieta, 1989, Gilroy	7	0.970
LA13	Northridge, 1994, Newhall	6.7	0.678
LA14	Northridge, 1994, Newhall	6.7	0.657
LA15	Northridge, 1994, Rinaldi RS	6.7	0.533
LA16	Northridge, 1994, Rinaldi RS	6.7	0.580
LA17	Northridge, 1994, Sylmar	6.7	0.570
LA18	Northridge, 1994, Sylmar	6.7	0.817
LA19	North Palm Springs, 1986	6	1.02
LA20	North Palm Springs, 1986	6	0.987

Interpretation of Inelastic Time-History Analysis Results

The RUAUMOKO computer program was used to carry out all the *inelastic dynamic time-history analyses* (**IDTHA**). The peak *interstorey drift ratio* (**ISDR**) within the structure, when subjected to each of the 20 earthquake records, was obtained. It was assumed that the distribution of peak ISDR is lognormal (Cornell *et al.* 2002), so the median and dispersion were found as per Equations 8 and 9.

$$\hat{x} = e^{\left(\frac{1}{n}\sum_{i=1}^{n}\ln(x_i)\right)} \qquad (8)$$

$$\sigma_{\ln x} = \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n}(\ln x_i - \ln\hat{x})^2} \qquad (9)$$

where x_i = peak interstorey drift ratio due to i^{th} record; and

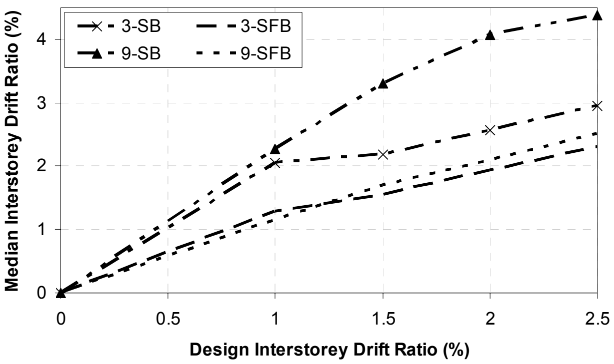
n = total number of earthquake records considered.

It should be noted that the label "Median interstorey drift ratio" on all the response plots in the following sections refers to the median peak interstorey drift ratio.

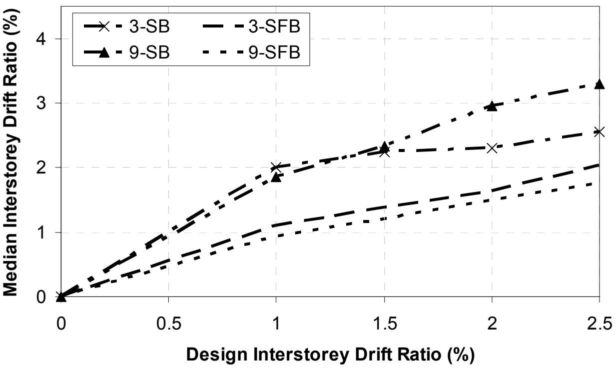
Comparison between SB and SFB models:

Median peak interstorey drift ratios obtained for 3, 9, 15 and 20 storey CISDR and CS-VSTG models, modelled as shear (**SB**) and shear-flexural vertical beams (**SFB**) were compared for each of the design interstorey drift ratios.

Figure 6(a) shows that when 3 and 9 storey regular CISDR structures are modelled as SB, there is significant increase in the median peak ISDR for all the design interstorey drift ratios compared to the corresponding SFB models. The median peak ISDR was on average 40% and 90% more than those obtained when the 3 and 9 storey structures were modelled as SFB respectively. For CS-VSTG regular structures, Figure 6(b) shows that the average increase in median peak ISDR for 3 and 9 storey structures modelled as SB is 53% and 94%, respectively. It was also observed that these average increases in drift increased with the structure's height for both the design models. Since neglecting the effect of column continuity can result in a likelihood of a soft-storey mechanism and high interstorey drift which would not be expected in realistic structures which have continuous columns over their height, the SFB model was used for all further analyses.

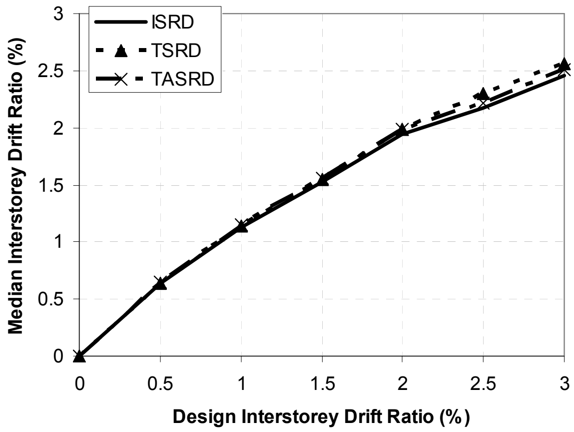


(a) CISDR Design Model

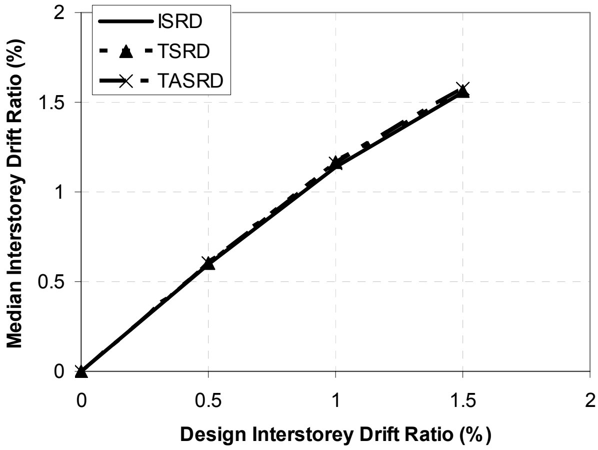


(b) CS-VSTG Design Model

Figure 6: Comparison between SB and SFB Models for Regular CISDR and CS-VSTG Design Models (structural ductility factor, μ = 4).



(a) Number of Storeys: 3



(b) Number of Storeys: 9

Figure 7: Comparison of Median peak ISDR due to different Damping Models – Regular CISDR Design Models (structural ductility factor, μ = 4).

Comparison between damping models:

The effect of damping model on median peak ISDR for 3 and 9 storey structures designed for a ductility of 4 in Wellington was investigated. The three types of Rayleigh damping models explained in the earlier section were considered for this sensitivity study.

Figures 7(a) and 7(b) show median peak ISDR for regular 3 and 9 storey CISDR design models respectively. Due to yielding of the structure and structural stiffness reducing, the TASRD produced on average 2% and 0.7% more median peak ISDR than the ISRD model for 3 and 9 storey structures respectively. The figures also show that there is no significant difference in drift due to TSRD and TASRD damping assumptions.

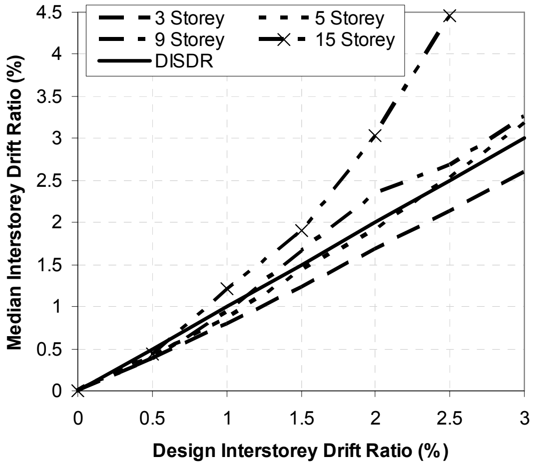
In the case of regular CS-VSTG design models, the TASRD damping assumption produced on average 2.9% and 0.6% more drift than the ISRD damping model for 3 and 9 storey structures respectively. Again there was no apparent difference in drift demands observed due to TSRD and TASRD damping models.

The choice of damping model is likely to be even less important when comparing the relative responses of the regular and the irregular structures. If it is not stated otherwise explicitly below, the TASRD damping model was used for all other analyses in this paper.

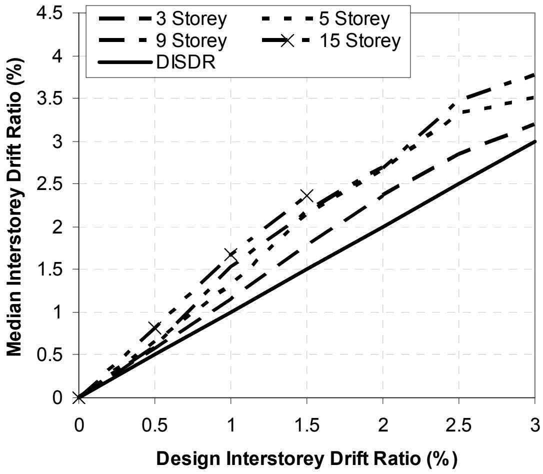
Comparison between IDTHA and code responses:

The NZ 1170.5 ES method is based on the assumption that the peak design interstorey drifts are comparable to those predicted using inelastic dynamic time-history analysis. If this assumption is true, the median peak interstorey drift ratio for the earthquake record suite will be close to the design interstorey drift ratio.

Figures 8(a) and 8(b) show the differences between the code and IDTHA responses for regular CISDR models designed for a structural ductility factor of 1 and 2 respectively. Figure 8(a) shows that when structures were designed for μ = 1, the drift demands for 5 storey structures nearly matched the code drift prediction. It can be seen that the ES method under-predicts the median peak ISDR for taller structures, and for shorter structures the median responses are over-predicted. For structures designed for a ductility factor of 2, Figure 8(b) shows that the ES method non-conservatively estimates the median peak ISDR for all DISDR values irrespective of the structure height. Design interstorey drift ratios for the 15 storey structure are not more than 1.5% because $C_{d,min}$ from Equation 2 controls. When μ was increased to 6, the ES method provided slightly non-conservative estimates of the interstorey drift ratio for DISDR < 2%. Results for higher DISDR values were not obtained because of $C_{d,min}$.



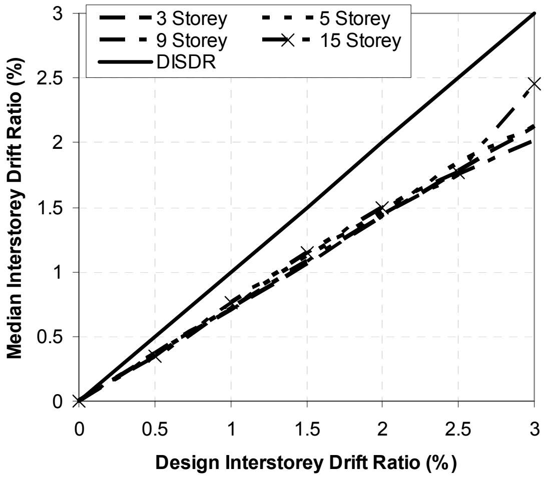
(a) Structural Ductility Factor, μ = 1.



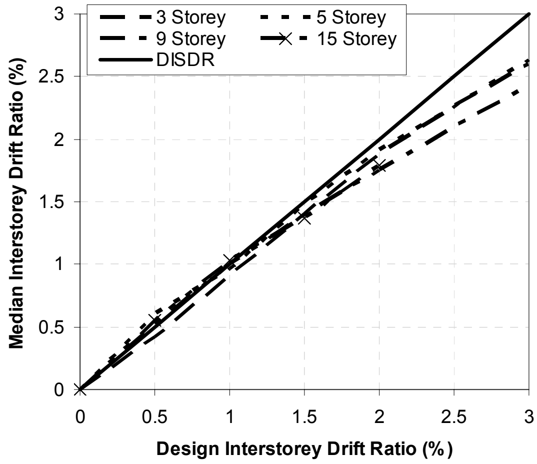
(b) Structural Ductility Factor, μ = 2

Figure 8: Comparison between Actual and Code Response for Regular CISDR Models.

For regular CS-VSTG structures designed for μ = 1, the ES method over-estimates the median peak ISDR for all the DISDR considered in this study as shown in Figure 9(a). Figure 9(b) shows that for any structure height designed for μ = 2, the median drifts closely matched the code prediction for DISDR < 1.5%. Again when μ was increased to 6, it was seen that the code over-estimates the drift demands for DISDR > 1.5%.



(a) Structural Ductility Factor, μ = 1



(b) Structural Ductility Factor, μ = 2

Figure 9: Comparison between Actual and Code Response for Regular CS-VSTG Models.

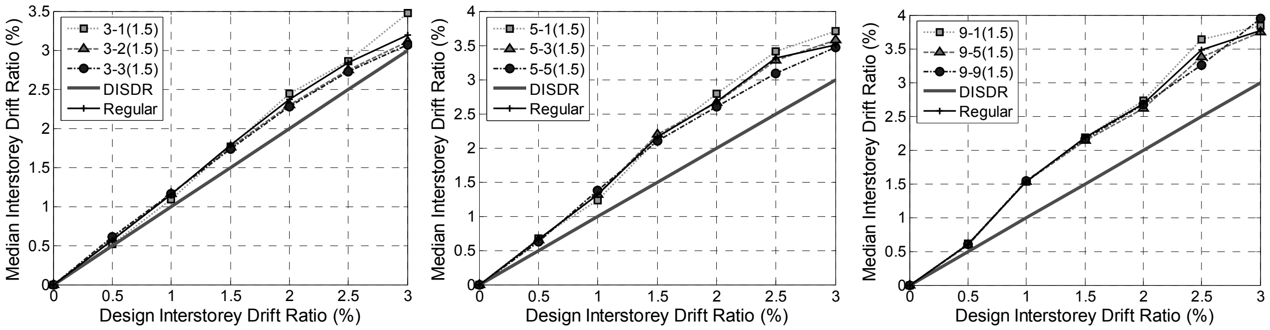
Effect of magnitude and floor level of mass irregularity:

3, 5, 9 and 15 storey structures having mass irregularity were designed for three cities (Wellington, Christchurch and Auckland) considering four structural ductility factors (1, 2, 4 and 6). Four mass ratios of 1.5, 2.5, 3.5 and 5 times the floor mass at the corresponding floor level of regular structure, were adopted in this study. The chosen mass ratios were placed separately at the first floor level, mid-height and at the topmost floor level. All the structures were modelled as a combination of vertical shear and flexural beams. Tangent stiffness Rayleigh damping based on the total equilibrium (TASRD) was used and inelastic time-history analyses were carried out to obtain peak interstorey drift responses. Median peak ISDR responses from irregular structures were then compared with those corresponding to regular structures to estimate the increase in drift demand for each of the design interstorey drift ratios.

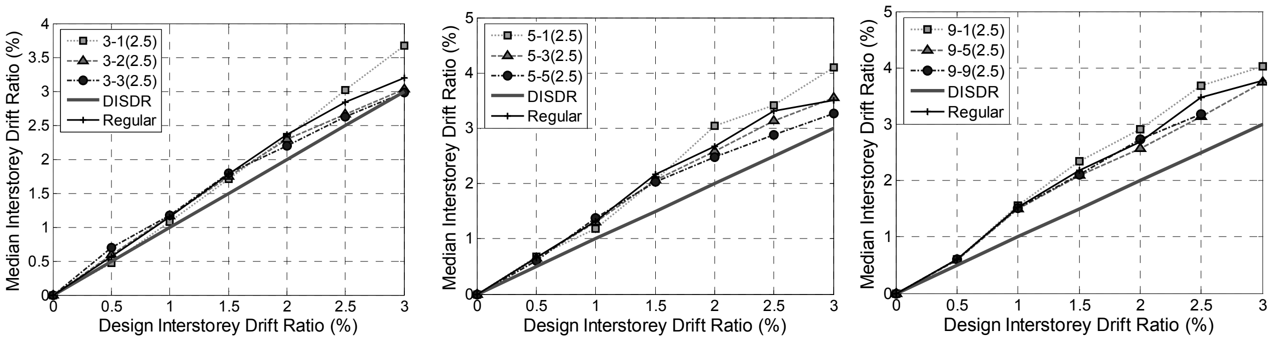
For brevity, only results for 3, 5 and 9 storey Wellington structures designed for a ductility factor of 2 are presented in the following plots to show the effect of mass irregularity. The ductility of 2 was chosen because the Wellington structures designed to this level are not generally governed by Equation 2; so many data points are available. The response plot labels in the following figures have the format "N-L(Q)", where N refers to the number of storeys in the structure, L refers to the floor level of the irregularity and Q defines the magnitude of irregularity (mass ratio).

Figure 10 shows the effect of magnitude and floor level of mass irregularity on the median peak ISDR for CISDR design models. For all structure heights, the maximum increase in median peak ISDR due to a mass ratio of 1.5 at the first and topmost floor level relative to that of regular structures is less than 9% and 6% respectively. For many design interstorey drift ratios, it can be seen that the regular structures produced slightly higher drift demands than when the mass ratio of 1.5 was applied at the mid-height of all structures. The maximum increase in median peak ISDR at this height over the regular structure was less than 2%. The mass ratio was increased from 1.5 to a maximum of 5 at the three chosen floor levels. When the increased mass was present at the top or bottom floor levels, it tended to produce higher drift demands than for regular structures. For taller structures the mass ratio at the mid-height produced lower median peak ISDR than those from regular structures. Maximum increases in median peak ISDR of 20% and 25% were observed due to any mass irregularity present at the first and the topmost floor level respectively. The increase in drift demands due to increased mass at the mid-height was less than 6%.

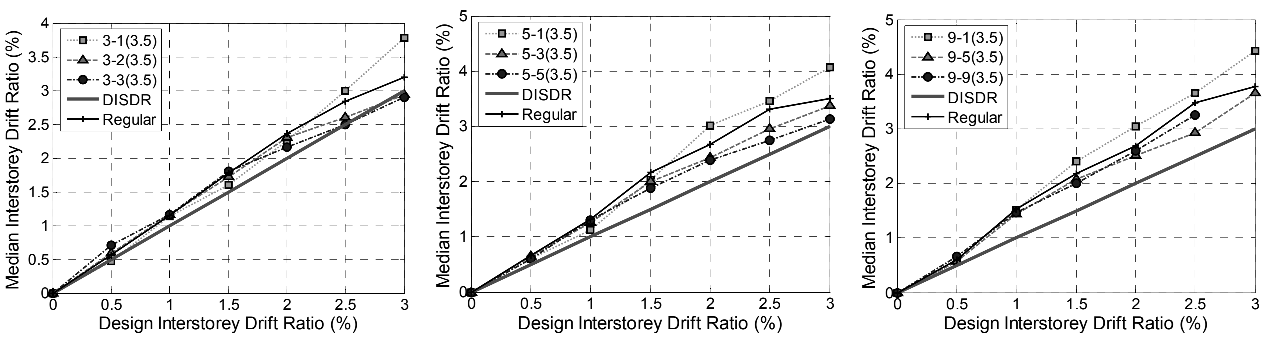
The effect of mass irregularities' magnitude and floor level on median peak ISDR for CS-VSTG design models is shown in Figure 11. It is seen that when a mass ratio of 1.5 is applied at the topmost floor level, it produces a maximum of 8% increase in drift demand due to irregularity considering all the structure heights. A mass ratio of 1.5 at the first floor level or at the



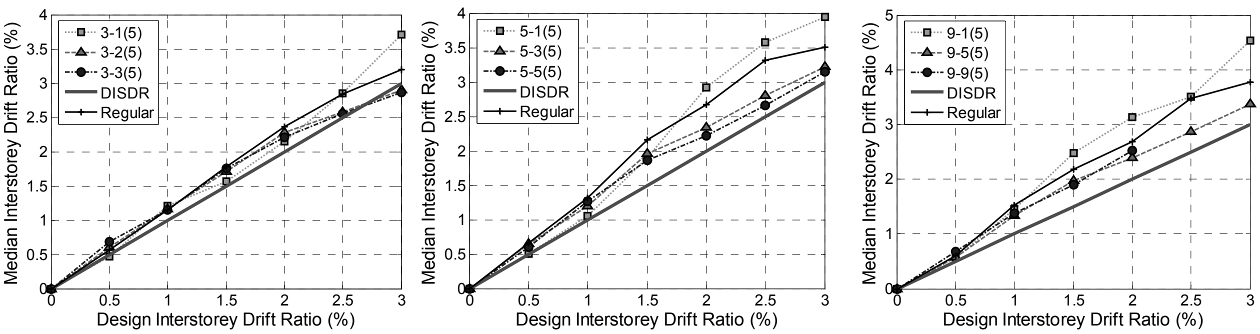
(a) Mass Ratio: 1.5



(b) Mass Ratio: 2.5



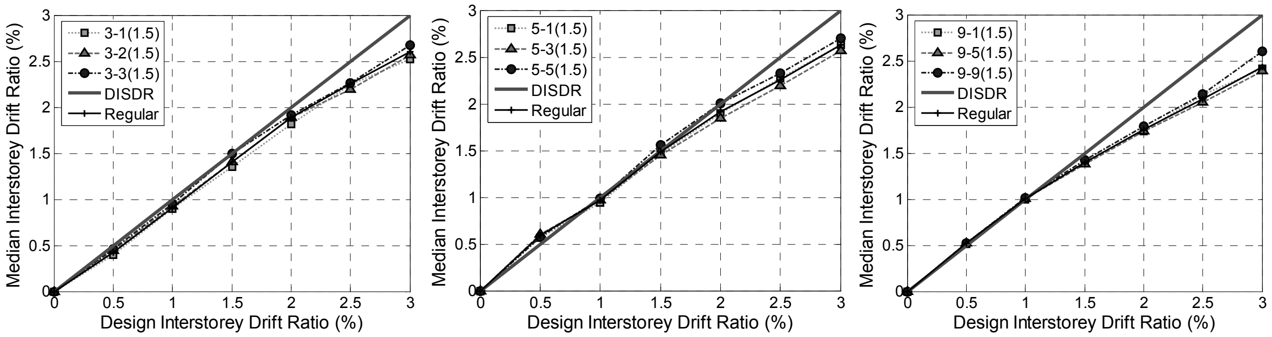
(c) Mass Ratio: 3.5



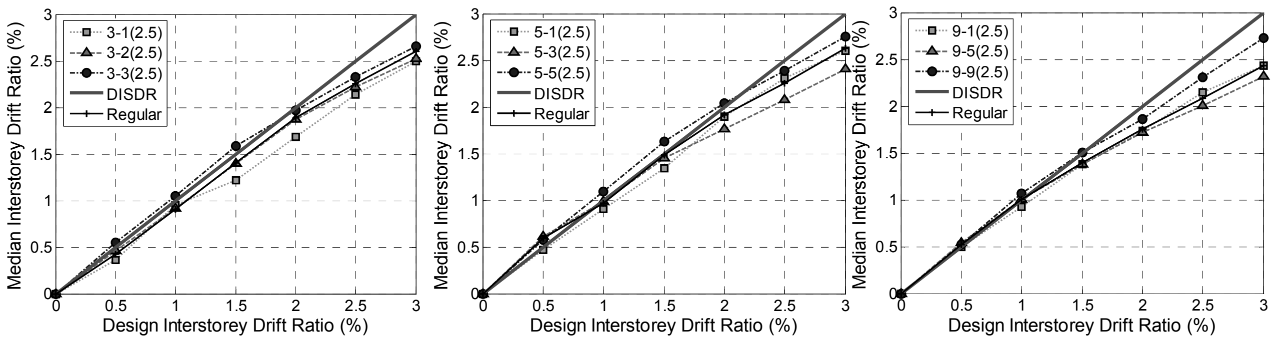
(d) Mass Ratio: 5

Figure 10: Effect of Magnitude and Floor Level of Mass Irregularity for CISDR Model (μ = 2, Z = 0.4).

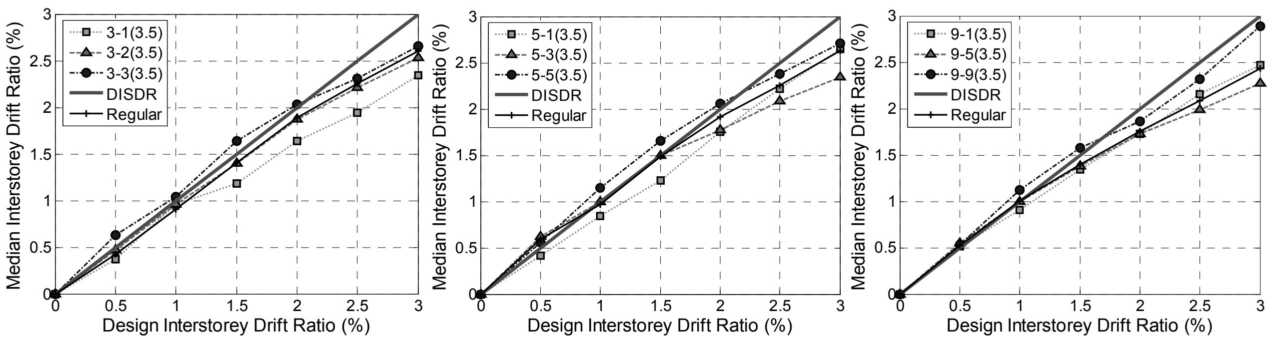
mid-height of all the structures generally tends to produce lesser drift demands than regular structures. When the mass ratio is increased from 1.5 to a maximum of 5, the maximum increase in median peak ISDR due to any mass ratio was found to be less than 6% and 19% for increased mass at the first floor level and the mid-height of structures respectively, and 56% for mass ratio at the topmost floor level. Again, the drift demands over those from regular structures tended to decrease for higher DISDR when any mass ratio was applied at the first floor level or at the mid-height.



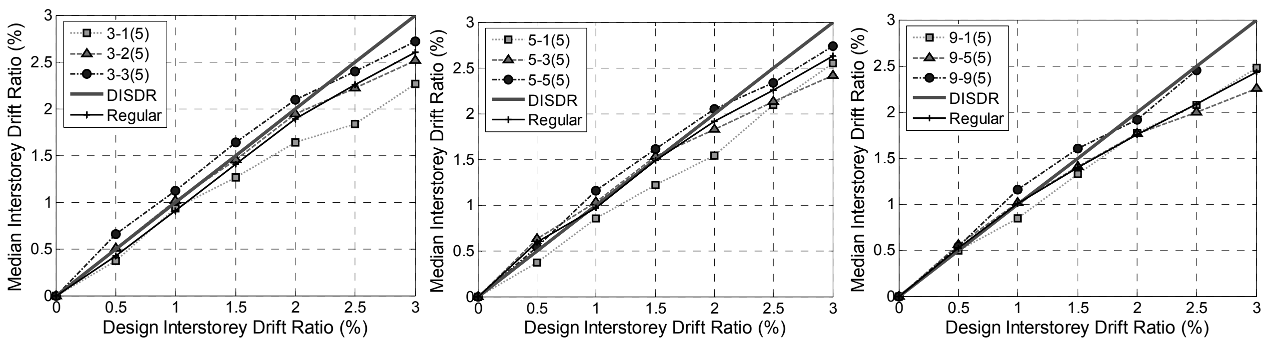
(a) Mass Ratio: 1.5



(b) Mass Ratio: 2.5



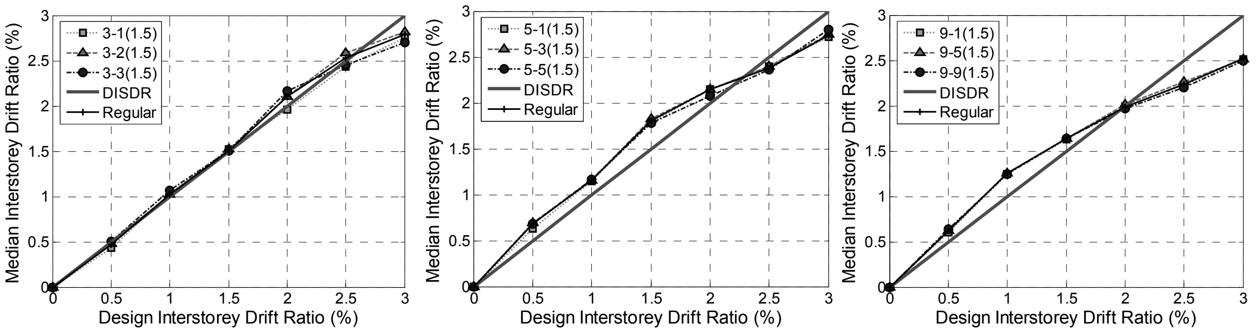
(c) Mass Ratio: 3.5



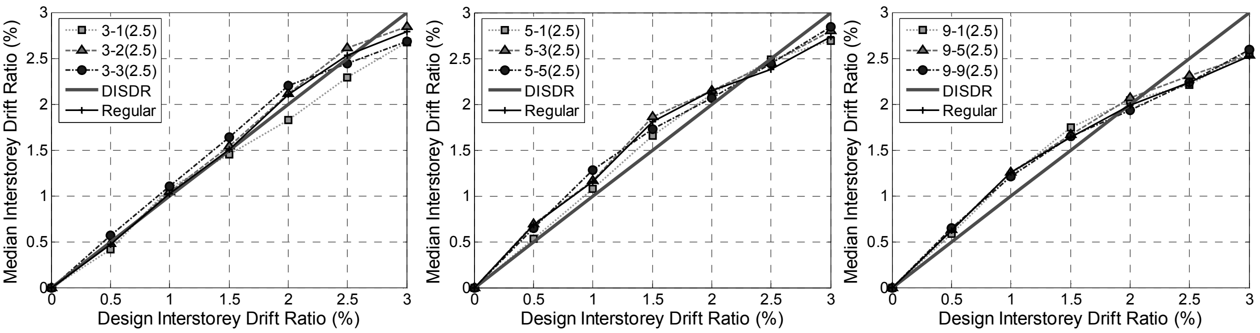
(d) Mass Ratio: 5

Figure 11: Effect of Magnitude and Floor Level of Mass Irregularity for CS-VSTG Model (μ = 2, Z = 0.4).

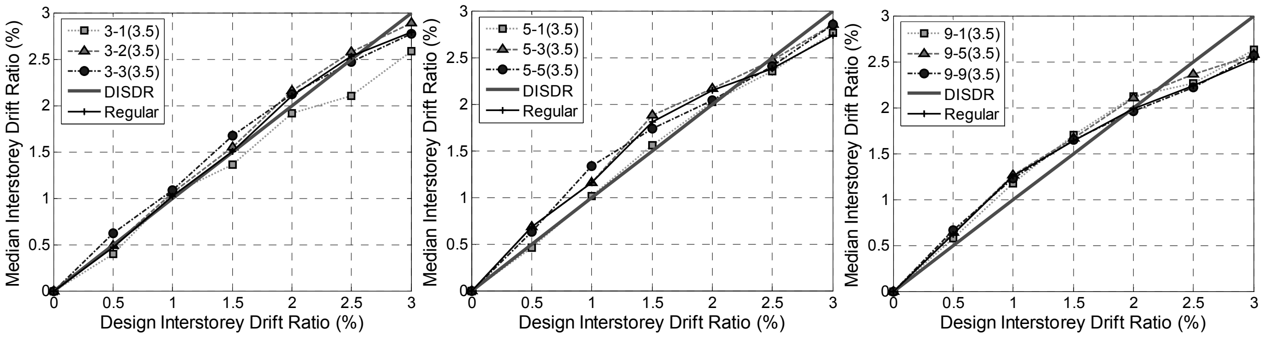
Figure 12 shows the effect of mass ratio and floor level on the CS-CSTG model, designed for structural ductility factor of 2. For a mass ratio of 1.5, Figure 12 (a) shows that the maximum increase in median peak ISDR is produced when the increased mass is present at the topmost floor level for all structure heights. When the mass ratio was increased to 5, the maximum increase in median peak ISDR is shown in Figure 12 (d) to be 40%, obtained from 3 storey structure having the increased mass at the roof. Figure 12 also shows that the effect of mass irregularity decreases as the structure height increases, irrespective of the magnitude of mass irregularity. It can also be seen that for many cases, the increased mass at the first floor level tended to produce lesser median peak ISDR than the regular structure.



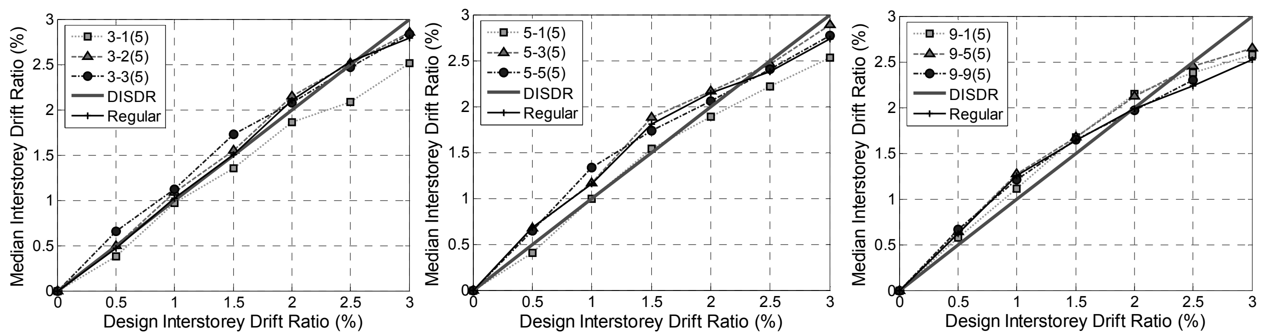
(a) Mass Ratio: 1.5



(b) Mass Ratio: 2.5



(c) Mass Ratio: 3.5



(d) Mass Ratio: 5

Figure 12: Effect of Magnitude and Floor Level of Mass Irregularity for CS-CSTG Model (μ = 2, Z = 0.4).

Determination of Irregularity Limit:

The following steps were used to compute the relationship between the increase in median peak interstorey drift ratio, $ISDR_{incr}$, due to irregularity and mass ratio:

Step 1. For a combination of structural form, structural ductility factor, design interstorey drift ratio, structure height, and the floor level of mass irregularity, the median peak ISDR for the regular structure, $ISDR_R$, is computed from the results of the structure to the suite of records. In a similar way, the median of peak interstorey drift ratio for the irregular structure, $ISDR_I$, is obtained. Thus, the increase in median peak ISDR due to irregularity, $ISDR_{incr}$, is calculated by Equation 10.

$$ISDR_{incr} = \left(\frac{ISDR_I}{ISDR_R} - 1 \right) * 100\ (\%) \qquad (10)$$

Step 2. Step 1 is repeated for all the combinations of structural form, structural ductility factor, design interstorey drift ratio, structure height, floor level of mass irregularity, and mass ratio.

Step 3. For each mass ratio, find the maximum value of $ISDR_{incr}$, $ISDR_{max\_incr}$.

Figure 13 shows $ISDR_{max\_incr}$ plotted against mass ratio for structures with specified structural ductility factors. Generally the structures with the greatest increase in drift due to irregularity were the CS models as shown in Figure 13. However, those with the greatest absolute drifts were the CISDR models as shown in Figures 10 to 12.

Equation 11 has been developed to estimate the increase in seismic demand due to irregularity for any structure. The relationship between *IRR* and mass ratio is shown in Figure 13. Here *IRR* is generally a conservative estimate of $ISDR_{max\_incr}$ because it is based on the most critical structural form, structure height, structural ductility factor, design interstorey drift ratio and floor level of irregularity.

$$IRR = 15(MR - 1) \qquad (11)$$

where *IRR* is the Irregular response greater than regular response; and *MR* is the Mass ratio.

Equation 11 may be used for design. For example, if it were decided that the mass irregularity should produce less than 15% additional interstorey drift, then Figure 14 shows that the mass ratio needs to be less than 2. The figure also shows that the code mass ratio of 1.5 corresponds to an increase in median peak ISDR of up to approximately 7.5%.

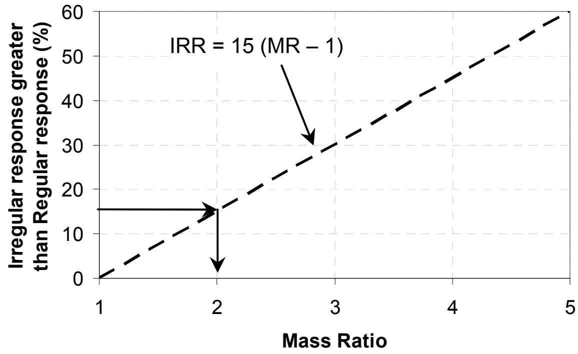
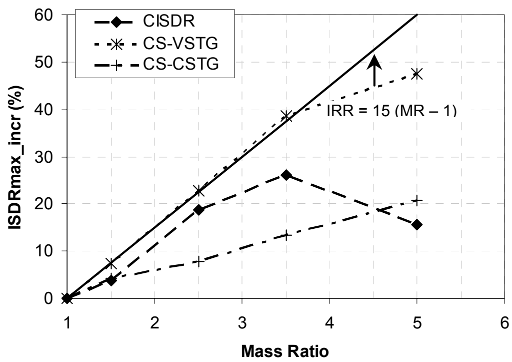
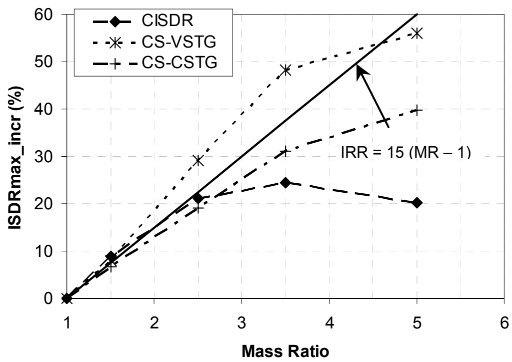


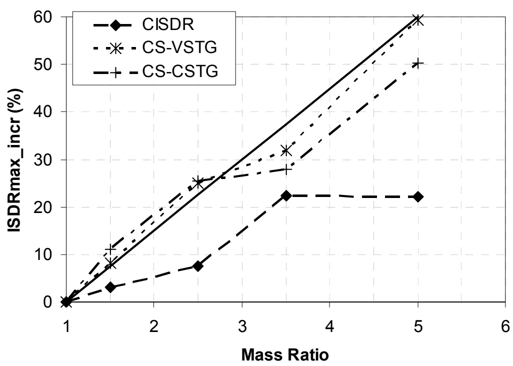
Figure 14: Determination of Mass Irregularity Limit.



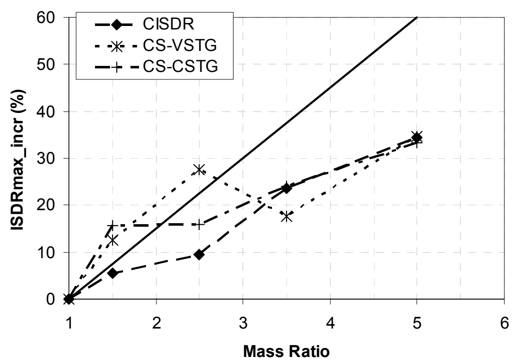
(a) Structural Ductility Factor, μ = 1



(b) Structural Ductility Factor, μ = 2



(c) Structural Ductility Factor, μ = 4



(d) Structural Ductility Factor, μ = 6

Figure 13: Increase in Median peak ISDR due to Mass Irregularities in Structures designed for different Structural Ductility Factors.

SUMMARY

This study on structural irregularity effects can be summarised as below:

1. Current regularity provisions in NZS 1170.5 are based on overseas irregularity recommendations. They are based on engineering judgement and lack rational justification.
2. Past research on vertical irregularities effects does not justify the appropriateness of regularity limits slated in NZS 1170.5. A better and more meaningful comparison is obtained if structures designed to a target drift are compared with the actual drift demand rather than tuning the structures to have the same period. Also, earlier works may not be appropriate for structures designed for New Zealand;
3. A method to quantify vertical irregularity effects was proposed for all irregularity types. This method was applied to evaluate the effects of mass irregularity on simple shear-type structures with 3, 5, 9 and 15 storey heights, assumed to be located in Wellington, Christchurch and Auckland, and designed for a range of structural ductility factors;
4. Regular structures were defined to have a constant floor mass at every floor level using the NZ 1170.5 Equivalent Static method. The structures were either designed to have constant interstorey drift ratios at all the floors simultaneously (CISDR), or to have a uniform stiffness distribution over the height of structure (CS). Various target interstorey drift ratios were considered. The CISDR models had constant strength-to-stiffness ratios at all floor levels. Two CS models were considered. One had storey shear strengths matching the design shears at every floor level (CS-VSTG). The other had constant shear strength over all the floor levels (CS-CSTG). The shear strength in this case matched the design shear in the first floor level of the frame. Irregular structures were created with floor masses of magnitude 1.5, 2.5, 3.5 and 5 times the regular floor mass. These increased masses were considered separately at the first floor level, mid-height, and at the roof. The irregular structures were designed in exactly the same way as the regular structures with the same target drifts. All the structures were then analysed using inelastic dynamic time-history analysis to obtain the change in the median peak interstorey drift demand due to mass irregularity;
5. The structures were modelled as a combination of vertical shear and flexural beam elements (SFB). The flexural beam, modelling the effect of all continuous columns in the structure, has been shown to represent 2-D frame behaviour well;
6. The choice of the type of Rayleigh damping model for the inelastic time-history analysis was shown to be not significant for assessing irregularity effects. This is because the absolute drifts were not sensitive to the damping model. Furthermore, when comparing both the regular and irregular structure response, it is the relative rather than the absolute response which is important. This difference is likely to be less than for the absolute response;
7. Median peak interstorey drift ratio estimates due to NZS 1170.5 ES method was found to be generally non-conservative for CISDR designs, and conservative for CS designs;
8. The drift demands were sensitive to both the magnitude and floor level of the mass irregularity. Increased mass, when present at either the first floor level or at the roof, produced higher drift demands than when located at the mid-height;
9. A simple equation was developed to provide a general conservative estimate of the increase in drift demand due to mass irregularity. The current code requirement of a maximum mass ratio of 1.5 corresponds to an increase in median response of approximately 7.5% according to this equation.

ACKNOWLEDGEMENTS

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