Influence of column base rotational stiffness and Non-structural elements on moment resisting steel frame building response to severe earthquakes

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ABSTRACT: Perimeter Moment resisting steel frames (PMRSFs) are a commonly used seismic resisting system, placed around the perimeter of the building for maximum torsional stiffness. They are typically designed as "strong column weak beam" systems with fixed column bases. When subjected to severe earthquake demand, sufficient to push the beams into the inelastic range, it is expected that plastic hinging at the column bases will occur. However, the response of PMRSF systems to the severe 2010/2011 Christchurch earthquake series did not generate column base hinging in systems which exhibited beam yielding.

In the first part of this research undertaken in 2014, representative 4 storey PMRSF building systems with varying column base rotational stiffnesses were subjected to inelastic time history analyses (ITHA) using three of the strong ground motions recorded from the 22 February 2011 M6.3 Christchurch earthquake. The influence of the column base stiffness on the performance of these systems is presented in this paper.

The second part of this project investigated the influence of non-structural wall strength and stiffness on the response of buildings. It was observed that the structural response of buildings which have large numbers of internal fire rated walls may have been influenced by these walls. Three internal wall layouts were considered; no walls, minimal walls representative of an open plan office and many walls typical of a hotel. The results showed that the "many wall" layout did significantly influence the building response while the "minimal wall" layout had little influence. Results are presented in the paper.

1.1 Background

Moment resisting frames are a form of seismic resisting system that utilises rigid frames to resist lateral forces. The fully rigid connections between the beams and columns mean that when the frame displaces laterally, due to the fixed geometry of the connection, rotation must occur in the beams and columns. This rotation can be either elastic or inelastic, depending on the extent of the rotational demand. When the superstructure is pushed into the inelastic range, it will form concentrations of inelastic yielding at specified locations within the PMRSFs.

The New Zealand Steel Design Standard (NZS 3404:1997) has provisions to ensure columns are designed to over strength moments and that the structure will form a strong-column-weak-beam mechanism, isolating inelastic demands to the beam ends and the column bases. The exact location of the plastic hinging and the extent of deformation will vary depending on the distribution of bending moment forces and the strength of the members used. Traditionally structural engineering practices have adopted two types of base detailing. A pinned connection, that assumes no base fixity and results in the moment demand being fully distributed into the column, or a fixed connection, that assumes infinite rigidity and distributes almost the entire moment demand at the column base into the foundations. Realistically, neither of these can be achieved, which is recognised by NZS 3404:1997, which limits the rotational fixity to lower *(equation 1)* and upper *(equation 2)* limits.

$$k_\theta = 0.10 \cdot \frac{E \cdot I_c}{L_c} \quad (1)$$

$$k_\theta = 1.67 \cdot \frac{E \cdot I_c}{L_c} \quad (2)$$

Where E, I_c & L_c apply to the column under consideration

The NZS 3404:1997 limits on rotational stiffness were developed from testing on representative fixed base portal frame columns which are relatively light members (C.4.8.3.4 NZS 3404). However recent testing undertaken on heavy seismic resisting column base connections (AISC 2012) has shown that the upper limit of elastic rotation in these column base connections is typically lower than the current standard limits. This increases the likelihood that the column bases will remain elastic. By allowing the column bases to remain elastic, it assists in allowing the structure to self-center and reduce the amount of residual drift to the structure.

Furthermore to this topic, it has been widely noted that the non-structural elements of a superstructure, in particular internal walls, may have a significant influence on the stiffness of the building. In commercial buildings, it is common for internal walls to be tied into the floor and roof in order to meet requirements for acoustic and fire ratings. This will provide additional lateral stiffness to the superstructure, whether it is intended or not. The New Zealand Loadings Standard (referred to as NZS 1170:2004 hereafter) does not require non-structural elements to be considered in the structural design of new buildings, nor does it provide any guidance on accounting for the additional stiffness exhibited in these elements.

1.2 **Christchurch earthquake observations**

The 2010/2011 Christchurch earthquake series was significant enough to push all of the multi-story steel structures in the CBD into the inelastic range (Bruneau et al. 2011), with yielding being exhibited in beams and active links of MRFs and EBFs, especially the latter. However no yielding was observed in the column bases of any modern steel framed buildings (Clifton et al. 2013), and as a result, column bases remained elastic which contributed to the structures being able to effectively self-center. The steel structures observed had been designed as fixed base connections, in accordance with the rotation limits of NZS 3404, and therefore it was anticipated to see some column base yielding. The lack of column base yielding indicates that either the current detailing of base connections is not developing the intended stiffness, or that the interaction between the soil and foundation is providing additional rotational flexibility or reduction in seismic demand that is not currently being accounted for.

The Christchurch earthquake series also highlighted the impacts of non-structural internal wall elements on the response of multi-story buildings, with the best example being the 23 storey Pacific Tower, for which the seismic-resisting system comprised of Eccentrically Braced Frames (EBFs) full height, but which comprised mix use levels of car parking, office, hotel and apartments, with widely varying cumulative lengths of non-structural internal walls. These impacted on the response of the building, concentrating inelastic demand into the active links over levels six to eight.

1.3 **Objectives**

The primary objective of this research was initially to determine the most appropriate/optimum value of column base rotational stiffness for MRSF's subjected to lateral loads such as those generated by seismic action. In order to define what is considered an "optimum" value, the following performance criteria were set,

- Avoid plastic hinge development in the column base during the ultimate limit state design earthquake.
- The distribution of bending moment is approximately equal at the top and bottom of the first storey column under the ultimate limit state design earthquake.

- The column size in the first storey and the beam size at level one are not controlled by the locally high moments at level one.
- The column does not form a soft storey mechanism under the maximum considered design earthquake.
- The recommended stiffness is realistically achievable using practical construction techniques.

The secondary objective has been to evaluate the effects of non-structural wall strength and stiffness on the seismic performance of MRSFs. To quantify the effects, direct comparisons will be made to a control analysis.

2 METHODOLOGY

2.1 Preliminary design

The analysis has been completed for an $1184m^2$, four story office building. The structure consists of a combination of moment resisting steel frames and gravity resisting systems.

Moment resisting members have been sized as per the capacity design procedures outlined in the code. The members have been designed to resist lateral loads determined via the equivalent static method highlighted in NZS 1170:2004. Columns have been designed to meet the over-strength moments of the primary beams, in order to ensure that a weak-beam-strong-column mechanism. MRSF members have been designed for a range of base fixities, ranging from 0.1 EI/L to 1.67 EI/L, in order to determine a realistic complying design.

2.2 Defining the linear model

The structure has been modelled two dimensionally in order to capture the response of the three bays of moment resisting frames along the short end of the building. Rigid diaphragms were applied at each level of the structure to emulate the effect of rigid floor diaphragms preventing displacement at each level.

In order to not overestimate the stiffness of the structure, it was important to capture the additional seismic mass of the structure that is not directly supported by the MRSF. Each group of three bays of moment resisting frames must resist half the mass of the structure.

In order to account for the seismic mass without reducing the moment capacity of the columns of the MRSF, "dummy columns" were adopted. The properties of the dummy columns were based on the compound value of the total gravity load carrying columns in the plane of action, and pinned at the connection to the ground, so that the column will not carry any moment and reduce the demand on the frame. The dummy column was connected to frame using rigid links with pin ends, in order to effectively transfer the seismic forces into the MRSF. The seismic mass of the structure that is not directly supported by the MRSF was then added as point loads onto the dummy columns.

To account for the rotational stiffness at the base connection, the column was modelled as a fixed connection onto a rotational spring connecting with the foundations. The rotational spring had a specified stiffness in units of Nmm/radian, calculated in accordance with the selected base fixity being analysed. The same treatment was given to the ends of the dummy columns, with these connections having pinned rotational stiffness. These springs were applied in both the plane of the frame and the perpendicular direction, using the appropriate column properties for each direction.

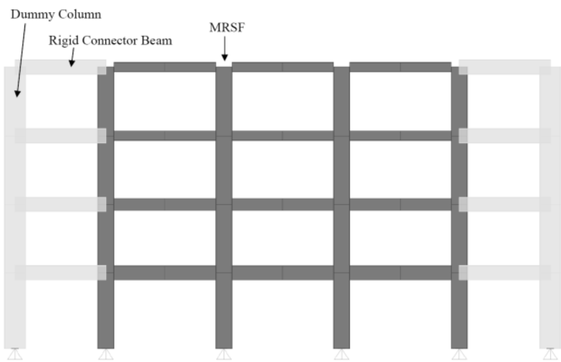

Figure 1. 2-Dimensional model including dummy members.

2.3 Defining non-linear properties

2.3.1 *Steel properties*

All preliminary designs were completed using the following properties for Grade 300 steel. The minimum yield stress was modelled using 300 MPa, and the minimum tensile strength modeled using 440 MPa.

These properties were modelled in SAP2000 to define the backbone curve of the steel members. The backbone curve refers to the relationship between force and deformation that is used to characterise the response of the material in non-linear analysis.

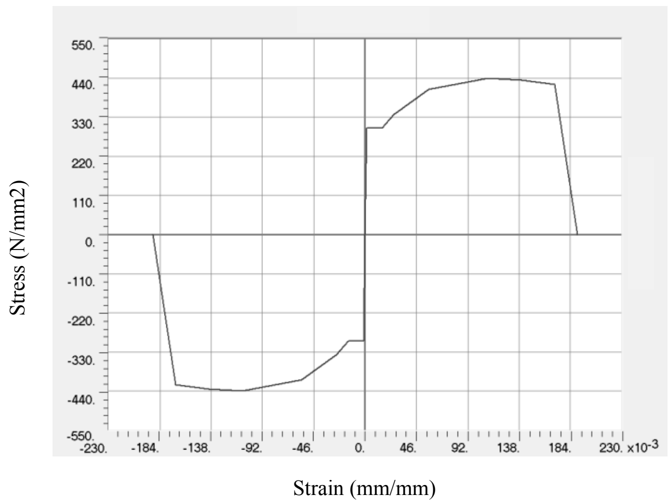

Figure 2. Stress-Strain plot of 300MPa steel.

The hysteretic properties of the steel have been modelled using the "Kinematic hardening" loop in-order to capture the strain hardening effect seen in ductile materials, as observed in metals, under inelastic cyclic loading. This captures the increase due to cyclic strain hardening.

2.3.2 *Non-linear hinges*

For this investigation, two different hinges have been used in the beams and columns. As columns are subject to high axial loads, those axial forces will reduce the moment capacity of the column, so a P-M3 hinge was adopted. This accounts for the interaction between the axial force and moment demand about the columns strong axis. Moment about the weak axis of the column is not accounted for as the frame will not be subjected to significant weak axis moments to warrant the extra computational analysis, as the frames in the perpendicular direction will resist the majority of the moment demand in the opposite direction.

For beams, M3 hinges have been used. These do not account for effect of axial forces on moment capacity, and as the beams are not subject to high axial load, represent an accurate model of the inelastic demand.

2.3.3 *Modelling non-structural walls*

This investigation has undertaken additional research to determine the effect from non-structural walls on the response of the frame, and determine if the energy dissipation from the non-structural walls is significant enough to reduce the inelastic demand on the superstructure.

In order to accurately model the non-structural walls, the model had to capture both the initial stiffness of the wall in the elastic range, and capture the degrading strength during reloading once the wall is pushed beyond the elastic range.

BRANZ conducted a study on the post-earthquake performance of passive fire protection systems (Collier 2005), which included experimental testing on non-structural walls to test the fire protection capabilities of those walls. The results of the experimental testing provided a range of hysteretic curves for a range of typical walls that are used in commercial structures. For this investigation, the hysteretic curve that relates to light gauge steel walls has been used, as that is the most common form of wall construction in commercial structures.

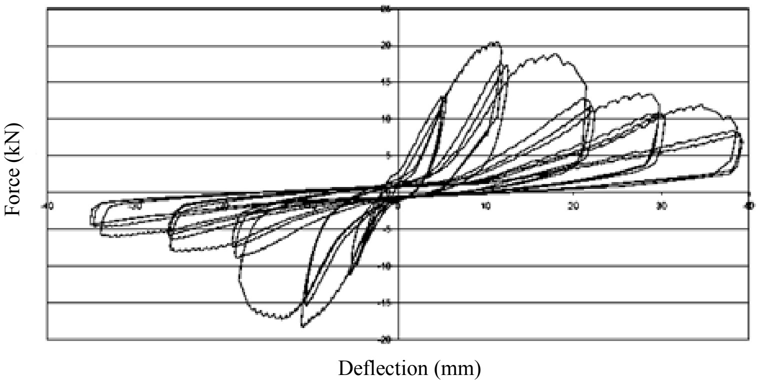

Figure 3. Force-deflection plot of steel framed non-structural walls from experimental testing.

Observing the backbone of the hysteretic curve, it can be seen that the wall undergoes a linearly elastic slope until it reaches the yield point. From which it then follows a relatively consistent degradation in strength, as a result of degradation between the connection of the linings to the frame, cracking of the linings and, at high deflections, damage to the internal framing of the wall. Overall the wall displays a significant ductility with degrading strength and works as an effective energy dissipating mechanism. The experimental data shows a significant amount of in-cycle stiffness degradation, particularly in the reloading phase. Also observed is a large amount of “pinching”, which refers to the low stiffness region that forms when the force changes direction.

In order to accurately simulate the effect of the non-structural walls, it is important to capture the observed features of the hysteretic curve to accurately replicate the level of energy dissipation. To simulate the non-structural walls, the non-linear link feature available within SAP2000 was utilised. The non-linear link element connects two nodes together, and enables the user to apply non-linear properties to that link. Testing on that link within SAP using a single degree of freedom “lollipop structure” set up between two node points was undertaken to tune the input properties of the hinge to accurately match the non-linear behavior described from the BRANZ testing. The “pivot” hysteretic loop option in SAP2000 was used for this representation, as it is best able to capture the degrading hysteretic behaviour observed experimentally.

3 RESULTS

3.1 Introduction

The results presented in this report consist of; the extent of inelastic demand, the structural drift and the moment distribution, for each base rotational stiffness tested. All results presented were obtained from analysing the structure under the GBGS west, unscaled load case (as described in the methodology). Lastly the results of the influence of non-structural walls on structural performance will be presented. The load case used here was the CBGS west case, but scaled down to the NZS1170 ULS design earthquake.

The frames for each rotational base stiffness were analysed under all load cases, both scaled and unscaled, and some reference will be made to results from other load cases. Due to the extremely large quantity of analytical data produced, only key results are presented here.

3.2 Inelastic Demand for different Base Stiffnesses

The response of all the frames, in terms of achieving the design failure mechanism, was strong column weak beam as expected. The video capability of SAP2000 allowed the real time viewing of the frames responding to the earthquake records. In all cases it could be seen that the beams started to yield from the first floor upwards, and in the case of the higher column base stiffnesses, 1.5 and 1.67 EI/L, the column bases were the last elements to yield.

The grey circles in the figure represent where a member has undergone plastic hinging. The severity of yielding increases with increasing darkness of dot colour. The lightest shade of grey represents immediate occupancy (such as all the column bases on the 1.5 and 1.67 EI/L frames). Where no hinge is shown, as such for the 0.1EI/L case at the column base, no inelastic deformation of the member has occurred, and the member has remained elastic.

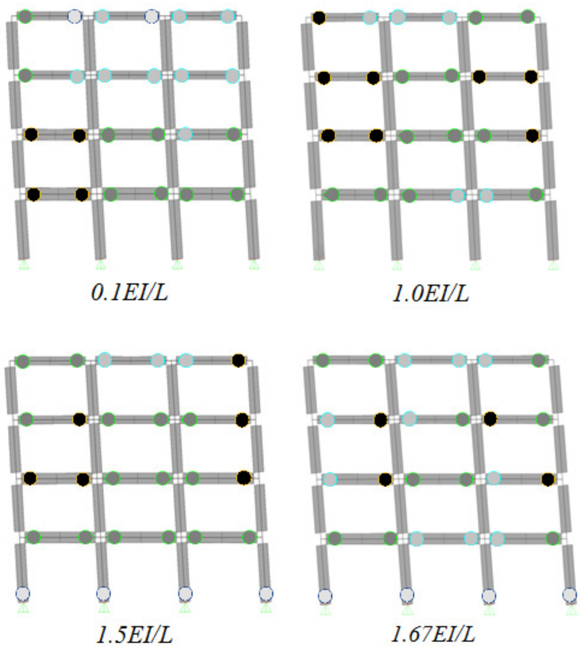

Figure 4. Inelastic demand in each frame for different base spring stiffnesses.

3.3 Structural Drift for Different Base Stiffnesses

The structural drift of the frame is the displacement from centre that it experiences during lateral loading. Two aspects of drift are assessed in the analysis; the absolute displacement of the top of the structure, and the inter-storey displacements.

The New Zealand Standard NZS1170 currently states that the maximum drift of a structure must not exceed 2.5% of the structural height. The frames modelled are 15.8m in height therefore the drift from centre must not exceed 395mm in either direction. The results show that each stiffness frame came under this allowable limit for the unscaled record. The peak interstory drift, which occurs over the first floor, decreases in the order of 10% for the greater base stiffness.

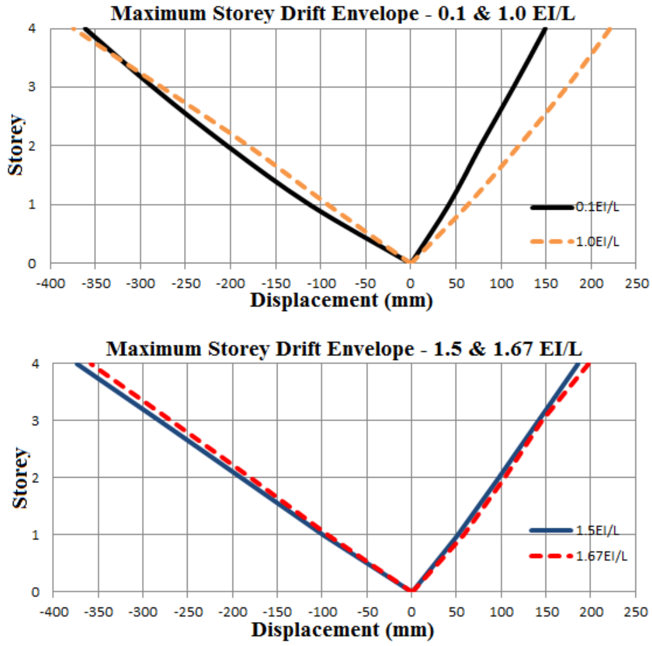

Figure 5. Storey Displacement.

3.4 Influence of non-structural walls

After the base rotational stiffness analyses were completed , non-structural walls were added using horizontal springs between the storey level. There were two scenarios modelled which were intended to represent the least and most lengths of non-structural wall in the direction of lateral load application. These cases represented a typical office style building and a hotel/apartment style building.

The non-structural walls were added to determine if they affected structural performance. Therefore, the same analyses was run on the 1.67EI/L model under the NZS1170 scaled CBGS west earthquake record for the models both with and without the non-structural walls so a direct comparison could be made. The non-structural walls were not added to the ground floor of the structure, to replicate a typical commercial floor plan where the ground floor is an open plan lobby area. The results for inter-storey drift, elastic rotation in the column bases and inelastic demand in the beams are shown below, with further numerical results in the appendix.

Table 1. Drift Results.

Peak Inter-storey Drift (mm)

Stories	None	Office	Hotel
4-3	38	33	24
3-2	39	36	29
2-1	39	38	34
1-0	45	44	41

Table 2. Elastic rotation of the column base hinge.

Elastic Column Base Rotation (radians)

Column	None	Office	Hotel
A	0.007	0.0069	0.0066
B	0.0073	0.0073	0.0069
C	0.0073	0.0073	0.0069
D	0.007	0.0069	0.0066

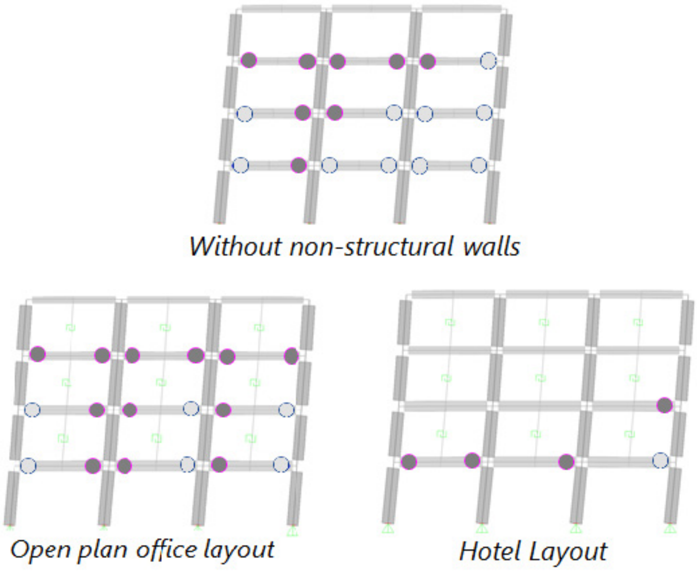

Figure 6. Hinge yielding – non-structural walls.

As before, the severity of yielding increases with increasing darkness of dot colour.

4 DISCUSSION

The absence of column base yielding in the 0.1 EI/L and 1.0 EI/L cases was desirable in assisting with self-centering.. The stiffer cases modelled of 1.5 EI/L & 1.67 EI/L have demonstrated almost identical responses, and were subject to a reasonable amount of base yielding. This would indicate that the optimal base stiffness that will meet the objectives outlined earlier in the report will lie in between the range of 1.0-1.5 EI/L.

The observed response of base yielding in the stiffer models is contrary to the observations made from the Christchurch earthquake series. This is an interesting observation, as the results from this research are what one would expect when considering the stiffness of the as built column bases. Therefore the structural responses observed in Christchurch suggests that the buildings were not developing their intended stiffness that the connections have been detailed for, or else the demand on the superstructure in practice was not as high as the models indicate. The first reason could be poor detailing and design resulting in the connection being weaker than anticipated, however there was no observed column base connection damage to support this. The second possibility is that the soil-foundation-structure interaction has reduced the base rotational stiffness and also potentially the seismic demand on the superstructure, thus reducing the moment demand on the column bases. The second theory has been a point of research recently among structural and geotechnical engineers and is highlighted by the fact that the only steel structures to be demolished in Christchurch due to earthquake damage have been a result of foundation failures.

The additional stiffness of the non-structural walls in the hotel/residential layout is consistent with the responses observed in the Pacific Tower in Christchurch. The results demonstrated a reduction in the first mode period of 20%, and a reduction in the inter-story drift of 22%. While the reductions observed in the analysis are significant, they would not account for the level of additional strength and stiffness shown in the Pacific Tower. The Royal Commission report (*The Performance of Christchurch CBD Buildings*) suggested the structure was twice as stiff and strong as originally designed. This could be a result of the structure being much taller and flexible than the structure modelled in this research; however it is more likely that the vertical car stacker in the lower levels was the other significant contributing factor to the response of the Pacific Tower. The car stacker required the bottom floors that would usually provide significant lateral restraint to be removed in sections, and in contribution to additional stiffness provided in the upper levels of the tower from the non-structural walls, the structure essentially acted as a semi-rigid box on top of a softer than usual base.

It was also observed from the research that under some cases of severe unscaled earthquakes, the non-structural walls actually increased the amount of inelastic deformation exhibited in the structure. This was a result of the addition of the non-structural walls increasing the stiffness of the upper floors of the structure during the first major pulses, resulting in larger demand being place on the columns, pushing them further into the inelastic range. As the first pulse from the earthquake was so large, it pushed the walls well into the inelastic range, and resulted in them having very little residual strength left. Therefore, when the next major pulse occurred during the CBGS west record, inelastic deformation had already occurred in the columns, weakening the structure. The walls provided little resistance against the second pulse as a result of the severe yielding from the first pulse. This resulted in the beams being subjected to very high moment demands on the second pulse, pushing them well into the inelastic range. As a result, both the columns and beams were subjected to unusually high demand than if the non-structural walls were not contributing to the stiffness of the structure.

This highlights the need to improve the current performance of non-structural walls to either provide better hysteric behavior that can dissipate greater amounts of energy throughout the entire loading cycle, or to provide a wall system that is less stiff, and allows the moment resisting frames to behave as if there was no additional elements, as they are modelled in the design office.

5 CONCLUSIONS

From the research, we can conclude that the optimal base stiffness that will achieve the greatest structural performance whilst maintaining more compact and economical section sizes will lie between the range of 1.0EI/L and 1.5EI/L. Further modelling of the structure under a larger range of base flexibilities, as well as modelling of a taller structure, more similar to some of the observed structures from the Canterbury earthquake, will help to provide more definitive information regarding the optimal base flexibility to use in the design process. The contribution of non-structural walls was determined to be significant, and will have an effect on the stiffness of the structure when large lengths of internal walls are used. However, due to poor residual strength of the walls once they have been permanently deformed, they cannot be reliably depended on as a seismic resisting element, and should not be incorporated into the initial structural design of the building. However, in extraordinary cases, where the walls will be placed in large lengths in some locations, and not in others, then consideration should be made for the effect of the non-structural walls. It is recommended in these cases that a non-linear analysis of the structure incorporating these additional elements is completed during the design procedure to determine the effect on the response of the structure.

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