STRENGTH AND STIFFNESS OF TYPICAL PRE-1980 HIGH-RISE CONCRETE BUILDINGS IN VANCOUVER

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ABSTRACT: An ongoing research project involves analyzing reinforced concrete buildings in Vancouver built before 1970 of seven or more storeys. As part of that project enough information was collected on hundreds of buildings to perform simple linear dynamic analysis to determine their strength and stiffness characteristics. This paper presents a descriptive overview of the buildings and a detailed series of analysis looking at their modal properties, displacement demands, and strength demands on individual lateral demand resisting components. It is found that overall the buildings are softer than equivalent buildings around the world, with correspondingly greater displacement demands. The components themselves are also weak relative to the demands expected by modern codes, considering that they are not detailed for significant ductility and will therefore likely have brittle failures. Torsional motions amplify both the displacement demands and the force demands. Lastly, a discussion of the significance of the results is presented, which concludes that the analysis is suitable as an estimate of the overall performance and of the variation between buildings.

1. Introduction

Understanding the potential for severe damage or collapse in older, nonductile reinforced concrete buildings is an important part of understanding the seismic risk of a region as a whole. The project discussed in this paper takes the approach of focusing on buildings in Vancouver built before 1980 of 7 or more storeys in height. In Yathon et al. (2014), the general characteristics of these buildings were summarized, and it was found that most of them use thin concrete walls as the seismic force resisting system. A summary of these findings is presented in Section 2.

Not discussed in that paper was the basic issue of whether the buildings had adequate strength and stiffness to resist the expected seismic demands. This is a complicated issue – the demands that these older buildings should be expected to resist are unclear, and the capacity of the components is difficult to determine. However, it is relatively simple to obtain a linear dynamic estimate of the demands using response spectrum analysis, and the capacities using basic flexural theory. The goal of this paper is to use these methods to determine approximately what kind of performance can be expected of the buildings, and what kind of variation there is from building to building.

In Section 3.1 the modeling approach is covered in more detail, and in Sections 3.2-3.4 the analysis results are discussed, from modal results to displacement demands to force demands and capacities. This detailed approach to the linear results will provide insight into the behaviour of the buildings and can identify those at the extremes that may be particularly vulnerable.

2. Basic properties of the buildings

In total, the structural drawings for 343 reinforced concrete buildings of 7 or more storeys with a date of 1980 or earlier were collected. Using these drawings, a database of the characteristics of the buildings was compiled, which consists of three main sections. The first summarizes the general information about the building, such as the age, height, and structural system. The second captures basic information about a critical column in the building, which is taken to be indicative of the gravity load resisting system as a whole. The final section contains information about all of the structural walls in the building at one floor, and gathers enough detail about each to be able to compute strengths and stiffnesses.

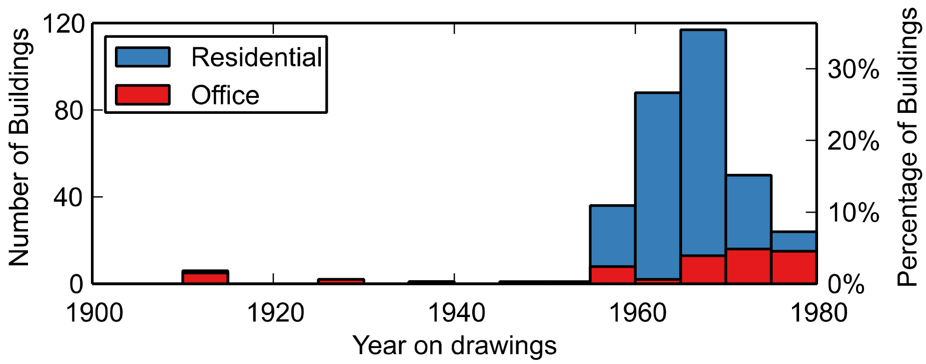

Fig. 1 – Distribution of age of the buildings in the database

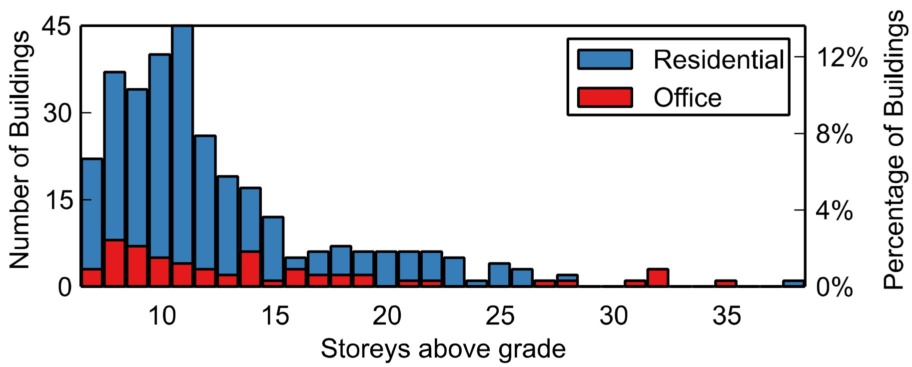

Fig. 2 – Distribution of the number of storeys of the buildings in the database

Figure 1 shows the distribution of the age of the buildings, along with their occupancy. It tells a clear story – most of the buildings were built from 1955-1980, and most of those are residential buildings. Although not represented in Figure 1, most of these buildings are also located close to each other geographically. In combination with Figure 2, which shows that most of the buildings are in the 7-15 storey range, this suggests a concentration of buildings with similar types of construction. This is problematic from a resilience perspective, because a single earthquake that causes damage to these types of buildings can remove a significant part of the housing.

Information was also collected on the gravity load resisting system (GLRS) and the lateral force resisting system (SFRS). In 79% of the buildings the GLRS is a flat plate system, and in 98% of the buildings the SFRS consists of structural walls, further reinforcing the uniformity of the buildings. The prevalence of structural walls is the reason that more information was collected about them than the columns, and it is instructive to summarize their properties.

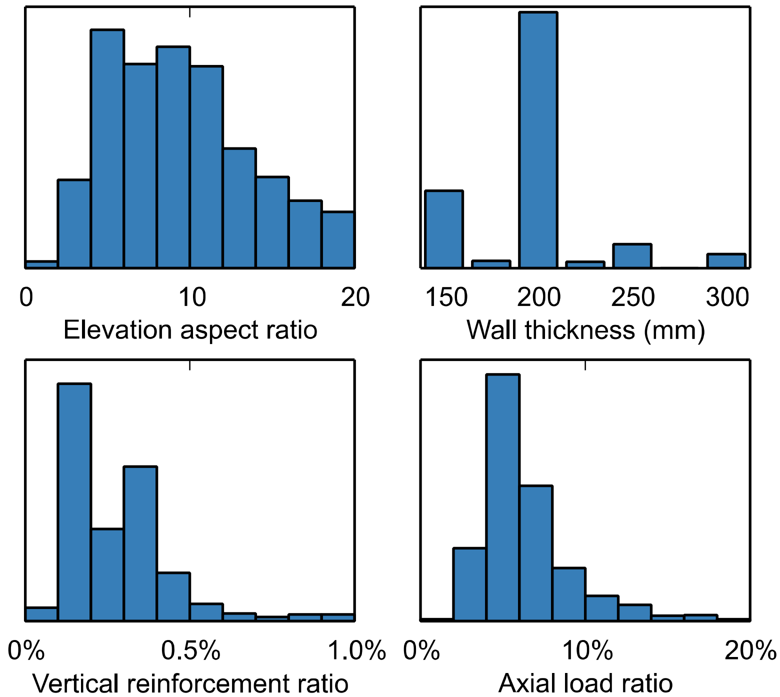

Fig. 3 – Key characteristics of structural walls in the database buildings

Figure 3 shows that the walls tend to be slender, thin, lightly reinforced, and have low axial load. Although not reflected in Figure 3, most of the walls have only a single layer of reinforcing and often no boundary steel. Some of the walls also have a flange on one side of the web only, which can result in very low ductilities when the flange is in tension due to the large compression depth in the web. In general, these types of thin walls have been seen to perform poorly both in recent earthquakes (Wallace et al., 2012; Elwood, 2013) and in laboratory experiments (e.g. Oh et al., 2002; Alarcon et al., 2014), often failing in compression before developing significant ductility.

Also captured by the database is information about other deficiencies in the buildings. Significant torsional eccentricity can amplify demands on edge elements beyond what is expected from a 2D analysis, and is present in approximately 25% of the buildings. Discontinuous walls occur when the area of a wall decreases at lower floors, and are present to some extent in approximately 35% of the buildings. These discontinuous walls can cause damage to the members below them (whether columns or a shorter wall), and can also result in a soft storey. Finally, architectural spandrel beams are present in approximately 70% of the buildings. These beams occur at the edges of buildings, and, depending on their connectivity to the vertical elements, can either initiate frame action in members not designed to resist such forces, or can shorten columns.

3. Linear modeling

3.1. Modeling assumptions and methodology

Any modeling approach must contain a number of simplifications and assumptions. In this paper, the general method for modeling the 3D linear response of the structures is to: i) simplify the actual structure to a fixed-base structure composed of interconnected rectangular walls oriented in the orthogonal axes, and ii) reduce the simplified structure to a stick model with two degrees of translational and one degree of rotational freedom at each node (representing a storey) by assuming a rigid diaphragm. For the 2D case, step ii) results in a stick model with one degree of freedom in translation at each node.

Step i) is implicit in the choice of information collected from the drawings, and assumes that the slab has no stiffness out of plane, the basement floors and soil are stiff compared to the structure, elements other than the walls do not contribute to the lateral response, and the stiffness and mass are entirely uniform over the height of the structure. For most of the structures in the database these are reasonable assumptions, and for the rest they are acceptable given that the goal of this paper is only to get a rough measure of the strength and stiffness.

Step ii) is a standard modeling approach to determine the response of concrete wall buildings for 3D response, and is described in more detail in Ghali et al. (2003). In this paper, warping and shear deformations are not considered, and the shear center of a wall assembly is taken as its center of mass. Because detailed information was not collected on coupling beams (and was sometimes not available), coupling is treated for stiffness by assigning some fraction of the fully coupled (i.e. no strain discontinuities) wall stiffness, and strength is calculated assuming a fraction of the capacity is due to the coupling action (i.e. the couple of axial loads).

Given this model, the dynamic characteristics can be computed. To perform response spectrum analysis (RSA) to determine the displacement and force demand requires that a spectrum be specified. The demand calculated is highly dependent on the spectrum, and for the sake of simplicity the National Building Code of Canada 2010 (IRC, 2010) design spectrum for Vancouver was chosen. The stiffness of the walls was taken to be 0.7 of the gross stiffness to account for cracking.

The strengths of the walls were calculated using basic flexural theory by assuming a linear strain profile across an entire wall assembly, using compatibility to determine the forces, and iterating to achieve equilibrium. It was assumed that the maximum concrete strain, ε_{cu}, was equal to the CSA A23.3 (2004) value of 0.0035, and that the concrete and reinforcement strengths were equal to what was shown on the drawings. For the thin walls observed in the buildings, this ε_{cu} may be unrealistic. This is discussed in more detail later.

3.2. Periods and mode shapes

In the 3D modeling approach, performing an eigenvalue analysis results in mode shapes that involve movement in the two horizontal directions and rotational about the vertical axis. In order to characterize these modes in a simple manner, the following terminology has been adopted:

β – the ratio of the maximum displacement at any point in the rigid diaphragm to the average displacement over the entire diaphragm.

θ – the angle of the displacement from either horizontal axis, expressed as a positive number between 0 and 45 degrees.

Lateral/torsional mode – Except for a building with a doubly symmetrical wall arrangement, there are no pure lateral or pure torsional modes. However, it is useful to attempt to identify which modes are primarily lateral and which are primarily torsional. This was done by assigning each of the modes with the largest mass participation factors to the direction in which it had the largest participation.

The designation of lateral or torsional modes is useful, and is a good approximation for many of the buildings in the database, which are often singly symmetrical or nearly doubly symmertical. Figure 4, displaying the β and θ values for the lateral modes, shows that only a small portion of the structures have significantly coupled modes (larger β) or modes that are not orthogonal to the walls in the building (larger θ). The modes calculated from a 2D lateral assessment of the buildings in each direction were also compared to the 3D lateral modes and it was found that in most cases the periods were similar, which serves as a validation of the 3D analysis.

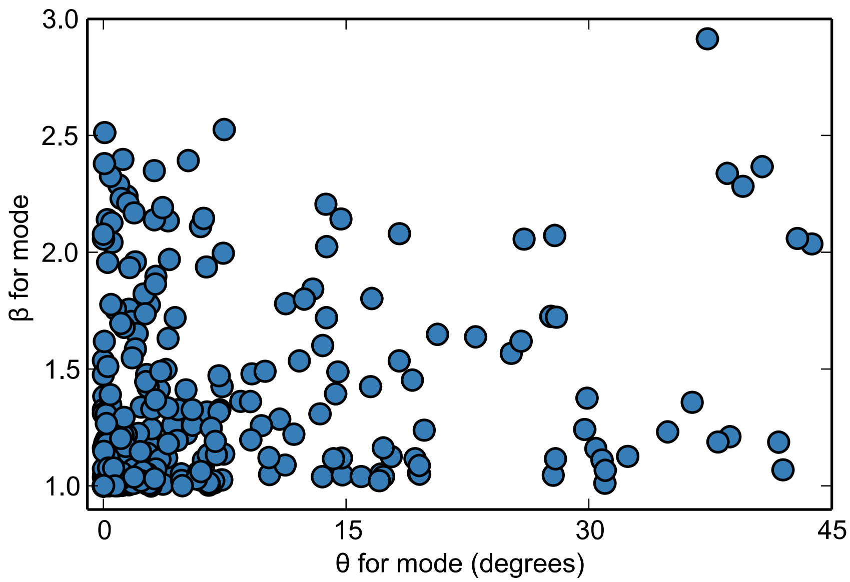

Fig. 4 – β and θ for each lateral mode for each building

Of chief interest is a way to relate the stiffness of the buildings to each other. While the actual stiffness matrix of the building was used to compute the periods, this is difficult to relate to the performance in a simple way. Instead, the lateral periods of the structures can be compared to one another to estimate their relative stiffness. Using the period alone as a proxy for stiffness is inadequate, however, since a shorter building and a taller building with the same period will behave differently with respect to the demands on the elements. A useful measure is the *wall height to lateral period ratio,* h_w/T, which relates directly to the drift demands in a building. The larger the ratio, the stiffer the building is expected to be, and the smaller the drift demands. Figure 5b shows the h_w/T ratio for the buildings in the database, where h_w is assumed to be the same as the total building height. Similar information can also be obtained by comparing the lateral periods to the number of storeys, shown in Figure 5a. The black line shows that estimating the period of the structure as N/10, where N is the number of storeys, is a reasonable approximation.

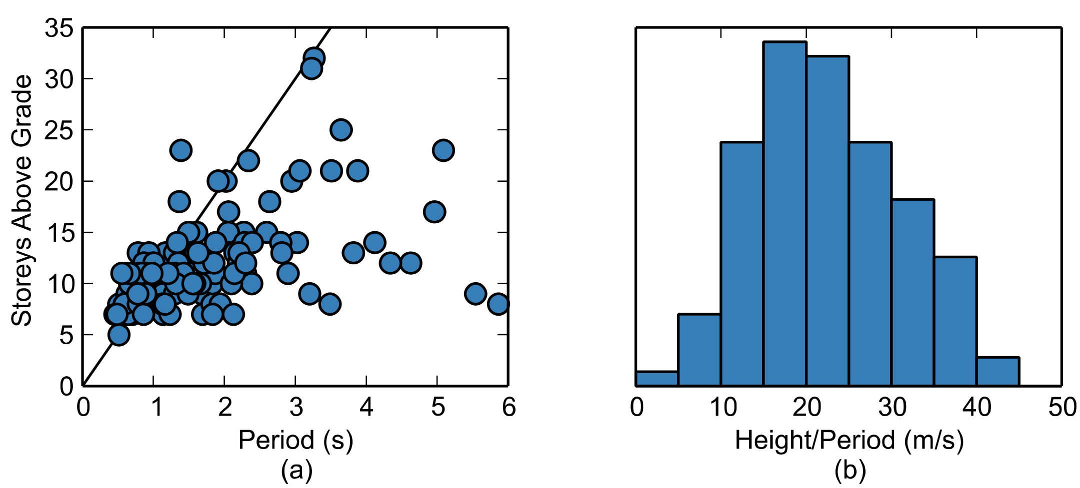

Fig. 5 – Relationship of first period to height for the larger lateral mode

These are relatively soft buildings. Typical buildings in Chile have h_w/T ratios on the order of 60 m/s (Jünemann et al., 2015), while the mean ratio of the Vancouver buildings is 20 m/s, indicating larger drift demands. While this will result in lower force demands on the walls, a major concern is the components of the gravity system, which were not designed to undergo large seismically induced drifts. Although not obvious from Figure 5, it is also indicative of the layout of the buildings, with the Chilean buildings featuring a large number of redundant walls, and the Vancouver buildings relying on a smaller number of more heavily loaded walls.

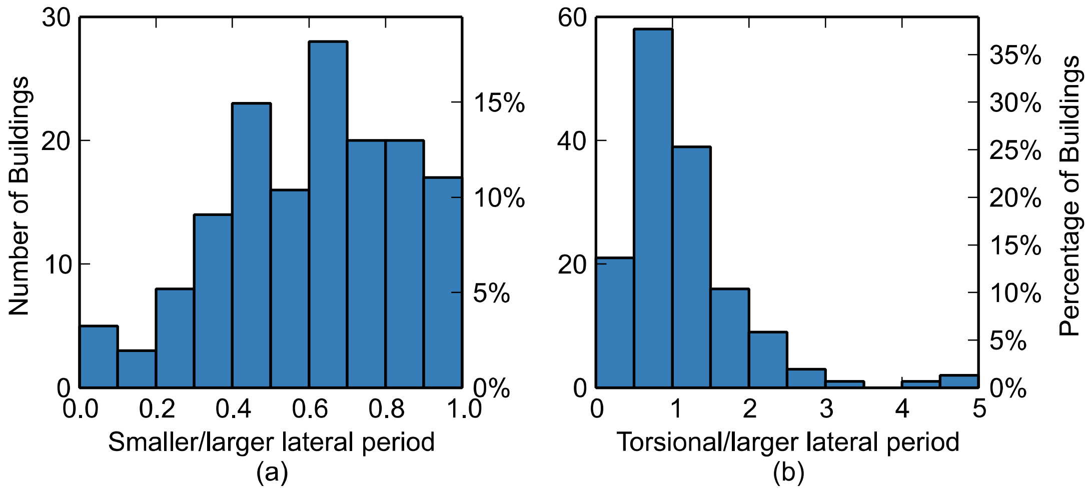

Fig. 6 – Ratios of the first periods

Of concern is the ratio of the periods; Figure 6a shows the ratio of the smaller lateral mode to the larger lateral mode, and Figure 6b shows the ratio of the torsional mode to the larger lateral mode. Two observations can be made, first that while the majority of buildings have similar lateral modes in either direction, there is an important minority that are significantly softer in one direction than the other. Second, many of the buildings have torsional periods that are close to the larger lateral period, suggesting that significant torsional response may occur simultaneously with the lateral response, amplifying demands on components. Cases where the torsional period is larger than the lateral period are also troublesome, indicating a building where any eccentric excitation may result in large demands.

3.3. Displacement demands

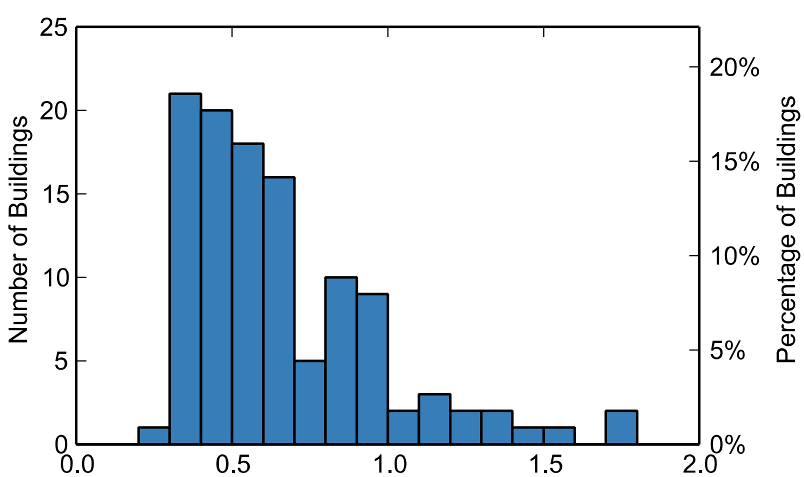

Fig. 7 – Global drift demands from a 2D RSA

While the period and mode shape information can provide an indication of the behaviour of the building, it stops short of calculating the actual demands. The displacement demands, expressed in this section in terms of global drift, give a good idea of the expected rotation demands at the critical section of the walls. It is helpful to first consider the average lateral demand on the entire system, neglecting torsional effects. This can be done by considering the global drift demands from a 2D analysis, taken as the maximum from an RSA applied in each principal axis, as shown in Figure 7.

Approximately 40% of the buildings have lateral drift demands below 0.5%, but there are a significant number that have larger drift demands. When a 3D RSA is performed, the drift demands on the individual components will be different due to the torsional motion. Figure 8a shows the maximum drift demand on any one component in the building due to a combination of 100% of the mass acting in one direction and 30% of the mass acting in the other. Both orthogonal axes were considered, and the maximum of those was taken.

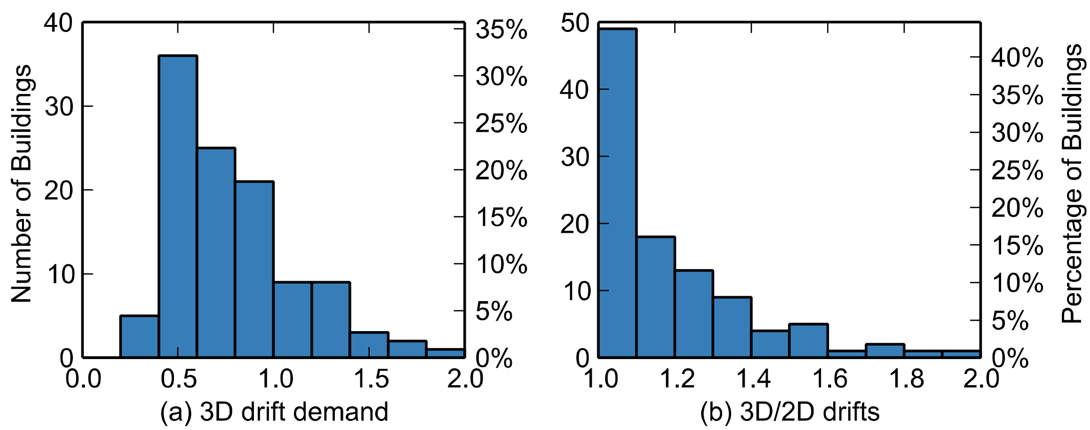

Fig. 8 – Global drift demands from (a) 3D analysis and (b) comparison to 2D

In addition, Figure 8b shows the amplification on the component from the 2D analysis, where the demands on all components are the same. As expected from the large β values seen in the previous section, the amplifications can be significant. In addition, accidental torsion was not considered in the analysis, which would amplify the maximum component demands even more. The question then becomes, what kinds of drifts can the structural elements undergo without failing? The typical model for the inelastic displacement capacity of a structural wall assumes a plastic hinge at the base of the wall, and the total drift capacity is then affected by the length of the plastic hinge and the curvature capacity in the plastic hinge. The poor reinforcement detailing in these older nonductile buildings will negatively impact both of these parameters, reducing the expected overall displacement capacity.

3.4. Force demands and capacities

While assessing the stiffness of the structure by using the displacement demands is useful, a fuller picture is provided by considering the strength demands and capacities as well. In a ductile member with a given displacement demand, the strength is less important. In a member with a brittle failure mechanism, which is likely to be the case with many of the thin walls, the strength is critical. As discussed in Section 3.1, the estimate being made of the strengths of the thin walls is based on flexural theory which assumes that the concrete can reach ε_{cu} =0.0035. Adebar & Lorzadeh (2010) have shown that this may not be the case, and so the strengths in the walls calculated here can be assumed to be an upper bound on the actual strengths.

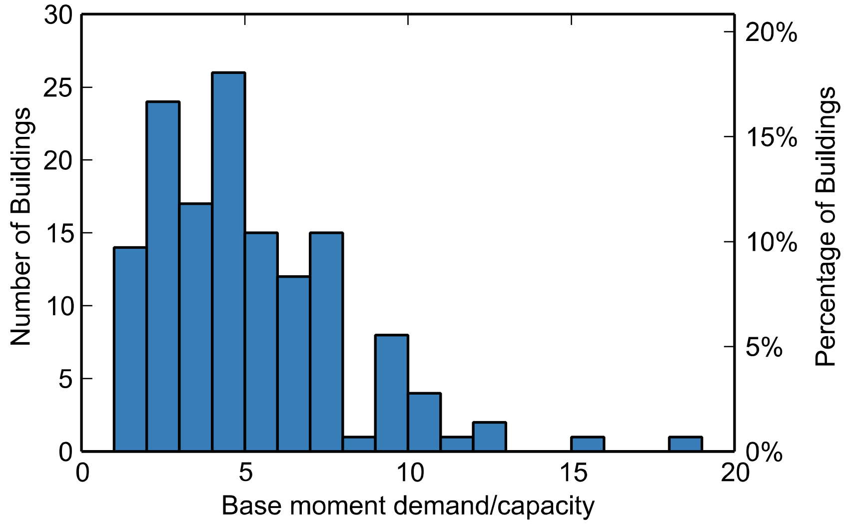

Fig. 9 – Base moment demand to capacity ratio for 2D RSA (maximum of orthogonal axes)

As before, it is first helpful to consider the 2D case, which assumes that the wall capacities in each orthogonal axis can be directly added together. Because strengths by themselves are not useful, Figure 9 shows the ratio of the base moment demand from RSA to the calculated strength. This can be interpreted as an "R" factor, or if equal displacement is assumed, the required ductility demand on the entire system of walls. While there are some extreme values, more concerning is that only a small number of buildings have ratios less that 2, which is probably a reasonable estimate for nonductile construction. Many of the structures have ratios in the range of 5, which is closer to the ductility expected of modern, well-detailed walls.

In the 3D case, the same demand/capacity ratio can be calculated, except for individual members. Because the strength was only calculated in the orthogonal axes, the ratio in each direction was taken and the square root sum of the squares of these two ratios is presented. Figure 10a shows the distribution of the maximum ratio of the wall components for each building in the database, while Figure 10b shows the "amplification" of the demands in the most highly stressed component from the 2D analysis. In some cases the amplification can be quite extreme, with components seeing over twice the demand that a 2D analysis would give, indicating significant torsional motion.

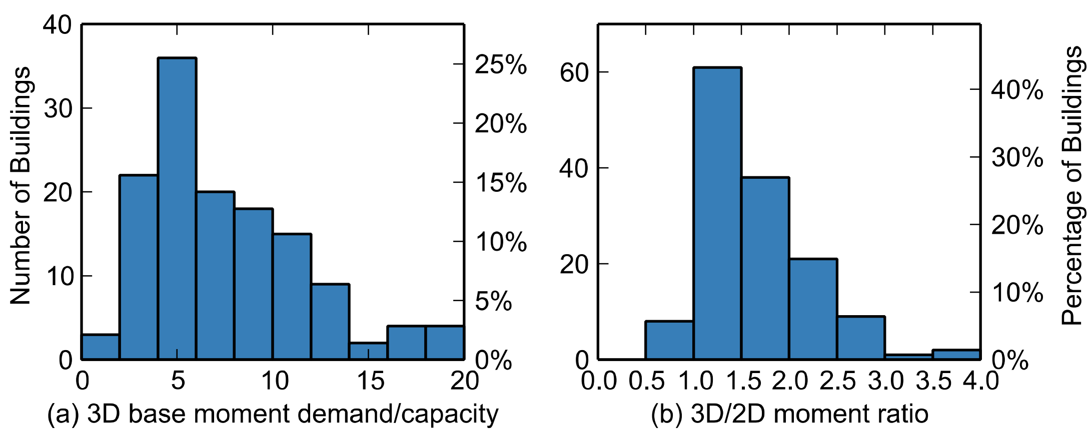

Fig. 10 – Base moment demand to capacity ratio for (a) 3D RSA (maximum of all wall assemblies for two different loading scenarios) and (b) comparison to 2D

Figure 10 is somewhat limited in that it shows neither the distribution of all the walls in the buildings, nor does it indicate whether the components experiencing maximum demands are critical to the integrity of the building as a whole. However, as shown in Section 2, these Vancouver buildings tend to have relatively fewer, higher stressed walls. This suggests that no matter which component experiences damage or failure, the implications for the system as a whole will be fairly significant, as the loads will have to be transferred to other axial and lateral load carrying members, which will in turn be vulnerable to failure. Furthermore, the components that will experience the largest demands will be those at the edge of the building, and these elements are almost never reinforced in a ductile manner.

3.5. Assessment of results

It has been seen that the buildings in the database are, in general, weaker than would be expected for a modern building. If the current design spectrum (which reflects a modern understanding of the expected demands) is applied to the buildings, the drift demands in combination with the low strengths will impose large ductility requirement on the members, which they likely cannot take. However, are these results accurate enough to label a building as vulnerable, or to deem it "unsafe"? Unfortunately, the simplicity of the model and the many assumptions render the findings suspect if one attempts to apply them to directly evaluate one building. For example, the buildings were analyzed by considering them completely uniform over their height. While many of these buildings are relatively uniform, it is common for the first storey to have a different arrangement of walls than storeys above. This could have the effect of shifting the critical section up to the second storey, which may reduce or increase demands. Other significant sources of error include the approximate way in which coupling was captured, the assumption of a linear strain profile in calculating strengths, and the difficulties in trying to capture nonlinear behaviour with linear analysis. Furthermore, it is difficult to say whether the net effect of these assumptions will be to reduce or increase demands.

Therefore, the analysis presented in this paper is not adequate to label a building as "safe" or "unsafe," or to assign a number to it such as meeting a percentage of current code. The results instead meet the two goals set out at the beginning of this paper. First, they give a rough idea of the vulnerability. If the results showed that the walls had significant excess capacity even with current demands, there would not be much cause for worry. That they show the opposite is enough to conclude that at least some of the buildings have low enough strength and stiffness to be of concern, especially when combined with the other deficiencies present.

Second, because the methods of analysis were consistently applied across a fairly uniform set of buildings, the *distributions* of the results are in and of themselves useful. In Figures 6-10, it can be seen that there is a consistent shape to the results, with many buildings displaying responses about the median, and then a few at the extremes. More refined analysis done on the buildings at a certain point in the distribution can be used to calibrate the results across the entire set of buildings.

4. Conclusion

This study has quantified several important aspects of older tall reinforced concrete buildings in Vancouver. Using linear response spectrum analysis, it has been shown that as a whole the buildings are softer than other wall buildings in other parts of the world, and will experience larger displacement demands. The buildings are also weak when compared with demands from a modern code. Torsional response has been shown to be a significant factor in many of the buildings, which imposes additional demands on the often poorly reinforced elements at the edges of the building.

As discussed in Section 3.5, the analysis done does not constitute a best prediction of the actual behaviour of the buildings, but rather a benchmark with which to compare the buildings to each other and to get a sense of the order of magnitude. To this end, the future research direction of this project will proceed by initially ignoring the variance between the buildings and instead focusing on small number of buildings that are representative of the whole (those near the median results). Incremental dynamic analysis will be performed to determine when significant damage or failure occurs in the buildings, which will involve defining a suitable model for thin walls. At this damage point, a parametric study will be performed which will address the variance seen in this paper, and bring in effects of torsion. Lastly, the results of this detailed analysis will be applied back to the database as a whole.

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