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OPINION PAPER

INTERSECTING TRADITIONS IN THE SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS

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ABSTRACT

This short manuscript describes how three different schools of thought, combining observation, pragmatism, physics, and judgment, have arrived at strikingly similar approaches to proportion RC building structures that can sustain intense earthquake demands without incurring excessive structural and non-structural damage. The examined traditions, which can be traced back to Japan, the University of Illinois at Urbana-Champaign, and Chile, were shaped by independent calibration processes. Yet they all share a common idea: structural robustness is crucial to reduce building earthquake damage. The success of this simple idea has been demonstrated repeatedly in the field. For that reason, New Zealand engineering students, at the very least, should be made aware of these practices, especially considering that the local tradition is radically different.

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INTRODUCTION

The Canterbury and Kaikoura Earthquakes of 2010, 2011, and 2016 made clear that attention to structural ductility can prevent collapse, but it cannot prevent widespread damage—structural or not—nor disruption of the functionality of buildings and entire cities. In contrast, especially after the Kobe Earthquake of 1995, the speed at which cities in Japan and Chile can return to nearly normal operation after powerful earthquakes is both admirable and hard to deny [1]. Similarly, field observations organized using ideas shaped at the University of Illinois at Urbana-Champaign show startling differences in the response of buildings of similar vintage and construction quality to apparently similar shaking intensities. What causes these radical differences? The case is made that structural robustness, which relates to lateral stiffness and drift control, is key to reduce earthquake damage according to the three schools of thought described next.

VIEWS FROM JAPAN

It can be said that systematic earthquake engineering research began in Japan in the late 1800s, with the pioneering work of John Milne and the founding of the Seismological Society of Japan in 1880. By 1923, the benefits of those efforts were demonstrated by the exceptional performance of buildings designed by T. Naito during the 1923 Kanto Earthquake [1]. Naito was perhaps the most salient of the second generation of Japanese engineers who were trained to heed systematic observation with rigor after socio-political reforms that occurred during the "Meiji Restoration." Naito promoted the use of structural walls and stiff structural systems in general. He began a tradition that would later be expressed in quantitative terms by Shiga, Shibata, and Takahashi [2] in their "Shiga Map" (Figure 1).

In Figure 1, the horizontal axis represents the ratio of wall cross-sectional area to total floor area, and the vertical axis represents nominal shear stresses acting on columns and walls. Dashed lines represent values of an index (called I_s) used to screen buildings in Japan, while solid lines represent the ratio of column cross-sectional area to total floor area. In Japanese practice, buildings with lower values of I_s are classified as being more vulnerable to earthquake damage.

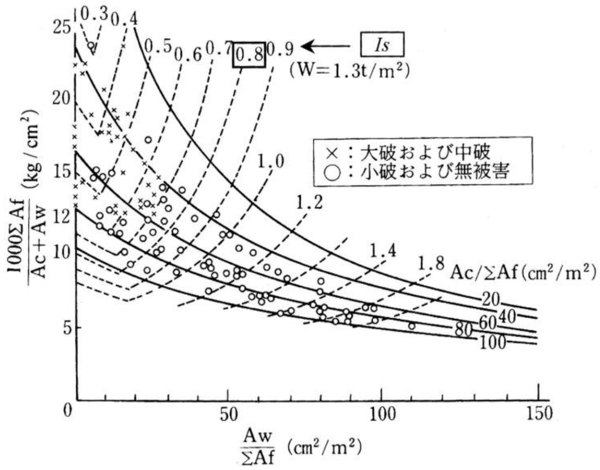

Figure 1: Shiga Map [2].

The basic idea in Shiga's proposal was to keep nominal shear stresses in columns and walls within a range that had been observed to lead to better performance in the field. Over the years, Shiga's ideas led to a remarkably simple design approach, which can be summarized as follows. First, define the total cross-sectional area of structural walls, A_w, aligned with the most critical of the principal floor-plan directions in a given storey. Then, define the total cross-sectional area of columns, A_c, for the same storey. Last, determine the total building weight above that storey (W). A building not taller than 20 metres is considered to have adequate lateral strength and stiffness to resist earthquake demands in most parts of Japan if it meets the following requirement:

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$$2.5\, MPa \cdot A_w + 0.7 MPa \cdot A_c > \frac{W}{\sqrt{f'_c/18\, MPa}} \quad (1)$$

The threshold expressed in Equation 1 applies to the first storey. Higher thresholds are imposed on upper storeys, but the reduction in weight, W, is in most cases larger than the increase in the threshold. As a result, the first story tends to govern the design. In Equation 1, f'_c represents concrete cylinder compressive strength.

A few assumptions are in order to facilitate comparisons. It is assumed that most new buildings in Japan are being designed with specified concrete strengths exceeding $f'_c = 30\, MPa$, so that the radical in the denominator on the right-hand side of Equation 1 is at least 1.3. It is also assumed that the building weight, W, is approximately 10 kPa times the total floor area above ground, $\sum A_f$. Reorganizing terms and defining the Wall Index as $WI = A_w/\sum A_f$ and the Column Index as $CI = A_c/[2\sum A_f]$ as suggested by Hassan & Sozen [3] leads from Equation 1 to Equation 2.

The factor 2 in the denominator of CI can be interpreted to be an early calibration coefficient or a way to indicate that while walls are counted in a single direction, the column is counted in two directions.

$$0.55\, CI + WI > 0.31\% \quad (2)$$

Together with detailing requirements, Equation 2 is assumed to be sufficient for the design of low-rise buildings in Japan, regardless of site or perceived hazard. No lateral-force analysis is required if the threshold it defines is met. That level of simplification is rare in earthquake engineering, and—remarkably—it has led to enviable results [1].

THE CHILEAN PERSPECTIVE

There does not appear to be consensus on the origin of the proportions used in Chilean buildings, known for their extensive use of structural walls. One possibility can be traced back to Rodrigo Flores and Julio Ibáñez, two engineers who investigated the damage from the 1939 Chillán Earthquake [4]. They reportedly observed that wall buildings performed better than frame buildings. A former student recalls Flores saying that they had misread a German standard—an accident that, by chance, aligned with the idea of using many structural walls to resist seismic demands. About that misstep, Flores is said to have remarked, "Anything good that happens in this country is just luck." Both Flores and Ibáñez taught structural design and trained their students to favour buildings with abundant and thick walls, starting a tradition that became a matter of national pride. In this sense, it is relevant to mention that the general public in Chile is not afraid of earthquakes. When asked why, they tend to say that they know how good their buildings are [4].

This tradition was tested by the Valdivia and Valparaíso Earthquakes of 1960 and 1985. The results were striking. Following the 1985 event, a detailed investigation [5] led M.A. Sozen [6] to conclude that the success of the Chilean building stock was largely due to the use of an average wall density—defined as the ratio of total wall cross-sectional area to typical floor area—of about 3% in each principal direction of the floor plan, or roughly 6% in total. This became known as the "Chilean Formula".

Figure 2 shows total wall densities reported by Riddell, et al. [5]. The plotted densities combine the two principal plan directions. The average appears close to 6%, consistent with the "Chilean Formula." There is, however, a discernible—though weak—trend for wall density to increase with the number of stories, at least up to 15 or 20 storeys. It is hard to interpret the data from taller buildings because most of the buildings surveyed by Riddell, et al. had fewer than 15 stories.

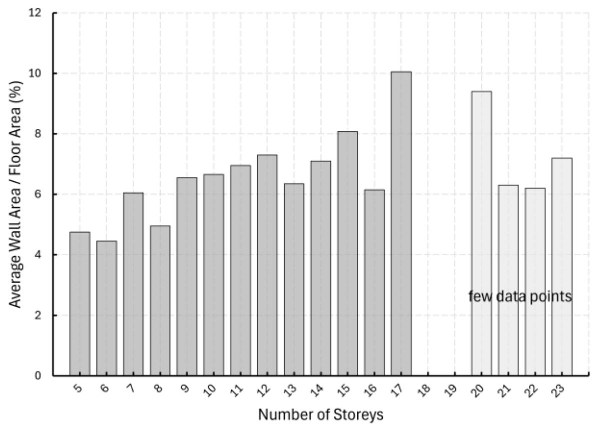


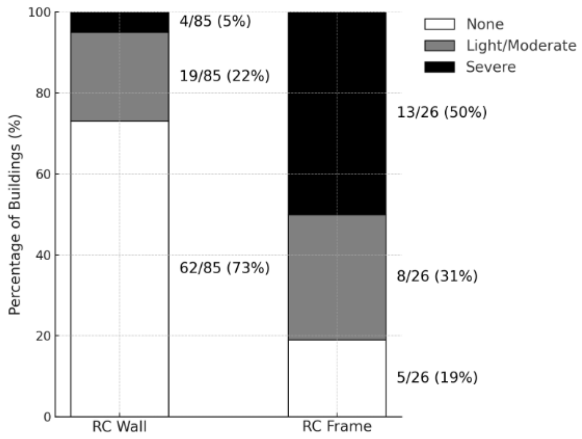
***Figure 2: Total wall densities reported by Ridell, et al. for Chilean buildings** [5].*

The 2010 Maule Earthquake tested tens of thousands of reinforced concrete wall buildings in Chile. The results were, once again, impressive. There were only four collapses and a few dozen buildings with structural damage [7]. Structural problems were more common in buildings with thin walls—as thin as 200 mm. The Chilean engineering community responded by tightening confinement requirements, although subsequent work by Ichinose and Takahashi [8] suggested that requiring thicker walls or thicker boundary elements might have been equally, if not more, effective. Overall, Chile's long-standing reliance on structural walls proved effective once again: cities remained operational, and most buildings continued to function.

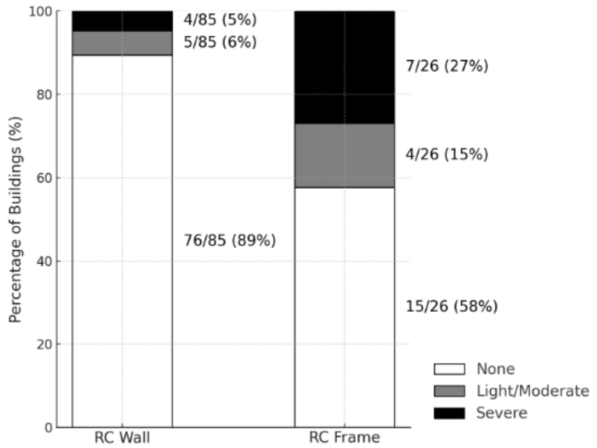
The effectiveness of Chile's reliance on structural walls can also be evaluated by examining post-earthquake reports from hospitals [9]. The Chilean Ministry of Health collects data on hospital performance after earthquakes, and the dataset is particularly useful because hospital structures in Chile include both wall systems and moment-resisting frames. Some hospitals appear to have been built with frame systems in response to constraints imposed by international lenders.

Figure 3.a shows that non-structural damage has occurred more frequently in frame buildings. Much of that difference stems from partition walls, which are more likely to be damaged in frames because they experience larger drifts. Figure 3.b, however, shows that components commonly classified not as "drift sensitive" but as "acceleration sensitive" have also been more prone to damage in frame buildings in Chile. This finding contradicts much of the prevailing professional opinion and deserves careful investigation. Nevertheless, quantitative and systematic data on non-structural performance remain limited. Most field observations in this regard are qualitative and often anecdotal. At the same time, city-to-city comparisons are difficult given the number of variables involved. Still, it is worth noting that recovery in cities like Sendai and Santiago de Chile—where buildings tend to be more robust—was considerably faster than in Christchurch, which relied mostly on ductile but flexible (i.e. lacking stiffness) frame buildings to meet seismic demands[1].

[1] One of the authors went to both Chile and Sendai within weeks from the Maule and Great Tohoku earthquakes. They were able to stay at central hotels and dine in central restaurants, while the Central Business District of Christchurch was cordoned off for two years.



a)



b)

Figure 3: a) Damage in non-structural building components, Reinforced Concrete (RC) structural walls v. RC frames, and b) Damage in non-structural building components commonly classified as "acceleration sensitive", RC structural walls v. RC frames [9].

LEGACIES FROM URBANA, ILLINOIS

At the University of Illinois at Urbana-Champaign, A. N. Talbot began a lasting tradition of effective simplicity—avoiding obsessive attention to exact analyses of approximate representations of real structures. This tradition was reflected by the work of D. A. Abrams, who recognized the critical importance of the water-to-cement ratio in concrete [10] and investigated bond in detail to arrive at modest conclusions questioning the reliability of some of our most basic assumptions [11]. The tradition continued with F. E. Richart, who proposed elegant yet simple methods for designing reinforced concrete columns [12], and who, after years studying shear, concluded soberly that it is "not susceptible to exact analysis" [13]. Westergaard demonstrated the power of such pragmatism by solving the fourth-order partial differential equation for flat plates and reducing it to a few design coefficients [14]. Likewise, Hardy Cross introduced "moment distribution," a method so powerful it extended to water networks and electrical circuits [15], which—he remarked—was too accurate for real structures. N. M. Newmark advanced the tradition with solutions to problems from soil mechanics to blast effects. His genius was perhaps demonstrated best by his idealization of earthquake spectra and their sharp variations as three unassuming straight lines on a "tripartite plot" [16], which we still use today. C. Siess added to the way of thinking that put Urbana on the engineering world map with ingenious insights such as the plastic hinge analogy [17].

The intellectual heritage that accumulated at Urbana since the turn of the 20th century was inherited by Mete Sozen in the 1950s. Among his many contributions, his work with T. Takeda yielded the first step-by-step solution of the nonlinear equation of motion for a reinforced concrete structure, supported by large-scale test results [18]. This work led directly to the development of SAKE, an early nonlinear dynamic analysis program [19], which is still in use—as STERA_3D—thanks to updates and improvements by T. Saito [20]. Despite—or perhaps because of—the theoretical, observational, and computational power at his disposal, Sozen created one of the simplest methods for classifying RC buildings in terms of their seismic vulnerability [3]. Inspired by Shiga's work, Hassan & Sozen [3] recast the problem of building screening in much simpler terms and initiated extensive calibration efforts that continue today. These efforts span diverse regions, construction practices, and earthquake ground motions [3, 21–23]. This calibration work was done independently from Shiga's research and subsequent Japanese developments. Fig. 4 presents data from 15 surveys [24, 25], covering more than 1,600 RC buildings, nearly 90% of which had seven or fewer stories. When Hassan and Sozen first published their vulnerability indices, they indicated that thresholds and calibration factors were to be revised to accommodate new data and local constraints. They proposed a preliminary threshold of $CI + WI = 0.25\%$, below which buildings should receive priority attention. Pujol et al. [26] analysed how changes to this threshold could affect outcomes and costs. Nevertheless, the data in Figure 4 suggest that most cases of severe damage and collapse occurred in buildings below a variation of the original threshold similar to the ones suggested by Ozcebe et al. [27] and Shah [28], as expressed in Equation 3.

$$CI/2 + WI = 0.25\% \tag{3}$$

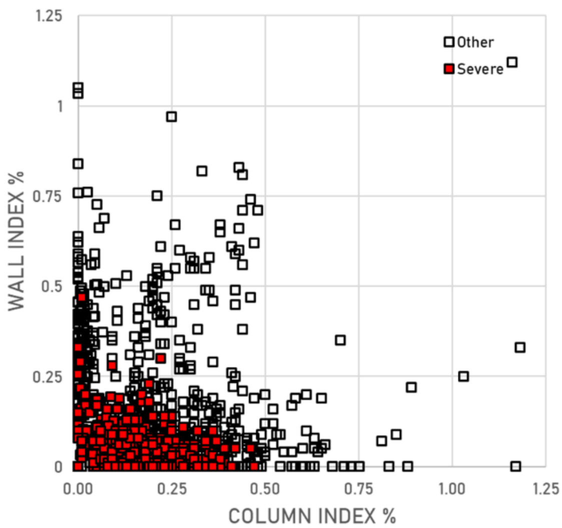


Figure 4: Hassan Indices collected from more than 1600 buildings.

INTERSECTING VIEWS

The Japanese experience, as summarized by Equation 2, and the findings from the surveys used to calibrate the index proposed by Hassan and Sozen [3], idealized by Equation 3, are compared in Figure 5. The similarity between the two is remarkable. Given how different the calibration processes and data leading to these thresholds are, it is difficult to believe that the similarity is coincidental.

If a system with no frames is to be used—which can be advantageous depending on available formwork—then $CI = 0$.

Assuming $\sum Af = N \times Af$, with N representing the number of stories, the thresholds shown in Figure 5 can be expressed in terms of total wall density by the straight lines in Figure 6. Except for more noticeable deviations at $N = 5, 7$, and 16, these lines appear to capture the overall trend in the data collected in Chile by Riddell et al. [5] for buildings with no more than 20 stories. Keep in mind that only a few taller buildings ($N > 15$) were included in Riddell's study, and that the data supporting both the Japanese and Hassan thresholds primarily come from low- to mid-rise buildings. It should also be acknowledged that extending Equation 2 and Equation 3 to high-rise buildings would lead to impractical designs, suggesting the need for a cap or taper on wall density as height increases.

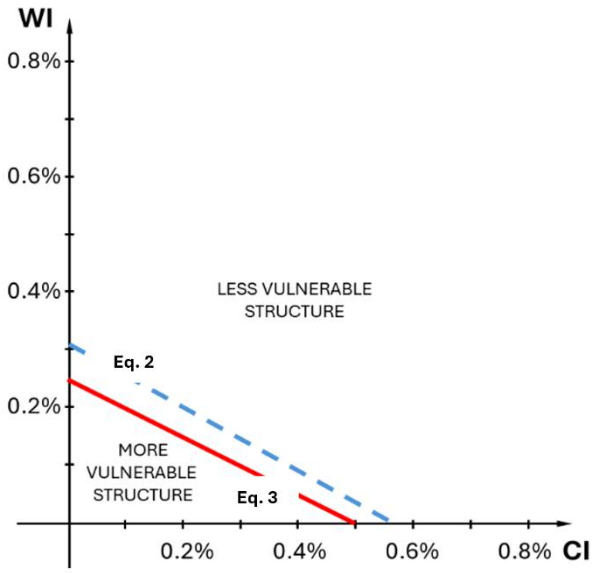


Figure 5: Comparison of Equation 1 and Equation 2.

Overall, Figure 6 illustrates how the ideas from Japan, Chile, and Urbana—developed more in parallel than in series—produced simple and congruent solutions to the complex problem of earthquake response, at least for RC buildings up to 20 stories.

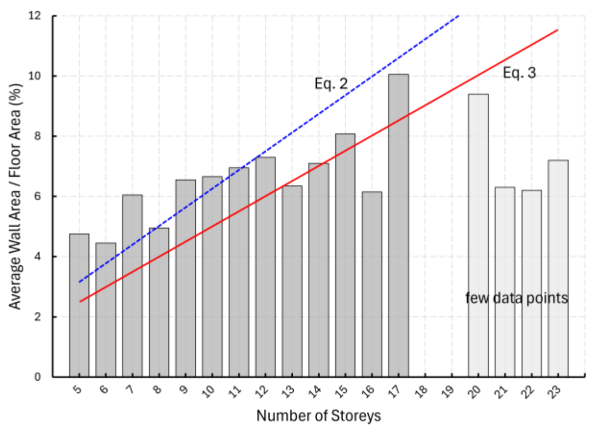


Figure 6: Comparison of results from outlined traditions.

THE DRIFT TRADITION

Views from Chile, Japan, and Urbana can also be compared in terms of drift ratio rather than relative areas of walls and columns. Any discussion of drift limits requires defining a shaking intensity. The comparisons that follow refer to what is commonly known as the "design earthquake." It must be understood that the definition of the design earthquake is as much a compromise as it is a decision informed by incomplete statistical information [29]. Still, a line must be drawn somewhere to support design practice. The idea that the design earthquake is likely to occur at intervals quantified in several hundred years—often 500—even if imperfect, at least allows comparisons of international practices.

In Japan, the design of low- and mid-rise structures (with height not exceeding 31 m), in general, focuses on formulations similar to the one represented by Equation 1. A drift check is imposed (at 0.5%) but only for moderate and nominally frequent motions identified with the label "Level-1 earthquake." Other requirements often control the design, such as requirements to keep the structure in its linear-elastic range of response in the Level-1 earthquake. Details are omitted here for the sake of clarity. On the other hand, for taller and special structures, the designer can be required to conduct more detailed studies including dynamic analyses. In such cases, and for the "design" (or "Level-2") earthquake—with a nominal return period of 500 years—the drift limit is set at 1% [30].

In Chile the drift limit is specified for "reduced" forces as 0.2%. The force reduction factor **R** used in Chile is defined in terms of site condition and building period. Nevertheless, if a factor **R**=7 is deemed representative, the corresponding drift limit is 0.2%x7=1.4%

It has been said that Sozen—arguably the last scholar from the school of thought started by Talbot at Urbana—was too clever to propose a design drift limit. That, however, is not quite accurate. After the early insights of Naito were largely forgotten—at least in the geopolitical West—Sozen was the first to argue that drift should be central to the design process [31]. In the classroom, he would address the need to limit drift as follows:

There is a drift limit past which the building loses its utility. Algan [32] surveyed engineers experienced in earthquake-resistant design about this limit. Approximately ¾ of the respondents thought that a building, to be considered as having been proportioned properly, should not sustain a drift ratio of more than 1.5% in the design earthquake. All agreed that a drift ratio of 2% or more would be unacceptable. Data from non-structural materials would suggest more conservative limits (smaller than 1%). Nevertheless, often non-structural elements sustain less drift than the frame. In that light, professional opinion appears plausible.

This view was preserved in print in the proceedings of a short course Sozen offered in Jakarta. When it came to critical facilities such as hospitals, the design recommendations that Sozen developed for the U.S. Veterans Administration [33] specified drift limits of 0.25% or 0.8%, depending on the "brittleness of contents." That is as crisp—and as stringent—a definition of a drift limit as one is likely to find, at least outside Chile and Japan.

The drift limits mentioned range from 0.25% to 1.5%. The range is wide and may be interpreted as a lack of consensus. That may be so, but there is agreement among the reviewed traditions that drifts under the "design earthquake" used by most professional communities should be much smaller than the 2% to 2.5% limits adopted by professional communities that emphasize ductility capacity over drift control. That idea aligns with the studied evidence and prior work [31, 32, 34] and it supports the view that robustness leads to better performance.

Despite the apparent power of the indices discussed, the key roles that detailing and regularity play in seismic response cannot be overlooked in any seismic design today.

THE COUNTERARGUMENTS

Counterarguments to the presented ideas favouring structural robustness often refer to cost and to the need to protect equipment and ceilings. The increases in cost associated with improved robustness have been shown to represent only a small fraction of total building cost [24, 35]. That fraction is far smaller than widely accepted expenses such as realtor fees and mortgage interest. Why should safety and functionality be sacrificed before the benefits associated with those other costs are sacrificed? As for the argument that equipment and ceilings are more often damaged in more robust structures, at least three comments are in order:

1. There is no systematic evidence supporting that idea, to the best of the author's knowledge;
2. Cities with robust buildings—such as Santiago and Sendai—have remained mostly operational after large earthquakes; and
3. The solution for protecting equipment and ceilings need not lie in the structure itself but can more easily be achieved through improved connections to it.

It also seems that some professional communities favour arguments rooted in analysis and theoretical constructs over ideas grounded in field evidence. Ideally, engineering should consider both. But if one side must be given greater weight, it should be the one supported by observation. After all, the final arbiter of any theory—or design method—is evidence. In the case presented here, three independent bodies of data and successful practices appear to converge or at least point in the same direction. That direction emphasizes robustness as the key not only to building resilience, but also to city resilience in regions of high seismicity. In regions of moderate and low seismicity, a transition to less demanding requirements may be justifiable. It should be understood, however, that such a compromise can lead to surprises—as was the case in Christchurch, New Zealand.

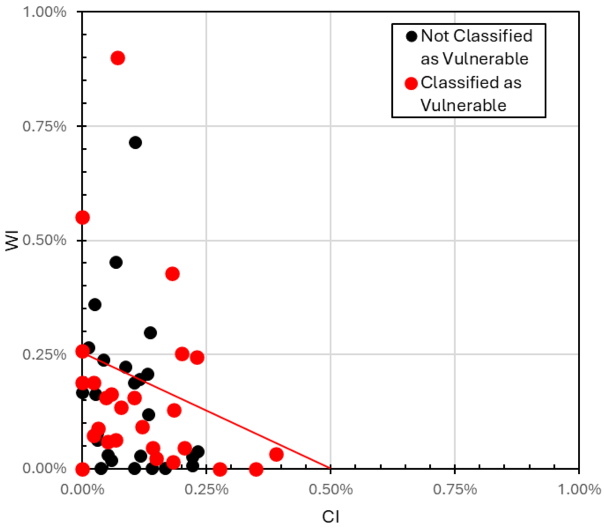
Professional societies educated to rely primarily on elaborate calculations in the design of building structures may not be ready to adopt—at least not directly—the rather blunt but effective ideas of Shiga, Flores, Ibáñez, and Sozen, which emphasize the selection of structural proportions before analysis. Local traditions and laws can also make such a shift difficult. Nevertheless, academic institutions should, at a minimum, expose students to the way of thinking that led to the remarkable convergence and success of the schools of thought reviewed here. This can be done without disregarding the importance of continuity, redundancy, uniformity, and detailing for ductility.

WHERE DO WE STAND?

To examine how the building inventory in New Zealand compares with the traditions described earlier, 75 RC buildings from Wellington were examined. Figure 7 illustrates column and wall indices calculated for those buildings. The buildings were constructed between 1911 and 2004, with 79% of them having 10 or fewer storeys, and with only one having more than 20 storeys. Of the 75 buildings studied, 51 were constructed before 1976, when an update to the building standard was introduced to require more attention to detailing to achieve ductility. In Figure 7, buildings are represented by symbols of different colours to indicate whether they have been classified through New Zealand's "earthquake-prone building legislation" as requiring intervention (as of the time of writing). The line in each plot represents Equation 3.

Figure 7 suggests that the New Zealand traditions for proportioning and assessing buildings differ radically from the traditions discussed earlier. The difference became even more radical after 1976. Clearly, ductility —when available— is the main line of defence against earthquake demands in the New Zealand tradition. But we must admit that for typical RC frame buildings, large ductility occurs together with large drift. Several questions should be asked in this regard:

1. What provides room for errors and/or uncertainty in the estimation of the earthquake demand? Ductility has a limit.
2. How can a city recover quickly if most of its buildings have partitions, facades, and finishes damaged heavily by drift (not to mention precast floors, which are common in New Zealand)?
3. What provides room for errors and tolerances in construction, and for the recurrent need to update detailing practices that are observed not to work as expected? Consider how many changes in detailing have occurred in the past 50 years.



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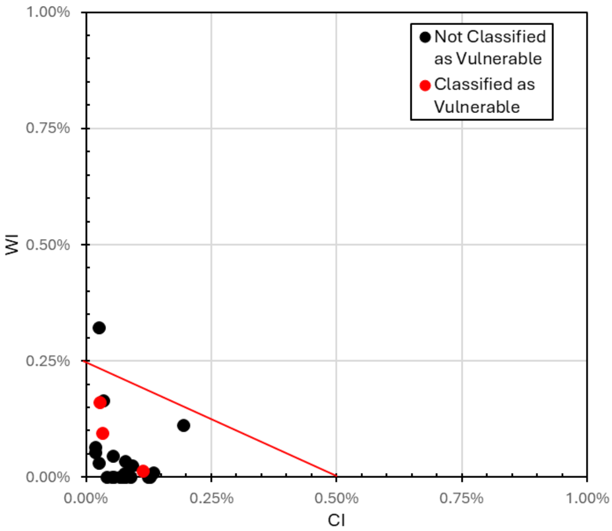

b)

Figure 7: Wall and Column Indices for Buildings in Wellington constructed before 1976 (a), and after 1976 (b).

CONCLUSION

The simple indices proposed by Hassan and Sozen, as presented here, align with the most fundamental premises of the two most successful traditions in earthquake engineering: the Chilean and Japanese traditions. It seems unlikely that this convergence is merely coincidental. Remarkably, these three approaches have arrived at essentially the same pragmatic solution—one that favours robustness instead of elaborate probabilistic estimates and nearly exclusive reliance on building deformability. Given the clear and repeated success of these intersecting traditions, the lack of attention they receive in most highly seismic regions—outside their places of origin—is difficult to justify. At the very least, engineering students should be made aware of the effectiveness of these practices, especially if we consider that the New Zealand tradition is radically different.

ACKNOWLEDGEMENTS

The data in Figure 3 were compiled by L. Pledger from information provided by the Chilean Ministry of Health. The data in Figure 4 include information from fifteen field surveys, with ACI Committee 133 leading a large number of those. Drawings and assessments of buildings in Wellington were provided by Wellington City Council.

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