
How simple is too simple: Estimating seismic drift demands of a hybrid irregular structure

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ABSTRACT

If drift, rather than strength, is to become the key parameter in seismic design, simple and reliable methods for estimating drift demands are required. Existing methods have often been calibrated for structural systems with a translational mode of response, raising the question of how effective they are for buildings with horizontal irregularities, in which the response is dominated by torsion. In addition, these methods have been typically developed for specific materials, such as reinforced concrete (RC) or structural steel, without explicitly addressing hybrid systems that combine multiple materials. This study evaluates the reliability of seismic drift demand estimates based on the modal properties of a structure and the intensity of a ground motion (expressed in terms of peak ground velocity). Estimates are compared against measured drift from 42 dynamic tests on a 12.6-m tall, five-storey, structural steel–RC hybrid building with horizontal irregularities. The specimen included two coupled RC walls forming a core offset from the centre of mass. In 16 dynamic tests, buckling-restrained braces were installed in a steel frame located opposite the core. Results indicate that drift demand estimates at the centre of mass have comparable statistical quality to estimations for translational structures. Drift demand estimates at the edges of the structure in the direction dominated by torsion were sensitive to the displacement history of the building.

1 INTRODUCTION

Force-based procedures have traditionally been used in the earthquake-resistant design of structures. The origin of these procedures can be traced back to the aftermath of the 1908 Messina earthquake. Although it was appreciated that the effect of earthquake demands on a structure was dynamic in nature, a static force-based approach was preferred because it resembled the design for gravity loading (Sozen, 2013). Current building codes still include force-based design approaches, in which base-shear strength is the main parameter of interest. Even though strength is essential, observations from experiments and post-earthquake surveys have shown that drift is one of the main causes of structural and non-structural damage during ground motions (Algan, 1982; Hassan & Sozen, 1997; Moehle & Sozen, 1980; Pledger, 2025; Pujol et al., 2022; Sozen, 1980). More specifically, seismic resistance of structures with elements not susceptible to decay in lateral strength or brittle failures is governed by drift (Sozen, 1980). These observations have led to

the development of drift-driven or displacement-based design procedures, in which drift, rather than strength, governs the design (Priestley et al., 2007; Pujol et al., 2022).

The effectiveness of drift-driven design methods relies on the ability to accurately estimate drift demands for a wide range of structures and ground motions. When estimating drift demands, two main difficulties arise: 1) although estimates of earthquake intensity have improved with time, they can easily be off by 50% or more (Pujol et al., 2022), and 2) current methods have often been calibrated for specific structural systems (Shah, 2021; Sullivan et al., 2003). For these reasons, in earthquake engineering, perhaps more than in other applications, the effort required by a design method should be proportional to the quality of the expected result.

Hybrid structures combining RC and structural steel elements have become increasingly common in modern construction (Bruneau & MacRae, 2017). Following the 2011 Canterbury Earthquakes, a survey of new developments in the Central Business District of Christchurch reported that approximately one-third of 74 new buildings have been constructed using a combination of RC and structural steel elements (Bruneau & MacRae, 2017). Similar design practices have also been documented internationally, particularly for mid- and high-rise buildings (Hou et al., 2008). This preference is driven by the stiffness and strength of RC elements, and the construction efficiency and high strength-to-weight ratio of steel framing (Pascua et al., 2023). Previous experimental studies on hybrid systems have primarily been conducted at the component level, focusing on the capacity of steel frame-RC wall connections. Large-scale experiments focusing on their seismic performance are yet to be completed.

An additional concern in the design of hybrid buildings is the possibility of torsional responses due to the combination of materials with different stiffnesses. Moreover, architectural constraints often lead to an irregular arrangement of vertical structural elements, which also results in a mode of response dominated by building torsion (Dragovich, 1997). Previous experimental studies on structures with horizontal irregularities have seldom focused on drift demands, which, as described previously, control the seismic resistance of a structure. This study evaluates the reliability of a method for estimating drift demands originally developed for RC structures with translational modes of response (Sozen, 2003). Estimates are compared against measured drift from 42 dynamic tests on a large-scale structural steel-RC structure with horizontal irregularities.

2 DYNAMIC TESTS OF A HYBRID IRREGULAR BUILDING

2.1 Specimen

The test specimen was a five-storey structural steel-RC hybrid building. Each floor measured 5.4 m × 5.4 m in plan. The nominal distance between steel columns was 5.0 m in both plan directions. Two coupled walls were located at the centreline of the south façade, offset 1.3 m from the centre of mass of the structure in the North-South (NS) direction, as shown in Figure 1a. The storey height of the first storey was 2.6 m, measured from the top of the column base plates to the top of the first-floor slab. The storey height for the upper storeys was 2.5 m, measured from top of slab to top of slab above. All elevated floors were 125-mm thick composite slabs. 750-mm thick RC foundation blocks were used to connect the test structure to the top platform of the equipment used during the tests. Non-structural components, including office furniture, partitions, ceilings, sprinklers, library shelves, and a curtain wall, were included in all storeys except at the roof level. The total weight of the non-structural components amounted to approximately 1kPa per floor.

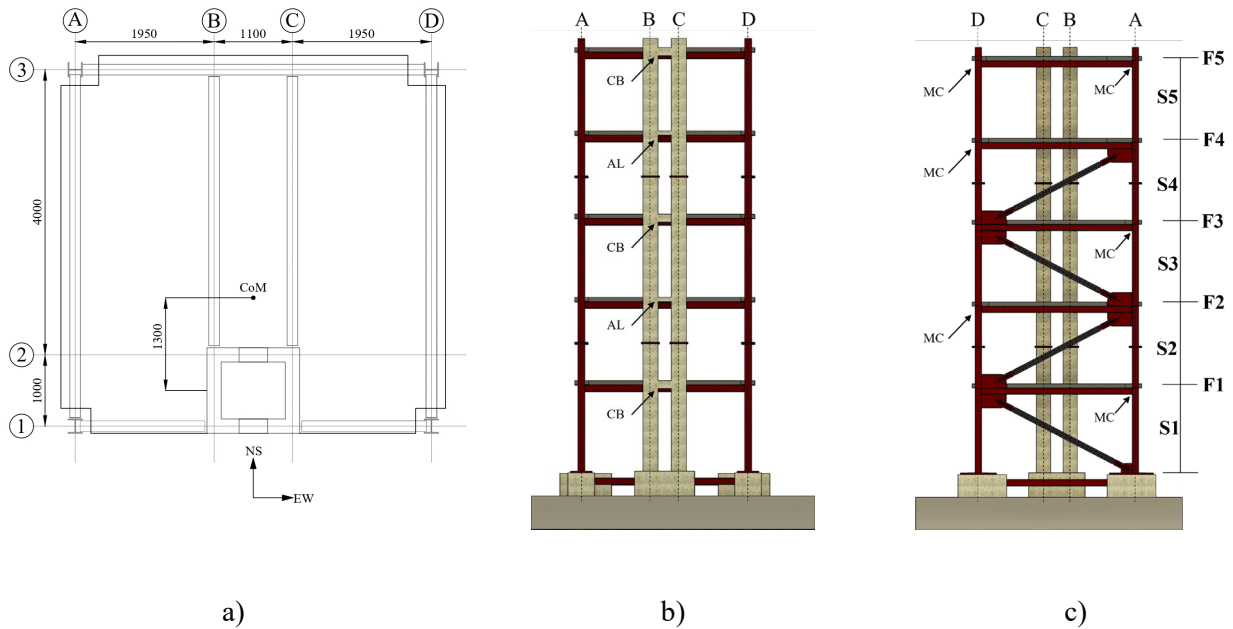


Figure 1: a) Typical plan view of test specimen, b) elevation of south façade showing the location of coupling beams (CB) and axial links (AL), and c) elevation of the north façade showing the location of moment-resisting beam-column connections (MC) and numbering of floors (F) and storeys (S).

The coupled walls consisted of two symmetric, C-shaped wall sections connected by coupling beams at storeys one, three, and five and by pinned axial links at storeys two and four (Fig. 1b). Each wall section had a web length of 1200 mm, flange width of 450 mm, and a uniform thickness of 200 mm. The span of each coupling beam was 400 mm, with a span-to-depth ratio of ~ 1.7 . Pinned axial links with a total depth of 125 mm were included to tie the diaphragm together, while limiting base-shear strength and overturning moment within the specifications of the earthquake simulator. The width of coupling beams and axial links width was 200 mm, matching the width of adjacent wall sections. All steel columns had H-sections of $200 \times 200 \times 8 \times 12$ (outer depth $\times$ width $\times$ web thickness $\times$ flange thickness), while all steel beams had H-sections of $194 \times 150 \times 6 \times 9$. In the NS direction, all beam-column connections were detailed as pinned. In the EW direction, moment-resisting beam-column connections were provided at joints 3A on floors 1, 3, and 5, and at joint 3D on levels 2, 4, and 5 (Fig. 1c). Because of construction and lifting constraints, the specimen was fabricated in three independent modules. 40-mm thick end plates were used to connect adjacent sections of the coupled walls, while 25-mm thick end plates were used to connect the steel columns of adjacent modules. End plates were located at mid-height of the 2nd and 4th storeys.

The structure was configured to allow modifications of its stiffness in the East-West direction (EW) while keeping a fixed configuration in the NS direction. Tests were completed in three phases, based on the building configuration used (Fig. 2). For the unbraced configuration, lateral resistance was provided by the weak-axis of the coupled walls and moment-resisting connections on gridline 3. For the braced configuration, buckling-restrained braces (BRBs) and gusset plates were installed on gridline 3 in the first four storeys. Lateral stiffness in the NS direction was provided by the strong axis of the coupled walls. Calculated storey weights for each phase are summarized in Table 1.

The average measured 28-day compressive strength for the coupled walls, coupling beams, and foundation blocks was 35 MPa, obtained from a total of 13-cylinder tests. For the composite slabs, the average measured 28-day compressive strength obtained from a total of 9-cylinder tests was 24 MPa. SS400A steel was used for all structural steel elements, with nominal yield and ultimate stresses of 205 MPa and 400 MPa, respectively.

Table 1: Calculated storey weights

Storey	Phase 1 [kN]	Phase 2 [kN]	Phase 3 [kN]
1	156	160	156
2	154	158	163
3	155	158	163
4	154	156	154
5	112	112	112

2.2 Design

The design of the structure roughly followed the proportioning procedure proposed by Browning (2001), and was governed by three main considerations: base-shear strength as determined by limit analysis, drift, and detailing to prevent brittle failure modes obtained from capacity design. Additionally, proportions, materials, and the position of structural elements were selected such that the specimen would be representative of new mid-rise buildings in New Zealand in terms of structural system, initial period, and base-shear strength.

Structural elements were designed to yield in flexure, without brittle failure modes, connection failures, and no shear or bond failures for RC elements. Longitudinal bars of the coupled walls were welded to end plates between modules and to a plate placed at the bottom of the wall footing for anchorage. Maximum shear demand was estimated assuming an effective lever arm equal to 50% of the wall height. In the direction of the coupling beams, a mechanism with plastic hinges was assumed to occur at all beam faces and wall bases, while in the perpendicular direction, hinging was assumed to occur only at wall bases. The minimum calculated base-shear strength coefficients ranged between 1/8 and 1/4, depending on the configuration.

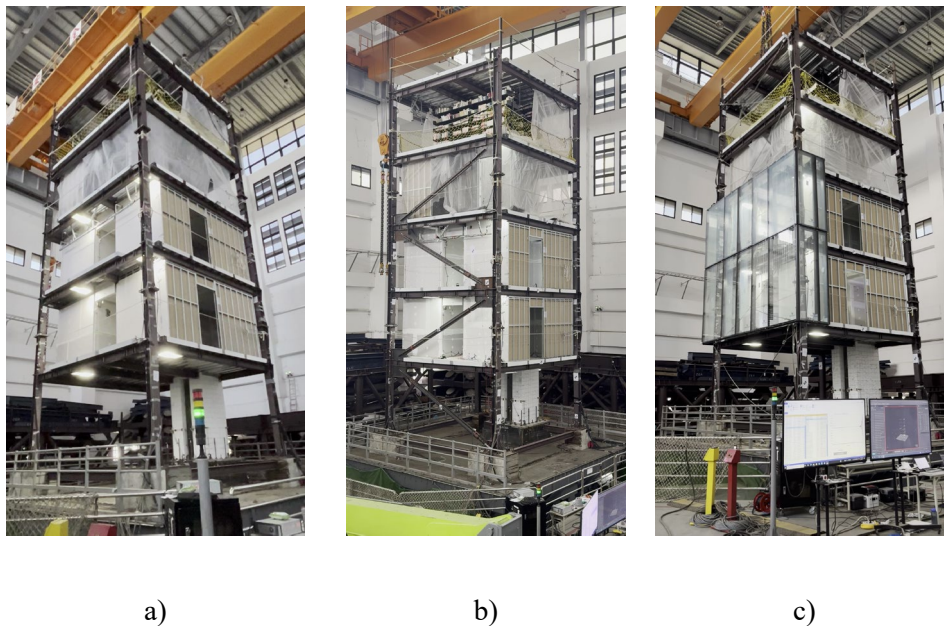


Figure 2: Building configurations: a) Phase 1: unbraced configuration, b) Phase 2: braced configuration, c) Phase 3: unbraced configuration.

The ‘design intensity’ corresponded roughly to a ground motion with a PGV of 40 cm/s, a PGA of 0.4 g, and a displacement spectrum reaching approximately 200 mm for a period of 1 s and a damping ratio of 2%. The corresponding spectral acceleration at the same ordinate was approximately 0.8 g. The calculated initial period for the unbraced configuration was approximately 0.6 s. The estimated roof drift ratio for this configuration was 2% for a ‘design intensity’ ground motion. The calculated initial period for the braced configuration was approximately 0.4 s, with an estimated roof drift ratio of 1% for a ‘design intensity’ ground motion. The calculated initial period for the NS configuration was approximately 0.4 s. The estimated roof drift ratio for this configuration was also 1% for a ‘design intensity’ ground motion.

2.3 Tests

The ground motions used during the experimental programme were selected based on two aspects: 1) scaled peak ground acceleration, velocity, and displacement had to remain within the limits of the equipment, and 2) spectral displacements had to increase with period inside a range of periods of interest (0.4 sec to 1.5 sec). Original ground motion records were scaled in amplitude to match the ‘design intensity’ described before. Based on these aspects, the ground motion recorded during the 1940 El Centro, California (RSN 6) was predominantly used during the tests. The ground motions recorded during the 1995 Kobe, Japan (KJMA), 1999 Chi-Chi, Taiwan (CHY006), and 2011 Christchurch, New Zealand (RE. HS.) earthquakes were used for two tests each. A total of 42 simulated earthquake excitations were performed.

The tests were divided into three independent phases, based on the building configuration used. Phase 1 consisted of 8 low-intensity earthquake simulations using the unbraced configuration, while Phases 2 and 3 consisted of 16 and 18 earthquake simulations using the braced and unbraced configurations, respectively. Before the start of Phase 3, a curtain wall was installed on the north façade of the building. Roof and storey drift ratios were calculated based on measured displacements obtained from an array of over 350 motion capture targets. Motion capture targets were located on the visible edges of the composite slabs, steel columns, and coupled walls on the north, west, and south façades of the specimen. Applied excitations were monitored using three triaxial accelerometers installed on the platform of the earthquake simulator, located at the projected centre of mass of the structure and near the north-east and south-west floorplan corners.

Overall, the test specimen was subjected to peak roof and storey drift ratios of 3% and 3.8%, respectively. The details used resulted in a structure exhibiting no brittle or unexpected failures. For ‘design intensity’ ground motions, average roof drift ratios measured at the centre of mass were 1.4%, 0.6%, and 1.9%, for the unbraced, braced, and North-South configurations, respectively. Results of the experimental programme show that a simplified design procedure focusing on drift, base-shear strength, and detailing to prevent brittle failure modes is an effective alternative for the earthquake-resistant design of structures.

3 DRIFT DEMANDS

The method evaluated in this study is known as the Velocity of Displacement (VOD) method, based on the title of the paper in which it was first introduced (Sozen, 2003). VOD states that an upper bound for drift demands can be estimated based on a linearly increasing displacement spectrum curve for single-degree-of-freedom (SDOF) oscillators. The displacement spectrum was assumed to be proportional to peak ground velocity (PGV), and to the effective period of the structure, calculated as the initial period (T_0) times a period amplification factor equal to $\sqrt{2}$. The period amplification factor was selected based on observations of the ratio of drift demands in linear vs. nonlinear structures (Bonacci, 1989; Lepage, 1997; Shimazaki & Sozen, 1984). The displacement spectrum was defined assuming 1) spectral displacements and velocities are related through circular frequencies, and 2) the spectral velocity of a ground motion can be approximated as the product of PGV and a constant amplification factor (F_v). For a damping ratio of 2%, the velocity amplification factor (F_v) was assumed to be equal to π (Newmark, 1973). Drift estimates for the substitute

SDOF are then projected to the multi-degree-of-freedom (MDOF) system by multiplying the SDOF drift by the modal participation factor (Γ) and the mode shape factor (ϕ), as shown in Equation 1. The method implicitly assumes that the deflected shape of a structure at peak drift is approximately equal to the dominant mode shape of the structure in the direction of interest.

$$\delta_i = PGV \cdot F_v \cdot \frac{T_o \sqrt{2}}{2\pi} \cdot \Gamma_j \cdot \phi_{j,i} \quad (1)$$

where δ_i = estimated drift at node i ; Γ_j = modal participation factor of the dominant mode j in the direction of interest; $\phi_{j,i}$ = mode shape factor of node i corresponding to mode j .

Figure 3 shows a comparison of the spectral displacement curve used by the VOD method for a PGV equal to 40 cm/s and the displacement spectrum calculated during the experimental programme for earthquake excitations reaching a PGV of approximately 40 cm/s and a damping ratio of 2%. Modal properties used to estimate drift demands were obtained using a linear model using gross-section properties for the RC elements (Table 2). Mode shape factors (ϕ) were normalized with respect to the mode shape factor at the centre of mass at roof level. In the East-West (EW) direction, roof drift ratios (RDR) and storey drift ratios (SDR) were calculated at the centre of mass (CoM), north façade (NF) and south façade (SF) of the structure. For brevity, RDR and SDR in the NS direction were only calculated at the centre of mass.

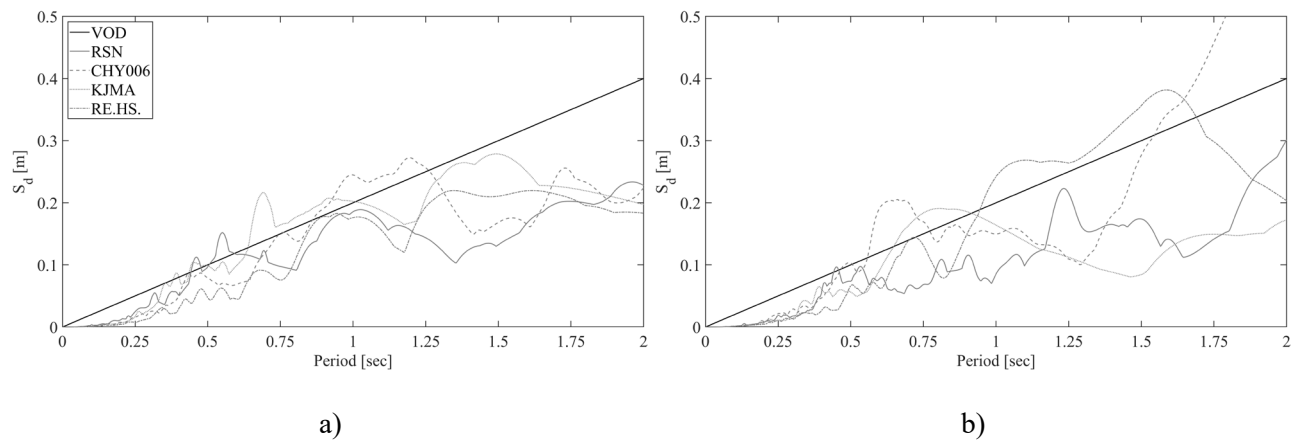


Figure 3: Comparisons of spectral displacement curves using the VOD method and spectral displacements measured during the experimental programme in the a) North-South and b) East-West directions.

RDRs were calculated using the mode shape factors (ϕ) at roof level and dividing drift estimates by the total height of the structure, while SDRs were calculated as the difference between estimated drift at the top and bottom levels of any storey, divided by the corresponding storey height. Figure 4 shows a comparison of estimated and measured RDR and SDR.

On average, the VOD method overestimated RDR and SDR at the south façade by approximately 7% and 57%, respectively, and underestimated RDR and SDR at the north façade by approximately 50% and 20%, respectively. In contrast, the method underestimated RDR and SDR at the centre of mass by approximately 10% in both directions. The coefficients of variation (COV) of the ratio of measured to estimated RDR and SDR at the centre of mass were 0.4 and 0.6, respectively. These ratios had similar means and COVs as those reported in previous studies for structures with translational modes of response (Laughery, 2016; Shah, 2021), suggesting that drift at the centre of mass was relatively insensitive to the displacement history of the test specimen and to the displacement amplitude. A comparison between estimated and measured RDR and SDR from a previous study on translational structures is shown in Figure 4 (Shah, 2021). The COVs of the ratio of measured to estimated RDR and SDR reported in that study are 0.3 and 0.4, respectively.

For both maximum roof and storey drift ratios, variations in estimates increased with increases in maximum measured drift ratios. For measured roof and storey drift ratios not exceeding 1%, approximately 18% and 16% of estimates, respectively, deviated from measurements by more than $\pm 0.5\%$, while for measured roof and storey drift ratios not exceeding 3%, approximately 29% and 35% of estimates, respectively, deviated from measurements by more than $\pm 0.5\%$. Overall, 89% of roof drift estimates and 84% of storey drift estimates deviated from measurements by less than $\pm 1\%$. Among these estimates, approximately half of the estimates were larger than measured values at the centre of mass in the NS direction and at the south façade in the EW direction. On the other hand, approximately 90% of the estimates were larger than measured values at both the centre of mass and at the north façade in the EW direction.

Table 2: Modal properties of each building configuration used to estimate drift demands

Property	Unbraced (EW)	Braced (EW)	North-South
T_o	0.57	0.44	0.39
Γ_j	1.1	1.3	1.4
$\phi_{j,CoM}^{RDR}$	1	1	1
$\phi_{j,NF}^{RDR}$	1.61	0.82	-
$\phi_{j,SF}^{RDR}$	0.52	1.15	-
$\phi_{j,CoM}^{SDR}$	0.28	0.33	0.36
$\phi_{j,NF}^{SDR}$	0.46	0.25	-
$\phi_{j,SF}^{SDR}$	0.15	0.39	-

T_o : Initial period, Γ_j : modal participation factor, $\phi_{j,-}^{RDR}$: mode shape factor used to calculate RDR at the centre of mass (CoM), north façade (NF), and south façade (SF), $\phi_{j,-}^{SDR}$: mode shape factor used to calculate SDR at the centre of mass (CoM), north façade (NF), and south façade (SF)

Difficulties in estimating drift demands at opposite façades of the test specimen in the direction controlled by a torsional response were a consequence of changes in the deflected shape caused by the displacement history of the specimen, compared to the initial deflected shape represented by the modal properties obtained from a linear model with gross-section properties. Changes in the deflected shape were quantified as the ratio of roof displacements in the North façade to roof displacements in the South façade measured during each test run. As shown in Figure 5, roof displacement ratios gradually decreased during the first 8 test runs (Phase 1) as a consequence of cracking in the RC walls in the South façade, while no softening occurred in the steel columns or composite slab near the North façade. During the following 16 tests (Phase 2), the ratio of roof displacements remained relatively constant at a value of approximately 0.4. After the removal of the BRBs, the roof displacement ratio gradually increased to approximately 1.2 as cracking of the composite slab extended to the North façade and steel columns yielded. In contrast, the ratios of roof mode shape factors in the North façade to roof mode shape factors in the South façade used to estimate drift demands were 3.1 for the unbraced configuration and 0.7 for the braced configuration, drastically different to average roof displacement ratios observed during the tests.

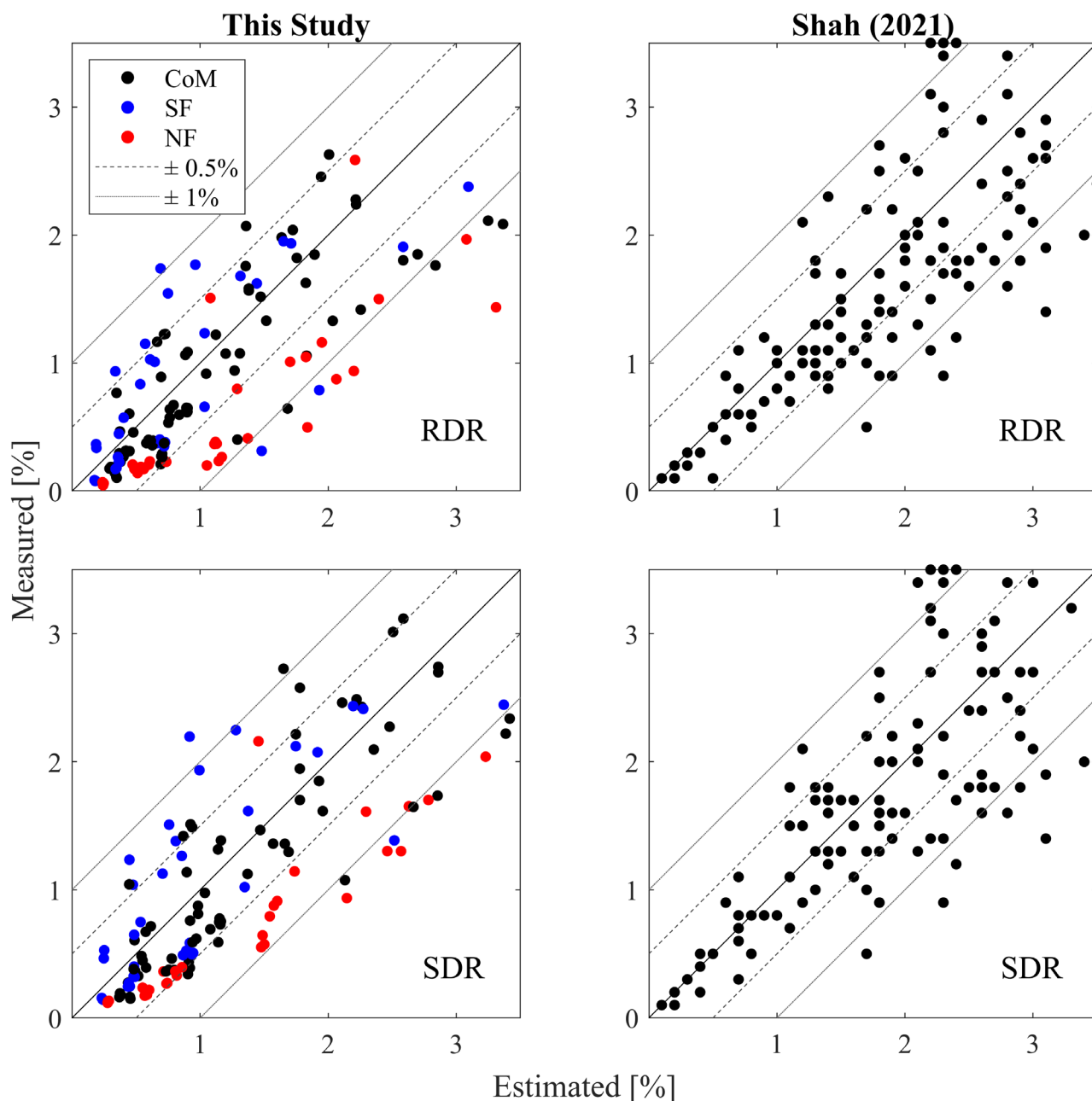


Figure 4: Comparisons of estimated and measured RDR and SDR at the Centre of Mass (CoM), North Façade (NF) and South Façade (SF) for this study (left), and comparisons of estimated and measured RDR and SDR for translational structures (right) from a previous study (Shah, 2021).

During the development of the VOD method, Sozen favoured the use of the initial period of a structure, calculated using gross-section properties, over the period calculated assuming all RC sections in a structure are cracked. Considering that both periods (or stiffnesses) are essentially ephemeral in earthquake response, neither option provides a clear advantage for idealizing a real structure with a linear model. An argument in favour of cracked stiffness is that the modal shape may better approximate the deflected shape of a structure. Consequently, a conservative approach is to compute drift demands assuming both gross-section and cracked-section properties and adopt the larger estimate.

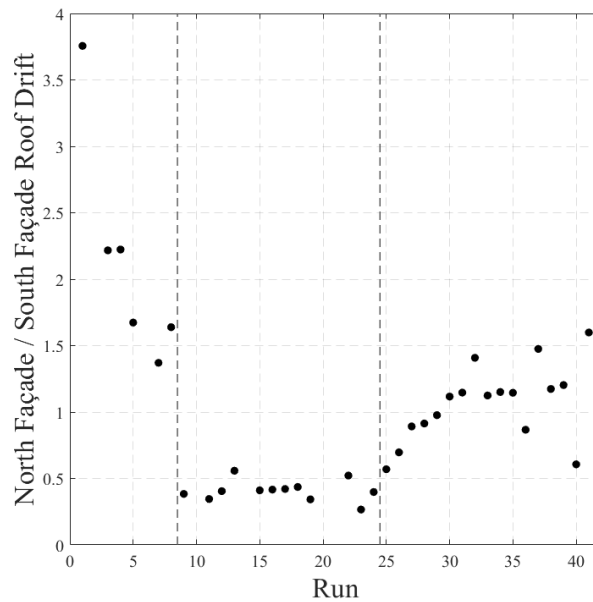


Figure 5: Progression of the ratio of roof displacements in the North façade to roof displacements in the South façade during the experimental programme. Vertical lines represent changes in the building configuration used (Phases of the experimental programme).

4 CONCLUSIONS

A five-storey, 12.6-m tall structural steel-RC hybrid structure with horizontal irregularities was subjected to 42 simulated earthquake excitations. The design of the specimen was governed by three main considerations: base-shear strength as determined by limit analysis, drift control, and detailing to prevent brittle failure modes obtained from capacity design. Two building configurations (i.e. braced and unbraced) were evaluated in the direction controlled by a torsional mode of response. Drift demands were estimated using the Velocity of Displacement method (Sozen, 2003), which suggests that drift is proportional to peak ground velocity (PGV) and to the initial period of a structure (T_0). Drift estimates were compared to measured drift ratios. Key conclusions are summarized here:

- The proportioning and detailing used in the large-scale, structural steel-RC hybrid building resulted in a structure exhibiting no brittle or unexpected failures, even after being subjected to peak roof drift ratios of 3% and peak storey drift ratios of 3.8%. A simplified design focusing on drift, base-shear strength and capacity design was effective for the design of an earthquake-resistant structure.
- Overall, the Velocity of Displacement method produced conservative drift estimates for the test structural steel-RC specimen with a torsional mode of response, even though the method was initially calibrated for RC structures with a translational mode of response. The ratios of measured to estimated roof and storey drift ratios at the centre of mass were similar to those reported in previous studies for translational structures, suggesting that drift estimates at the centre of mass were relatively insensitive to the displacement history of the structure and the displacement amplitude of each test run.
- Drift demands at opposite façades in the direction controlled by a torsional response were sensitive to the displacement history of the specimen. Measured roof displacement ratios at opposite façades changed depending on the building configuration and irregular softening of both RC and structural steel elements. Considering that the quality of the estimates could be improved by reducing the stiffness of the RC elements (i.e. using cracked-section stiffness), a conservative approach is to

compute drift demands assuming both gross-section and cracked-section properties and adopt the larger estimate.

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