Seismic axial collapse of short shear span RC shear walls above transfer structure

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Abstract. A special class of short shear span RC shear wall exists in the form of a sub-structure at the base of slender walls in tall buildings, was found common in the low-to-moderate seismicity region. These non-seismically designed short shear span RC walls are highly stressed in axial load under gravity and lateral actions, and have limited deformability. Thus, there is a need for structural engineers to re-examine the seismic performance of this special class of short shear span RC walls under a rare earthquake event. Recently, a Modified Mohr’s Axial Capacity Model (MMACM) was developed by the authors to estimate the axial load capacity of these walls prior to axial collapse after shear failure. The model underpinned the assumption of seismic axial collapse of these walls is closely associated with out-of-plane buckling. A numerical example of a RC building with transfer structure in Malaysia, subject to seismic loading is modelled to demonstrate the use of the MMACM in estimating seismic axial collapse of short shear span RC shear walls.

1. Introduction
A special class of short shear span reinforced concrete (RC) shear walls is identified with short shear span, attributed to the high coupling degree in the elastic design of tall buildings for controlling drift. These walls are often not seismically designed with capacity hierarchy and are located in low-to-moderate seismicity regions. The short shear span walls are essentially a sub-structure at the base of slender shear walls and hence are often heavily reinforced with 1% to 2% reinforcement ratio. The short shear span effects are more significant for walls sitting on transfer structure. This is due to the out-of-plane bending of the transfer structure induces additional shear force into the walls, by means of in-plane floor strut and tie forces adjacent to the transfer level. This scenario is named as the shear concentration effects [1, 2].

Figure 1(a) to 1(c) show the histograms of the shear span-to-length ratio (SLR) of short shear span RC walls adjacent to the transfer level, where the shear span \((a)\) is defined as the lateral moment over base shear \((a = M/V)\). Figure 1 also shows the axial load ratio (ALR) of those walls for three tall RC buildings with transfer structure in Malaysia, analysed by using ETABS [3]. Assuming non-composite actions exist in some of the flanged walls by taking all walls as individual rectangular, it was found that under gravity at neutral position, the average ALR is about 0.15 to 0.20 (except for Building Y.
which has a higher ALR). Under wind load, the SLR in walls can be as low as 1.5. The short shear span characteristic precludes plastic hinge formation and increases the susceptibility of shear failure during seismic events. Past experiences with earthquakes [4, 5] demonstrated the risk of shear wall failure under axial loading after experiencing shear failure.

![Graphs showing SLR and ALR](image)

Figure 1. Reconnaissance of SLR and ALR of shear walls in tall RC buildings in Malaysia with transfer structure (a) Building X (b) Building Y (c) Building Z.

It is noted that shear damage alone does not indicate collapse by default. The authors had carried out experiments on the axial collapse (beyond shear failure) of the short shear span walls with concurrent axial permanent actions and lateral seismic actions in Looi et al. [6]. Based on the observation during the experiments and the post-processed results of test data, the Modified Mohr’s Axial Capacity Model (MMACM) was recently developed in Looi and Su [7] for walls with axial-to-shear stress ratio of less than 2 ($p/v < 2$). This paper introduces the use of the MMACM model on a tall RC building with transfer structure in Malaysia, considering the seismic actions for Malaysia proposed in Looi et al. [8], together with the spectral shape of Model B adopted in the Malaysia National Annex (NA) for Eurocode 8 (EC8) [9].

2. The Modified Mohr’s Axial Capacity Model
The ALR capacity prior to collapse is formulated based on Mohr’s circle framework [10], but modified with the inclusion of reinforcement buckling. The MMACM associates axial collapse with out-of-plane buckling. The out-of-plane buckling scenario was observed in recent earthquake damage such as in Christchurch [11] and in the authors’ experiments [6] (see Figure 2).
2.1. Formulation

Figure 3 illustrates the vertical axial capacity formulation expressed in a Mohr’s circle framework, with normalised normal stress ($\sigma/f_c'$) as the abscissa and normalised shear stress ($\tau/f_c'$) as the ordinate. Figure 3(a) is the envelope circle of steel reinforcements provided by horizontal steel reinforcement in tension and vertical steel reinforcement in compression buckling. Figure 3(b) plots the contribution of cracked concrete, where the cracked concrete is assumed to have zero tension capacity at the onset of collapse.

Equation (1) is a mathematical representation of Figure 3 at the compression stress axis.

$$P_{caps} = \rho_v \sigma_{buck} + \nu$$

$$\frac{P_{caps}}{f_c'} = \rho_v \frac{\sigma_{buck}}{f_c'} + \frac{\nu}{f_c'}$$

$$ALR_{cap} = \frac{\rho_v \sigma_{buck}}{f_c'} + \frac{\nu}{f_c'}$$

Figure 3. Schematic illustration of ALR limit based on Mohr’s circle framework (a) envelope of steel contribution (b) cracked concrete contribution (c) applied ALR.
where $\rho_{\text{cap}}$ is the axial compression stress capacity, $\text{ALR}_{\text{cap}}$ is the ALR capacity of the wall, $\rho_v$ is the vertical reinforcement ratio at the wall edge, $\sigma_{\text{buck}}$ is the compression buckling stress of the reinforcement, $f'_c$ is concrete cylinder strength and $\nu f'_c$ is the normalised shear stress capacity. The compression buckling stress capacity ($\sigma_{\text{buck}}$) given by the vertical reinforcement can be estimated by using the Dhakal and Maekawa model [12], where the maximum residual tensile strain ($\varepsilon_{\text{res}}$) of the longitudinal reinforcement in the preceding cycle before buckling occurs can be estimated using the Chai and Elayer model [13]. The shear stress capacity ($\nu$) can be readily estimated using proposed equation in Looi et al. [6]. For Figure 3(c), the applied axial stress ($p$) is more critical at the wall edge and should be added with the vertical component of the diagonal shear stress coming from the lateral seismic actions, assuming an angle corresponds to the SLR. Readers of this article are suggested to refer to those original articles [6, 12-13] and the concise summary in Looi and Su [7].

2.2. Assumptions and limitations
Assumptions were made in developing the MMACM, below are the known limitations:

- the RC walls with short shear span are not designed with a strength hierarchy, have an SLR that is less than 1.5, vertical reinforcement ratio that is more than 1%, and diagonal shear failure precedes the formation of plastic hinges;
- axial collapse behaviour is assumed to be closely related to out-of-plane buckling;
- the MMACM should be only used to check walls with $p/\nu < 2$;
- walls with short shear span (with any value of $p/\nu$) should be checked for collapse by examining the limits of drift using recommendation in Looi et al. [6];
- cracked concrete with zero tension capacity is assumed; and
- post-peak shear sliding failure is assumed does not occur at the base of walls with sufficient dowel action. This failure mode was observed from the experiment in Looi et al. [6], with insignificant pinching of hysteresis loop induced by shear sliding.

3. A case study of RC building with short shear span wall sitting on transfer structure in Malaysia
A unique performance based seismic design (PBSD) approach for tall buildings was proposed for regions of low-to-moderate seismicity [14]. Considering the geometry irregularity and complexity of tall buildings (i.e. transfer floor, elevation and plan setbacks), simply fully adopting an internationally recognised prescriptive earthquake design codes (for example EC8 [15]) with stringent clauses may impose heavy penalties on buildings in regions that previously did not consider seismic actions. Alternative option is sought and the PBSD appears to be an attractive avenue.

This approach allows the structural engineers to first analyse and design structural members under conventional gravity and wind actions. This is called a Model-W, where no failure of members is expected as this is the common design routine under elastic condition. The approach is followed by a two-level cracked stiffness scheme to model a softer damaged concrete behaviour by implicitly consider the RC material non-linearity in a pseudo inelastic secant stiffness response spectrum analysis. They are called Model-E1 and Model-E2. Structural engineers are required to check the response of the members under seismic action in Model-E1 with the proposed initial cracked stiffness. If the members exceed the code capacity, the cracked stiffnesses are lowered down within a limit to form Model-E2 for further checking. Under a very rare earthquake scenario (e.g. 2475-year return period), this approach capitalises the material expected strength and unity value of material safety factor to avoid over-conservatism considering non-collapse performance state. For design of new building structure when a seismic code is legally enforced by the local authorities in the low-to-moderate seismicity region, the capacity design philosophy can be incorporated with identification of force-controlled and deformation-controlled actions on members, allowing over-strength factor to
elevate the demand on force-controlled members. Simplified global and local component acceptance criteria are suggested with associated procedures and limit.

In this paper, an unidentified 46-story RC tall building with transfer structure in Malaysia (codenamed as Building X, see Figure 1(a)) is used as an example to further demonstrate the proposed PBSD approach for seismic checking of existing building (not for new design). The short shear span walls adjacent to the transfer structure in Building X are checked for seismic axial collapse. The building is expected to be subject to a notional 475-year return period earthquake scenario, and hence material safety factor is included. Capacity design is not considered, to reflect the actual building design scenario now in Malaysia.

3.1. General building structural description and site information
Building X is a 46-storey RC tall building with a transfer plate supporting 40 storeys above. The building measures about 60 m x 18 m on plan and stands at a height of 158 m above ground level. The lateral force resisting system consists of a 40-storey shear wall-frame structure tower above a 6-storey core wall-moment frame structure podium. A 2.5 m thick plate exists to transfer the load from the tower to the podium. The typical story height is 3.2 m with more headroom at the podium levels, approximately 4.8 m. Concrete grade ranging from C30, C40 and C50 concrete (measured on characteristic cube strength $f_{cu,k}$) are used for slab, beam, column and wall. Reinforcements are typically of grade 250 MPa and 460 MPa. The slab thickness is in the range of 150 to 250 mm. The beam size (width x depth) is in the range of 150 x 350 mm to 800 x 600 mm. The column size varies from 300 x 300 mm to 1200 x 3000 mm. The core wall thickness is about 400 mm at the base and the shear wall thickness is 300 mm at transfer level. The foundation details are intentionally not included, but the building is located at a relatively deep soil site with sediment more than 30 m in Peninsular Malaysia, correspond to a site natural period of $T_s = 0.7$ s (in between ground type C and D in accordance to Model B in the NA [9]). Readers of this article are encouraged to refer to Looi et al. [16] for example of site natural period calculation based on borehole records.

3.2. Structural analysis model
Building X was modelled and analysed in three-dimensional finite element program ETABS [3]. There are two main categories of models, namely Model-W, an elastic model (uncracked concrete stiffness) for gravity and wind analysis; and Model-E1 and Model-E2, models using initial and further cracked concrete stiffness for earthquake response spectrum analysis. The frames are modelled as line elements, shear walls as shell elements with minimum out-of-plane stiffness (0.25 cracked stiffness was used in this example to avoid attracting large out-of-plane forces) and typical floor slabs as shell elements. The transfer plate is modelled as shell element with thick plate characteristic to consider shear deformation. Semi-rigid floor diaphragms are assigned to three floors above and below the transfer plates to account for shear concentration in the shear walls, while rigid diaphragm behaviour is assumed for the rest of the floors. The supports are modelled as pinned. The building density is about 5 kN/m$^3$. Design wind pressure is estimated from the local wind code MS 1553 [17] with basic wind speed of 20 m/s. The building is subjected to the design spectrum of acceleration ($S_d$) of Ground Type D in Figure 4, with Importance Class III ($\gamma_I = 1.2$) and a behaviour factor $q = 1.5$. The Malaysia EC8 NA [9] limits the spectrum at 4 s, however Building X in Model-E1 has a first mode period of 6.7 s. By assuming a constant displacement after the second corner period ($T_{d}$), the spectrum is extrapolated to 7 s by using Equation 3.16 of EC8 [15].
3.3. Analysis and design results

Model-W for Building X was first analysed under elastic condition. The building passed the global drift criteria and the members are designed accordingly. The elastic first mode period of Building X is about 4.3 s. The detailed results such as drift and stress check of Model-W are not shown in this article due to page limit requirement.

Model-E1 was constructed based on the cracked stiffness in Table 1. There is a period lengthening effect of about 1.5 from Model-W to E1 due to the softening effects in concrete. The mass participation ratio of 95% is achieved after 12 mode shapes. The corresponding period of mode 1 and 3 are superimposed on the acceleration-displacement response spectrum (ADRS) diagram in Figure 5 for the convenience of engineers in understanding the dynamic behaviour of Building X and estimating the acceleration and displacement demand. The local stress component check for Model-E1 was carried out for bending moment, axial-moment interaction and shear capacity check against the seismic demand.

| RC members | Flexural ($E_i I_g$) | Axial ($E_c A_g$) | Shear ($G A_g$) | Compatibility Torsion ($G J$) |
|-------------|---------------------|------------------|-----------------|-------------------------------|
| Model       | E1                  | E2               | E1              | E2                           | E1 or E2 |
| Coupling beam | 0.35               | 0.12             | -               | -                             | -        |
| Column      | 0.70               | 0.50             | 0.60            | -                             | -        |
| Wall        | 0.60               | 0.30             | 0.60            | 0.30                          | 0.50     |
| Slab        | 0.20               | -                | -               | -                             | n/a      |
| Transfer Plate | 0.35             | -                | -               | 0.50                          | -        |

Notes: $E_i$ is the modulus of elasticity of concrete, $G$ is the shear modulus of concrete, $I_g$ is the moment of inertia of the section, $A_g$ is the sectional area and $J$ is the torsional constant.

Figure 4. The design acceleration spectrum

Figure 5. Building X structural periods of Model-E1 superimposed on the ADRS diagram
Further cracked section factor to account for further softening of concrete can be fine-tuned within the limits of Table 1, the final model is named as Model-E2. This approach is justified by redistribution mechanism of internal forces and moment demand to other stiffer supporting members, hence Model-E2 needs to be re-evaluated for ductility demand in global level and in local component strength check. Natural periods of Model-E2 is higher than Model-E1 (with a gentler slope), hence it is not shown in Figure 5 for clarity.

This paper focuses on the behaviour of short shear span RC shear walls above transfer structure, and hence only those walls are described. The walls were assumed reinforced with at least 2% longitudinal reinforcement, without provision of boundary confinement. The seismic actions were considered for the results of the most onerous directional combination. Figure 6 shows the location of the axial-moment (PM) critical wall (coded as P103), with a utilisation ratio (defined as demand over capacity) of 0.93. The wall was stressed to the limit in the PM check after considering seismic action in Model-E2, even though it passed with sufficient buffer in the earlier PM check in Model-W. This can be explained by the redistribution of internal forces and the automatic inclusion of the vertical component of lateral seismic action in the continuum shear wall finite element model, which imposed higher demand on this wall.

Figure 6. Location of P-M-critical shear wall at transfer level

Figure 7 shows the locations of the identified shear critical walls (coded as P102 and P122), checked with the web crushing limit due to shear without considering axial compression. The walls did not crush in the web, with a utilisation ratio of 0.85. However, these walls will be further examined by the seismic axial collapse check.

Figure 7. Locations of shear-critical shear walls at transfer level, checked under web crushing without considering axial compression
4. Seismic axial collapse check of short shear span walls using the MMACM

Further checking on seismic axial collapse for the final Model-E2 is demonstrated in this section. All walls were found to have SLR of less than 1.5 (refer to SLR summarised in Figure 1(a)). Figure 8 shows the locations of the critical short shear span walls above the transfer structure. The shear stress capacity (ν) of the walls were estimated based on the proposal in Looi et al. [6]. By using the MMACM in Equation 1, wall P102 and P121 were identified. Wall P102 has a \( p/ν < 2 \) and wall P121 is with \( p/ν > 2 \). Wall with \( p/ν < 2 \) may experience cyclic tension-compression excursion at the wall edge prior to collapse [7]. Wall P102 was found reaching its peak diagonal shear capacity with a utilisation ratio of 0.95 but survived in the seismic axial collapse check with utilisation ratio of 0.60. Wall P121 is critical due to its \( p/ν > 2 \), where the wall may experience cyclic compression-compression excursion at the wall edge [7]. It was discovered in previous experiment [6] that the short shear span walls with \( p/ν > 2 \) would collapse with out-of-plane buckling with lateral drift capacity as small as 0.47%. For Building X, wall P102 and P121 have a high possibility failing in seismic axial collapse. However, the location and the dimensions of the walls revealed that these walls are not considered as main shear walls. Nonetheless, caution should be taken for post-earthquake progressive collapse after internal gravity stress redistribution.

![Figure 8. Locations of seismic axial collapse critical short shear span walls at transfer level, checked with the MMACM](image)

5. Conclusion

It is noted that shear and axial failures can simultaneously occur at relatively low drift levels under high ALRs. The minimum ultimate drift capacity associated with the shear and axial failures of walls with a short shear span is found to be as low as about 0.47% under an ALR of 0.4 [6]. This means that building occupants are given no warning before axial failure occurs. Engineers must be therefore able to identify shear walls that may experience this catastrophic failure mode, and focus more on the effects of the load path of gravity systems rather than only the lateral load resisting systems. In this paper, a method (i.e., the Modified Mohr’s Axial Capacity Model) to quantify the seismic axial collapse of short shear span shear walls were presented with a numerical model of a tall building with transfer structure in Malaysia. Engineers should be aware of the limitations of the model for axial collapse check of short shear span RC walls in practical design.

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