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Simplified progressive collapse simulation of RC frame–wall structures

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ABSTRACT

A macromodel-based approach to enable post-event progressive collapse analysis of reinforced concrete (RC) frame–wall structures is investigated. A simplified shear wall model is developed to simulate the inelastic behavior of a multi-story frame–wall system due to the sudden loss of a significant portion of the shear wall at the first story. Detailed finite element analyses are employed not only to provide modeling insights but also as a tool to verify the accuracy of the developed shear wall model. Two perimeter frame–wall systems designed for different seismic zones are modeled using the proposed approach and numerical simulations following the sudden loss of a portion of the shear wall at the lowest story are compared and evaluated. Although no signs of collapse are evident in either system, detailed investigation of force variations in structural members shows that the seismically designed frame–wall system (SDC-D) is a more robust system compared to a system designed for much lower seismic demands due to the effectiveness of its structural layout and seismic detailing. The simplified methodology is a suitable approach for preliminary progressive collapse investigation of RC frame–wall structures.

1. Introduction

Among the practical design approaches to enhance structural resistance against progressive collapse are increasing structural integrity and/or redundancy. Procedures to incorporate progressive collapse considerations into the design process are available in guideline documents published by the US General Services Administration [1] and the Department of Defense [2]. However these documents do not provide sufficient information on procedures, particularly numerical modeling guidelines, to carry out progressive collapse studies of buildings. Numerical simulations investigating progressive collapse have been carried out by several researchers [3–6]. In addition to the fact that several simplifying assumptions are introduced to enable global response prediction, these studies are limited to frame structures. Simple modeling approaches to perform reliable analyses of reinforced concrete frame–wall structures in particular are still forthcoming. Part of the reason for the lack of progressive collapse studies of large scale wall–frame structures can be attributed to the lack of reliable macromodels that incorporate both wall and frame components.

The literature on beam–column modeling for frame systems is vast and does not need reiteration. Shear wall modeling, on the other hand, has seen limited advancement and has evolved through three fundamental methods: approaches derived from beam–column type models in which flexure is the dominant mode of response [7–11], multi-spring macromodels [12–15] and finite element models [16]. While shear effects can be incorporated by aggregating an inelastic shear spring in series to the flexural beam–column element, true shear–flexure interaction is not accurately modeled. Inelastic action in beam–column elements can be represented through lumped plasticity or by distributing inelastic behavior along a finite length. Calibration of model parameters is critical in achieving reasonable simulations. Multi-spring models are composed of a collection of discrete springs and other macro-elements that enable a better representation of the strain distribution across the section as well as the migration of the neutral axis under lateral cyclic loading. A more recent effort by Massone [17] extends the technique proposed by Colotti [15] to incorporate RC shear panel behavior into a displacement-based beam–column element thereby facilitating shear–flexure interaction. Finally it should be mentioned that in all cases the shear wall is modeled only in the 2D plane and out-of-plane 3D effects are not considered.

An ideal modeling methodology should enable a large scale simulation in a computationally efficient manner while essential and critical effects of large displacement response of both wall and frame components are still adequately represented. In this study, a simple shear wall model is proposed to enable a progressive collapse analysis when a significant part of a shear wall is removed at the lowest story. A macromodeling approach that essentially builds on existing models is employed to achieve the objectives of the study and comparative simulations are carried out on two dual (frame–wall) systems to investigate the effectiveness of seismic

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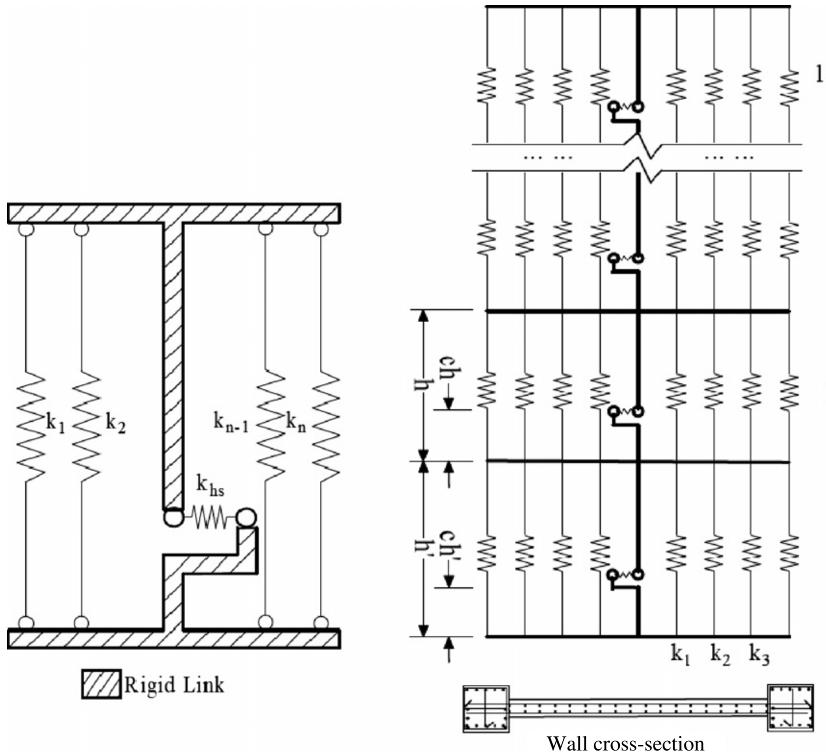


Fig. 1. Single MVLEM element and typical shear wall model.

design and detailing in effectively enhancing the performance of dual RC systems.

2. Proposed approach and limitations

Generally, progressive collapse simulation is carried out using either direct analysis wherein the imposed loading is modeled explicitly or through indirect analysis in which the actual loading event leading to structural damage is not modeled though the effects of the damage resulting from the loading event are evaluated. If the type of loading and the affected region of a structure can be well defined, direct analysis can be used to provide an accurate representation of actual performance in a damaging event. Given the uncertainty in identifying the exact nature and location of loading, threat-independent approaches are often used to evaluate the progressive collapse resistance of a structure so as to assess the redundancy in the gravity load resisting system. The alternate load path method (APM) is a threat-independent method recommended in the GSA [1] guidelines. For the purpose of this study, the APM is adopted as the analysis approach to evaluate the progressive collapse resistance of RC frame–wall systems.

A comprehensive progressive collapse simulation of a large structural system using detailed finite element models can be computationally prohibitive since it involves geometric and material nonlinearity as well as dynamic effects. Therefore, developing simplified yet reliable structural models is important for cost-effective collapse simulations. In this study, macromodeling techniques are used to simulate the large-deformation response of structural components such as beams, columns, joints, and shear walls. Physical phenomena at local level are represented by reduced models which are calibrated through high fidelity finite element analysis. The open-source platform OpenSees [18] is used to implement and demonstrate the proposed macromodeling method. Beams and columns in the structural system are modeled using integrated beam elements with fiber sections. Detailed discussion on modeling of beam–column joints can be found in previous studies by the authors [19]. The model for partially damaged shear wall is developed based on the multi vertical line element model (MVLEM) originally proposed by Kabeyasawa et al. [20] and enhanced by Vulcano et al. [21]. Details of the proposed modeling approach will be described in the following sections. Co-rotational transformation, available in OpenSees [18], is used to perform an exact geometric transformation of beam/column elements under large displacement conditions.

Since this study primarily focuses on evaluating the performance of frame–wall systems under gravity loads, the materials are considered to be rate independent. However, rate dependency must be considered if the direct effects of high velocity impact need to be investigated. Floor slabs are not included in current study although its effects are expected to influence the overall resistance of progressive collapse. The incorporation of slab effects requires the development of a three-dimensional model which is the subject of ongoing research.

3. Shear wall modeling

Two types of shear walls are considered in this study: an intact shear wall and a partially damaged shear wall. The intact wall model needs to have the ability to represent the primary failure modes: flexural or shear sliding. The partially damaged wall model, on the other hand, should represent local effects which may influence overall response as well as the failure mechanism under the assumed damage scenarios. In this work, emphasis was placed both on simplicity and the need to develop a methodology for progressive collapse analysis of a complete frame–wall system. While the study assumes that exactly half of the lower floor wall is damaged due to an extreme loading condition, the process described can be extended to other scenarios involving partial wall damage. Both regular and irregular damage boundaries are considered in the simulations.

3.1. Modeling of intact shear wall

The modeling of an intact shear wall is based on the multi-vertical-line-element model (MVLEM), shown in Fig. 1, in which the shear wall behavior is represented by a number of vertical–parallel springs and one horizontal spring to simulate the inelastic axial, shear, and flexural response of the element while rigid elements are used to represent the physical size of the wall.

A simple form of the multi vertical line element scheme was first introduced by Kabeyasawa et al. [20]. Improved versions of this approach have since been developed by other researchers [14,15] who have demonstrated that this approach captures important features of wall response that other simplified models fail to incorporate, for example, migration of the neutral axis, and provides the ability to include refined material models to describe important effects such as axial, flexure and shear interaction. The model parameters investigated by Orakcal et al. [14] include the number of macro wall elements stacked on the top of each other along the height of the wall (m), the number of vertical elements within each wall element (n), and the center of rotation parameter (c). Their findings indicate that the simulated global response is not very sensitive to the selection of m or n, provided that reasonable values are chosen to represent the overall physical characteristics of walls. Increasing m or n depends on how much detail is desired in the analytical results. The change of the center of rotation parameter c will affect the prediction of the wall strength and lateral stiffness, but this influence can be diminished by stacking more wall elements along wall height, especially in the highly inelastic region to reduce the change in curvature within each wall element [22]. In this paper, the value of c is assigned to be 0.4 as recommended by Vulcano et al. [21]. For modeling convenience, the wall segment within one story height is presented by a single MVLEM element. The web cross section is presented by six vertical elements and two wall edge columns are represented by two additional vertical elements.

3.2. Finite element analysis of partially damaged shear wall

To understand the influence of local effects on overall response and the failure mechanism due to the loss of a half wall section at the first floor, a detailed finite element analysis was carried out. Since the response of interest is primarily due to gravity loads, a pushdown simulation under displacement controlled loading is expected to provide the necessary information on model development. The prototype shear wall is taken from a ten-story dual system office building designed by Ghosh and Associates [23]. The same building is also used to investigate system responses later in the paper under sudden removal of a section of the lower wall.

3.2.1. Elements and material models

The finite element analysis is carried out using the commercial software DIANA [24]. Eight-node quadrilateral isoparametric plane stress elements (CQ16M) are used for modeling the shear wall panel since out-of-plane stresses are generally small and can be ignored without significantly affecting the accuracy of the simulation. Concrete behavior is described through the total strain crack model. The model represents concrete cracking using the smeared fixed or rotating crack approach and is generally considered to be more reliable than continuum modeling to simulate the post-peak response of concrete. Concrete in the shear wall web are assumed to be unconfined, but the effect of transverse confinement is considered for core concrete in the edge columns. The compressive stress–strain relationship of concrete is based on the model available in DIANA that was developed by Thorenfeldt et al. [25]. Concrete tension stiffening is considered, and the descending portion of stress–strain curve uses the softening function based on the model-I fracture energy introduced by Hordijk [26]. Steel bars in the wall are modeled as smeared reinforcement with perfect bond assumed between steel and concrete. This is a reasonable assumption for shear wall modeling since the influence of bond–slip is less significant than for beam–column joint regions. The material response of steel is modeled using Von Mises isotropic plasticity model with an assumed hardening ratio of 1.0% of the initial stiffness.

3.3. Modeling of partially damaged wall

The results of detailed finite element analysis show that a highly nonlinear zone is concentrated at the lowest two stories, while the remainder of the shear wall mostly remains elastic during the entire simulation (Fig. 2).

Two segment rotations are compared along the wall height as the wall deforms under the applied load: one is the rotation of the unsupported wall noted as θ_a and the other is the rotation of the supported wall noted as θ_b. Significant difference between θ_a and θ_b is observed in the story directly above the damaged story. This indicates the shear wall cross section in the highly nonlinear zone is no longer in the same plane. Therefore, simply removing vertical elements which represent the damaged wall does not reflect the local effect on the behavior of the damaged shear wall. A development of a simplified model of the unsupported wall section at second story level is shown in Fig. 2. Based on the stress distribution, the effective area of the unsupported second story wall is assumed to be the upper triangular region which is represented by a diagonal spring k_s. The outside column is modeled using an elastic element and the lower triangular region is represented by a horizontal spring k_h. These spring properties are calibrated in the present study with results from the finite element analysis. The simplified model for the entire damaged shear wall model is described in Fig. 3.

The model parameter c is assumed to be 0.1 at the second story because the center of rotation is expected to be located closer to the bottom due to stress concentration. This parameter was further verified by additional parametric studies in which the wall section configuration was altered to represent varying height-to-depth ratios. The shear wall web of the lowest two stories is modeled using ten vertical elements in order to capture the detailed progress of failure in these highly nonlinear regions.

3.4. Validation of wall model

The high fidelity finite element analysis is used not only to help developing the simplified shear wall model but also as a tool to validate the proposed macromodel by comparing the simulated responses using the two different modeling approaches.

3.4.1. Macromodel simulation using OpenSees

The simplified shear wall model described in the previous section and shown conceptually in Fig. 2 is simulated using the OpenSees [18] platform. In the proposed macromodel, finite length elements (vertical elements or the diagonal spring) are modeled using truss elements with fiber sections. The horizontal shear springs are represented by zero-length elements. Rigid elements are modeled through multi-point constraints (rigidLink option in OpenSees). Concrete is modeled using a uniaxial constitutive model with linear tension softening (Concrete02 model in OpenSees). For the purpose of this comparison study, model parameters, such as compressive strength, crushing strength and their corresponding strains, are the same values used in FE simulation (Table 1). Reinforcing steel bars are modeled using a modified Menegotto–Pinto model (Steel02). The horizontal shear spring is modeled using a uniaxial material model (PINCHING4) which incorporates degradation and pinching behavior [27]. The envelope of the force–deformation curve for the shear spring is derived based on the modified compression field theory by Vecchio and Collins [28].

3.4.2. Comparison between detailed model and macromodel

The load versus displacement responses obtained from the detailed FE analysis and the macromodel simulations are compared in Fig. 4. The damaged shear wall is also modeled by simply removing

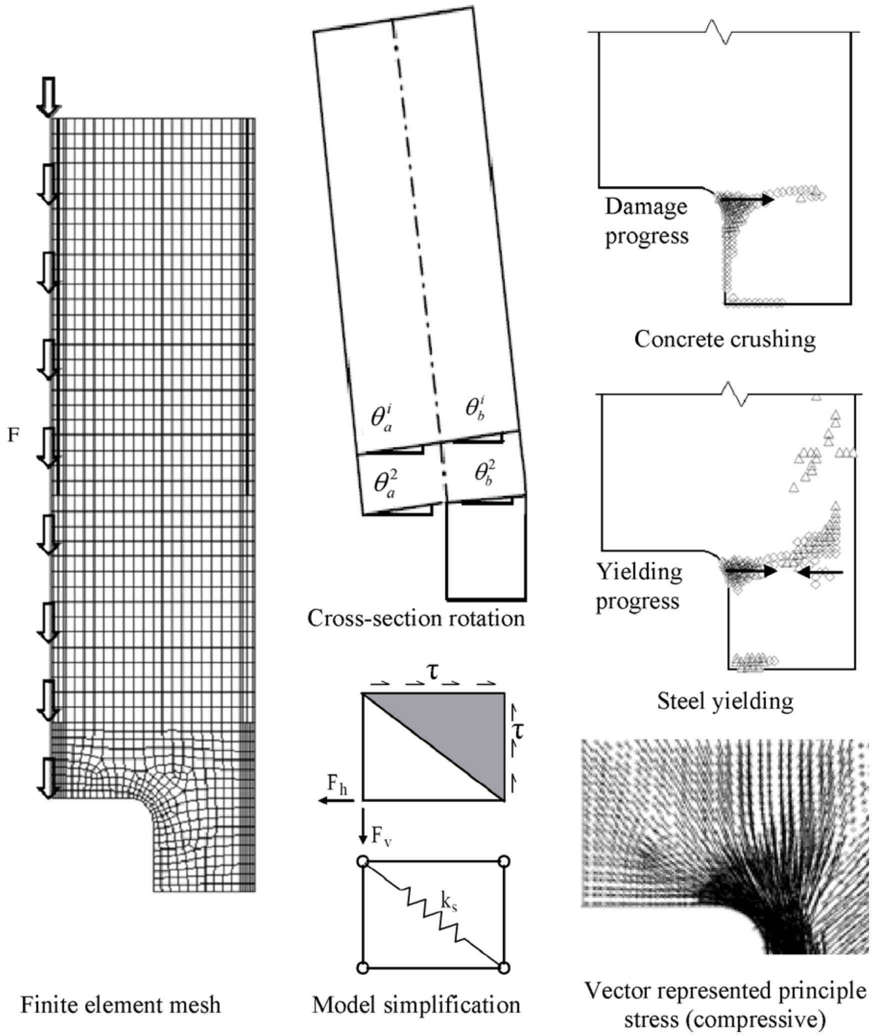


Fig. 2. Finite element analysis and simplified model development.

Table 1
Material properties.

Concrete	f'_c (MPa) −27.6	f'_{cu} (MPa) $0.1f'_c$	E_c (MPa) 2.5E4	f_t (MPa) $-0.1f'_c$
Steel	f_y (MPa) 413.8	E_s (MPa) 2.0E5	E_h (MPa) $0.01E_s$	ϵ_u 0.20

the vertical elements, representing the damaged part of the shear wall, in the intact shear wall model (shown as "Simplified model B" in Fig. 7). The load displacement curve obtained from such a simplification shows a stiffer pre-peak response which is a result of the inaccurate representation of local effects. The proposed model is shown as "Simplified model A" in Fig. 4. Generally, all pushdown curves exhibit a softening post-peak response and a flexural failure at the lowest story is expected when the prescribed displacement continues to increase. The simulated result by using the proposed macromodel shows good agreement with the detailed FE simulation. For the purpose of studying the response of a partially damaged shear wall under gravity loading, the simplified shear wall model is expected to provide a reasonable measure of the overall building behavior.

4. Effect of damage boundaries on wall response

The damaged wall section presented above was represented by a regular rectangular shape. The effect of this simplification as well as the consequence of alternate section profiles is examined in this section. Four damage patterns are considered in the simulations as shown in Fig. 5. Pattern SW-D0 is the assumed pattern for this study. Three additional patterns are introduced to investigate the variation in the force–displacement response due to the assumed damage profile. These three patterns are selected based on keeping the unsupported wall section to be the same as SW-D0 (50% of the original wall). Results of the detailed finite element simulations are presented in Fig. 6. The change of the peak values of the applied vertical loads for the different patterns is insignificant while the corresponding displacements to peak loads are somewhat different.

The location of the critical section of flexural failure was found to vary according to the assumed damage patterns: top of the first story wall in SW-D0; middle of the first story wall in SW-D1 and SW-D2; bottom of the first story wall in SW-D3. Although the locations are different, the effective area of the critical section remains nearly constant (50% of the original wall cross section) for all cases which may explain the small variation on peak loads. These observed variations can be reasonably captured by suitably modifying the proposed model. Two possible simplifications are illustrated in Fig. 7. Simplified model B is the extreme case of Simplified model C. Previously (Fig. 4), simplified model B was demonstrated to not adequately represent damage pattern SW-D0. If sufficient support is provided through the remaining wall fragment (as in SW-D3), model B can be used.

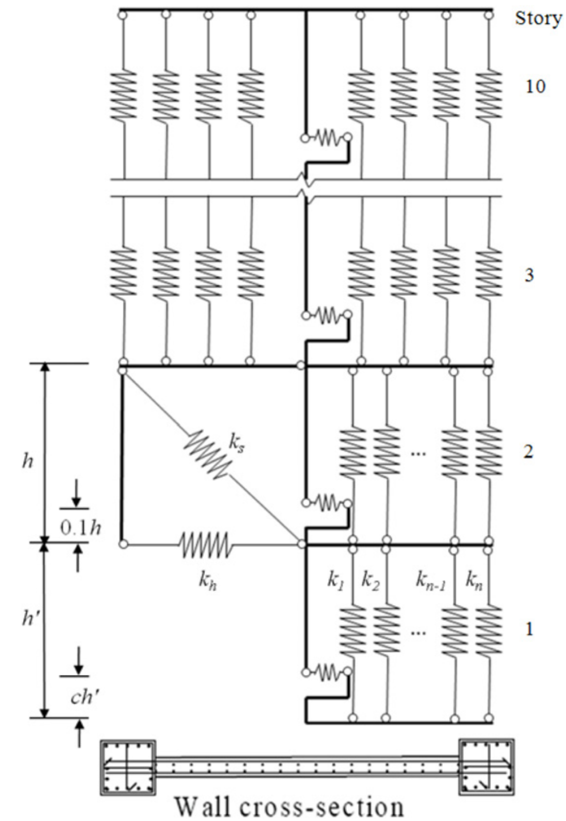


Fig. 3. Simplified damaged shear wall model A.

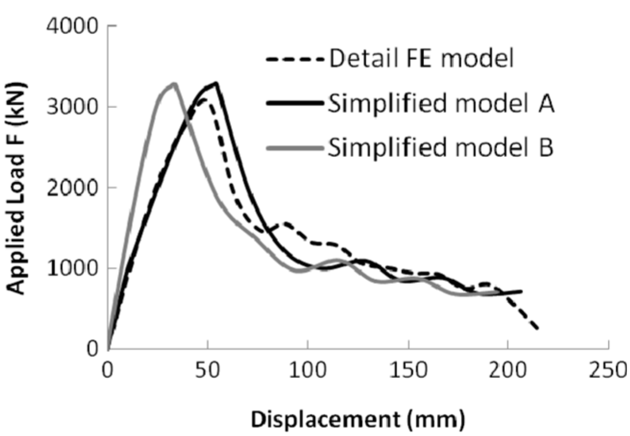


Fig. 4. Load versus displacement: FE model and macromodels.

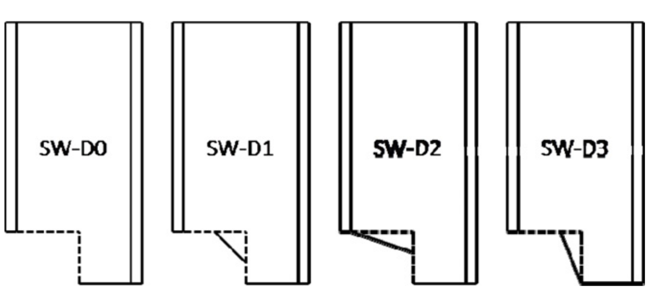


Fig. 5. Different wall damage patterns investigated in the study.

Other intermediate cases can be represented by model C. The resulting load versus displacement curves generated by using each of the simplified models is plotted in Fig. 8. Despite the slight differences in the displacement magnitude at peak force, the overall force–deformation response at the limit stage of collapse is quite similar for all cases.

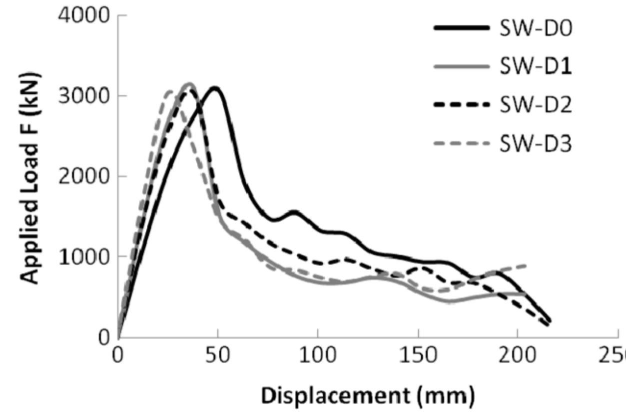


Fig. 6. Load versus displacement: different damage patterns.

4.1. *Irregular versus regular profiles*

Finally, the effect of smoothing the damage boundary is studied. To be consistent with the previous simulations, 50% of the original wall is assumed to be unsupported. Pushdown analysis is carried out on the damaged shear wall with irregular boundary. The irregular boundary is then replaced by a smooth boundary made of piece-wise straight lines. The simulated responses are shown in Fig. 9 together with the responses from SW-D0 and SW-D2. All of load–displacement curves are fairly close to each other suggesting that smoothing process does not affect the response significantly.

5. Response of RC dual systems

Two reinforced concrete dual systems designed by Ghosh and Associates [23] to comply with the requirements of a low to moderate seismic zone (SDC-C) and a high seismic zone (SDC-D) are considered in collapse simulation studies following the loss of 50% of the shear wall at the lowest level of the systems. Building plans and the elevations of the dual systems are shown in Fig. 10, and shear wall reinforcement details are listed in Table 2.

The perimeter frames in Line F of the two building systems are modeled using the macromodels presented in this paper for intact and partially damaged shear walls whereas the beam–columns and the joint regions are modeled using advanced macromodels developed by the authors in another study [19]. Detailed description of beam–column joint model and the nonlinear beam–columns with fiber sections can be found in the paper by Bao et al. [19].

The connection between beams and the edge columns of shear wall are modeled using the beam–column joint model proposed in [19] without shear distortion at the joint panel. Applied loads are calculated from design specifications with a load combination corresponding to dead load plus 25% of live load (Table 3). The process of simulating the sudden loss of part of the lower level shear wall is achieved as follows:

1. Static analysis of the intact system is carried out under the design gravity loads to obtain element forces of the wall section to be removed.
2. The obtained element forces along with the gravity loads are applied to the damaged system (where the shear wall section is removed).

Table 2
Shear wall reinforcement detail.

		Story 1–2	Story 3–5	Story 6–8	Story 9–10
Shear wall (9.804 m × 0.203 m) in building SDC-C	Longitudinal bars in boundary elements	16#10; #4, 3 legs @203 mm	8#10; #4, 2 legs @279 mm	12#7; #4, 2 legs @279 mm	12#7; #4, 2 legs @279 mm
	Vertical bars in web	2 curtains #4 @457 mm	2 curtains #4 @457 mm	2 curtains #4 @457 mm	2 curtains #4 @457 mm
	Horizontal bars in web	2 curtains #4 @203 mm	2 curtains #4 @203 mm	2 curtains #4 @305 mm	2 curtains #4 @457 mm
		Story 1–2	Story 3–5	Story 6–8	Story 9–10
Shear wall (9.957 m × 0.254 m) in building SDC-D	Longitudinal bars in boundary elements	24#10; #4, 7 legs @152 mm	16#10; #4, 5 legs @127 mm	8#10; #4, 3 legs @127 mm	8#10; #4, 3 legs @127 mm
	Vertical bars in web	2 curtains #5 @457 mm	2 curtains #5 @457 mm	2 curtains #5 @457 mm	2 curtains #4 @457 mm
	Horizontal bars in web	2 curtains #5 @305 mm	2 curtains #5 @356 mm	2 curtains #5 @457 mm	2 curtains #4 @457 mm

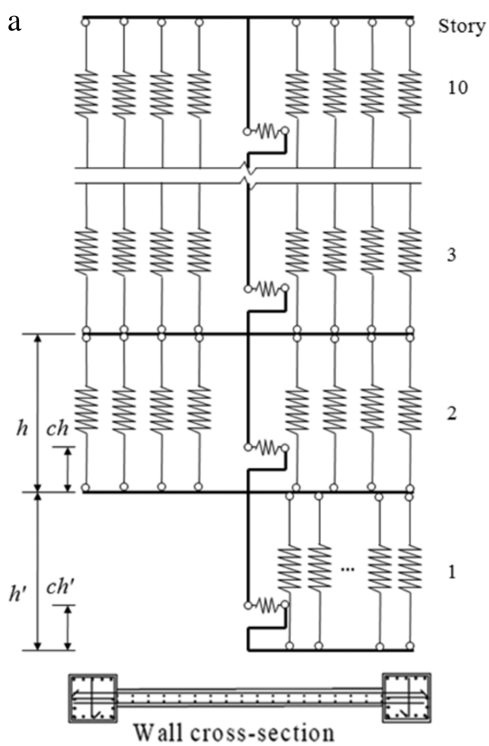


Fig. 7. Simplified damaged shear wall models: (a) Model B and (b) Model C.

Table 3
Loads.

Dead loads		Live loads	
Roof (kN/m^2)	Floor (kN/m^2)	Roof (kN/m^2)	Floor (kN/m^2)
68.97	206.90	172.41	689.66

3. The sudden loss of the wall segment is simulated by applying forces with the same magnitude as the forces calculated in step 2 but in opposite direction during a very short duration (shown as Δt in Fig. 11). The wall damage is therefore simulated as a dynamic load.

Fig. 11 demonstrates the above steps considering the example of removing 50% of the shear wall at the first floor. In all cases, 5% mass-proportional damping is specified in the dynamic analyses carried out in OpenSees [18].

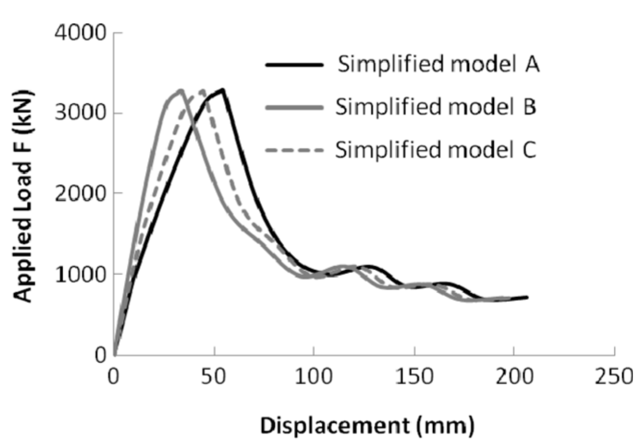


Fig. 8. Load versus displacement: different simplified models.

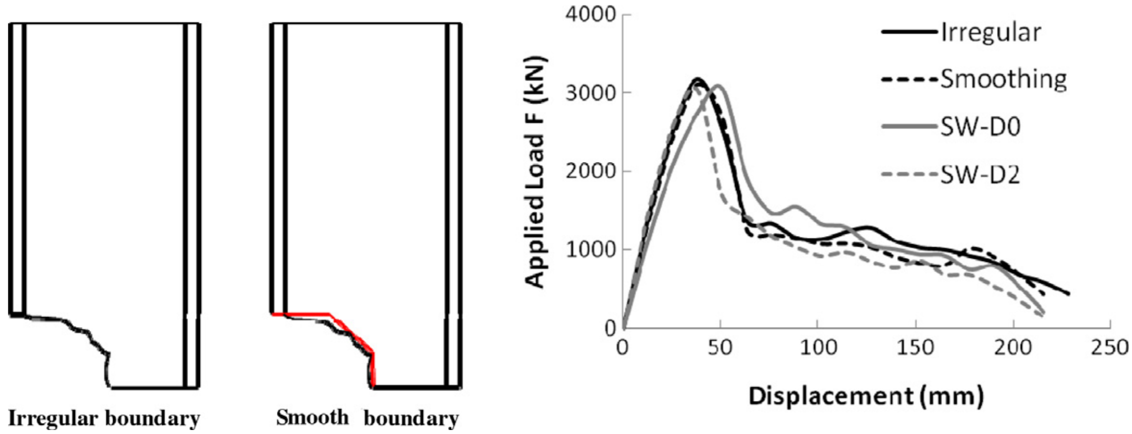


Fig. 9. Effect of damage boundary smoothing.

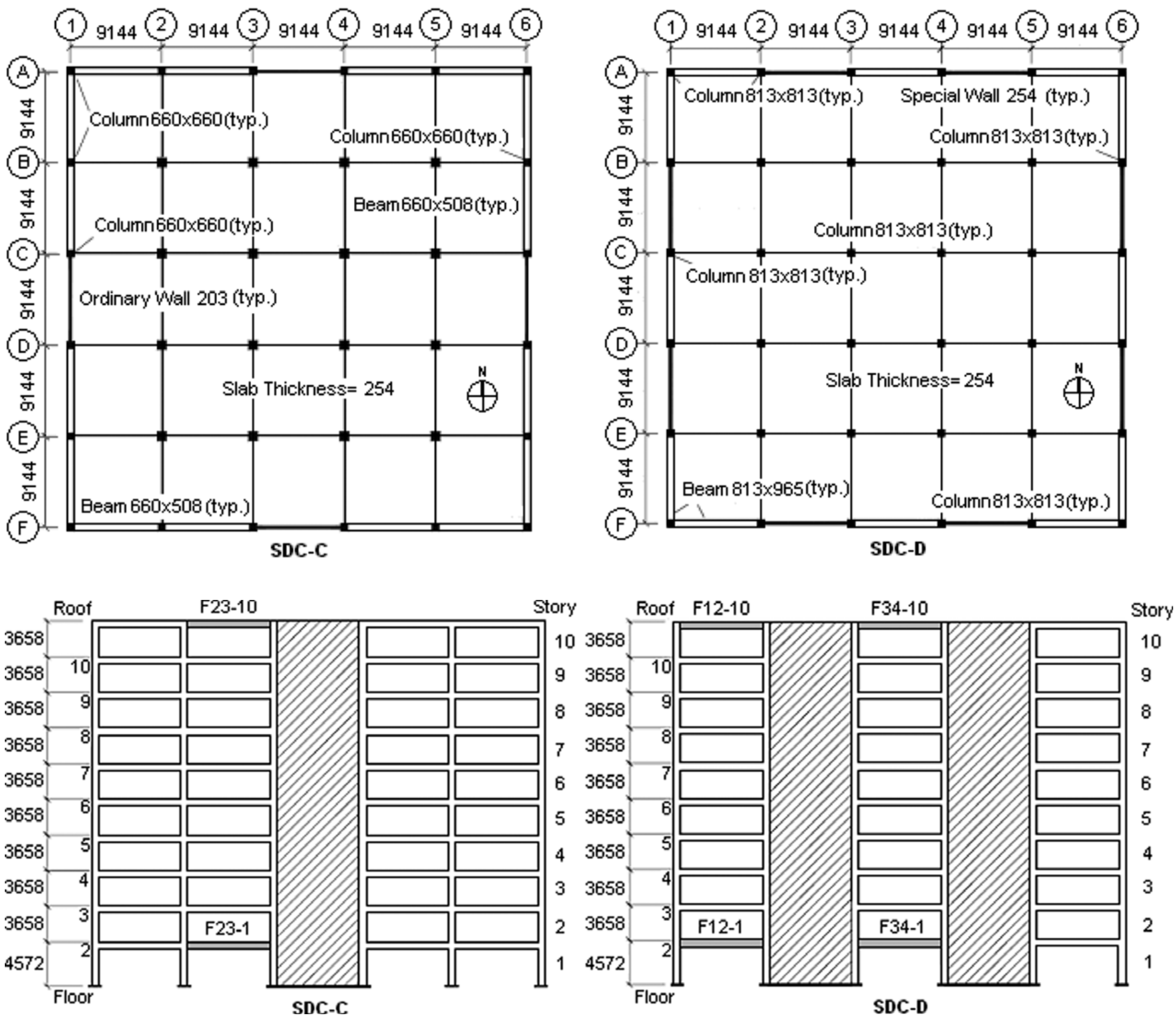


Fig. 10. Plan and elevation of building SDC-C and SDC-D (Unit: mm).

5.1. Response of dual system: SDC-C

The responses of selected elements in the building to sudden loss of half the wall section (including the edge column F3) at the first floor are summarized in Fig. 12. A peak vertical displacement of 11.5 mm is observed at the top of column F3. It damps out quickly to a residual displacement of 6.7 mm. The axial force in column F4 changes from compression (650 kN) to tension (76 kN), while only very small variations in the axial forces is observed in other columns at the first floor. Beam F23-1 (refer to Fig. 10) reaches a peak tensile axial force of 282 kN, then stabilizes to a tension of 160 kN. The rotation of the wall due to the loss of the partial wall block at the bottom tends to push the left-side frame further to the left, therefore compressive axial force is observed in

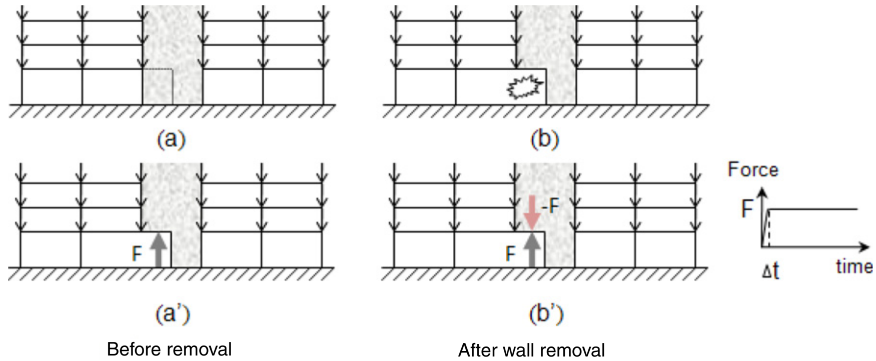
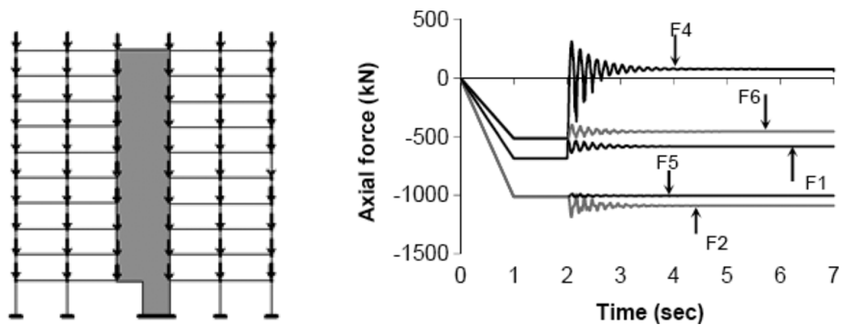
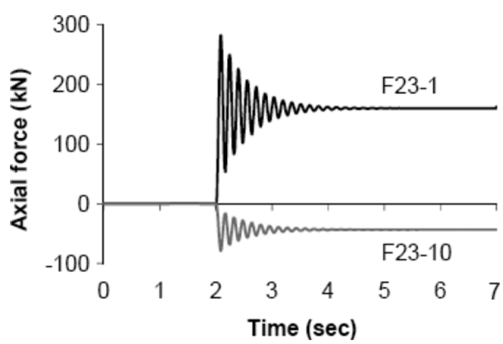


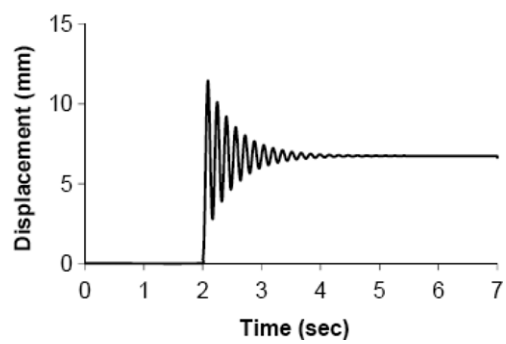
Fig. 11. Loading scheme to simulate member loss.



(a) Axial forces in columns.



(b) Axial forces in beams.



(c) Vertical displacement.

Fig. 12. Response of dual system SDC-C due to removal of partial wall.

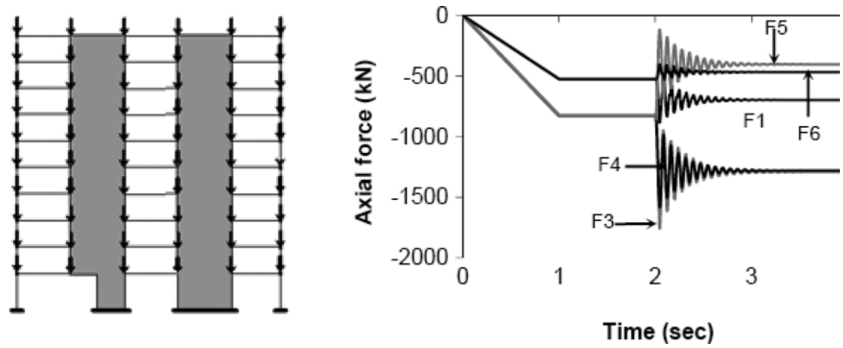
roof beam F23-10. The overall system however remains stable as the dynamic vibrations diminish quickly due to inherent system damping.

5.2. Response of dual system: SDC-D

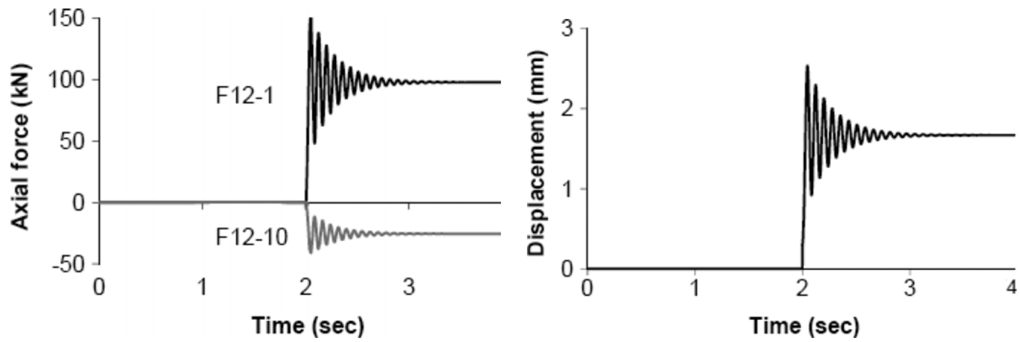
System responses due to sudden removal of 50% of the shear wall section at the first story are summarized in Figs. 13 and 14. In the first scenario (Fig. 13) the left-half of the left wall is removed while in the latter the right-half of the same wall is removed. Very small vertical displacements (between 2 and 3 mm) are observed at the tops of the edge columns that formed part of the original undamaged wall. Under the two different wall removal scenarios, gravity loads that were carried by the lost/damaged portion of the shear wall are redistributed to the remaining part of the wall or the neighboring frame column while the developed overturning moment is sustained primarily by the shear wall which remains intact. Such load transfer and redistribution can be best captured through axial force variations in the first story columns (Figs. 13(a) and 14(a)). Axial forces in the story beams (Figs. 13(b) and 14(b)) along the height exhibit similar responses as observed in the case of SDC-C (Fig. 12(b)). The systems are seen to remain stable under the assumed damage scenarios.

5.3. Comparison between system responses of SDC-C and SDC-D

Although no system instability occurs under the assumed component damage scenarios, it is obvious that frame SDC-D exhibits better performance than frame SDC-C both in terms of residual displacement and axial forces in critical elements. One reason for this observation can be attributed to the better structural layout of frame SDC-D compared with frame SDC-C. Two parallel shear walls are used in frame SDC-D. When one of the two shear walls is damaged, loads and overturning moments can be transferred to the intact shear wall if the two walls are well connected through story beams. When the only available shear wall is damaged (as in the case of frame SDC-C), loads carried by the removed shear wall part are mostly redistributed within the damaged shear wall instead of being transferred to the connected



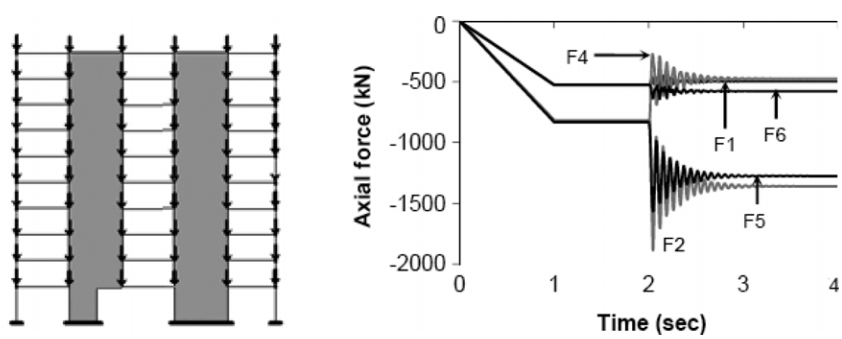
(a) Axial forces in columns.



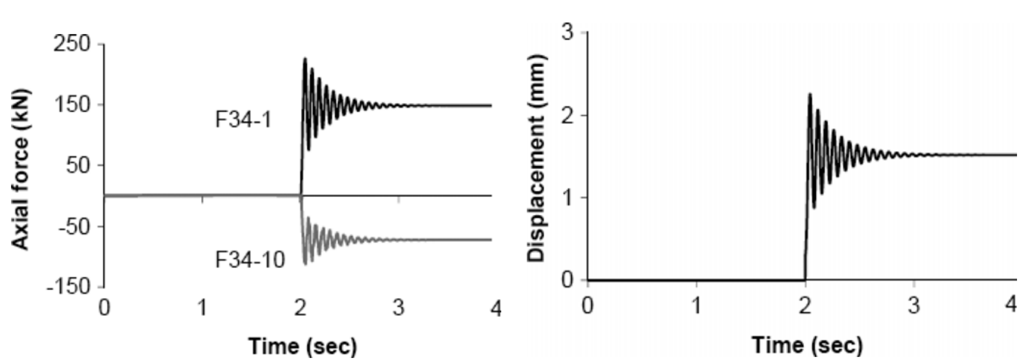
(b) Axial forces in beams.

(c) Vertical displacement.

Fig. 13. Response of dual system SDC-D due to removal of 50% wall (left).



(a) Axial forces in columns.



(b) Axial forces in beams.

(c) Vertical displacement.

Fig. 14. Response of dual system SDC-D due to removal of 50% wall (right).

frame due to the significant difference in stiffness between the shear wall and the connected frame. This increases the risk of system instability. Such instability is bound to amplify if a greater portion of the wall is damaged.

6. Summary of findings and concluding remarks

A macromodel-based approach used for progressive collapse simulation of RC frame–wall systems is presented in this paper. Due to the lack of available experimental studies, detailed finite element analysis is employed in this paper not only to assist developing a simplified shear wall model with a pre-assumed damage pattern, but also as a tool to verify the developed model. By comparing the simulated results of the detailed FE model with the developed macromodel, it is demonstrated that the simplified model has the capability to represent the failure mechanism as well as the local effects in the overall response under the given damage pattern. It is important to point out that the methodology outlined in the paper provides a means to model a damaged wall. Hence the approach can be extended to the development of partially damaged wall models in any general frame–wall system which can then be used in collapse simulations of the overall system.

Two frame–wall systems designed to comply with different seismic requirements (SDC-C and SDC-D) are modeled using the simplified models. Although the simulations show that both systems remain stable under the given component damage scenarios, load redistribution and variation of forces in critical structural members indicate system SDC-D performs better than system SDC-C because of its enhanced seismic design and resulting structural layout. Progressive collapse simulation using simplified analytical models can provide helpful information to determine the possibility of structural instability; however, quantifying the robustness of a certain type of structure requires additional detailed evaluation (with 3D models and slab effects) as well as considerable engineering judgment. The work presented in this paper should be viewed as one of many ongoing efforts contributing to the larger and more complex issues of understanding progressive collapse of building structures.

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