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Coupling Ratio Verification of Direct Displacement Based Design Procedure for RC Coupled Walls

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ABSTRACT

In coupled wall systems energy dissipation is scattered over a more extensive region of the structure because inelastic deformations are occurred not only at the bases of the walls but also at both ends of coupling beams along the elevation of the shear walls. Since the progression of plastic behavior on coupling beams in tall buildings does not occur simultaneously, the degradation of beam capacities along the elevation will also be irregular. Therefore, the coupling ratio is always variable over height and against the lateral drift of the wall system.

In this research study, actual coupling ratio transition against lateral drift of a reinforced concrete coupled wall system is evaluated. The examined wall system selected as a case study that is designed with Direct Displacement Based Design (DDBD) approach. The DDBD procedure suggests assuming a coupling ratio a priori as the first step of the design procedure. The real coupling ratio over the height of the structure has been investigated and compared with the initially assumed coupling ratio. The nonlinear behaviour of the system is then examined by a simple pushover analysis. The conclusions obtained with the numerical verification are summarized and comparisons are made between the observations and the DDBD assumptions.

1. INTRODUCTION

With the swift urbanization process, a rapid growth in the number construction of tall structures has taken place. Reinforced concrete walls are common lateral load resisting elements in multi-storey buildings since they are generally preferred conventionally to resist seismic demands. Coupled structural walls are frequently

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preferred in multistory reinforced concrete structural designs because they maintain effective mechanisms for resisting earthquake loads. By combining the adequate lateral stiffness of shear walls with properly proportioned coupling beams that can provide the most of the energy-dissipative mechanism during the response to the earthquake motions, an exceptional behavior of structural system can be achieved. Since plastic hinges are intended to form not only at the base of the walls but also even before those they would form also at both ends of coupling beams, such that energy dissipation is distributed over a wide-ranging region of the structure with the result having more ductile behavior than is the case with cantilever walls. On the other hand, careful design of the beams is essential to achieve the desired degree of coupling and the level and the order of plastic hinge formation for required energy dissipation. The degree of coupling of the system significantly affects the structural system response against lateral forces (Harries, 2000).

Several experimental researches have been carried out to evaluate the load-deformation response of coupling beams [(Paulay, 1971), (Paulay and Binney, 1974), (Barney et al., 1980), (Tassios et al., 1996), (Xiao et al., 1999), (Galano and Vignoli, 2000), (Kwan and Zhao, 2001), (Fortney, 2005)]. Major variables in these studies were the ratio of the beam clear span to the beam depth (commonly referred to as the beam aspect ratio) and the beam rebar layout. In a majority of these studies the load-deformation behavior of coupling beams with low-aspect ratio (1.0 to 1.5) constructed with longitudinal reinforcement were compared with beams constructed with diagonal reinforcement. Concrete compressive capacities for most experimental results were differ from 25 MPa to 30 MPa. Although these tests provided valuable information, they do not address the issues for current tall building constructions, where beam aspect ratios are typically between 2.0 and 3.5 and concrete strengths are in the range of 40 MPa to 55 MPa (Naish, 2010).

The use of diagonal reinforcement in coupling beams with aspect ratio (clear length to total depth) less than four was introduced into ACI 318-99. Two bundles of diagonal rebar are placed such that they intersect at the center of the beam (Fig. 1). These two groups of diagonal bars and the concrete they cover, are counted as a truss, with one group serving as the tension member and the other as the compression member. To increase the compression and deformation capacities of the diagonal truss members as well as to prevent buckling of the diagonal bars, use of transverse reinforcement around the diagonal bar groups is required (ACI 318-99).

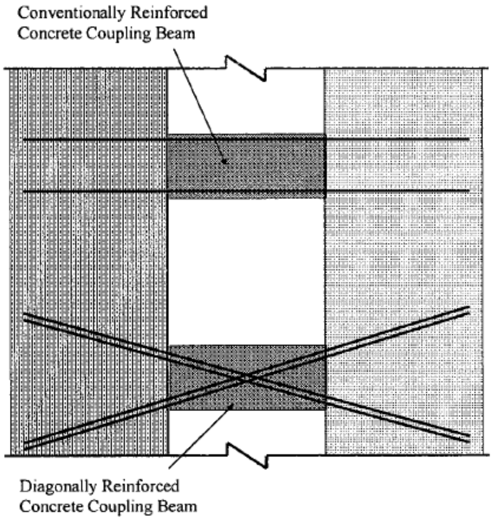
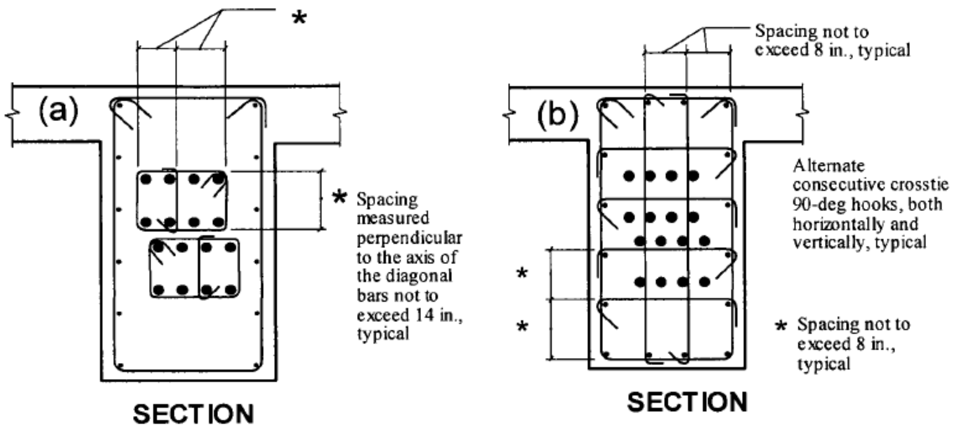


Fig. 1. Rebar pattern for conventional and diagonal reinforcement coupling beams.

On the other hand, providing transverse reinforcement around the diagonal bar bundles as detailed in ACI 318-05 S21.7.7 (Fig. 2a) brings a dilemma with regards to constructability. Placing required hoops around the diagonal rebar groups is difficult where they intersect at mid-span, specifically for shallow beams (aspect ratio greater than 2.5), for which the intersection of the bars is much longer. As well, it is also very difficult to place hoops around the diagonal bundles at the beam-wall interface, particularly for deep beams (aspect ratio less than 2.0) due to interference with the wall boundary vertical reinforcement. To avoid these construction difficulties, ties or hoops are preferred to be placed around the entire intersection region, rather than each bundle individually; on the other hand, it is undetermined whether if the modified detailing meets the acceptance criteria of the code or not.



(a) (b)

Fig. 2. Confinement alternatives provided in ACI 318-05 and ACI 318-08.

To overcome these issues, ACI 318-08 S21.9.7 introduced an alternative detail, where transverse reinforcement is placed around the entire coupling beam cross section to provide confinement and prevent buckling, and no stirrup reinforcement is provided directly around the diagonal bundles (Fig.2b). Use of this detailing option avoids the problems mentioned above where the diagonal bars intersect and at beam-wall interface, reducing the construction time for a typical floor by a day or two. While the volumetric ratio of steel used for this detail may increase, but the overall cost would be lower due to the construction time.

The procedures for the design of diagonally-reinforced concrete coupling beams are given in ACI 318-08 S21.9.7. Specifically, coupling beams with aspect ratio less than 2.0 and expected shear stress greater than $0.35\sqrt{f_c' MPa}$ must be reinforced with diagonally-placed bars. The strength of beams with diagonal reinforcement is determined by ACI 318-08 Equation 21-9, as in the following

$$V_n = 2A_{vd} f_y \sin\alpha \leq 0.85\sqrt{f'_c} A_{cw} \tag{1}$$

where A_{vd} and A_{cw} are the diagonal bundle rebar area and coupling beam gross cross-section area, respectively. f_y and f'_c represents reinforcement steel yield and concrete compression strengths.

2. DIRECT DISPLACEMENT-BASED DESIGN FOR RC COUPLED WALLS

The method titled as Direct Displacement-Based Design (DDBD) has been used and developed over the past fifteen years in order to reduce the inadequacies and uncertainties in force-based design (Priestley et al, 2007). The fact behind the approach is to design a structure which can achieve a required performance limit state under a given seismic demand. The procedure determines the capacities required at designated plastic regions to acquire the design deformation objectives.

The methodology simulates a single-degree-of-freedom representation and it is applicable to all structural types, including coupled structural walls. The bi-linear behaviour of the lateral force-displacement response of the single-degree-of-freedom system is shown in Fig. 3. An initial elastic stiffness K_i is followed by a post yield stiffness of rK_i. While the force-based seismic design characterizes a structure in terms of elastic, pre-yield properties, DDBD method evaluate the structure by secant stiffness K_e at the maximum displacement Δ_d.

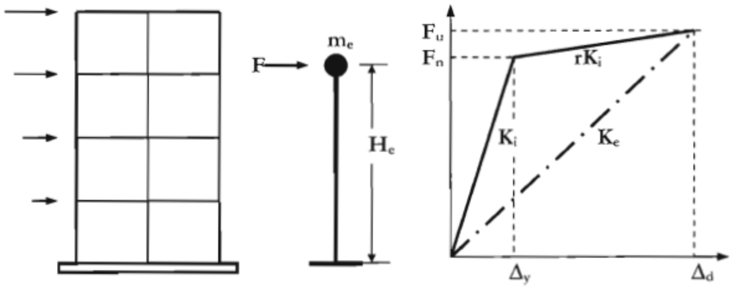


Fig. 3. DDBD Single-DOF representation (Priestley et al., 2007)

With the design displacement at maximum response determined, the effective period T_e at maximum displacement response, measured at the effective height H_e can be determined from a set of displacement spectra for different levels of damping. The effective stiffness K_e of the equivalent single-degree-of-freedom system at maximum displacement can be found by inverting the normal equation for the period of a single-degree-of-freedom oscillator, given by Eqn. 2 as follows

$$K_e = 4\pi^2 m_e / T_e^2 \quad (2)$$

where m_e is the effective mass of the structure participating in the fundamental mode of vibration. From Fig. 3, the design lateral force, which is also the design base shear force, can be calculated by Eqn. 3

$$F = V_{base} = K_e \Delta_d \quad (3)$$

In coupled walls, the transfer of shear forces through the coupling beams introduce tensile and compressive forces in each wall pier, triggering a couple moment which resists a portion of the total overturning moment. The proportion of system overturning moment resisted by the coupling action is defined as the coupling ratio.

Previously, the coupling ratio percentage has been calculated with initial stiffness force-based design based on elastic analysis and the recommended range of coupling ratio is between 25% and 75% (Paulay, 2002). On the other hand, further studies has revealed that elastic analysis is not sufficient to provide an accurate information for evaluating the distribution of forces in coupled walls and coupling beams.

3. MULTIPLE VERTICAL LINE ELEMENT AS MACRO MODEL

Post-elastic reaction of RC wall systems can be simulated by using either micro models based on a comprehensive interpretation of the local response, or by using phenomenological macro models which takes considers entire behaviour within acceptable range. Despite the complexity of micro models, macro models are practical and efficient, although their application is restricted based on the simplifying assumptions upon which the model is based.

Usage of column beam line element at the wall centroid axis is a well-known modelling approach. In this case, an equivalent column is used to model the properties of the wall, and beams with high stiffness are bound to the column at each floor level. The rotations of a beam-column element always develop about the centroid axis of the element; therefore, the shifting of neutral axis along wall cross-section during lateral loading and unloading is not taken into account. As a result, over-turning of the wall and its interaction with any connecting elements, both in the plane of the wall and out-of-plane of the wall, cannot be adequately considered.

Previous extensive research results relieve that the multiple-vertical-line element model (MVLEM) proposed by Vulcano et al. (1988) has been able to successfully balance the simplicity of a macroscopic model and the refinements of a microscopic model. The MVLEM captures essential response characteristics (e.g., shifting of neutral axis, and the effect of a fluctuating axial force on strength and stiffness), which are commonly ignored in simple models, and offers the flexibility to incorporate refined material constitutive models and important response features (e.g., confinement, progressive gap closure and non-linear shear behaviour).

The model in Fig. 4a is an illustration of the typical two-dimensional MVLEM wall element. The flexural response is simulated by a series of uniaxial elements connected to rigid beams at the top and bottom floor levels. The two external fibres (k_1 and k_n) embody the axial stiffness of the boundary columns, while the interior elements represent the axial and flexural stiffness of the central panel. The force-deformation behaviour back-bone plot of vertical fibres is summarized in Fig. 4b (Orakcal et al., 2006)

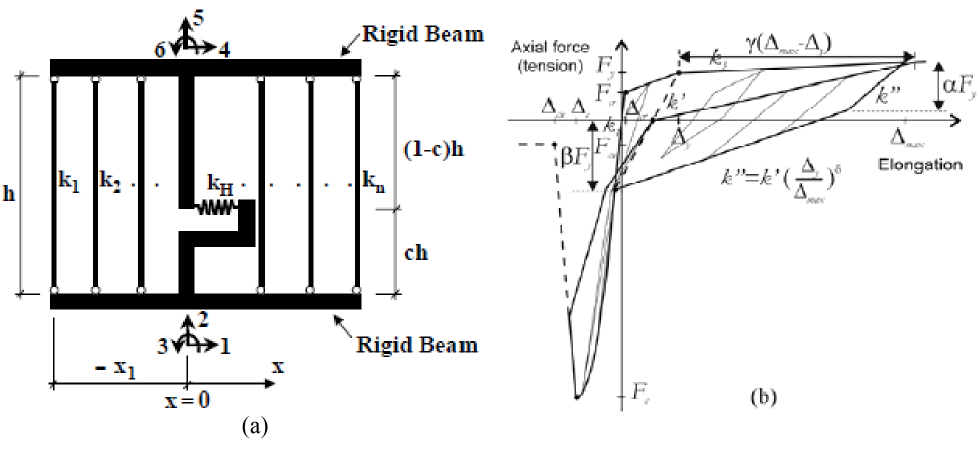


Fig. 4. Multiple Vertical Line Element Model (MVLEM) (Orakcal et al., 2006)

Together with the idealized flexural response, the shear model was used to envelope the three possible failure probabilities for coupling beams under lateral loads. The shear capacity of a given element is not constant through the deformation cycles.

Extensive tests (Miranda et al., 2005) have proven that the shear capacity is function of the ductility, something that renders the shear force-deformation behaviour of an RC member tri-linear, as shown in Fig.5.

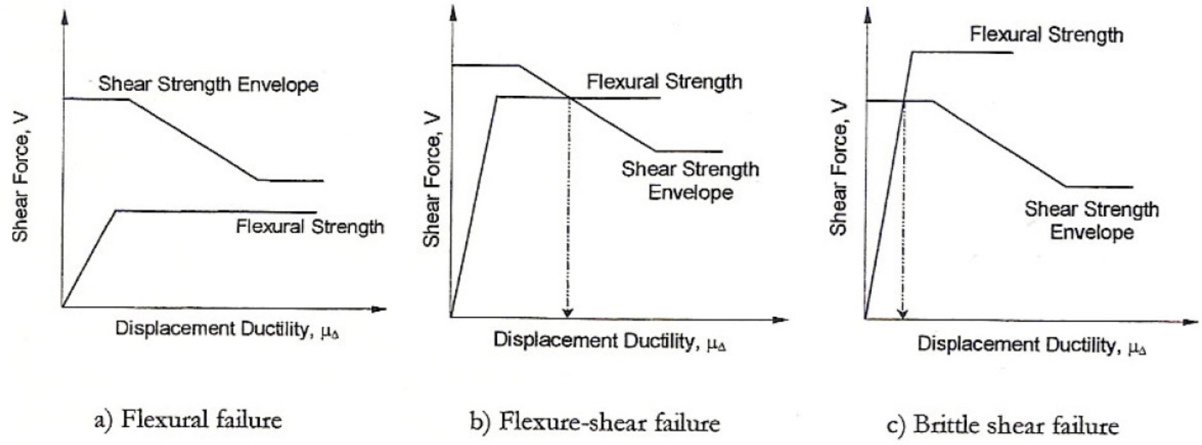


Fig. 5. Coupling Beam Failure Modes (Miranda et al., 2005)

Flexure failure takes place if the shear force corresponding to the nominal flexural strength is less than the shear capacity for any value of ductility. A typical flexural failure situation is shown in Fig. 5a. A flexure-shear failure occurs when the coupling beam reaches its nominal flexural capacity first, but as ductility increases the corresponding shear force exceeds the shear strength envelope. Shear failure is triggered at the point where the shear resistance goes below the flexural resistance. This situation is presented graphically in Fig. 5b. Finally, Fig. 5c. shows a brittle shear failure, which occurs when the shear capacity of the coupling beam is reached prior to the development of the flexural strength.

4. NUMERICAL ANALYSIS (CASE STUDY)

The examined coupled wall system is selected as a case study that is designed with Direct Displacement Based Design (DDBD) approach. The DDBD procedure suggests assuming a coupling ratio a priori as the first step of the design procedure. The actual coupling ratio of the wall system has been investigated and compared with the initially assumed coupling ratio. The non-linear behaviour of the system is then examined by a simple pushover analysis with uniform and triangular loading patterns.

The sample coupled wall system consists of two identical 8 storey walls of 4.00m x 0.30m cross-sections connected with 2.00m long and 1.00m deep beams (aspect ratio = 2) at each floor level. 3D representation of the model is shown in Fig 6.

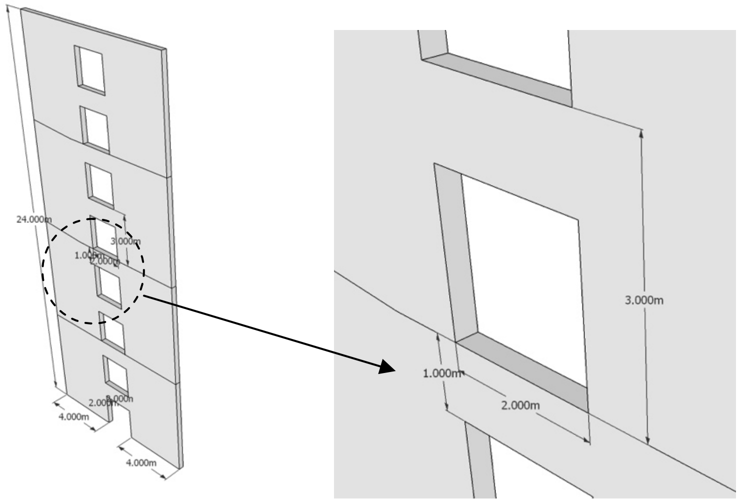


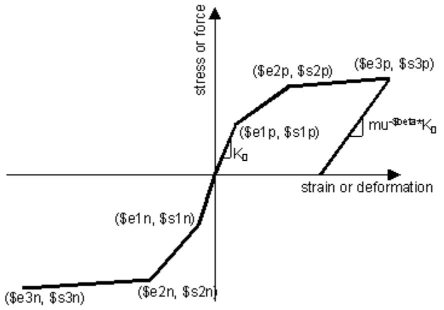
Fig. 6. Evaluated coupled wall system

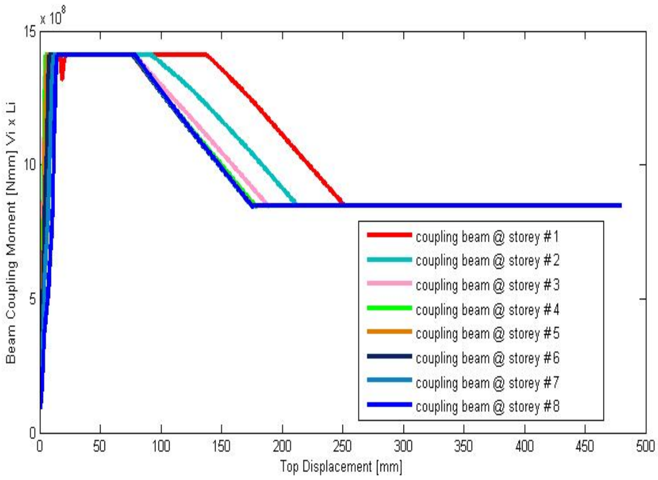
Direct Displacement-Based Design procedure starting with an initial coupling ratio assumption of 40% results with beam design shear strength of 235kN. The nonlinear shear-deformation behaviour is modelled in OpenSees software (OpenSees, 2012) by using hysteretic model. The default properties of hysteretic force-deformation model are summarised in Fig. 7.

It should be noted that, for the sake of simplification and abiding by the modern design concepts, the shear failure is omitted in the behaviour of the RC walls used in the case study. This is an issue that requires more work since coupling flexural and shear behaviours in RC walls is an open research topic that attracts much experimental and analytical work.

In order to adopt shear strength envelope given in Fig. 5a, pinching effect is disabled from the default hysteretic back-bone curve. Additionally, post-yielding stiffness represents 40% of strength degradation. Resultant coupling moments (Vi x Li) for each individual story beam is summarized in Fig. 7. It is observed that coupling beams located on lower stories have strength degradation later than the ones on upper floors.

Structural wall system coupling ratio analysis result is plotted on Fig. 7. The graph shows that actual coupling ratio starts with approximately 70% under elastic behaviour region. Then, it rapidly decreases down to 40% (DDBD assumption) at 100mm displacement. The actual coupling is 25% lower than the assumed DDBD coupling ratio under design displacement value of 296mm.





(a) **(b)**

Fig. 7. The Hysteretic Model in OpenSees (left) and the beam moment distribution (right)

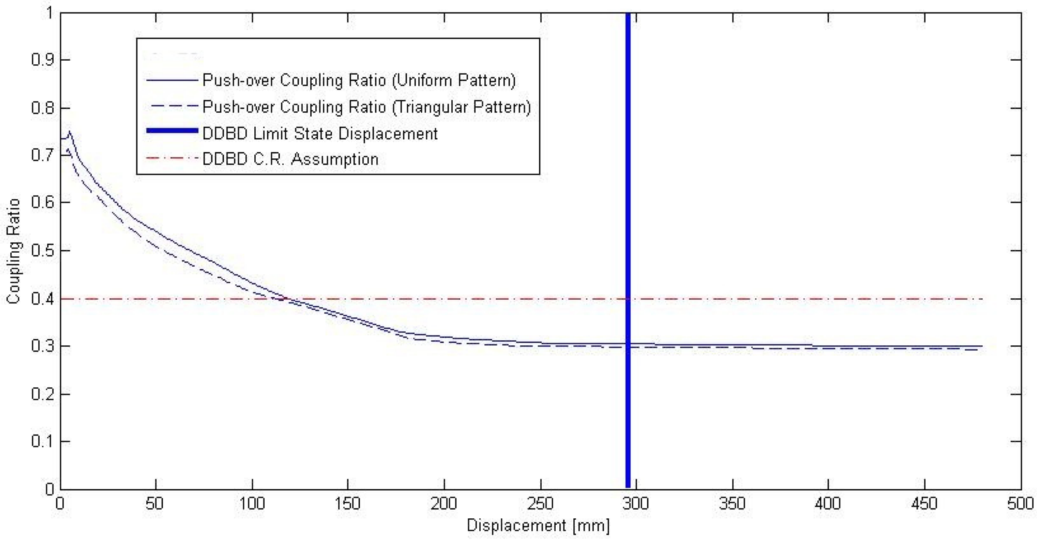


Fig. 8. Coupling ratio vs. top displacement

The plot in Fig. 8 suggests that the coupling ratio of the beams, their contribution in other words, is much higher than the assumed initial design ratio, which was 40%. The increasing displacement demand on the structures, thus on these beams, lead to a drastic drop in the coupling ratio of these beams. For the given example, the coupling ratio, which is the most basic design parameter and assumption, drops below the initially assumed value before the design displacement is reached.

The findings in Fig. 8 exhibit a variable behaviour of the beam-coupling ratio over the increasing deformations, which is an important issue that has to be considered during design maybe by implementing different coupling ratios in different limit states and it requires further research.

CONCLUSION

Reinforced concrete coupled wall systems are frequently used in medium and high-rise building construction. The coupling action is beneficial, because it reduces the moments to be resisted by individual walls. Secondly, the plastic deformation (energy dissipation) is extended along the height of the wall rather than being only at the base of cantilever walls. The selection of analysis method to be used for designing of coupled wall systems is very important. Since the progression of plastic behaviour on coupling beams in tall buildings does not occur simultaneously along elevation, the coupling ratio of the coupled wall system cannot be expected as constant during the whole process of ground motion.

In this research study, an 8-story coupled wall system which is designed with Direct Displacement Based Design (DDBD) approach is used as case study. A desired coupling ratio in DDBD procedure is assumed initially as the first step of the design. Then, the actual coupling ratio of the wall system has been investigated and compared with the initially assumed coupling ratio. The non-linear behaviour of the system is also examined by a simple pushover analysis with uniform and triangular loading patterns. The conclusions obtained with the evaluations of the numerical solutions are summarized below:

1. The initial coupling ratio (before coupling beams undergo plastic) is over two times than the coupling ratio value at design displacement. Therefore, analysing coupled wall systems under linear-elastic procedures would give unrealistic results.
2. The coupling ratio alteration is not significantly affected by the type of the lateral load pattern (triangular or uniform) for coupled wall systems under 8 storeys.
3. The actual coupling ratio at design displacement is 25% lower than the DDBD-assumed coupling ratio value for the case study examined herein. The number of samples must be increased in order to further validate this finding.
4. The coupling beams at higher stories experience strength degradation initially. The coupling beams at the first floor reach strength degradation significantly later than all other beams.
5. Further investigation should be performed for examining the effect of the coupling ratio going below the initially assumed value, in terms of its effects on the validity of the design.

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