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Evaluation of the response modification factor of RC structures constructed with bubble deck system

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Bubble deck;
Response modification factor;
Nonlinear static analysis;
Seismic behavior;
Reinforced concrete.

Abstract. Since the concrete material has been eliminated from the locations situated around the middle of the cross-sections of Bubble Decks (BDs), the BD-type slabs are lighter than the traditional slabs. In the recent researches, Response Modification Factor (RMF) is generally determined for Reinforced Concrete (RC) structures with Moment-Resisting Frame (MRF) and dual systems. The dual system mainly comprises MRF with Shear Wall (MRFSW) and flat slab chiefly with BD system. In this study, evaluation of the RMF values of RC structures using BD system was the main concern. The obtained results indicated that lateral strength of buildings increased by increasing the span length to story height ratio (L/H). Besides, the variations of span length and the number of stories had more significant effects than the variation of usage category of building on the RMFs of structures. Furthermore, the effect of span length was greater than that of the number of stories in determining RMF of an MRF. Finally, amongst the buildings with dual system structures incorporating MRFSW, the low-rise building structures had an RMF equal to 5 and both the mid-rise and high-rise building structures had an RMF of 7.

1. Introduction

The use of innovative structural systems has increased in today's world. As an instance of these structures, Bubble Deck (BD) system can be noted. The behavior of this type of structural systems is like the behavior of a light-weight two-way slab. Plastic Spherical Hollow Cores (PSHCs) are used in the BD system instead of concrete. These PSHCs are mainly used in the central zone of the cross sections around the mid-span of the slabs, where the shear stress is relatively small in comparison with the supports. PSHCs create hollow spaces in the slabs. In the BDs, recycled plastic-made spheres are used to create air voids and provide strength through the arch action. There are three-dimensional voids inside BD slabs in two horizontal directions, which decrease self-weight of the slab. Bubble diameter to slab thickness ratio affects the behavior of a BD slab (see Figure 1).

In the middle of spans, where the plastic balls are located, the design is controlled by flexural and direct shear stresses. On the other hand, in the supports, the solid deck (without PSHC) is used and the design is controlled by the punching shear stress. Shear reinforcements may be used to prevent the slab from shear failure (if required) (see Figure 2).

The most important advantage of using BD sys-

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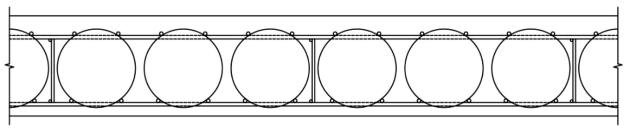


Figure 1. Scheme of the cross-section of a Bubble Deck (BD) slab.

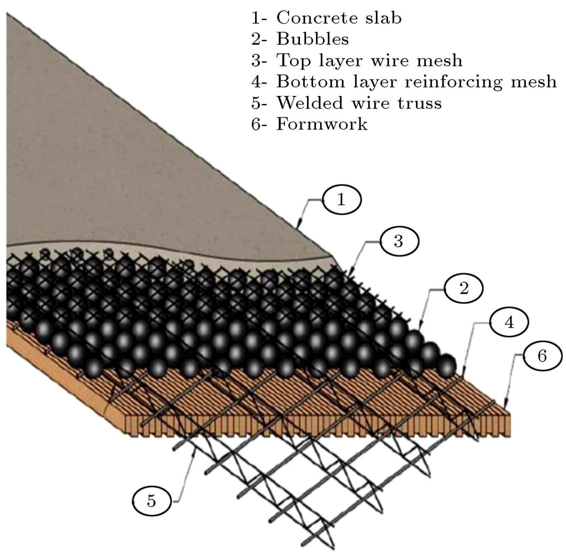


Figure 2. A view of a Bubble Deck (BD) slab.

tem is the reduction in the concrete and steel quantities required for the construction of a building. This reduction affects the weights of both the slabs and the whole structure, in turn reducing the earthquake forces and the cost of construction. Reduction in working time and costs, increase in the span lengths, reduction in the number of columns, improving the architectural plans and drawings, and providing more spaces can be pointed out as further advantages of using BD system.

The shear strength of the BD slabs has been studied by some researchers. Aldejohann and Schnellenbach-Held [1] as well as Schnellenbach-Held and Pfeffer [2] tested the shear strength of BDs experimentally and investigated the effect of voids on the punching shear. Chung et al. [3] investigated the shear strength of the hollow slab using the donut type hollow sphere. The results of this investigation showed that material and shape of the hollow spheres would affect the shear strength of the slabs. Bindea et al. [4] investigated the shear strength of BD and proposed a formula for controlling it in this type of slabs. Bindea et al. [5] also performed experimental tests to find the shear strength of DB slabs. Their obtained results showed that in the studied BD slabs, the ultimate shear force was around 97% of that in solid slabs with the same thicknesses.

The punching shear and flexural capacities of the BD system were theoretically studied by some researchers such as Schmidt et al. (1993) [6], Schnellenbach-Held and Pfeffer [2], Schnellenbach-Held et al. [7,8], and Gudmand-Hoyer [9]. Based on the performed bending tests, the BD slabs have greater ultimate flexural strength than the theoretically considered values for solid slabs. Also, the effective value of shear strength of a BD slab is at least about 70% of the shear strength of a solid slab with the same thickness. Lai [10] analyzed the behavior of BD slabs and recommended using them in the light-weight bridge decks. An experimental program dealing with concrete slabs with spherical voids for a full-scale test was presented by Calin and Asavoaie [11]. They examined the deformation, cracking, and failing characteristics of slabs subjected to static gravitational loadings. Teja et al. [12] discussed various properties of BD slab based on various studies. The results of this research indicated that flexural strength of the BD slabs was about 6% lower than that of the solid slabs. Deflections of the BD slabs were about 6% greater than those of the solid slabs with the same size despite the fact that stiffness was reduced due to the effect of hollow parts in this type of slabs. Based on their investigations, shear strength of BD slabs was about 60% of that the solid slabs with the same thickness. However, the required shear strength could be achieved by providing transverse reinforcement. The weight reduction was about 35% compared to the solid slab. Based on experimental investigations carried out in laboratories, Terec and Terec [13] stated that, with the same quantities of used concrete and reinforcement as in a solid slab, the BD configuration and performance allowed obtaining an improved ultimate flexural strength and stiffness while the shear strength of the BD slabs would be reduced to about 70% that of a solid slab, realizing 30–50% concrete economy in comparison with the solid slab. Churakov [14] studied different types of hollow slabs. Dowell and Smith [15], Olsen [16], Calin and Asavoaie [17], and Gajen [18] investigated the BD slabs and compared their design regulations, stiffness, deformation, and shear strength with those of solid decks through numerical modeling or experimental tests. Some other novel techniques have been adopted to improve the strength of two-way slabs. Behzard et al. [19] carried out an experimental program to investigate the effectiveness of a novel near surface mounted technique using innovative manually made Carbon Fiber Reinforced Polymer (CFRP) rods and manually made CFRP strips for flexural strengthening of Reinforced Concrete (RC) two-way slabs with low clear cover thickness. Four full-scale RC slabs were tested under monotonic four-point bending. The behavior of slabs strengthened by this technique was compared to the behavior of a slab strengthened with Glass Fiber Reinforced Polymer (GFRP) rods. The test results confirmed the feasibility and efficacy of this technique in improving the flexural behavior of RC two-way slabs. Li et al. [20] proposed a novel tuned rolling mass damper, embedded in voided biaxial RC slabs, to act as an ensemble passive damping device mitigating structural response. The promising control efficacy observed in the analysis confirmed the potential application of their proposed control device.

In general, the BD slabs behave similarly to the flat slabs. Consequently, the RC structures constructed with BD slabs allow a significant reduction in heights of stories and a great flexibility in architectural plan design compared to the conventional MRF structures. Their casting work is simple and offers superior constructability. The existing design codes of practice [21,22] allow employing flat slabs in the low to moderate seismic risk zones as a lateral force-resisting structural member. The flat slabs are normally used together with the lateral force-resisting structural members like shear walls or Moment-Resisting Frame (MRF) structures [23]. Mohammad et al. [24] studied two 3-D RC framed structures designed according to ACI-318-14 and IS 2800-14 codes employing Linear Response History Analysis (LRHA) as well as Non-linear Response History Analysis (NRHA) under an ensemble of 11 near-fault ground motions. The obtained results revealed that the design spectrum of IS 2800-14 was incompatible with near-fault spectra and underestimated demands in long-period ranges. They also found that the implementation of LRHA using Response Modification Factor (RMF) and deflection amplification factor would lead to insufficient inter-story drift ratios. Akbarzadeh Bengar and Mohammadalipour Aski [25] studied the influence of an increase in building height on the nonlinear seismic behavior of dual structural systems in the form of RC frames accompanied by RC coupled shear walls. Their experiments were carried out once with concrete and then with steel coupling beam by modeling 7-, 14-, and 21-story buildings containing RC coupled wall systems with concrete and steel coupling beams under pushover analysis with different load patterns. Some seismic parameters, such as ductility factor, RMF due to ductility, over-strength factor, RMF (R), and displacement amplification factor (Cd), were studied. The obtained results indicated that the RMF values for the mentioned structural system were higher than the values used in codes of practice for seismic-resistant design of buildings. In addition, the displacement amplification factor and the RMF increased as the height of the structure decreased and the values of these factors in steel coupling beam structures were higher than those in concrete coupling beams. Hashemi et al. [26] studied the ductility of RC structures constructed by BD system. The existing studies carried out on the BDs, principally in the small-scale slab models, have been concentrated chiefly on the shear, flexural, and punching shear ultimate strength

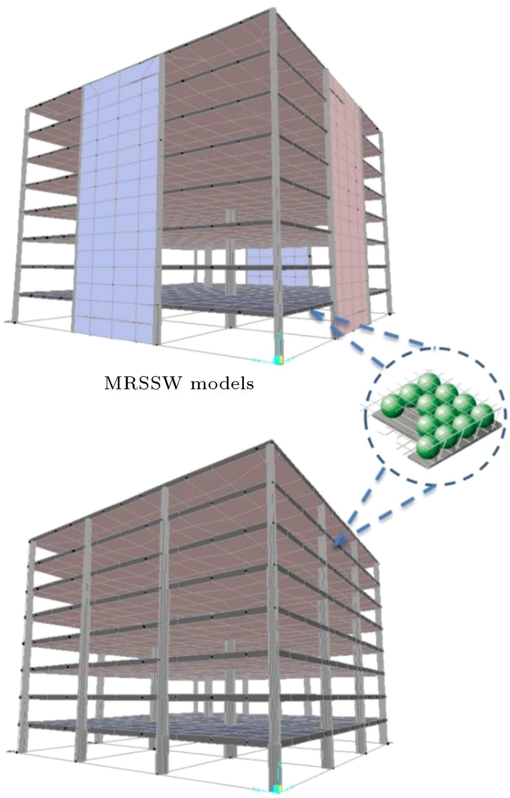


Figure 3. Two views of the lateral resisting models used in this research.

and the lateral strength and seismic behavior of RC structures with BDs have not been studied adequately. Moreover, the structural codes of practice have not proposed clear regulations for determining the RMF for these type of structures [27,28]. In many texts, RMF is called R factor. Therefore, further studies and investigations are necessary for evaluation of the seismic response of full-scale RC structures constructed with BDs. Accordingly, the results of investigations performed into RMF (R) of the MRF with BD slabs along with the influences of the span length to story height ratio (L/H) and the number of stories are submitted in this paper. Along with the MRFs, the dual systems including the combination of MRF and Shear Wall (MRFSW) have also been evaluated (see Figure 3).

In this research, nonlinear static analysis is adopted to determine the RMF. This type of analysis approach is selected because it is fast and has acceptable accuracy in the analysis of the structures with short natural periods [29]. This method of analysis simulates the behavior of structures numerically by computing strength and the related deformation taking into account the design earthquake specifications. In this method, static lateral load is gradually applied to the structure and increased until the displacement of control point reaches a target quantity [30].

2. Basis of the research

2.1. The method adopted to compute the RMF

It is observed that in the case of strong earthquakes, most of the structures have nonlinear behavior. Similarly to the linear responses, the nonlinear responses are controllable. In other words, the length of the horizontal plateau of the base shear-displacement curve, when some methods are used, significantly increases. By applying some specific measures, taken in the design process of hinge composition, the horizontal plateau of the pushover curve, starting with the formation of the first plastic hinge and continuing up to the collapse mechanism, can be enhanced. This means that some measures can be taken in a way that the initial plastic hinge remains safe during the formation of the next plastic hinge and it is not crashed. This is the main philosophical point of the seismic design of structures. In Figure 4, the overall response of a structure with one degree of freedom is depicted in the form of base shear-horizontal displacement curve. The response curves of the actual and bilinear idealized responses are shown in this figure. The vertical and horizontal axes show the base shear and the relative lateral displacement of the roof, respectively [26].

Since the inelastic analysis and design of structures is complex and time-consuming, most of the regulations, under some conditions, replace the elastic analysis and use RMF (R) to determine the design resistance, which reduces the elastic force to design force. Over-strength of the designed structures and their ability to dissipate the energy imposed by an earthquake (ductility) are two important factors related to RMF [31].

According to FEMA-450 [32] regulations, the remaining strength between the actual level of structure yield (V_y) and level of design force (V_s), in Load and Resistance Factor Design (LRFD) method, is expressed in over-strength factor (Ω_0). This factor is determined by Eq. (1) and by the type of structural system. It depends on these parameters: system over-strength factor (Ω_S), material over-strength factor (Ω_M), and design over-strength factor (Ω_D).

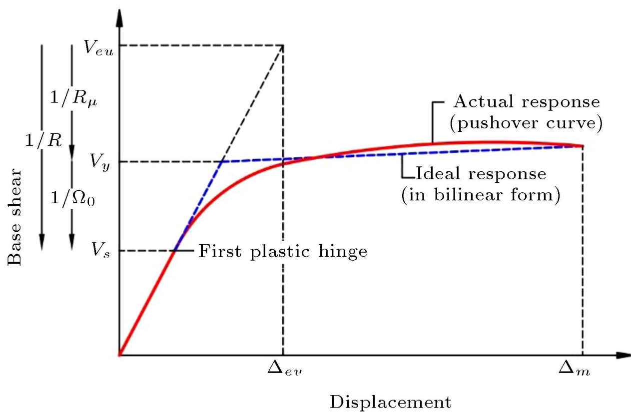


Figure 4. The seismic design parameters of the structures adopted for the pushover curve [32].

$$\Omega_0 = \Omega_S \Omega_M \Omega_D = \frac{V_y}{V_s}. \tag{1}$$

The base shear V_s is used in the LRFD design method that indicates the forming of the first plastic hinge in structure. Members enter into the plastic area with a further increase in lateral force. The RMF of the structure is obtained from the product of ductility reduction factor (R_μ) and over-strength factor (Ω_0) through Eq. (2). More details about the parameters are given in Figure 4.

$$R = \frac{V_{eu}}{V_y} \cdot \frac{V_y}{V_s} = \frac{V_{eu}}{V_s} = R_\mu \Omega_D. \tag{2}$$

2.2. The method employed to model the Reinforced Concrete (RC) columns

One of the approaches to modeling the nonlinear responses of the RC structural members is to assign the response of plastic hinges taking into account their specified lengths to their locations, which most probably demonstrates a nonlinear static response and nonlinear behavior for the structural member. There are different types of hinges, but from the modeling point of view, they are classified into two leading categories as described below [26]:

1. The hinges that comprise the entire cross-sections of the components as the points with characteristic geometry and material [26];
2. The hinges that divide the cross-sections of the components into smaller sub-components. Each of them has a length equal to the length of the hinge with an individual nonlinear loading and a response. The overall response of the component is determined according to the responses of the sub-component series. Each fiber could undergo only the longitudinal stress. Therefore, by means of these hinges, only the nonlinear responses of the components under the axial load and bending moment can be investigated. The force exerted on each fiber is the sum of the stresses times their allocated surface area on the main cross-section. In fact, each fiber acts as a rod under the axial load. This type of modeling is known as fiber or layer theory, sometimes called by other similar names [26].

The fiber theory is employed in this research taking into account the combination of concrete and steel behaviors along with the fiber plastic hinges (see Figure 5) [33,34]. Different test results of determining the plastic hinge length (L_p) show major scattering,

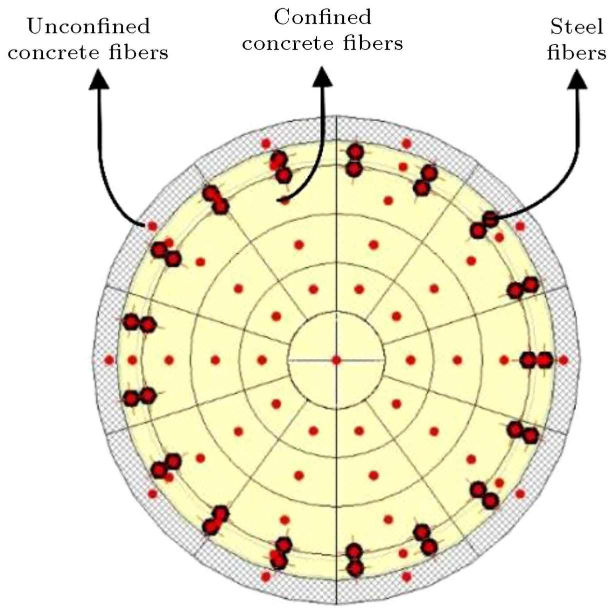


Figure 5. Scheme of the fiber model used in the analysis of Reinforced Concrete (RC) columns.

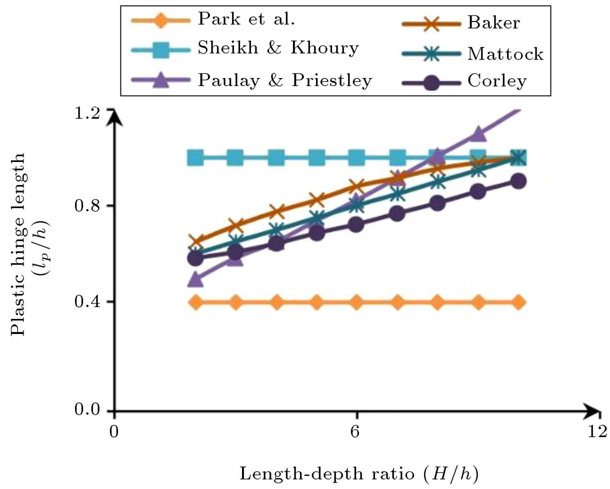


Figure 6. Proposed relations by different researchers for L_p [35].

because there are considerable differences between different existing methods proposed. Different equations have been proposed by researchers to calculate plastic hinge length [35]. Figure 6 illustrates a comparison between different existing models for determining the plastic hinge length. In Figure 6, H and h represent the height and width of the cross-section of the column, respectively.

Eq. (3) proposed by the Paulay and Priestley [36] is used in modeling of the structures in this paper:

$$L_P = 0.08L + 0.022 d_b f_y, \tag{3}$$

where L_P (in mm) represents the plastic hinge length, L (in mm) represents the column length, d_b (in mm) represents the diameter of longitudinal reinforcement, and f_y (in MPa) represents the yield strength of the reinforcement.

2.3. The method employed to model the Bubble Deck (BD) slabs

A number of methods exist to model the nonlinear behavior of RC slabs [23,37,38]. In this research, the equivalent nonlinear shell layered element is employed to model the sections of the slab in the numerical simulation. As shown in Figure 7, in this type of elements, the slab cross-sections are divided into some layers. The thickness of each concrete layer in the element of the model is considered equal to the existing concrete area in the prototype one [26].

2.4. The stress-strain models applied to reinforcement and concrete

In this paper, a three-phase stress-strain model is used to simulate the behavior of reinforcements. Linear elasticity, perfectly plastic region (plateau), and strain hardening phases are considered in this model (see Figure 8(a)) [39]. The well-known constitutive law of Mander et al. [40] for confined and unconfined concrete is used in this research (see Figure 8(b)). The reason for adopting this constitutive law in the present research is its validity, which has been confirmed by Sadeghi [41–44], Sadeghi and Nouban [45], and other researchers by comparing the results of experimental tests and their proposed analytical models with Mander et al. constitutive law. In addition, the application of Mander et al. model is relatively simpler and easier than other models. The compared models in other studies have led to more or less similar results and they have shown a good agreement. As a final proof, Mander

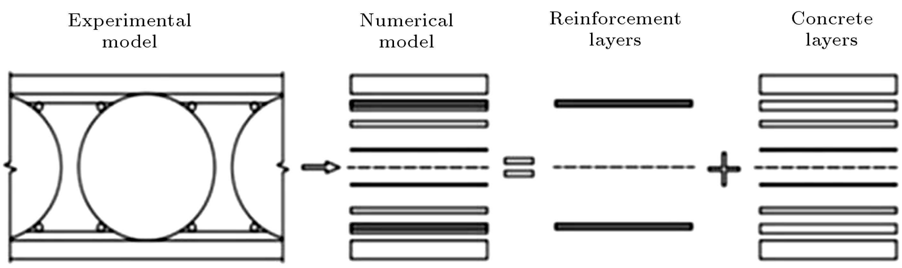


Figure 7. Scheme of the equivalent nonlinear shell layered elements used in the modeling of Bubble Decks (BDs) [26].

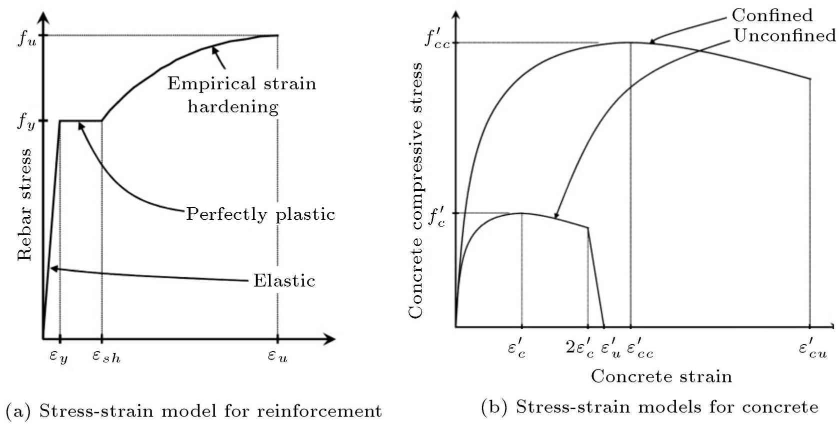


Figure 8. Employed stress-strain models of concrete and reinforcements [39].

et al. law has been employed in some structural design software such as SAP2000 that is used in this research.

3. The applied numerical modeling

In the current research, the same structures and modeling of 36 buildings considered in another paper published by the authors on ductility evaluation [26] have been employed. These structures have the same plan and three spans in the main two horizontal directions have been modeled. The MRF and MRFSW lateral-resisting systems have been selected for these structures. The shear walls have been considered to be positioned on the middle spans of the outside structural frames of the buildings. The models have 4, 8, and 12 stories that can be considered as representatives of low-rise, mid-rise, and high-rise building structures, respectively. Story height of all structures is 3.5 m and there are three types of span length to story height ratio (L/H). The models have been selected in three usage categories, namely commercial (C), administrative (A), and residential (R), applying the relative live loads in the design processes. For these usage categories, live loads of 500, 350, and 200 kgf/m^2 have been chosen, respectively. The details of naming and specifications of the models are presented in Table 1, considering a variety of specifications. For example, "12A2.5W" indicates a 12-story structure in administrative usage with the span length to story height ratio of 2.5; "W" at the end of the model name indicates dual systems including MRFSW as lateral resisting system. For the 12-story structure models, the design and evaluation of MRF have not been done, because the dimensions of columns became too large for providing the lateral stiffness requirements and seemed impractical [26].

In the design process of the structures, it was assumed that all the models were located in a zone with very high seismic risk and the peak ground acceleration of the design base earthquake was 0.35 times the gravitational acceleration. In addition, the structures were designed based on ACI 318-14 [21] with compressive strength and elasticity modulus of concrete equal to 25 MPa and 24222 MPa, respectively. The yield stress of 400 MPa was considered for longitudinal reinforcements and 340 MPa for transverse reinforcements in both slabs and columns. In the

Table 1. Specifications of models and the used abbreviated names.

Stories no.	Live load (kgf/m^2)	MRFSW model			MRF model		
12	350	12A3.5W	12A2.5W	12A1.5W	—	—	—
	200	12R3.5W	12R2.5W	12R1.5W	—	—	—
8	350	8A3.5W	8A2.5W	8A1.5W	8A3.5	8A2.5	8A1.5
	200	8R3.5W	8R2.5W	8R1.5W	8R3.5	8R2.5	8R1.5
4	500	4C3.5W	4C2.5W	4C1.5W	4C3.5	4C2.5	4C1.5
	350	4A3.5W	4A2.5W	4A1.5W	4A3.5	4A2.5	4A1.5
	200	4R3.5W	4R2.5W	4R1.5W	4R3.5	4R2.5	4R1.5
L/H		3.5	2.5	1.5	3.5	2.5	1.5

Table 2. Thicknesses of the decks in models.

L/H	Deck thickness in the MRFSW model (mm)	Deck thickness in the MRF model (mm)
1.5	230	230
2.5	340	390
3.5	390	600

design process of structures, four types of the deck were used and all types of decks were selected based on the span length to story height ratio (L/H) and the lateral resisting systems (see Table 2) [26].

Circular section columns with confinement reinforcements of spiral type and concrete covers equal to 45 mm were selected. Columns of each structure were designed in three types, namely corner, exterior, and middle columns, with different diameters for different stories, as presented in Tables 3–7 [26].

4. Modeling verification

Due to the issues of numerical nonlinear modeling, SAP2000 software has been used to model, design, and analyze the structures [39]. To verify the modeling method, the experimental work of Ibrahim et al. [46], performed on a BD pushed with five concentrate loads, has been selected and used in this paper. After loading on slabs with specified dimensions shown in Figures 9 and 10, pushover curve is drawn for a point located in the middle of slab span. Experimental test specifications of slabs are given in Table 8 [26].

The obtained results from the proposed numerical simulation method are compared with the experimental test results and illustrated in Figure 11. As this comparison demonstrates, there is a good agreement between the responses of the proposed simulation method and the experimental test for strength as well as stiffness. Due to the good speed of analysis compared to the finite element microscopic model analysis, the proposed modeling method is recommended to be employed in the evaluation of the response of structures having several spans and stories.

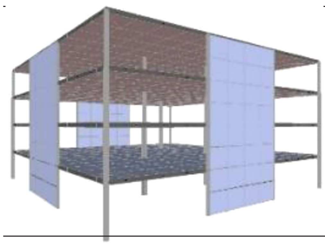
5. Seismic responses

In order to assess the seismic behavior of structures, first modeling and then nonlinear static analysis are performed. The obtained responses for different structural conditions are found and finally, the seismic parameters are computed after idealization of the responses.

5.1. *Low-rise building structures (4-story structures)*

Figures 12 and 13 illustrate the pushover curves of the 4-story models. As it can be seen in these figures, by increasing the value of the span length to story height ratio (L/H), structural strength increases. Note that by increasing the L/H ratio, stronger columns and walls are required. Hence, the lateral strength and stiffness increase. The analysis of seismic parameters has been carried out for all of the 4-story models. RMFs (R) were calculated by idealizing pushover curves. The parameters of R_μ, Ω_0, and R are presented in Table 9.

Table 3. Diameters of columns for different stories of the 4-story Moment-Resisting Factor with Shear Wall (MRFSW) models (cm).



Model	4C3.5W			4C2.5W			4C1.5W			4A3.5W			4A2.5W			4A1.5W			4R3.5W			4R2.5W			4R1.5W		
Column type	C*	E*	M*	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M
Story number 1	60	50	100	40	30	60	30	30	35	55	50	85	40	30	60	30	30	35	50	50	80	30	30	55	30	30	35
2	60	50	80	40	30	55	30	30	35	55	50	80	40	30	55	30	30	35	50	50	70	30	30	50	30	30	30
3	55	50	60	35	30	45	30	30	30	50	50	60	35	30	45	30	30	30	50	50	55	30	30	40	30	30	30
4	50	50	40	35	30	30	30	30	30	50	50	40	35	30	30	30	30	30	50	50	40	30	30	30	30	30	30

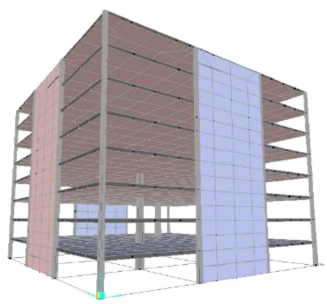
*: C = Corner; E = Exterior; M = Middle

Table 4. Diameters of columns for different stories of the 4-story Moment-Resisting Frame (MRF) models (cm).

Model	4C3.5			4C2.5			4C1.5			4A3.5			4A2.5			4A1.5			4R3.5			4R2.5			4R1.5		
Column type	C*	E*	M*	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M
Story number 1	80	100	110	60	65	80	40	45	50	80	85	120	60	65	80	40	45	50	80	85	120	55	65	80	40	40	50
2	70	100	100	55	65	70	40	40	50	70	85	100	55	60	70	35	40	45	70	85	100	55	60	65	35	40	45
3	65	85	85	50	60	65	35	40	45	65	80	85	50	60	60	35	40	40	65	80	85	50	60	60	35	40	40
4	60	80	65	45	55	45	30	35	35	60	70	65	45	50	50	30	35	35	60	70	60	45	50	50	30	35	35

*: C = Corner; E = Exterior and M = Middle.

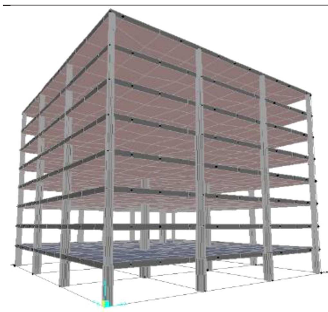
Table 5. Diameters of columns for different stories of the 8-story Moment-Resisting Factor with Shear Wall (MRFSW) models (cm).



Model		8A3.5W			8A2.5W			8A1.5W			8R3.5W			8R2.5W			8R1.5W		
Column type		C*	E*	M*	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M
Story number	1	60	70	120	40	55	80	30	40	45	55	70	120	35	55	80	30	40	40
	2	60	60	120	40	50	80	30	35	40	55	60	105	35	50	70	30	35	40
	3	55	55	105	35	45	70	30	30	40	55	55	100	30	45	65	30	30	35
	4	55	55	100	35	35	65	30	30	35	50	55	100	30	35	60	30	30	35
	5	55	55	85	35	35	55	30	30	30	50	55	80	30	35	55	30	30	30
	6	50	55	80	35	35	50	30	30	30	50	55	70	30	35	45	30	30	30
	7	50	55	60	30	35	40	30	30	30	50	55	55	30	35	40	30	30	30
	8	50	55	40	30	35	35	30	30	30	50	55	40	30	35	35	30	30	30

*: C = Corner; E = Exterior and M = Middle.

Table 6. Diameters of columns for different stories of the 8-story Moment-Resisting Frame (MRF) models (cm).



Model		8A3.5			8A2.5			8A1.5			8R3.5			8R2.5			8R1.5		
Column type		C*	E*	M*	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M
Story number	1	100	120	155	70	85	100	45	55	60	100	110	145	65	80	100	45	55	55
	2	85	110	135	60	80	100	40	50	55	80	105	125	60	80	85	40	50	55
	3	80	100	120	60	70	85	40	50	55	80	100	120	60	70	80	40	45	55
	4	80	100	120	60	70	80	40	45	55	80	100	110	55	70	80	40	45	50
	5	80	100	105	55	65	80	40	45	50	80	100	100	55	65	80	40	45	50
	6	70	85	100	55	65	70	35	40	50	70	85	100	55	60	70	35	40	45
	7	65	80	80	50	60	60	35	40	40	65	80	80	50	55	60	35	40	40
	8	65	70	65	50	55	50	35	35	35	65	80	65	50	55	50	35	35	35

*: C = Corner; E = Exterior and M = Middle.

Table 7. Diameters of columns for different stories of the 12-story Moment-Resisting Factor with Shear Wall (MRFSW) models (cm).

Model		12A3.5W			12A2.5W			12A1.5W			12R3.5W			12R2.5W			12R1.5W		
Column type		C*	E*	M*	C	E	M	C	E	M	C	E	M	C	E	M	C	E	M
Story number	1	65	100	150	45	80	100	30	55	50	60	100	140	40	80	100	30	55	45
	2	65	100	145	45	80	100	30	55	50	60	100	135	40	80	100	30	50	45
	3	65	85	135	40	70	100	30	50	45	60	85	125	40	70	85	30	50	40
	4	60	80	135	40	65	85	30	45	45	55	80	120	35	65	80	30	45	40
	5	60	70	120	40	60	80	30	40	40	55	70	120	35	55	80	30	40	35
	6	55	60	120	35	50	80	30	35	40	55	60	105	35	50	70	30	35	35
	7	55	55	105	35	45	70	30	30	35	50	55	100	35	45	65	30	30	35
	8	55	55	100	35	40	65	30	30	35	50	55	100	35	40	60	30	30	30
	9	50	55	85	35	40	55	30	30	30	50	55	80	35	40	50	30	30	30
	10	50	55	70	35	40	50	30	30	30	50	55	70	35	40	45	30	30	30
	11	50	55	60	35	40	40	30	30	30	50	55	55	35	40	40	30	30	30
	12	50	55	45	35	40	40	30	30	30	50	55	45	35	40	35	30	30	30

*: C = Corner; E = Exterior; and M = Middle.

The usage category application was employed by applying different live loads, which led to building dissimilar models with different specifications. The variations of RMF in function of live load as well as the L/H values are illustrated in Figure 14. As demonstrated in Figure 14, the span length variations have more significant effects than the live load variations on the RMF values; moreover, the average values of various usage categories can be considered in the seismic loading cases.

Note that to emphasize the changes in parameters, in Figure 14, the common parts of the names of the models have been deleted (as an example, in 4...1.5, the emphasis is on 4 and 1.5).

As Figure 14 and Table 9 specify, RMF variations with various L/H ratios in MRF structures are

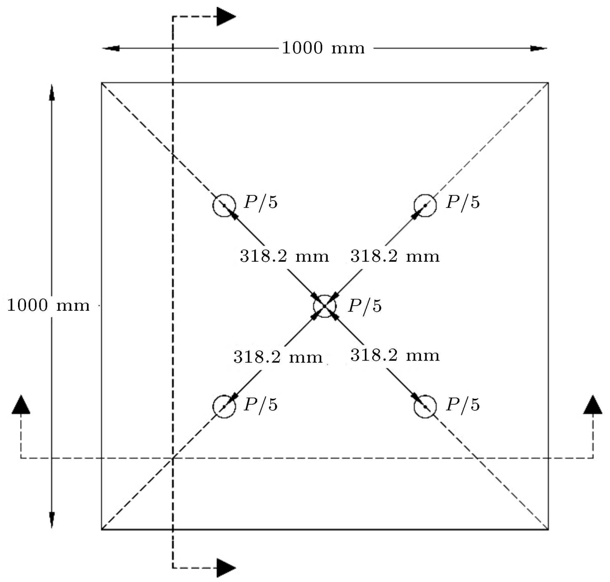


Figure 9. Specifications of the experimental test specimen [46].

higher than those in MRFSW. This is due to more participation of columns in lateral load resisting in MRF, while in MRFSW model, shear walls have a more important role.

5.2. Mid-rise building structures (8-story structures)

The pushover curves for the 8-story models are submitted in Figures 15 and 16. As indicated in these figures, by increasing the L/H ratio, structural strength increases. Note that there is a similar manner to that of the 4-story structures. By increasing L/H ratio in the design process, stronger columns and walls are required. Thus, lateral strength and stiffness increase (Tables 5 and 6).

The seismic parameters were analyzed for all the 8-story models. Then, the RMFs (R) were computed by the idealization of the pushover curves. The parameters of R_μ, Ω_0, and R are presented in Table 10.

Variations of RMF in function of live load and L/H variations are illustrated in Figure 17. Note that, to emphasize the changes in parameters, in this figure, the common parts of the names of the models have been deleted (as an example, in 8...1.5, the emphasis is on 8 and 1.5). As indicated in Figure 17, the live load variations have higher effect than the span length variations on the RMF of structures and the average values of various usage categories can be considered in seismic parameters. As Table 10 and Figure 17 indicate, also as mentioned for the 4-story models, the RMF variations for various L/H ratios in MRF structures are higher than those in MRFSW structures. In addition, L/H has greater effect than the number of stories on determining RMF in MRF models, while in MRFSW models, the number of stories has a more significant effect on RMF.

5.3. High-rise building structures (12-story structures)

The pushover curves for the12-story structural models are prepared similarly to those for the other models

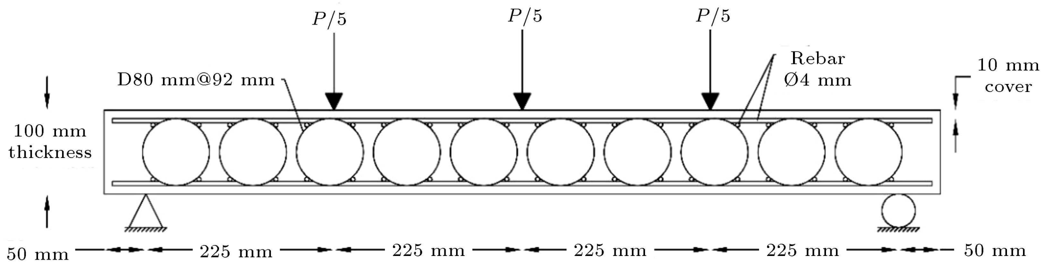


Figure 10. Scheme of the cross-section of Bubble Deck (BD) used in the experimental test [46].

Table 8. Experimental test specimen specifications [46].

Reinforcement diameter (mm)	Reinforcement area of cross section (mm^2)		Reinforcement yield stress (MPa)		Reinforcement ultimate strength (MPa)	
4	12.566		557		835	
Length of slab (mm)	**Width of slab (mm)**	**Thickness of slab (mm)**	**Diameter of ball (mm)**	**No. of balls**	**Concrete compressive strength (MPa)**	**Percentage of reinforcement ρ (%)**
1000	1000	100	80	100	33.34	0.503

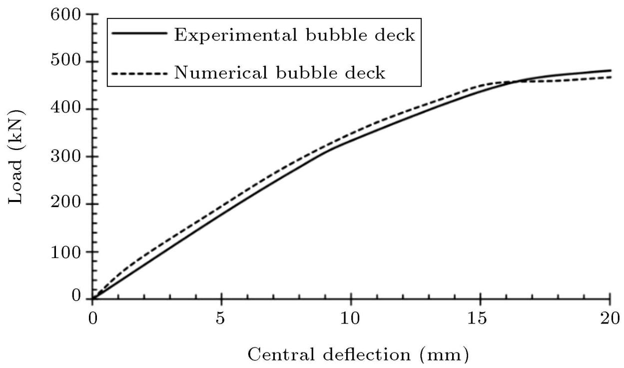


Figure 11. Comparison of simulated and tested pushover curves for the point at the middle of the Bubble Deck (BD).

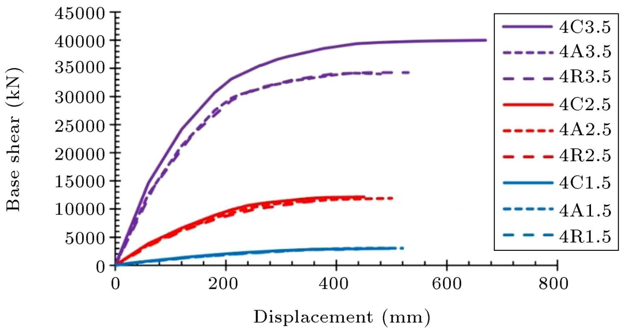


Figure 12. Pushover curves of the 4-story Moment-Resisting Frame (MRF) models.

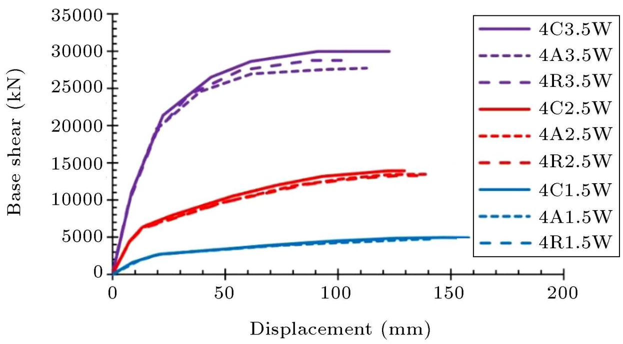


Figure 13. Pushover curves of the 4-story Moment-Resisting Factor with Shear Wall (MRFSW) models.

and presented in Figure 18. In a similar clarification given for the other structural models, as it can be seen in Figure 18, by increasing the value of the span length to story height ratio (L/H), the structural strength increases. Note that by increasing the L/H ratio, stronger columns and walls are required. Therefore, lateral strength and stiffness increase (Table 7). The seismic parameters were analyzed for all the 12-story models and then, RMFs (R) were computed by the idealization of the pushover curves. The parameters of R_μ, Ω_0, and R are presented in Table 11.

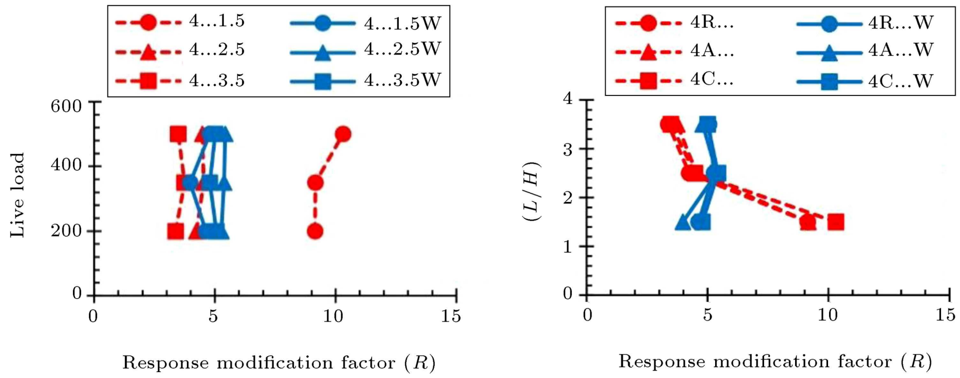
As indicated in Figure 17, the span length variations have greater effect than the live load variations on the RMF of structures and the average values of various usage categories can be considered in seismic parameters. As Table 11 and Figure 19 illustrate, also as mentioned for other structures, variations in the number of stories affect the RMF in MRFSW models more significantly.

Table 9. Seismic parameters of the 4-story models.

Model	R_μ	Ω_0	R
4R1.5	2.4	3.8	9.15
4A1.5	2.11	4.35	9.16
4C1.5	2.27	4.53	10.3
4R2.5	2.2	1.92	4.24
4A2.5	1.81	2.51	4.54
4C2.5	1.88	2.39	4.48
4R3.5	1.64	2.06	3.37
4A3.5	1.71	2.19	3.75
4C3.5	1.52	2.29	3.48
4R1.5W	2.63	1.76	4.63
4A1.5W	2.5	1.59	3.97
4C1.5W	2.38	2.01	4.78
4R2.5W	2.75	1.91	5.26
4A2.5W	2.75	1.95	5.38
4C2.5W	2.78	1.95	5.43
4R3.5W	2.23	2.27	5.06
4A3.5W	2.26	2.12	4.79
4C3.5W	2.21	2.27	5.01

Table 10. Seismic parameters of the 8-story models.

Model	R_μ	Ω_0	R
8R1.5	2.32	3.33	7.75
8A1.5	1.9	3.97	7.55
8R2.5	2.42	1.84	4.46
8A2.5	2.29	1.95	4.47
8R3.5	2.37	1.63	3.85
8A3.5	2.08	1.8	3.75
8R1.5W	5.06	1.44	7.26
8A1.5W	5.06	1.46	7.4
8R2.5W	3.98	1.61	6.4
8A2.5W	3.77	1.69	6.37
8R3.5W	3.47	2.04	7.06
8A3.5W	3.43	2.08	7.13



(a) RMF variations in function of live load

(b) RMF variations in function of L/H

Figure 14. Comparison of Response Modification Factors (RMFs) for the 4-story models and variations of live load and L/H.

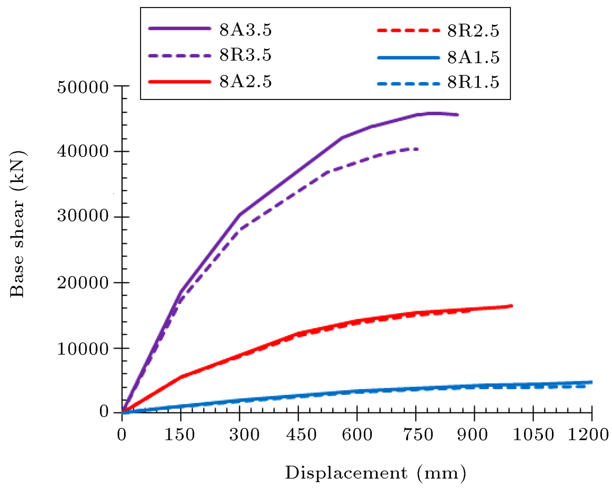


Figure 15. Pushover curves for the 8-story Moment-Resisting Frame (MRF) models.

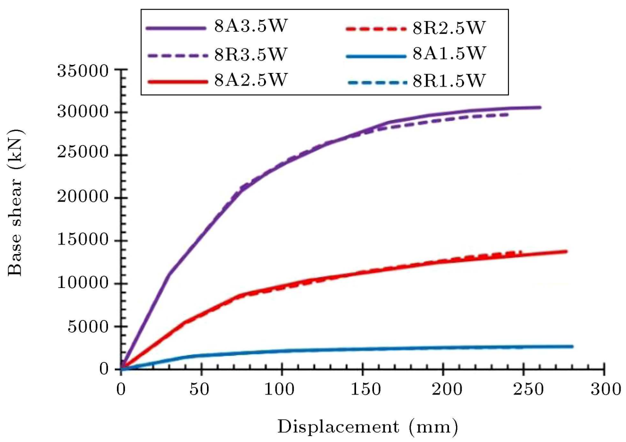


Figure 16. Pushover curves for the 8-story Moment-Resisting Factor with Shear Wall (MRFSW) models.

5.4. *Comparison of Response Modification Factor (RMF) structures*

As stated before, the span length variations have greater effect than the usage category variations on the RMF of the structures. Figure 20 shows the variations of RMF in administrative usage as a function of the number of stories. RMFs were analyzed for all the models studied in this paper. According to the values specified in Tables 9–11, the average values of seismic parameter R are given in Table 12 for various building categories.

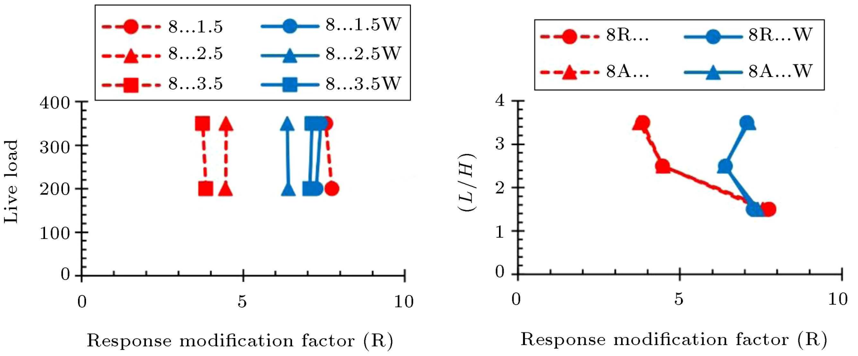
Table 11. Seismic parameters of the 12-story models.

Model	R_μ	Ω_0	R
12R1.5W	3.34	2.66	8.9
12A1.5W	3.37	2.95	9.96
12R2.5W	3.29	1.99	6.56
12A2.5W	3.2	2.15	6.88
12R3.5W	3.56	1.77	6.3
12A3.5W	3.37	1.9	6.4

It can be seen that the span length has a more significant effect than the number of stories on determining RMF in MRF, while in the MRFSW models, the number of stories has a greater effect on RMF. Hence, to determine the RMF of MRF structures, as a conservative approach, for the structures with the L/H ratio of 1.5, RMF of 7 and for larger ratios, RMF of 4 are recommended. Also, to determine the RMF values for MRFSW structures, as a conservative approach, for 4-story structures (as representatives of low-rise building structures), the RMF of 5 and for the 8- and 12-story structures as representatives of mid-rise and high-rise building structures, respectively, the RMF of 7 are recommended.

6. Conclusions

Thirty-six Reinforced Concrete (RC) Response Mod-



(a) RMF variations in function of live load (b) RMF variations in function of L/H

Figure 17. Response Modification Factor (RMF) variations for the 8-story models and variations of live load and L/H.

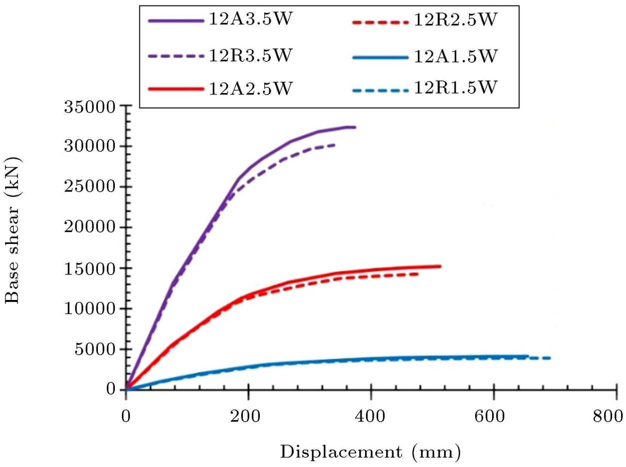


Figure 18. Pushover curves for the 12-story models with Moment-Resisting Factor with Shear Wall (MRFSW).

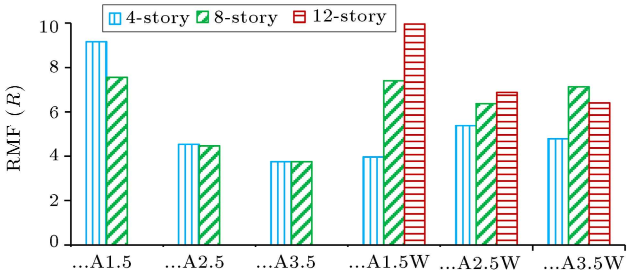
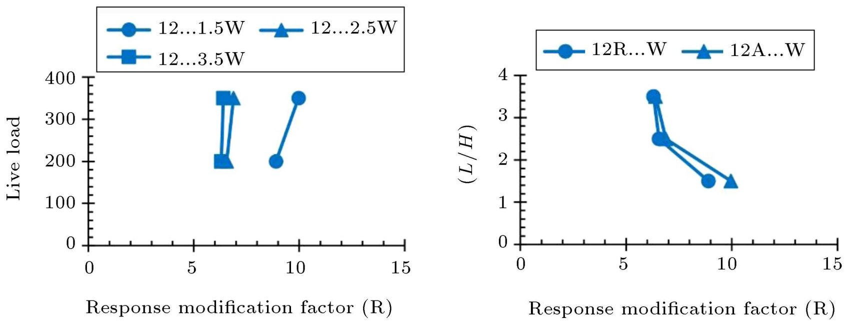


Figure 20. Comparison of Response Modification Factors (RMFs) for different models.

ification Factor (RMF) structures constructed with the Bubble Deck (BD) slabs were evaluated and the obtained results were presented in this paper. The influences of the usage category of structures, span length to story height ratio, and the number of stories on two lateral-resisting systems of Moment-Resisting Fame (MRF) and MRF with Shear Wall (MRFSW) were evaluated and the main obtained results were reported. The conclusions achieved in this research are summarized as follows:

- Lateral strength of the structure increases by increasing the span length to story height ratio (L/H);
- Variations of span length and number of stories have greater effects than variation in usage category on the RMF of structures. Also, span length has a more significant effect than the number of stories on determining RMF in an MRF, while in MRFSW, variation in the number of stories has a greater effect on the RMF;
- For the case of MRFSW structures, the RMF of 5 is proposed for 4-story structures as representative of the low-rise building structures and the RMF of 7 is proposed for 8- and 12-story structures as representatives of mid-rise and high-rise building structures, respectively. For the case of MRF structures with



(a) RMF variations in function of live load (b) RMF variations in function of L/H

Figure 19. Response Modification Factor (RMF) variations for the 12-story models and variations of live load and L/H.

Table 12. Seismic parameters (average values) of structures.

Number of stories	L/H	R (for MRFSW)	R (for MRF)
4 (Low-rise building structures)	1.5	4.46	9.54
	2.5	5.36	4.42
	3.5	4.95	3.53
8 (Mid-rise building structures)	1.5	7.33	7.65
	2.5	6.39	4.47
	3.5	7.1	3.8
12 (High-rise building structures)	1.5	9.43	Not recommend
	2.5	6.72	
	3.5	6.35	

the L/H ratio of up to 1.5, the RMF of 7 and for larger L/H ratios, the RMF of 4 are recommended.

References

1. Aldejohann, M. and Schnellenbach-Held, M. "Investigations on the shear capacity of biaxial hollow slabs - test setup and test program", *Darmstadt Concrete: Annual Journal on Concrete and Concrete Structures*, **17**, pp. 1–8 (2002).
2. Schnellenbach-Held, M. and Pfeffer, K. "Punching behavior of biaxial hollow slabs", *Cement & Concrete Composites*, **24**(6), pp. 551–556 (2002).
3. Chung, J.H., Choi, H.K., Lee, S.C., and Choi, C.S. "Shear capacity of biaxial hollow slab with donut type hollow sphere", *Procedia Engineering*, **14**(12), pp. 2219–2222 (2011).
4. Bindea, M., Moldovan, D., and Kiss, Z. "Flat slabs with spherical voids. Part I: prescriptions for flexural and shear design", *Acta Technica Napocensis: Civil Engineering & Architecture*, **56**(1), pp. 67–73 (2013).
5. Bindea, M., Moldovan, D., and Kiss, Z. "Flat slabs with spherical voids. Part II: experimental tests concerning shear strength", *Acta Technica Napocensis: Civil Engineering & Architecture*, **56**(1), pp. 74–81 (2013).
6. Schmidt, C., Neumeier, B., and Christoffersen, J. "Bubble slab, abstract of test results. Comparative analysis bubble slab-solid slab", AEC Technical University of Denmark, Department of Structural Engineering, Denmark (1993).
7. Schnellenbach-Held, M., Ehmann, S., and Pfeffer, K. "Bubble deck - new ways in concrete building", *Darmstadt Concrete: Annual Journal on Concrete and Concrete Structures*, **13**, pp. 93–100 (1998).
8. Schnellenbach-Held, M., Ehmann, S., and Pfeffer, K. "Bubble deck design of biaxial hollow slabs", *Darmstadt Concrete: Annual Journal on Concrete and Concrete Structures*, **14**, pp. 145–152 (1999).
9. Gudmand-Hoyer, T. "Note on the moment capacity in a bubble deck joint", Technical University of Denmark, Report No. BYG-DTU R-074, Denmark (2003).
10. Lai, T. "Structural behavior of bubble deck slabs and their application to lightweight bridge decks", Massachusetts Institute of Technology, Massachusetts, USA (2010).
11. Calin, S. and Asavoaie, C. "Experimental program regarding bubble deck concrete slab with spherical gaps", *Intersections/Intersect II*, **4**(7)1, pp. 34–40 (2010).
12. Teja, P.P., Kumar, P.V., Anusha, S., and Mounika, CH. "Structural behavior of bubble deck slab", *IEEE-International Conference on Advances in Engineering, Science and Management (ICAESM-2012)*, Nagapattinam (March 2012).
13. Terec, L. and Terec, M. "Bubble deck floor system: a brief presentation", *Constructii: Journal of Civil Engineering Research*, **14**(2), pp. 33–40 (2013).
14. Churakov, A. "Biaxial hollow slab with innovative types of voids", *Construction of Unique Buildings and Structures*, **6**(21), pp. 70–88 (2014).
15. Dowell, R.K. and Smith, J.W. "Structural tests of precast, prestressed concrete deck panels for California freeway bridges", *PCI Journal*, **51**(2), pp. 76–87 (2006).
16. Olsen, O., *Beregning, Dimensioning and Execution of Biaxial Hollow Core Elements*, 6th Edn., Jjj Consult (Dr. Techn. Jens Jacob Jensen AS), in Norwegian (2009).
17. Calin, S. and Asavoaie, C. "Method for bubble deck concrete slab with gaps", *Bulletin of The Polytechnic Institute of Iasi*, **2**(1), pp. 63–70 (2009).
18. Gajen, N. "Investigation of moment behavior and shear strength in two-way slabs", M.Sc. Dissertation, Engineering Faculty, Yasouj University, Iran (in Persian) (2012).

19. Behzard, P., Sharbatdar, M.K., and Kheyroddin, A. "A different NSM FRP technique for strengthening of RC two-way slabs with low clear cover thickness", *Scientia Iranica, Transactions A*, **23**(2), pp. 520–534 (2016).

20. Li, S., Fu, L., and Kong, F. "Seismic response reduction of structures equipped with a voided biaxial slab-based tuned rolling mass damper", Hindawi Publishing Corporation, *Shock and Vibration*, **2015**, Article ID 760394 (2015). https://doi.org/10.1155/2015/760394

21. ACI-318-14, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, MI, USA (2014).

22. ASCE/SEI-41-06, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia, USA (2007).

23. Kim, K., Choi, S., Ju, H., Lee, D., Lee, J., and Shin, M. "Unified equivalent frame method for flat plate slab structures under combined gravity and lateral loads - Part 1: Derivation", *Earthquakes and Structures*, **7**(5), pp. 719–733 (2014).

24. Mohammad, M.H., Massumi, A., and Meshkat-Dini, A. "Near-fault effects on RC buildings' demand in linear and nonlinear analyses", *Scientia Iranica, Transactions A*, **26**(1), pp. 188–201 (2019).

25. Akbarzadeh Bengar, H. and Mohammadalipour Aski, R. "Effect of steel and concrete coupling beams on seismic behavior of RC frame accompanied with coupled shear walls", *Scientia Iranica, Transactions A*, **24**(5), pp. 2227–2241 (2017).

26. Hashemi, S.Sh., Sadeghi, K., Vaghefi, M., and Siadat, A. "Evaluation of ductility of RC structures constructed with bubble deck system", *International Journal of Civil Engineering*, **16**(5), pp. 513–526 (2017).

27. IBC2015, *International Building Code*, International Code Council Inc., Washington, DC, USA (2015).

28. ASCE/SEI-7-10, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia, USA (2010).

29. Mwafi, A. and Elanshai, A. "Static pushover versus dynamic collapse analysis of RC buildings", *Engineering Structures*, **23**(1), pp. 407–424 (2001).

30. FEMA-365, *Pre-Standard and Commentary for the Seismic Rehabilitation of Building*, Federal Emergency Management Agency, Washington DC, USA (2000).

31. Samimifar, M. and Vatani-oskouei, E. "Determine the response modification factor of reinforcement concrete moment frame with nonlinear dynamic analysis", *6th National Congress on Civil Engineering*, Semnan, Iran, (in Persian) (2012).

32. FEMA-450, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Building Seismic Safety Council, Federal Emergency Management Agency, Washington DC, USA (2004).

33. Hashemi, S.SH. and Vaghefi, M. "Investigation of bond slip effect on the P-M interaction surface of RC columns under biaxial bending", *Scientia Iranica, Transactions A*, **22**(2), pp. 388–399 (2015).

34. Hashemi, S.SH., Vaghefi, M., and Hemmat, M. "Evaluation the effects of stirrup spacing and buckling of steel reinforcing bars on the capacity of RC columns", *Scientia Iranica, Transactions A*, **25**(3), pp. 1140–1151 (2018).

35. Mortezaei, A. and Zahrai, S.M. "A proposed equation for the estimation of plastic hinge length of RC columns subjected to ground motions", *Journal of Computational Methods in Engineering (Esteghlal)*, **31**(2), pp. 39–59 (2013).

36. Paulay, T. and Priestley, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, New York (1992).

37. Park, Y.M., Han, S.W., and Kee, S.H. "A modified equivalent frame method for lateral load analysis", *Magazine of Concrete Research*, **61**(5), pp. 359–370 (2009).

38. Choi, S., Lee, D., Oh, J., Kim, K., Lee, J., and Shin, M. "Unified equivalent frame method for flat plate slab structures under combined gravity and lateral loads - Part 2: Verification", *Earthquakes and Structures*, **7**(5), pp. 735–751 (2014).

39. Computers and Structures, Inc. SAP2000 advanced 14.2.0, Berkeley, California, USA (2010).

40. Mander, J.B., Priestley, M.J.N., and Park, R. "Theoretical stress-strain model for confined concrete", *Journal of Structural Engineering*, **114**(8), pp. 1804–1825 (1988).

41. Sadeghi, K. "Numerical simulation and experimental test of compression confined and unconfined concretes", Technical Report Submitted to Water Resources Management Organization, Ministry of Energy, Concrete Laboratory of Power and Water University of Technology, Tehran (2002).

42. Sadeghi, K. "Analytical stress-strain model and damage index for confined and unconfined concretes to simulate RC structures under cyclic loading", *International Journal of Civil Engineering, Transactions A*, **12**(3), pp. 333–343 (2014).

43. Sadeghi, K. "Nonlinear numerical simulation of reinforced concrete columns under cyclic biaxial bending moment and axial loading", *International Journal of Civil Engineering*, **15**(1), pp. 113–124 (2017).

44. Sadeghi, K. "Nonlinear static-oriented pushover analysis of reinforced concrete columns using variable oblique finite-element discretization", *International Journal of Civil Engineering*, **14**(5), pp. 295–306 (2016).

45. Sadeghi, K. and Nouban, F. "Behavior modeling and damage quantification of confined concrete under cyclic loading", *Structural Engineering and Mechanics*, **61**(5), pp. 625–635 (2017).

46. Ibrahim, A., Ali, N., and Salman, W. "Flexural capacities of reinforced concrete two-way bubble deck slabs of plastic spherical voids", *Diyala Journal of Engineering Sciences*, **6**(2), pp. 9–20 (2013).

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