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

S.Sh. Hashemi\textsuperscript{a,}\textsuperscript{*}, K. Sadeghi\textsuperscript{b}, M. Vaghefi\textsuperscript{a}, and S.A. Siadat\textsuperscript{c}

\textsuperscript{a.} Department of Civil Engineering, Persian Gulf University, Bushehr, P. O. Box 75169-13817, Iran.
\textsuperscript{b.} Department of Civil Engineering, Near East University, ZIP Code 99418, Nicosia, North Cyprus, Mersin 10, Turkey.
\textsuperscript{c.} Department of Civil Engineering, Islamic Azad University of Bushehr, Varzesh Street, Bushehr, P. O. Box 75196-1955, Iran.

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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.

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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-
system 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 lightweight 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 failure 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, Terez and Terez [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 Mohammadali Pour 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 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 (MRSFW) 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-horizonal 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 ($V_e$), in Load and Resistance Factor Design (LRFD) method, is expressed in over-strength factor ($\Omega_0$). This factor is determined by Eq. (1) and by the type of structural system. It depends on these parameters: system over-strength factor ($\Omega_s$), material over-strength factor ($\Omega_m$), and design over-strength factor ($\Omega_D$).

$$\Omega_0 = \Omega_s \Omega_m \Omega_D = \frac{V_y}{V_e}$$

The base shear $V_y$ 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 ($R_y$) and over-strength factor ($\Omega_0$) through Eq. (2). More details about the parameters are given in Figure 4.

$$R = \frac{V_{ey}}{V_y}, \frac{V_y}{V_e} = \frac{V_{ey}}{V_e} = R_y \Omega_0.$$  

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.
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:

\[
L_p = 0.08L + 0.022d_b f_y, \tag{3}
\]

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 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

Figure 7. Scheme of the equivalent nonlinear shell layered elements used in the modeling of Bubble Decks (BDs) [26].
et al. law has been employed in some structural design software such as SAP 2000 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 MRF-SW 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/m² 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, “12A 2.5W” 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 MRF-SW 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. 

| Stories no. | Live load (kgf/m²) | MRFSW model | MRF model |
|------------|-------------------|-------------|-----------|
| 12         | 350               | 12A 3.5W    | 12A 2.5W  | 12A 1.5W | —         | —         | —         |
|            | 200               | 12R 3.5W    | 12R 2.5W  | 12R 1.5W | —         | —         | —         |
| 8          | 350               | 8A 3.5W     | 8A 2.5W   | 8A 1.5W  | 8A 3.5    | 8A 2.5    | 8A 1.5    |
|            | 200               | 8R 3.5W     | 8R 2.5W   | 8R 1.5W  | 8R 3.5    | 8R 2.5    | 8R 1.5    |
|            | 500               | 4C 3.5W     | 4C 2.5W   | 4C 1.5W  | 4C 3.5    | 4C 2.5    | 4C 1.5    |
| 4          | 350               | 4A 3.5W     | 4A 2.5W   | 4A 1.5W  | 4A 3.5    | 4A 2.5    | 4A 1.5    |
|            | 200               | 4R 3.5W     | 4R 2.5W   | 4R 1.5W  | 4R 3.5    | 4R 2.5    | 4R 1.5    |

$L/H$ | 3.5 | 2.5 | 1.5 | 3.5 | 2.5 | 1.5

**Figure 8.** Employed stress-strain models of concrete and reinforcements [39].
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, SAP 2000 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].

### 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. RMF's (\( R \)) were calculated by idealizing pushover curves. The parameters of \( R_p, \Omega_0, \) and \( R \) are presented in Table 9.

| Column type | 4C3.5W | 4C2.5W | 4C1.5W | 4A3.5W | 4A2.5W | 4A1.5W | 4R3.5W | 4R2.5W | 4R1.5W |
|-------------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Model       | 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 C E M |
| Story number| 1 60 50 100 40 20 40 30 35 55 50 85 40 30 60 30 30 35 50 50 80 30 30 35 |
|             | 2 60 50 80 40 30 55 30 30 35 55 50 80 40 30 55 30 30 35 50 50 10 30 30 35 |
|             | 3 55 50 60 35 30 45 30 30 30 30 50 50 60 35 30 45 30 30 30 50 50 55 30 30 30 |
|             | 4 50 50 40 35 30 30 30 30 30 30 50 50 40 35 30 30 30 30 30 30 30 30 30 30 30 |

*: C = Corner; E = Exterior; 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 |
| 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 60  | 30 35 40 | 30 30 30 | 50 55 55  | 30 35 40 | 30 30 30 |
| 7       | 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 |
| 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 85 | 40 50 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 |
| 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 55 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
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 $R_u$, $\Omega_u$, 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 the 12-story structural models are prepared similarly to those for the other models.

![Figure 9](link)

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

![Figure 10](link)

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

| Reinforcement diameter (mm) | Reinforcement area of cross section (mm²) | Reinforcement yield stress (MPa) | Reinforcement ultimate strength (MPa) | Concrete compressive strength (MPa) | Percentage of reinforcement $\rho$ (%) |
|-----------------------------|------------------------------------------|----------------------------------|--------------------------------------|-----------------------------------|--------------------------------------|
| 4                           | 12.566                                    | 557                              | 835                                  | 33.34                             | 0.503                                |

| 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 $\rho$ (%) |
|---------------------|--------------------|------------------------|-----------------------|--------------|------------------------------------|--------------------------------------|
| 1000                | 1000               | 100                    | 80                    | 100          | 33.34                              | 0.503                                |
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 $R_{\mu}$, $\Omega_{0}$, 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.

| Model  | $R_{\mu}$ | $\Omega_{0}$ | $R$  |
|--------|-----------|--------------|------|
| 8R1.5  | 2.32      | 3.33         | 7.75 |
| 8A1.5  | 1.90      | 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      | 1.80         | 3.75 |
| 8R1.5W | 5.06      | 1.44         | 7.26 |
| 8A1.5W | 5.06      | 1.46         | 7.26 |
| 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 |

Table 9. Seismic parameters of the 4-story Moment-Resisting Frame (MRF) models.
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.

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-
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 (MRF SW).

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

- 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 MRF SW, variation in the number of stories has a greater effect on the RMF;
- For the case of MRF SW 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...
Table 12. Seismic parameters (average values) of structures.

| Number of stories | L/H  | R (for MRFSW) | R (for MRF) |
|-------------------|------|---------------|-------------|
| Low-rise building structures |      |               |             |
| 4                 | 1.5  | 4.46          | 9.54        |
|                   | 2.5  | 5.36          | 4.42        |
|                   | 3.5  | 4.95          | 3.33        |
| Mid-rise building structures |      |               |             |
| 8                 | 1.5  | 7.33          | 7.65        |
|                   | 2.5  | 6.39          | 4.47        |
|                   | 3.5  | 7.1           | 3.8         |
| High-rise building structures |      |               |             |
| 12                | 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.

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Biographies

Seyed Shaker Hashemi was born in Kazeroun, Iran, in 1981. He received a BSc degree in Civil Engineering from the Power and Water University of Technology (PWUT) in 2003 and MSc and PhD degrees in Earthquake and Structural Engineering from Tarbiat Modares University, Tehran, Iran, in 2005 and 2009, respectively. He is currently Assistant Professor in the Department of Civil Engineering at Persian Gulf University, Bushehr, Iran, and his research interests are in the areas of structural and earthquake engineering, especially nonlinear numerical analysis of reinforced concrete, and the design of structures.

Kabir Sadeghi is currently a Professor of Structural Engineering at the Near East University. He received the PhD and DEA degrees in Civil (Structural) Engineering from the University of Nantes/Ecole Centrale de Nantes, France, and the MSc degree in Structural Engineering and the BSc degree in Civil Engineering from Amir kabir University of Technology, Tehran, Iran. He has published two books and over 85 articles in SCI journals, international journals, and conferences. His research interest is the nonlinear numerical simulation of RC buildings.

Mohammad Vaghefi was born in Shiraz, Iran, in 1973. He received BSc and MSc degrees in Civil Engineering from Shiraz University, Iran, in 1997 and 1999, respectively, and PhD degree in 2009 from Tarbiat Modares University, Tehran, Iran. He is currently Associate Professor of Civil Engineering. He has published about 70 journal papers and presented about 220 papers at national and international conferences.

Seyed Alireza Siadat was born in Bandar-e Emam Khomeyni, Iran, in 1990. He received BSc degree in Civil Engineering from Islamic Azad University, Abadan Branch, Khoozestan, Iran in 2013 and MSc degree in Structural Engineering from Islamic Azad University, Bushehr Branch, Iran, in 2016. His research interests are in the areas of structural engineering.