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# The Effect of the Arrangement and Length of the Concrete Shear Walls on the Response Modification Factor (R)

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ABSTRACT: In equivalent static analysis procedure, the design seismic force is affected by the response factor related to the inelastic behavior of the structure. This factor, whose value is used to calculate the amount of the energy damping, absorbed by the structure, depends on some parameters such as ductility and over strength. It is regarded as a constant coefficient for each type of structural systems in seismic codes. However, in dual structural systems, the effect of features like the geometry of the structure and lateral forces resistance system on this coefficient is not taken into account. This research is aimed at investigating the effect of the arrangement and length of the shear wall in the plan on the response modification factor in dual concrete systems. To do this, 15 concrete buildings as high as 30 to 45 meters were analyzed by nonlinear static method, using Perform 3D software. The initial estimation of the shear wall length was based on resisting 75 percent of the design base shear force by the shear walls. Next, the over strength factors, ductility coefficients, and subsequently the response modification factors of the models were determined and compared to the value used in the design procedure, presented in ASCE7 code. The results indicated that the value of response modification factor, in comparison with the presented value in ASCE7 code, was varied between -18% to +25% over changing the arrangement of the shear walls, and increased for up to +32% by increasing the length of the shear wall in the plan as much as 100% in proportion to the original model.

*Keywords: Response Modification Factor, Ductility, Over Strength, Concrete Shear Wall Length and Arrangement*

## 1 INTRODUCTION

In dual concrete systems (moment frame with shear wall) there are various factors influencing the system's performance and lateral forces distribution among the elements of the structure. In this system the behavior of the frame and the shear wall individually is totally different from their interactive behavior [1]. In this regard, the geometrical shape, length, thickness, and the location of the shear walls inside the frames all affect the system's performance. Also knowing these factors and their effects plays a significant role in optimizing the structural design.

In dual concrete systems, the shear wall can be located either in the frame plane or outside of it. If inside the plane, the wall can be connected to the frame's columns in its both sides, which is called the "attached wall" in this paper, or separated from the column in its one or both sides which is named "detached wall" here (Fig.1). In 1986, Kayal made some surveys on the frame-wall interference in reinforced concrete structures under the effect of the gravity and lateral loads and studied the effect of four factors which are the stiffness ratio of column to shear wall, the stiffness ratio of beam to column, the columns' slenderness ratio, and the ratio of lateral to gravity loads [2]. In 1991, Coul and Smith investigated the effect of decreasing the height of both the shear wall and the central super column on the stiffness of frame-wall system [3].

In the structural design process, the architecture imposes some limitations on the location of the required shear walls on the structure's plan. These limitations cause different behaviors of the shear walls depending on their length-to-height ratios which include moment, shear, and interactive behavior. Moreover, the value of the seismic parameters like ductility, over strength, and response modification factor for each case of walls attached to and detached from columns would be different.

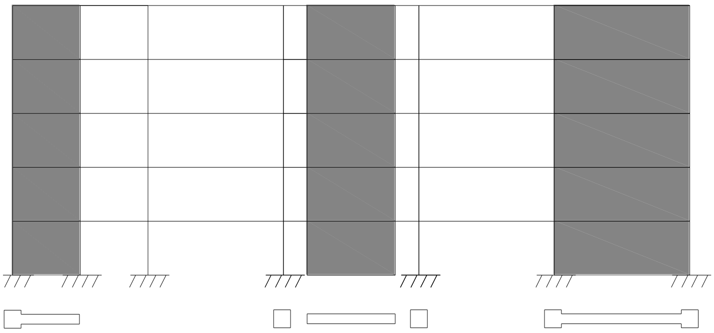


Figure 1. Shear walls attached to or detached from the frame's columns

In dual concrete systems, the effect of features such as the length of the wall and its arrangement in the structure's plan on the value of response factor is generally ignored, and the only obligation in codes like ASCE7-10 is that the moment frames must be individually able to resist at least 25% of the design seismic force [4] so that if this condition is not satisfied, the system is not considered a dual system.

The aim of this study is to investigate the effect of the arrangement and length of the shear wall in the structure's plan on the seismic parameters and determining the response modification factor of the structures with dual concrete systems (moment frame with shear wall). To do this, structures with 10 and 15 stories having different wall arrangements were studied as follows:

a. Locating the shear wall in the frame plane, completely in one span and attached to the nearby columns
b. Locating the wall in the frame plane but in several spans and detached from the nearby columns.

Afterwards, the results were presented in a table and compared to the value of the response modification factor which was used in design process of the structural models according to the code requirements. Then, the results were discussed using diagrams. Finally a 9-story constructed building was examined to compare the values of the response modification factor in two different directions.

## 2 ESTIMATION OF THE SHEAR WALL PRIMARY LENGTH FOR DESIGN OF THE SRUCTURE

In 1995, Wallace examined some concrete buildings whose structural systems were a combination of moment frames and T-shaped shear walls, using the nonlinear analysis method [5]. He developed an equation based on the UBC97 code for estimating the needed in-plan area of such walls. Kheyroddin in 2010, using a program called NONLACS and both linear and nonlinear analyses, examined some reinforced concrete structures which were designed based on the requirements in codes ASCE7 and ACI318. He suggested an equation for estimating the minimum needed area of shear walls in a reinforced concrete building with symmetrical rectangular walls [6].

The method used in this paper for estimating the primary length of the shear wall is based on the shear strength definition of a section. The wall must be able to resist the shear force exerted from the lateral seismic load in all directions. According to the code ACI318, the shear strength for a section (the nominal shear capacity of the section or Vn) is calculated from the summation of the shear strength for the concrete (Vc) and the steel (Vs) [7].

$$V_u = \varphi\, V_n \ \le \varphi(V_c + V_s) \tag{1}$$

where:

$$V_c = \frac{1}{6}\sqrt{f'_c}\,.\,t_w\,.\,d \tag{2}$$

$$V_s = \frac{A_v\, f_y\, d}{s} \tag{3}$$

$$A_v = \rho.\,t_w.\,s \tag{4}$$

In these equations, ***d*** is the effective depth and is equal to $0.8l_w$ according to ACI318-05 [7] where $l_w$ is the horizontal length of the shear wall, $s$ is the shear bars space, $A_v$ shear reinforcement area, $\rho$ the shear reinforcement ratio and $t_w$ the shear wall thickness.

After replacing $V_c$ and $V_s$ with their equations based on the ACI318-05 and considering the least value for the shear reinforcement of the shear walls ($\rho$ =0.0025) and simplifying, the equation (5) is derived [7].

$$\varphi V \leq \varphi(\tfrac{1}{6}\sqrt{f'_c} \times t_w \times 0.8\, l_w + 0.002 \times t_w \times f_y \times l_w) \quad (5)$$

In this equation, V is the base shear force in the wall which is 75 percent of the total shear force (seismic design force), and φ is equal to 0.75. In addition, according to ACI318-05, the base shear force should be increased by a factor of 1.4 for ultimate strength design [7]. Therefore, equation (5) can be simplified to reach equation (6).

$$l_w \geq \frac{1.05V}{\left(0.133\sqrt{f'_c} + 0.002 f_y\right) t_w} \quad (6)$$

where $l_w$ is in millimeters, V in kN, and $f'_c$ and $f_y$ in MPa.

The final length and location of the shear wall in each direction in the plan may be designed smaller or bigger than this estimated value depending on the structural and architectural considerations.

In this research, the required shear wall length of the structural models was calculated using equation (6). Then the shear walls' thickness was set to satisfy the code requirements for designing the structures.

## 3 THE DERIVATION PROCEDURE OF THE RESPONSE MODIFICATION FACTOR

The seismic codes apply the Response Modification Factor in definition of the lateral forces so that the seismic design force is calculated by dividing the seismic elastic force by the response factor (equation (7)) [8,9].

$$V = \frac{V_e}{R} \quad (7)$$

where V is the design base shear force, and $V_e$ is the base shear force corresponding to the elastic response of the structure or the maximum base shear.

In nonlinear static analysis, the response factor of the structure can be determined using the shear strength-displacement diagram and the design spectrum of the code. In 1991 Uang, using the structure's capacity curve (Fig.2), introduced the equation (8) for calculating response factor [8].

$$R = \Omega . R_\mu \quad (8)$$

Ω is known as the over strength which is calculated through the equation (9). In this equation $V_y$ is the overall yield strength and Vs is the first yield strength of the structure.

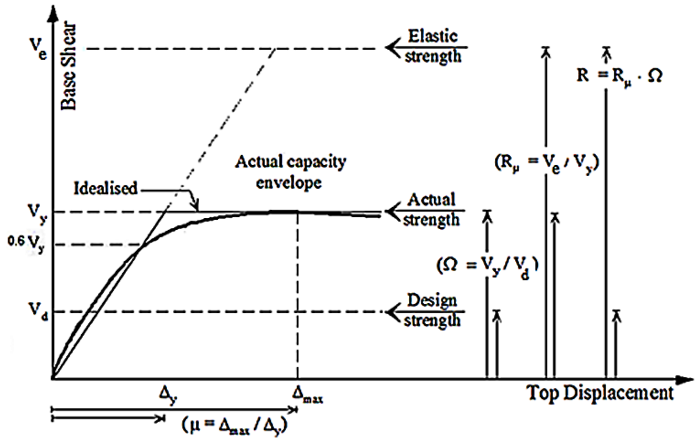



Figure 2. Relationship between force reduction factor (R), over strength ($\Omega_d$), ductility reduction factor ($R_\mu$) and displacement ductility factor (μ) [8]

$$\Omega = \frac{v_y}{v_s} \quad (9)$$

Rμ is the force reduction factor which depends on the total ductility of the structure (μs) and is calculated through equation (10).

$$R_\mu = \frac{v_e}{v_y} \quad (10)$$

μ is the capacity of energy damping for the whole structure. This coefficient is defined as the ratio of the maximum lateral displacement ($\Delta_{max}$) to the yield lateral displacement ($\Delta_y$):

$$\mu = \frac{\Delta_{max}}{\Delta_y} \quad (11)$$

There are several studies by Newmark, Krawinkler, Uang, Miranda, and Bertro carried out for numerical calculation of the ductility factor (μs) and the force reduction factor due to ductility (Rμ). In the present study, with presumption of 5% damping and basement with sedimentary soil, equation (12) from Miranda[10] and equation (13) from Krawinkler [11] are used which are more suitable for the models under the study.

$$R_\mu = \frac{\mu-1}{\varphi} + 1\,, \varphi = 1 + \frac{1}{12T-\mu T} - \frac{2}{5T} e^{-2(\ln(T)-0.2)^2} \quad (12)$$

$$R_\mu = [c(\mu-1)+1]^{\frac{1}{c}} \;,\; c\,(T,\alpha) = \frac{T^a}{1+T^a} + \frac{b}{T} \quad (13)$$

where T is the fundamental period of the structure, and α is the post-yield stiffness which is expressed as a percentage of the elastic stiffness; a and b are the coefficients depending on α. Table 1 shows the values for these coefficients suggested by Krawinkler and Nasr for different values of α.[9]

Table 1. Values for the coefficients “a” and “b” suggested by Krawinkler and Nasr [9]

| α | a | b |
|---|---|---|
| 0.0 | 1.0 | 0.42 |
| 0.02 | 1.0 | 0.37 |
| 0.1 | 0.8 | 0.29 |

## 4 MODELING THE SHEAR WALLS IN DUAL CONCRETE SYSTEMS FOR NON-LINEAR ANALYSES

Software programs used for non-linear analyses have different features and limitations. In some, like ETABS or SAP2000, approximate methods and equalization are used, which is easy in practice but makes it difficult to access the real performance of the structure and its components [12].

PERFORM3D, however, provides the possibility of non-linear modeling using the shell elements in the wall and removes the need for the frame elements while modeling different parts of the wall. These elements are defined using some fibers which are suitable for modeling the geometry of the wall section as well as considering the non-linear behavior of materials. It is possible to enhance the model’s accuracy using the fiber model without needing precise analysis of the finite elements. The important modeling parameters, which must be defined, include the material properties for the reinforcement, concrete core separated by shear reinforcement and concrete cover [13].

PERFORM3D has some advantages as follow:

a. It is able to show the non-linear behavior of the structure due to inelastic behavior of the materials.
b. It is possible to consider the non-linear structural behavior created by inelastic behavior of the elements including stiffness, strength, and the capacity of deformation in elastic and plastic zones regarding M-φ diagram, seismic performances of the life safety, and collapse prevention.

To model the elements in this software program the features of the constituent layers must be introduced which include axial/bending layer, diagonal concrete layer, and shear concrete layer. Fig. 3 shows the wall element separated into these layers and the direction of their operations [13].

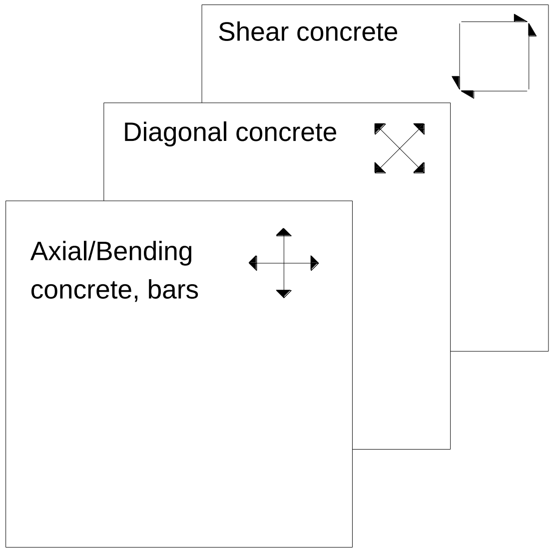



Figure 3. Wall element separated into layers and the direction of their operations

In the software, the solution procedure was done using matrix analysis method and solving the Duhamel integral was based on the Newton-Raphson method.

- Calibrating the software results with the experimental test results

To predict the non-linear behavior of the shear walls under the lateral forces, some simple analytical models are needed which can accurately show the non-linear behaviors of the shear walls in comparison to experimental samples. To do this, two investigated experimental samples were selected whose geometrical and mechanical characteristics are listed in Table 2 and their test results are presented in Table 3. The first sample (named wall1) was experimented by Wiradinata and Sachioglu [14] and the second one (M4) is taken from Grifenhagen’s experiment [15].

In Table 2, $h_w$ is the wall’s height, $l_w$ the wall’s length, t the wall’s thickness, $h_b$ the height of top beam, $w_b$ the width of the top beam, $\rho_v$ the longitudinal reinforcement ratio, $\rho_h$ the shear reinforcement ratio, $f_{vy}$ the yield stress of the longitudinal reinforcement, $f_{hy}$ the yield stress of the horizontal reinforcement, and $f'_c$ the compressive strength of the concrete.

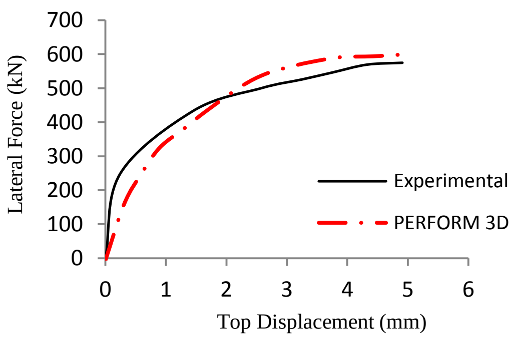
Table 2. Geometrical and mechanical characteristics of the material used in experimental tests

| Specimen | $h_w$ (mm) | $l_w$ (mm) | t (mm) | $h_b$ (mm) | $w_b$ (mm) | $\rho_v$ % | $\rho_h$ % | $f_{vy}$ (MPa) | $f_{hy}$ (MPa) | $f'_c$ (MPa) |
|---|---|---|---|---|---|---|---|---|---|---|
| Wiradinata, Saatciuglu (wall1) [14] | 1000 | 2000 | 100 | 300 | 500 | 0.8 | 0.25 | 435 | 425 | 25 |
| Greifenhagen ($M_4$) [15] | 610 | 900 | 80 | 500 | 350 | 0.3 | 0.3 | 504 | 745 | 24.4 |

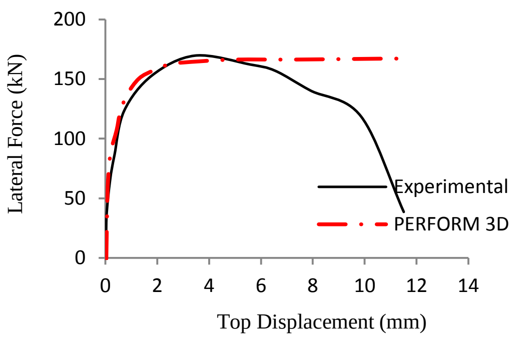
Table 3. The experimental tests' results

| Specimen | Axial Force (kN) | Failuremode |
|---|---|---|
| Wiradinata Saatciuglu (wall1) [14] | 0 | Diagonal tension (Horizontal bars' yeild) |
| Greifenhagen (M4) [15] | 76 | Buckling (Longidudinal bars' yeild) |

The stress-strain curves which were used for the concrete and reinforcement in PERFORM 3D were elasto-plastic based on the Mander pattern in the software settings. Also to model the concrete behavior two types of layers were used: the first one showed the shear behavior, and the second served to model the axial behavior of the concrete. Fig.4 shows the response curves (lateral force-displacement) for both the experiment and the analysis.




4-a: Wall1




4-b: M4

Figure 4. The response curves resulted from the experiment and the analysis

In these analyses, the limit of displacement for stopping the analysis was introduced to the software program as much as the limit of displacement in the experiment.

In the first sample, whose response curve from both the analytical sample and the experimental one are presented in Fig. 4-a, the overall diagram and the strength are estimated very well. Although the stiffness in the elastic zone of the analytical curve is derived below the real curve, the ultimate strength in the analysis is almost identical with that of the experiment.

In the second sample, in the analyses where the loading distribution applied to the model is uniform, it is possible to calculate the amount of change in the applied force in each step based on the model stiffness of each previous step. This change in some cases is decreasing and is shown as the degradation of the strength in the response curve.

The push-over curve derived from experimental model M4 (Fig. 4-b) will experience a collapse at the end of the loading, but in the analytical model the deterioration of strength in this zone does not happen because of the lack of required accuracy in the non-linear analysis of the layer element in estimating the amount of deformation estimating. Despite all these, both diagrams have an acceptable maximum strength point which is before the experimental curve starts to descend. Regarding the criteria used for the destruction of the components, which will be discussed later, the analytical models within the required range of the response curve show the behavior of the shear wall very well compared to the experimental results.

## 5 PREPARING MODELS FOR NON-LINEAR ANALYSES

### 5.1 The structural models in the study

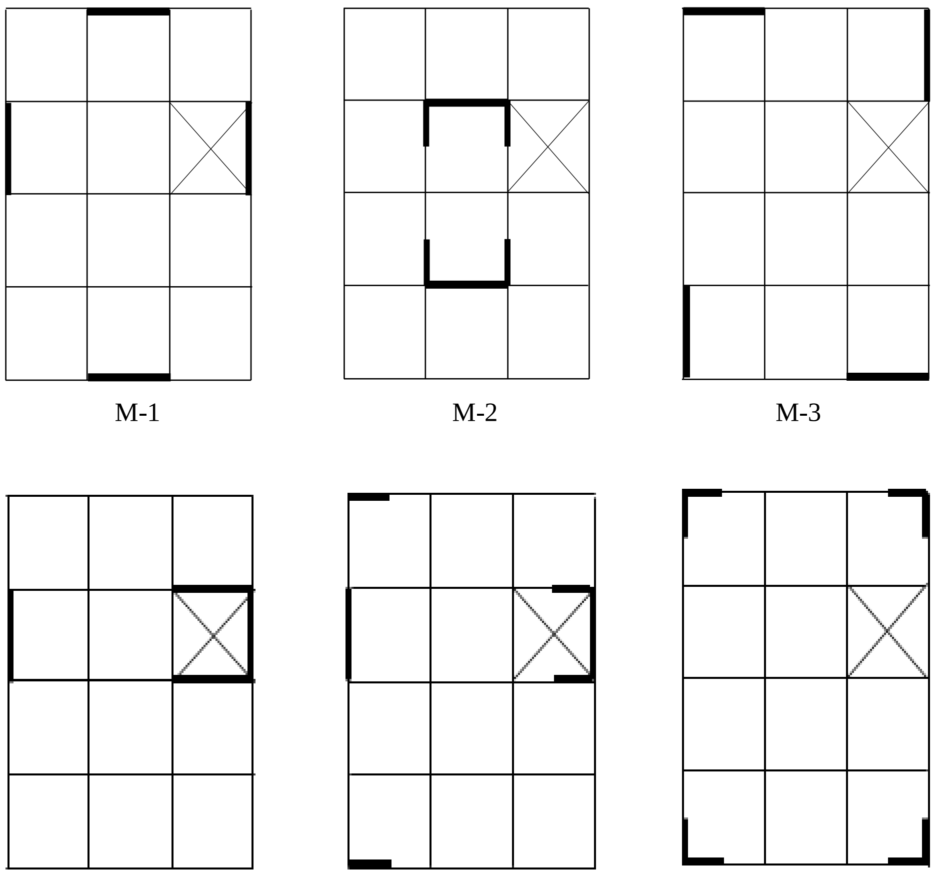



Fig. 5: The designed structures' plans used for investigating the effect of the arrangement of shear walls on the response factor

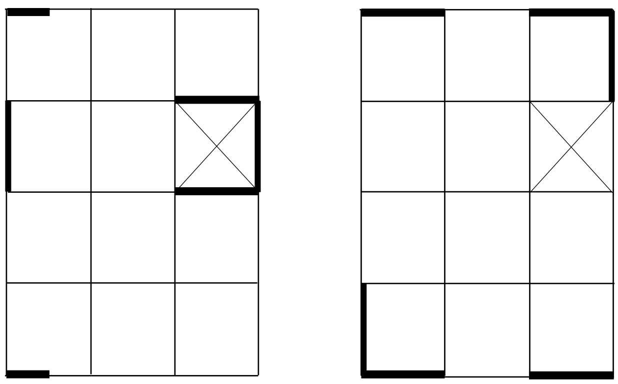



Fig. 6: Plans of the designed structures used for investigating the effect of the arrangement of shear walls

To investigate the effect of the arrangement of shear walls in RC dual systems, 6 structures with 10- and 15-stories were selected as shown in Fig. 5 in which the height of each story is 3 meters, the spans along X and Y axes are respectively 4 and 5 meters. Also to study the effect of increasing the in-plan length of shear walls on the response factor, two 10-story structures were selected as in Fig. 6. Loading of the structures was done according to ASCE7 code; the seismic design load factor was calculated presuming the ground soil type D and the zone category of high relative risk for earthquake. The used material properties are given in Table 4.

Table 4. The properties of the used material (MPa)

| $f'_c$ | $E_c$ | $f_y$ | $E_s$ |
|---|---|---|---|
| 30 | 26700 | 400 | 210000 |

Designing each model was done separately using ETABS v.9.5 and the concrete code ACI 318-2005 for sway intermediate systems. It should be noted that in structures with 10 and 15 stories with the same arrangement and length of the shear walls, it is clear that the width of the walls and length of the boundary element as well as the dimensions of the frame's components will be different.

## 5.2 The Characteristics of the Non-Linear Static Analysis

One of the most important stages in non-linear static analysis is selecting the suitable distribution of lateral load. During an earthquake, the story shear force distribution along the height of the structure can be different in various times. Therefore codes offer various types of lateral load distributions for analysis. In the present study, the first type of distribution in FEMA356 instruction was used which is the triangular load pattern [16]. After designing the required structures, PERFORM 3D V.4 was used for the non-linear static analysis (push-over analysis), drawing the response curves of the structures, idealizing the curves (bilinear form), and finally calculating the seismic parameters and the response factor.

The plastic hinges were determined to form at the beginning and end of the beams, columns and shear walls. The M-ϕ diagram of each element type was derived from the section designer in SAP2000 software and then imported to PERFORM 3D manually.

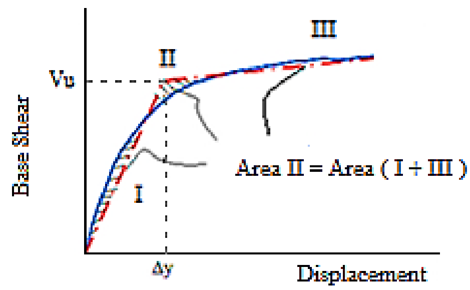
## 5.3 Optimizing and Simplifying the Capacity Curve

To simplify the non-linear behavior of the structure, in the relation between the base shear and the displacement of the control point, it is necessary to idealize the response curves. To do this, the bilinear form can be used. Therefore, using these idealized curves and according to the code criteria, the amounts of the yield and collapse limits are determined.

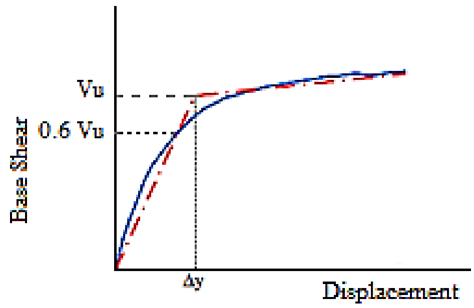
Researchers use different definitions to idealize the curves for estimating the yield displacement in structures. The most popular definitions which are referred to in FEMA356 instructions are as follow [16]:

a. The yield displacement in the idealized elasto-plastic system whose energy absorption is similar to real one. In this model, like Fig. 7-a, the area under the bilinear curve is equal to the area under the initial non-linear behavior curve [9,17].
b. The yield displacement in the idealized elasto-plastic system when the reduced stiffness is reached by rotating the line of the elastic zone. In this definition, as seen in Fig. 7-b, the effective yield shear (Vy) should be determined so that the crossing point of its elastic zone line and the curve from the real behavior occurs at 0.6 $V_y$ [9,17].

In this study, both criteria were used simultaneously using trial and error method so that the final bilinear curve satisfied both criteria.




7-a




7-b

Figure 7. Two methods of idealizing the response curves [9,17]

The failure criteria used in the present study are as follow:

a. The criterion of destruction of the element (this criterion is the element's curvature exceeding its allowable value)
b. Stability criterion (by controlling the stability index)
c. The criterion of the relative displacement between the stories. In the FEMA356 instructions [16] and also in Table 12-12-1 of the ASCE7 code [4] for the structures with shear wall and the performance level of LS, the maximum lateral displacement of the roof control point is limited to 1.5% of the structure's total height.

All the above criteria were used to determine element failure through the analyses. In other words, the first criterion occurrence was regarded as the element failure.

## 6 RESULTS OF THE NON-LINEAR STATIC ANALYSES

After analyzing the models, the response curves (the base shear vs. roof displacement diagram) were drawn along with their bilinear response curves for X direction as shown in Fig. 8.

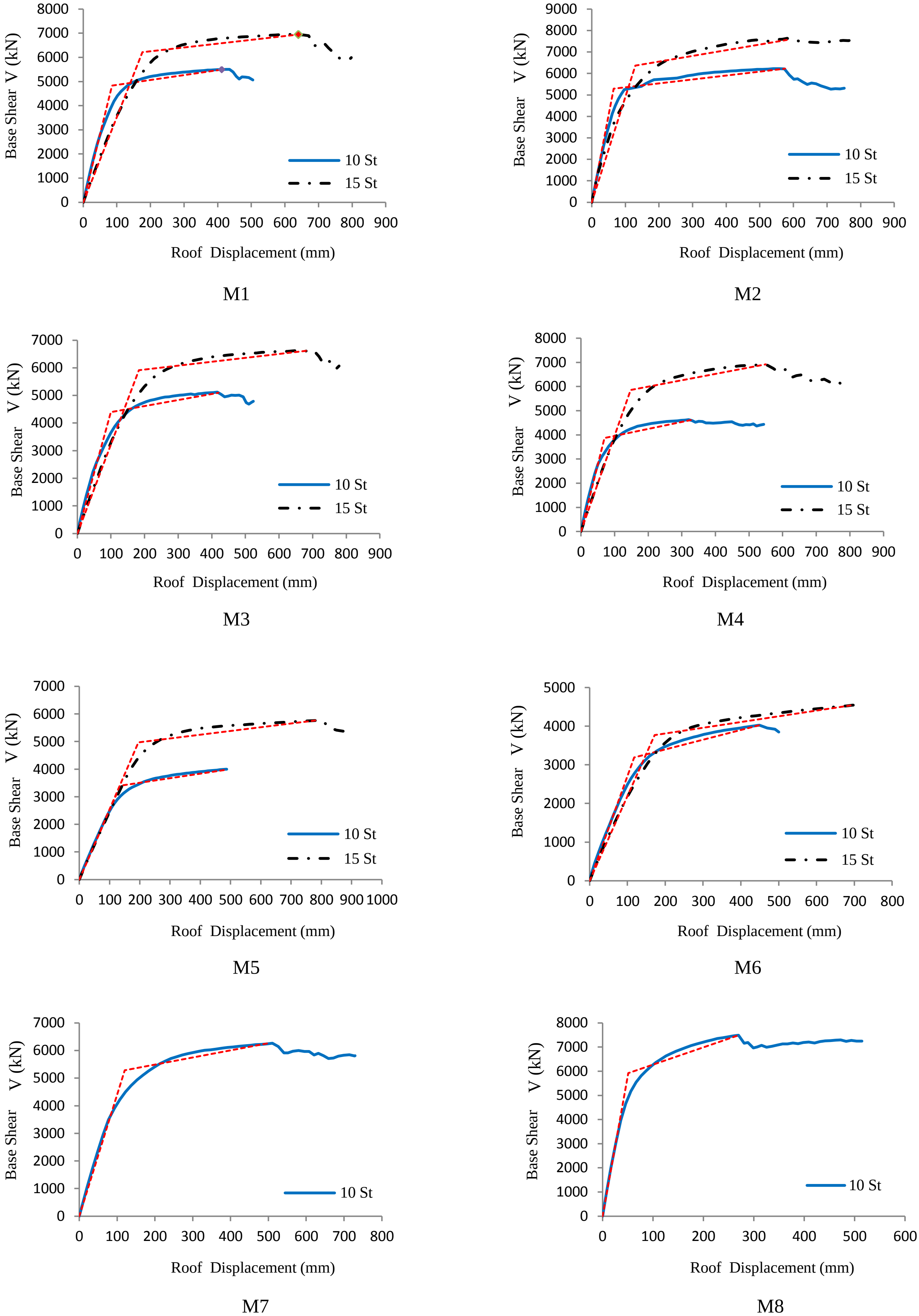



Fig. 8: Response curves with their bilinear response curves for X direction

## 6.1 Calculating the response modification factor (R)

Having the structures' response curves, it is possible to calculate the over strength factors, ductility, and response factor through equations (8) to (13). To calculate the response factor, the average amount of reduction factor due to the ductility obtained from both Miranda [10] and Krawinkler [11] methods in section 3 was used. The response factor calculated for each structure in X direction as well as the used response factor from the code for designing the structures is presented in Table 5.

## 6.2 Discussion on the results

The changes in seismic parameters with the change in the arrangement and length of the wall are shown for each model in Fig. 9. The different values of the response modification factor for various models shows that in the dual concrete system, this factor depends directly on the length and arrangement of the shear walls.

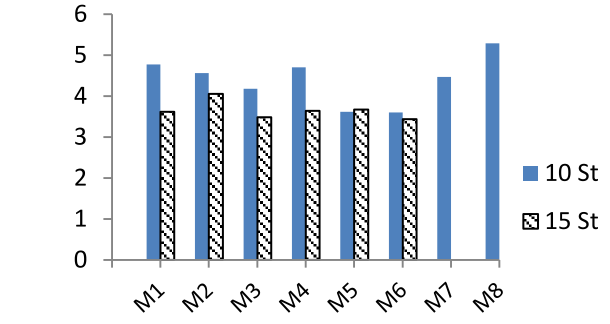
- The effect of arrangement of the shear wall

Regarding the results, locating the shear walls in a symmetrical position so that their rigidity center can be placed close to the mass center of the whole structure leads to a higher value for the response modification factor. In this situation, locating the walls on the perimeters of the plan in a way that there is the maximum distance between them or placing them near the center of mass and rigidity

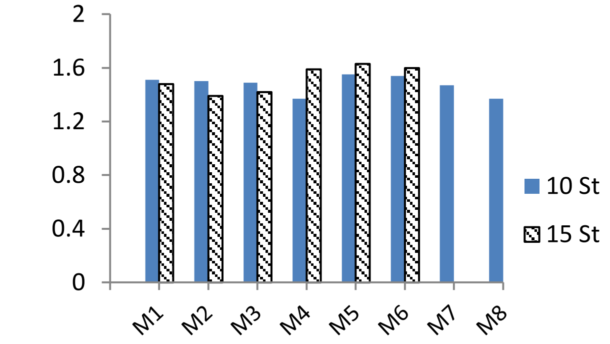
Table 5: Seismic parameters resulted from nonlinear analyses for X direction and comparing the computed R with the value of R from the code

| Model | μ | $R_\mu$ | | $R_\mu$(AVG) | Ω | R | $\frac{R}{R_{(ASCE)}}$ |
|---|---|---|---|---|---|---|---|
| | | Miranda | Krawinkler | | | | |
| M1-10st | 4.77 | 5.58 | 4.38 | 4.97 | 1.51 | 7.5 | 1.25 |
| M1-15st | 3.62 | 4.49 | 3.45 | 3.97 | 1.6 | 6.3 | 1.05 |
| M2-10st | 4.56 | 5.3 | 4.24 | 4.77 | 1.5 | 7.2 | 1.2 |
| M2-15st | 4.05 | 4.82 | 3.4 | 4.11 | 1.63 | 6.7 | 1.12 |
| M3-10st | 4.18 | 4.92 | 3.82 | 4.37 | 1.49 | 6.5 | 1.08 |
| M3-15st | 3.48 | 4.24 | 4.36 | 4.3 | 1.59 | 5.9 | 0.98 |
| M4-10st | 4.7 | 5.34 | 4.28 | 4.81 | 1.369 | 6.7 | 1.12 |
| M4-15st | 3.64 | 4.51 | 3.47 | 3.99 | 1.42 | 5.7 | 0.95 |
| M5-10st | 3.62 | 4.28 | 3.28 | 3.78 | 1.55 | 5.9 | 0.98 |
| M5-15st | 3.67 | 4.0 | 3.02 | 3.51 | 1.39 | 4.9 | 0.82 |
| M6-10st | 3.6 | 4.24 | 3.66 | 3.95 | 1.54 | 6.1 | 1.02 |
| M6-15st | 3.44 | 3.95 | 3.45 | 3.7 | 1.48 | 5.5 | 0.92 |
| M7-10st | 4.47 | 5.15 | 4.65 | 5.08 | 1.47 | 7.5 | 1.25 |
| M8-10st | 5.29 | 6.13 | 5.39 | 5.76 | 1.37 | 7.9 | 1.32 |

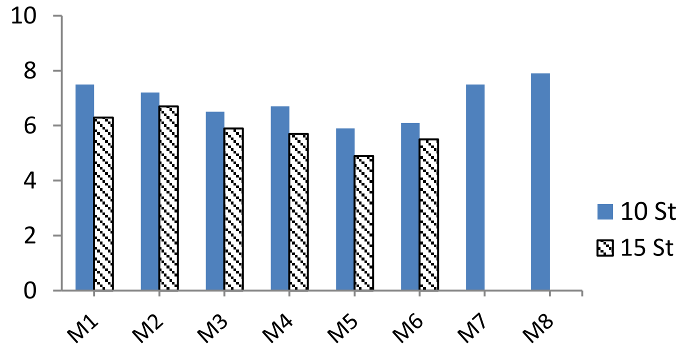
(that is, the walls act like a central concrete core), will lead to maximum response factor value.




9-a. Ductility factor




9-b. Over-strength factor




9-c. Response modification factor

Figure 9. The comparison of seismic parameters in the models

• The effect of attaching the shear walls to or detaching them from columns

Table 5 and Fig. 9 illustrate the effect of the wall location on the response factor in two states; the wall attached in both sides to the columns (M1, M2, M3, M4) and the detached walls (M5, M6). After comparing the results in models with similar height, it was clear that in models with detached walls in some spans, the reduction factor due to ductility (which affects the response factor) was below the values from the models with complete walls in spans. However, the over strength factor, which is another important factor in determining the response factor, was highest in models with detached shear walls. In this case, the variation of response factor value doesn't follow any specific pattern.

In the model M6 with detached walls, although the shear walls have an optimum arrangement and the whole structure satisfies the code requirements (about lateral displacement, accidental torsion, etc.) better than the models M3 and M4 with attached walls during the design process, the value of response factor is attained less than that of M3 and M4.

• The effect of structure's height

In the models with attached walls, with increasing the structure's height, the over strength factor increases, while the reduction factor due to the ductility decreases. But in the models with detached walls, these two factors do not follow any fixed pattern because the amounts of change in the seismic parameters affecting this factor are not the same. However, in most cases, the value of the response factor decreases as the number of the levels increases.

• The effect of increasing the shear wall's length

The model M7 is derived from increasing the wall's length of the model M4 or M5 in X direction as much as 1.5 times (increasing 4 meters to the shear wall) and then re-designing the new structure. In this situation, as shown in Fig. 9, the over strength factor decreases while the ductility factor increases. This amount of change in seismic parameters is as follow: with 50% increase in the length of the shear wall in these models, the numerical value of the response factor increases as much as 25% on average. Also, with 100% increase in the shear wall and converting the structural model M6 to model M8, the factor increases for 30% of its original amount. In comparison, the response factor in model M8 compared to model M7 increases for only 5%. To state the matter differently, the response factor increases directly with lengthening the walls in the structure's plan but this upward trend does not have a constant rate. The change rate of this factor will go down with the increase in the length of the wall to reach a stage where the factor stays constant for any increase in the shear wall length.

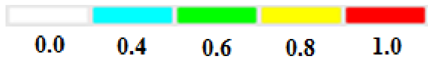
• The process of developing the plastic hinges through the Analysis Procedure

Fig. 10 shows the effect of the attached and detached walls in the plan on the process of developing the plastic hinges in the structure's elements through the analysis procedure. In models M5 and M6 with detached shear walls in the plan, although the structure has not yet reached the mechanism and instability in the end point of the curve, the non-linear analysis was aborted based on the displacement criterion of 1.5%. The process of formation of the plastic hinges in the frame's elements and the wall was as follow: first, the shear walls in the first three levels were yielded and their hinges were formed, then the beams adjacent to the walls reached the destruction criterion, that is, the end rotation of the components regarding the seismic performance in the instruction FEMA356 exceeded the allowable level of seismic performance of LS. The process of developing the hinges in the beam elements was from lower levels towards the upper ones in the frames including walls. After that the other beams in the frames without a wall started to form plastic hinges with the same sequence. In the end, the columns adjacent to the shear walls began to form plastic hinges. It should be noted that in the process of formation of the hinges, the beams which are linked to the detached walls in the plan did not reach the seismic performance level because in designing the model M5, the linking points of these beams were designed as flexural plastic hinges due to the structural limitations.

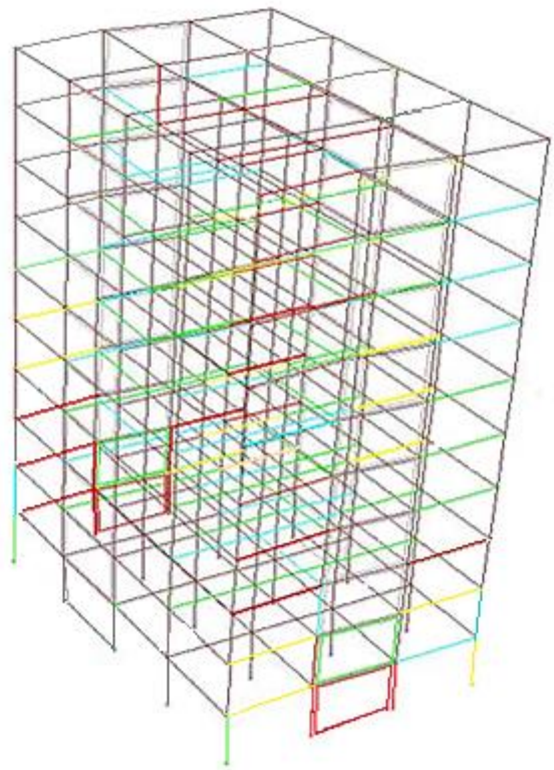
Investigating the process of developing the plastic hinges in the structure model M1 indicated that after the structure entered the non-linear zone, the plastic hinges in the elements of the frame in seismic performance level LS were formed first in the shear walls of the first two levels and then in the beams. The beginning of this process in the beams was from the beams adjacent to the shear wall in the lowermost and uppermost frames in the plan along the X direction and from lower levels to the upper ones, then in the beams next to the stairs (opening) in the X direction and next to the walls in Y direction, and finally the plastic hinges were formed in the columns. When designing the whole structure some of the beams in the first two levels were designed very strong due to their position in the structure; therefore, these beams did not reach the seismic performance level until the end of the analysis.

## 7 COMPARING THE RESPONSE FACTOR IN BOTH DIRECTIONS

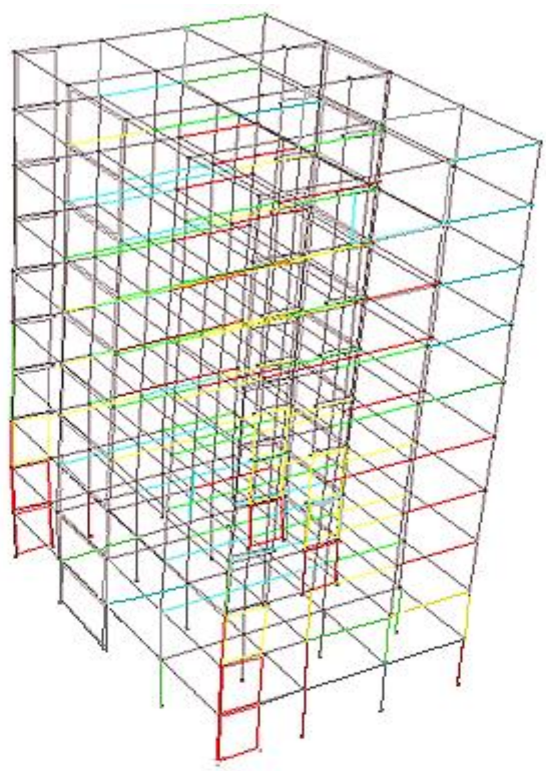
In this part, the response factors in both directions X and Y are calculated for a 9-story structure which is already designed and constructed. Although the lateral resistant force systems in both directions of this structure are the same, the arrangement and length of the shear walls are different in each direction due to the architectural limitations. Therefore, it is reasonable to compare the response factor in two different directions of a structure.




Minimum usage ratio in Limit State for LS performance level



10-a. M1-10 St



10-b. M5-10 St

Figure 10. The plastic hinges in the structure's elements through the analysis procedure in X direction

The surveyed structure's plan and its shear walls' arrangement are shown in Fig. 11. According to the equation (6), the required wall length for each direction of this structure is estimated 11.5 meters, while the designed length in X direction is 9.5 meters and in Y direction is 13 meters.

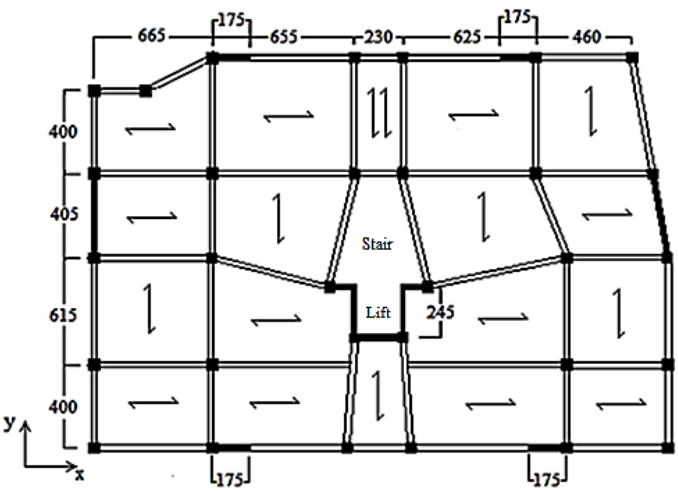



Figure 11. The surveyed structure's plan and its shear walls' arrangement to compare the response factor in two different directions

The results from calculating the seismic parameters are presented in Table 6.

In this structure, the walls' length in Y direction is 42% longer than that in X direction. Also the walls in Y direction are attached to the columns in both sides while the X direction's walls are mostly detached. Therefore, despite using the same structural system and a relatively optimum arrangement for the shear walls in both directions, the value of the response factor in Y direction is 40% more than that in X direction.

The results show that it is not reasonable to assume an identical value of response factor for both directions merely because of using the same dual concrete structural system in two directions. The value of this factor in the structure depends on other factors including the location, length and the thickness of the shear walls in each direction and their effects on the structure's behavior in the orthogonal direction, the number of frames and spans, and length of each span. However, the code considers the response factor for both directions identical.

## 8 CONCLUSIONS

In this study, after designing the structural models based on the code's offered response modification factor, calculating their response factors using non-linear analysis and comparing the results with the code's suggested factor, the following conclusions were reached:

a. In the models with detached walls in some spans, the reduction factor due to ductility, which affects the response factor, were less than the values from the walls with complete walls in spans. However, the over strength factor, which is another important factor in determining the response factor, was highest in models with detached shear walls. In this case, the variation of response factor value does not follow any specific pattern.

b. In the models with attached walls, with increasing the structure's height, the over strength factor increases, while the reduction factor due to the ductility decreases. But in the models with detached walls, these two factors do not follow any fixed patterns because the amounts of change in the seismic parameters affecting this factor are not the same. However, in most cases, the value of the response factor decreases as the number of the levels increases.

c. Regarding the results, locating the shear walls in a symmetrical position so that their rigidity center can be situated as close as possible to the rigidity center of the whole structure leads to higher value for the response modification factor. In this situation, locating the walls on the perimeters of the plan in a way that there is the maximum distance between them or placing them near the center of mass and rigidity (like a central concrete core), leads to maximum value for the response modification factor.

d. The response factor increases directly with increasing the walls' length in the structure's plan but this upward trend does not have a constant rate. The change rate of this factor will go down with the increase in the length of the wall to reach a stage where the factor stays constant for any increase in the shear wall length.

e. It is not practical to assume an identical value of response factor for both directions merely because of using the same dual concrete structural system in two directions. It is affected by other factors such as the location, length and the thickness of the shear walls. However, the current codes present an identical response factor for both orthogonal directions.

f. Since no seismic design code has determined a specific size for the shear wall's length in dual systems, it is secure to use equation (6) for a primary estimation of its length, because in the 10-story models which are designed using this equation, the numerical values of the response factors calculated from non-linear analysis show conservative results compared to the values suggested by the code.

Table 6: Seismic parameters for 2 different directions of the structure

| Direction | μ | $R_\mu$ | | $R_\mu$(AVG) | Ω | R | $\frac{R}{R_{(ASCE)}}$ |
|---|---|---|---|---|---|---|---|
| | | Miranda | Krawinkler | | | | |
| x | 3.16 | 3.27 | 3.39 | 3.33 | 1.46 | 4.9 | 0.82 |
| y | 3.76 | 3.83 | 4.11 | 3.97 | 1.73 | 6.9 | 1.15 |

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