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Original Article

Determination of Seismic Response Modification Factor for RC Wall-Frames Structural Systems

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Abstract *- In this research, the response modification factor for various reinforced concrete wall-frames systems is calculated using the pushover analysis, which considers seismic loads exceeding their limitation. Different moment-resistant frames with rectangular shear walls, L-shape shear walls, one core, a combination of rectangular shear walls and one core, and L-shape shear walls and one core are among the load-resisting configurations explored. Three, five, seven, nine, eleven, and thirteen stories are investigated. The vertical and lateral loads are examined for all systems under consideration. The Egyptian Code for Calculation of Loads and Forces for Buildings ECP-201 (2012) and the Egyptian code for designing and constructing reinforced concrete buildings ECP-203 (2007) are used to develop these systems. The SAP2000 Version 14.2 software package is used for modelling and analysis. ECP-203 considers concrete and reinforcing steel (2007). The columns and beams are represented using frame elements, with plastic hinges at their ends according to FEMA 450 requirements; the shear walls are modelled with multi-layer shell elements. The R-factor is calculated by plotting the force-displacement relationships (pushover curves), with the study's main goal to determine the R-factor and its components.*

Keywords *- Response modification factor, Nonlinear analysis, Wall-frames system, Plastic hinge formation, Multi-layer shell element.*

1. Introduction

Buildings are built to withstand many loads, such as static and dynamic loads, to prevent them from collapsing. The fundamental objective of the design regulations is to make structures safe under the effect of their dead and live loads, wind loads, and seismic loads, which cause structural failure. Economic restrictions in building construction can influence design philosophy, allowing structural and non-structural components to be damaged based on the structure's capacity to absorb energy through plastic deformations. The building's real strength is lower than its elastic strength due to this design approach. Since the size and mass of the members are reduced, the plastic deformations cause fractures in concrete and yielding in steel. Meanwhile, damage levels must be set to a certain extent since any higher amount of damage in the members might fail. As a result, the design regulations define a fixed factor to lower the building's strength to meet earthquake safety and financial economy.

The response modification factor, which indicates the ratio between the needed base shear force to maintain the structure elastic during the earthquake and the design base shear force considering the structure's inelastic behaviour, is code-specified. Many design codes include this component, which varies substantially from one to the next but serves the same purpose of representing the ratio of decreased strength force to dissipated energy in inelastic deformations. This factor was established to aid in the design process. It has been estimated throughout the years in various studies using laboratory experiments and computer models, taking into account a variety of parameters such as building material, structural system, type of soil, and seismicity.

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1.1. Background

The earthquake-resistant design concept states that a structure should withstand ground motion without collapsing, albeit with some damage. The structure is built for significantly lower base shear pressures than would be necessary if the building were to stay elastic despite severe shaking at a location, in keeping with this principle. The Response Modification Factor "R," which has been used since the end of the 1970s and was first introduced by the

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applied technology council [3] in 1978 to reduce the elastic seismic force obtained by elastic analysis to the seismic design force, is primarily responsible for these large reductions. Furthermore, it had not been extensively updated until the mid-1980s, when Uang and Bertero [4] and Whittaker et al. [5] constructed the formula exactly from 1986-1987 at the University of California at Berkeley. Those formulations will be discussed in the next paragraphs in order.

2. Literature Review

2.1. Response Modification Factor

The Response Modification Factor is a factor that is used to minimize the seismic force that is incorporated in the design based on the flexibility and overstrength of the structure to guarantee that an economic design is present.

Ductility, overstrength, and redundancy are the three key factors determining the Response Modification Factor. Using Bertero [4] and [5] did laboratory experiments to comprehend and record the seismic behaviour of steel braced frames structure buildings across their seismic response, beginning in the United States at Berkeley city and the University of California. Submitting base shear - roof displacement relationships timed to each earthquake's maximum base shear by using these data to graph the base shear versus roof displacement relationship (pushover curve) for every model and using the earthquake platform's acceleration-response history to generate the elastic acceleration response spectrum. Furthermore, the researchers used the findings data to characterize R as the combination of three components that account for ductility, strength, and damping forward through the equation below;

$$R = R_{\mu} . R_{s} . R_{\zeta} \tag{1}$$

Where

R_{μ}: Ductility reduction factor.

R_s: Overstrength factor.

R_{ξ}: Damping factor.

The three components R_{μ}, R_s, and R_{ξ}, account for various properties of the structure, including energy dissipation and absorption through plastic deformations, internal force redistribution in the inelastic region, and structural damping via extra viscous damping devices.

For more than one researcher, further study concentrated on giving a developed formula to acquire the "R" factor, such as [5], [7], and so on. The most effective was the update made by [5] splitting R into three factors; flexibility, strength, and redundancy instead.

The most effective was the update made by [6] splitting R into three factors; flexibility, strength, and redundancy instead.

$$R = R_{\mu} . R_{s} . R_{R} \tag{2}$$

Where

R_R: Redundancy factor.

Excluding the redundancy factor, this formulation is comparable to those provided by University of California academics.

Furthermore, several studies proposed a different formulation of the "R" factor, such as those proposed by [5], [7], who divided "R" into a product of two factors; the ductility reduction factor R_{μ}, multiplied by the overstrength component R_s.

$$R = R_{\mu} . R_{s} \tag{3}$$

The ductility reduction factor R_{μ} decreases the elastic force to the structure's yield strength, and the overstrength factor R_s accommodates for overstrength provided in code-designed structures. The one employed in the current thesis is Eq. (3). The structural system's properties determine these parameters, ductility and overstrength, as shown in Fig. 1.

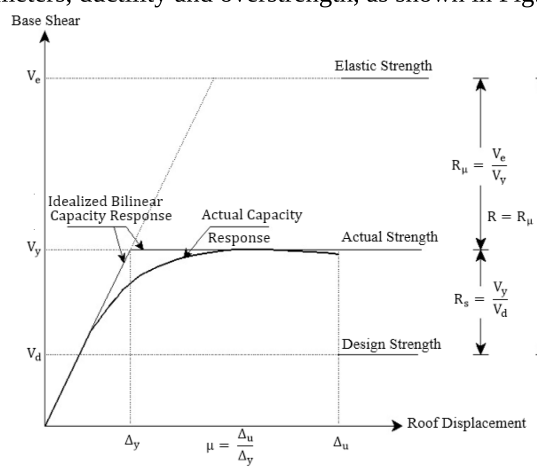


Fig. 1 Relationship between base shear V and roof displacement Δ, displaying response modification factor and its components; the ductility reduction factor R_{μ} and the overstrength factor R_s.

2.1.1. Ductility Reduction Factor

The ductility reduction factor is a property that describes a structure's ability to dissipate hysteretic energy by experiencing plastic deformations with acceptable stiffness loss. It is expressed as a factor that decreases the elastic force demand to the idealized yield strength of the structure as the proportion of the elastic failure force V_e to the actual yield one V_y.

$$R_{\mu} = \frac{v_e}{v_y} \tag{4}$$

This factor can also be represented as the elastic strength demand to keep the system elastic. At the same time, Fy (μ = μt) is the inelastic yield strength necessary to keep the displacement ductility demand equal to the target ductility μt.

$$R_{\mu} = \frac{F_y(\mu = 1)}{F_y(\mu = \mu_t)} \quad (5)$$

This factor is determined by the Ductility factor, a structural characteristic. It can be expressed mathematically as the ratio of the maximum deformation Δ_u to the yield deformation Δ_y at an assumed collapse point.

$$\mu = \frac{\Delta_e}{\Delta_y} \quad (6)$$

The ductility reduction factor has also been studied and expressed in several formulae that connect it to the structure's ductility and period, as studied by many researchers [8] ~[18].

Miranda and Bertero [19] introduced equations predicated on the inelastic strength from elastic strength based on the results of thirteen different studies on strength reduction factors due to nonlinear behaviour conducted over the last thirty years and put them in a common format, allowing for easier comparison and unification of the main parameters that affect R. They concluded that the maximum displacement flexibility, the structure's period, the input seismic forces, and the soil conditions all impact the strength decrease factor. They gave the most commonly used formulae, where the suggested R relations were described by splitting the period into two linear parts;

$$R_{\mu} = (\mu - 1)\frac{T}{T_c} + 1 \qquad T<Tc \quad (7)$$

$$R_{\mu} = \mu \qquad T \geq Tc \quad (8)$$

2.1.2. Overstrength Factor (R_s)

The overstrength factor is a parameter that depicts a structure's capacity to avoid collapse. It is expressed as a factor that shows that the real strength of the structure is considerably higher than that of the design strength as that of the proportion between the yield force V_y and the design force V_d.

$$R_s = \frac{v_y}{v_d} \quad (9)$$

The overstrength factor has also been studied and expressed in several formulae that connect it to the structure's elastic and design forces, as studied by many researchers [20]~[33].

Overstrength in buildings can be investigated both globally and locally.

Local Overstrength

Overstrength results from the design procedures and the use of stronger members than are necessary. The following are the most typical causes of local overstrength;

- The strength level of the actual material exceeds the design code's requirements.
- Create a coding approach (e.g., working vs ultimate) and make use of the capabilities of the members.
- Actual floor loads are lower than those required by the building code.
- Design load scenarios and load combinations are governed.
- Design code minimum standards for reinforcement and member size.
- Structural member deflection restrictions
- The structural story's drift restrictions.

Global Overstrength

The structure's response to lateral stresses causes this form of overstrength. The emergence of plastic hinges due to yielding and distribution of internal stresses of parts in the inelastic region allows the entire structure to withstand forces substantially greater than the design stresses. The following are the most prevalent causes of global overstrength;

- Overlooked elements throughout the design stage (e.g. slab under lateral action).
- Non-structural elements (e.g. stairways, cladding, walls, etc.).

The sources cited to show that measuring the overstrength factor has been challenging due to several complicated components and criteria.

The overstrength factor has also been investigated and numerically quantified. The structural base shear vs displacement graph is obtained using nonlinear static analysis (pushover). It is reformed into a bilinear curve to produce the yield base shear, which is then compared to the design base shear.

2.2. Nonlinear Static Pushover Analysis

The force-displacement relationship of MDOF systems is utilized in pushover analysis for the design and performance assessment of global limit states of reinforced concrete structures. This evaluation comprises two or three-dimensional computer simulations models of the structure; the vertical load is applied initially. Then, a specified monotonous lateral load is applied and dispersed as the structure height increases. The members' active lateral forces are steadily raised until they approach the yield stage (plastic hinge formation). The mathematical model dealt with the stiffness change and resisted increasing base shear forces until further parts yielded. When the final displacement or goal displacement is attained, the analysis is complete.

This study may be used to estimate maximal strength and deformation and investigate weak zones and soft tales in the structure.

There are several popular ways for idealizing pushover curves to convert them to bilinear force deformation curves, one of which is employed in this paper;

2.2.1. Park Method [34]

According to the structural behaviour, this technique proposed an idealization based on four possible definitions for the yielding displacement Δ_y and maximum displacement Δ_u; 1) the displacement caused by the yielding of longitudinal steel reinforcement (the first yielding displacement). 2) The corresponding elastoplastic system's yield displacement with much the same energy absorption as the genuine system. 3) The secant stiffness at 75% of the maximum lateral load of the real system was determined to be the yield displacement of the comparable elastoplastic system with decreased stiffness. 4) The yield displacement of an analogous elastoplastic system with about the same elastic stiffness and maximum load as the genuine system is the most accurate definition for reinforced concrete structure yield displacement, as illustrated in Fig. 2.

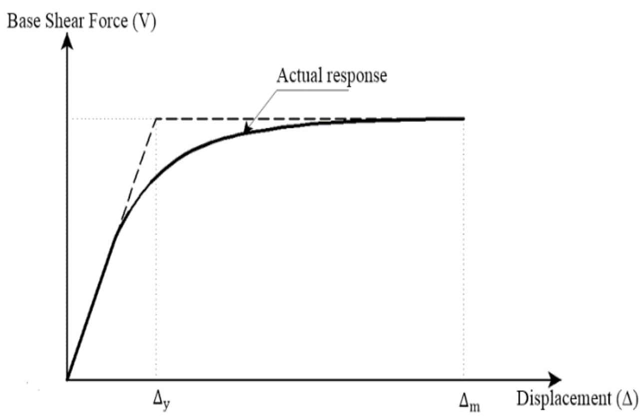


Fig. 2 The idealized equivalent elastoplastic yield force-displacement relationship

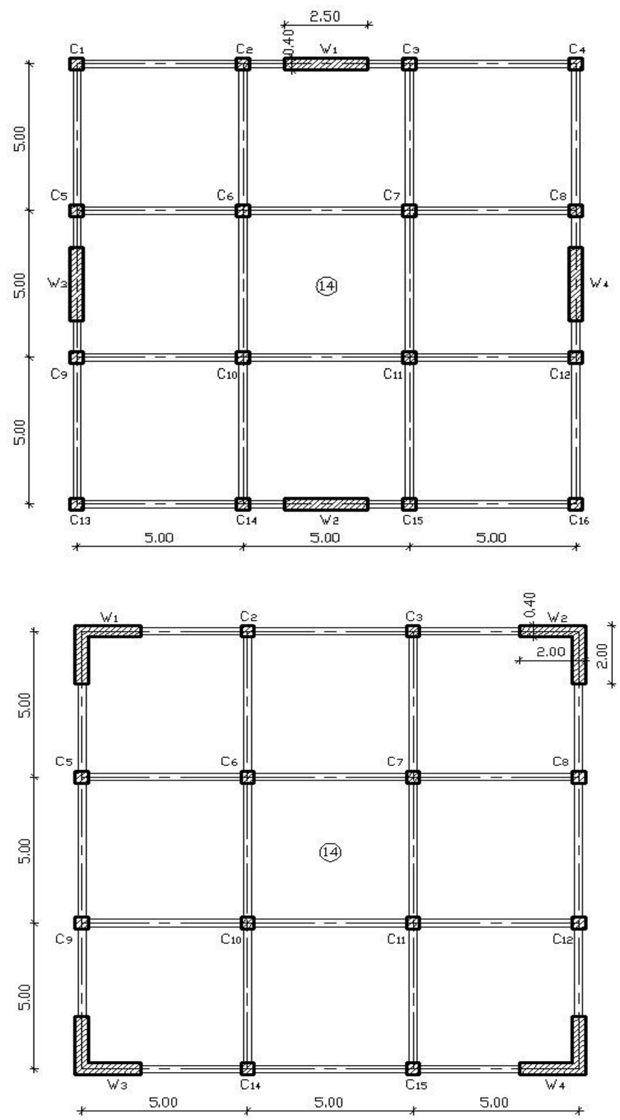
3. Methodology

3.1. Description of structural models

3.1.1. Configuration

Different forms of idealized reinforced concrete statical systems for residential buildings are investigated, with the primary wall-frame systems chosen. In both plan and elevation, the systems are symmetrical and regular. According to the wall section and its location, the mathematical models that reflect the real systems in computer simulation are categorized into five systems shown in Fig. 3;

- Wall-frames systems with rectangular shear walls at the centre of the exterior frames.
- Wall-frames systems with L-shape shear walls at the corner of the exterior frames.
- Wall-frames systems with box-section core shear walls at the centre of the structure.
- Combination of systems 1&3; wall-frames systems with rectangular shear walls at the centre of the exterior frames with wall-frames systems with box-section core shear walls at the centre of the structure.
- Combination of systems 2&3; wall-frames systems with L-shape shear walls at the corner of the exterior frames with wall-frames systems with box-section core shear walls at the centre of the structure.



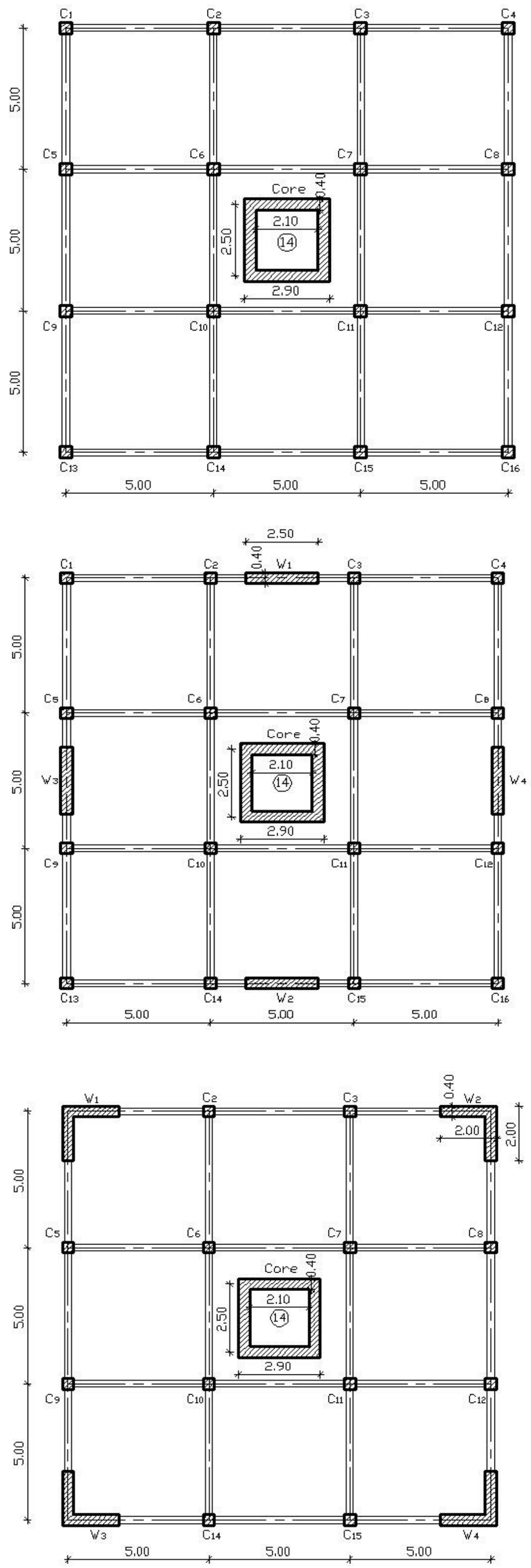


Fig. 3 The plan arrangement for the five studied systems

3.1.2. Geometry

All types of configuration systems have a varied number of stories; three, five, seven, nine, eleven, and thirteen, with a typical story height of 3.0 m. For all versions, the standard bay length is 5.0 m, with three bays for each direction to guarantee symmetry. All of the columns have a squared cross-section and a 1% per cent reinforcing ratio. Rectangular, L-section, and box-section shear walls have L/b ratios of 6.25, 6.25, and 5.0 (greater or equal to 5.0), respectively, with a concentrated reinforcement ratio of 3.0 per cent in the edge length of the wall and a distributed reinforcement ratio of 0.40 per cent in the middle length of the wall (greater or equal to 5.0). The foundation is considered fixed by the columns and walls.

3.2. Design Process Description

3.2.1. Design Codes

The aforementioned systems are built in accordance with the Egyptian Codes of Practice for the computation of vertical and seismic loads on residential structures and design guidelines [1] and [2].

3.3. Analytical Modelling Process Description

3.3.1. Methodology for Software Modelling

SAP2000 [35] is a graphical interface finite element modelling application that employs the object-based paradigm to depict the actual structure as a realistic model. Traditional finite element joints (nodes), lines (frames) elements, and shells (areas) elements are used in the model to make it simple to describe practically any structure in two or three dimensions. SAP2000 turns the object-based model into an element-based model that would be utilized for analysis when the analysis is run. The analysis model is component-based. The study findings are presented on the model once the analysis is completed. SAP2000 also includes an analysis tool for nonlinear static pushover analysis, which is performed by subjecting a given force or target displacement to an increasing analysis time while user-defined parameters are observed throughout the analysis.

3.3.2. Material Modelling

Materials nonlinearity must be considered in the modelling process by creating specific constitutive materials or modifying the existing ones to match the experimental provisions of the design code, including such confined concrete, unconfined concrete, and high-grade steel bars, to perform nonlinear static analysis. Material type, symmetrical directional type (isotropic or uniaxial), specific gravity, modulus of elasticity, Poisson's ratio, and the most effective nonlinear stress-strain curve characteristic are all included.

Confined Concrete

The Egyptian code ECP-203 [2] included provisions in its stress-strain curve to account for the behaviour of constrained concrete. Confining the concrete using square or rectangle stirrups proved ineffective since there was no increase in the compressive strength of concrete owing to confinement; hence the maximum stress achieved by confined concrete maintained the same as the maximum stress reached by unconfined concrete, resulting in a second-degree parabola representing the ascending portion of the curve. Only the slope of the curve's post-peak portion was influenced by confinement. For the climbing part of the curve, Eq. (10) is employed till a strain value of 0.002 is obtained. The steady-state section is then calculated using Eq. (11) until the curve's strain values of 0.003 are attained.

$$f_c = f_c^* \left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002} \right)^2 \right] \varepsilon_c < 0.002 \qquad (10)$$

$$f_c = f_c^* \qquad 0.002 \le \varepsilon_c \le 0.003 \qquad (11)$$

$$f_c^* = \frac{0.67 f_{cu}}{\gamma_c} \qquad (12)$$

This material will be utilized for columns, beams, and shear wall concentrated reinforcing zones.

Unconfined Concrete

ECP-203 [2] does not specify this form of the concrete stress-strain curve, yet it is required for the modelling procedure. Kent and Park [36] established a stress-strain equation for unconfined and confined concrete. The ascending section of this stress-strain curve is characterized by the same constrained second-degree parabola defined by Eq. (13) as the descending segment. Because confinement influenced the slope of the curve's post-peak section, the post-peak segment was considered a straight line with a slope specified principally as a function of concrete strength and described by Eq. (14) until the curve's stress of 0.2 f*c was attained.

$$f_c = f_c^* \left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002} \right)^2 \right] \varepsilon_c < 0.002 \qquad (13)$$

$$f_c = f_c^* \left[1 - Z\left(\varepsilon_c - 0.002 \right) \right] \qquad \varepsilon_c \ge 0.002 \qquad (14)$$

In which

$$Z = \frac{0.5}{\varepsilon_{50u} - 0.002} \qquad (15)$$

Where ε_{50u} denotes the strain equivalent to a tension of 50% of the ultimate concrete strength in unconfined concrete.

$$Z = \frac{3 + 0.29 f_c^*}{145 f_c^* - 1000} \qquad (16)$$

This material will be utilized for distributed reinforcement regions of the shear walls.

High-Grade Steel Bars

The Egyptian code ECP-203 [2] included provisions in its stress-strain curve to account for reinforcing steel behaviour. The elastoplastic behaviour is shown in this curve. This stress-strain curve is divided into two parts, with the rising section represented by a straight line described by Eq (17). The steady-state section of the curve is then calculated using Eq. (18).

$$f_s = \varepsilon_s . E_s \qquad \varepsilon_s < \varepsilon_y / \gamma_s \qquad (17)$$

$$f_s = \frac{f_y}{\gamma_s} \qquad \varepsilon_s \ge \varepsilon_y / \gamma_s \qquad (18)$$

The reinforcement of the columns, beams, and shear walls will be made of steel.

3.3.3. Finite Element Modelling

Frame Element

As previously stated, the plastic hinges, together with the frame element, are allocated to specified positions to explore the behaviour of columns and beams outside the elastic limit. The yielding of plastic hinges for structural components is investigated by installing hard plastic hinges at the sites where yielding is predicted, as yielding is most likely to occur at the ends of frame components exposed to lateral loads. Between the two stiff plastic hinges, the section of a frame element stays elastic. These hinges are thought to be the source of all inelastic deformations. M3 hinges are allocated to beams in SAP2000, while P-M3 hinges are assigned to columns. Relying on the plastic hinge idea and a bilinear model, [37] expanded this one-component approach. Based on the plastic hinge notion and a bilinear moment-rotation connection [37], expanded this one-component model as shown in Fig. 4.

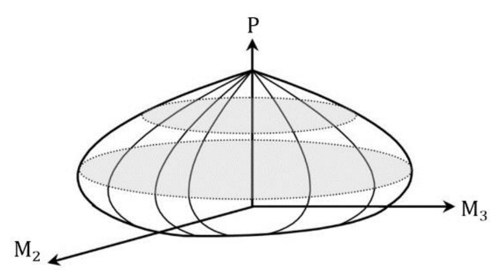


Fig. 4 Plastic (M &P-M Interaction) hinge

Multi-layer Shell Element

The shear wall is defined by a coupled in-plane/out-plane bending and a coupled in-plane bending-shear nonlinear behaviours of reinforced concrete shear walls, as cited in [38]. The shear wall is modelled by such a fine mesh of multi-layer shell elements, it is based on composite material mechanics principles, and it could be defined by a coupled in-plane/out-plane bending and just a coupled in-plane bending. The shell element comprises multiple layers of varying thicknesses and material qualities that are allocated to distinct layers. The reinforcing rebars are represented by one or more layers. To illustrate the concrete material model, idealized stress-strain relationships are used, with compressive strain at working stress of 0.002 and ultimate strain of 0.003 for confined concrete and compressive strain at working stress of 0.002 and ultimate strain of 0.0025 for unconfined concrete. The idealized elastoplastic model is chosen for rebar steel, with a stain at beginning strain hardening of 0.0015 and an ultimate strain capacity of 0.003. The reinforcement is considered a distinct layer in longitudinal and transverse dimensions. Two layers are assumed for each direction to account for top and bottom reinforcement in the cross-section, as clarified in Fig. 5.

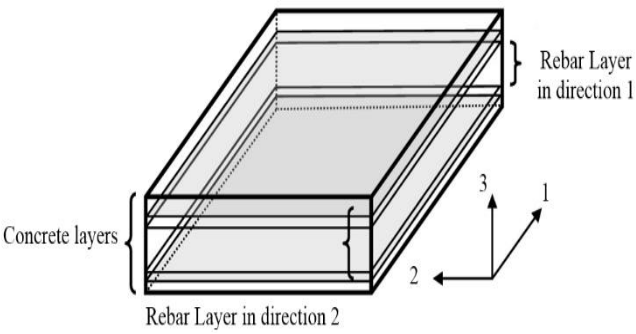


Fig. 5 Multi-layer Shell Element

Nonlinear Static Pushover Analysis

Using the force-displacement connection of multi-degree freedom systems, this sort of study is used to design and assess the performance of global response characteristics of reinforced concrete structures. Either to focus on the analysis or to impose displacements, the mathematical formulation of the examined system is exposed to an increasing forcing expressed as horizontal forces (simulation of inertial forces together with the height of the building). The analysis is finished whenever the monitored displacement or final limit state is achieved. When the earthquake forces the building to fail, the monitored displacement reflects the greatest building displacement. The pushover analysis is crucial for determining the structure's maximal strength and deformation capacity.

3.4. Response Modification Factor Calculation Process

To get the R factor, first, compute the ductility reduction factor R_μ as well as the overstrength factor R_s, then combine both of them as indicated in Equation 2.3.

The ductility reduction factor R_μ is derived using the Miranda and Bertero [8] method as mentioned before in Eqs. (7) and (8), using the value of T as the fundamental time period of the system given from the SAP model, and the value of 0.25 for T_c as taken from ECP-201 [1] item 8.4.2.2. Like that of the two components of the R-factor, Eq. (9) is used to derive the overstrength factor R_s previously shown in Fig. 1.

4. The Numerical Analysis Result

4.1. The Results of the First System

This system comprises three-dimensional frames with rectangular shear walls at the external frames' centres. Six mathematical models of reinforced concrete wall-frames systems with three, five, seven, nine, eleven, and thirteen-story models are constant three bays structures. Nonlinear static pushover analysis was performed on these systems. The results of the study are reported for every model in the system.

4.1.1. Determination of R-factor Parameters

To calculate the pushover curve for each of the six building designs, nonlinear static pushover analysis was done utilizing a lateral load pattern in accordance with the design acceleration response for parametric analysis. The process for doing nonlinear static pushover analysis and computing the R-factor has previously been detailed in item 2.2. The findings of the pushover analysis, as well as the pushover curve for the initial system buildings with each configuration, have already been detailed in item 2.2. as graphed in Fig. 6.

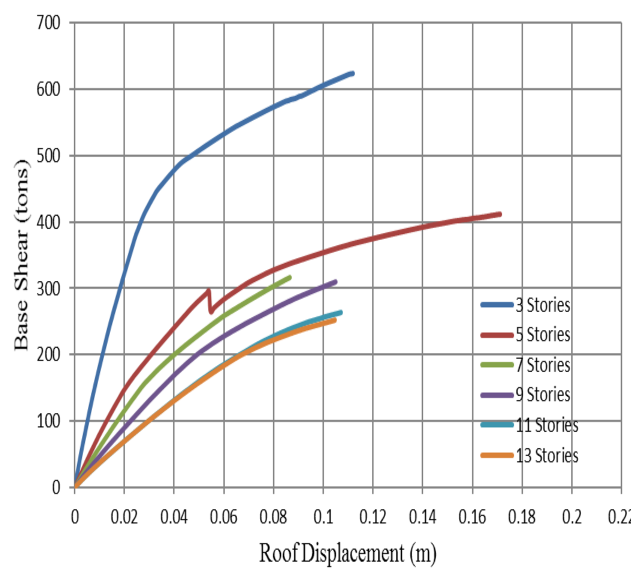


Fig. 6 Base shear versus roof displacement for the first system models

Table 1 gathers the pushover curve parameters (R-factor and values of its parameters) for many models of the first system so that they may be compared.

Table 1. Response modification factor parameters versus the number of stories for the first system models

No. of stories	δ_y (m)	δ_u (m)	μ	R_μ	V_y (tons)	V_d (tons)	R_s	R
3	0.031	0.112	3.566	3.566	624.077	59.187	10.544	37.598
5	0.049	0.171	3.456	3.456	412.023	67.250	6.127	21.175
7	0.049	0.086	1.759	1.759	315.839	73.152	4.318	7.595
9	0.063	0.105	1.675	1.675	309.891	77.895	3.978	6.664
11	0.071	0.107	1.507	1.507	264.045	81.903	3.218	4.851
13	0.063	0.105	1.649	1.649	251.623	85.395	2.947	4.858

The number of stories affects the ductility reduction factor. For 3-, 5-, and 7-story structures, the ductility reduction factor drops by 46%. The factor is nearly unchanged for structures with nine, eleven, and thirteen stories. The overstrength factor is indeed impacted by the number of stories, with the overstrength factor decreasing by 58 per cent for 3-, 5-, and 7-story structures. The factor is then reduced by 21% for structures with nine, eleven, and thirteen stories. For 3-, 5-, and 7-story structures, the Response Modification factor (the combination of the two factors) is reduced by 78%. Then, for structures with nine, eleven, and thirteen stories, this factor is reduced by 20%.

4.2. The Results of the Second System

This system comprises three-dimensional frames with L-shaped shear walls at the external frame corners. Six numerical simulations of reinforced concrete wall-frames systems were chosen; they are three-, five-, seven-, nine-, eleven-, and thirteen-story versions with constant three bays. The same analyses and parameters were used to run and show these systems.

4.2.1. Determination of R-factor Parameters

The results of the pushover analysis, as well as the pushover curve for each configuration of the second system models as graphed in Fig. 7.

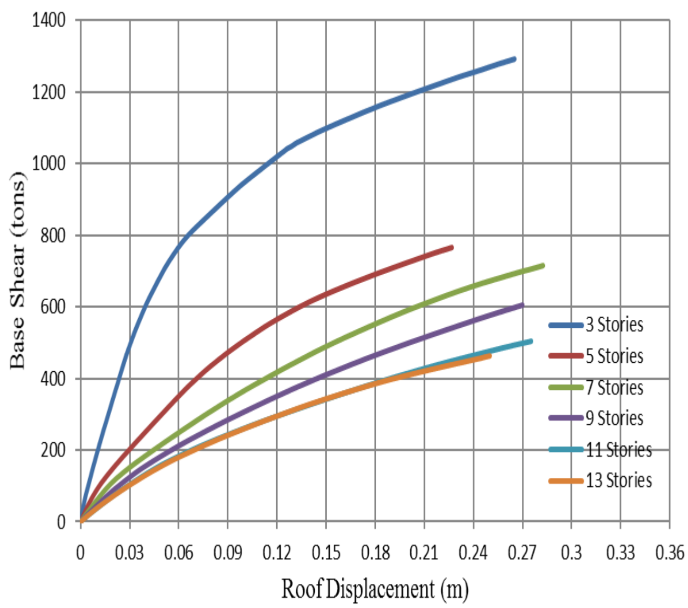


Fig. 7 Base shear versus roof displacement for the second system models

Table 2 compiles the pushover curve parameters for various models of the second system to compare them.

4.3. The Results of the Third System

Table 2. Response modification factor parameters versus the number of stories for the second system models

No. of stories	δ_y (m)	δ_u (m)	μ	R_μ	V_y (tons)	V_d (tons)	R_s	R
3	0.066	0.265	4.044	4.044	1290.27	59.187	21.800	88.155
5	0.084	0.227	2.703	2.703	765.004	67.250	11.376	30.747
7	0.117	0.283	2.422	2.422	716.470	73.152	9.794	23.722
9	0.133	0.270	2.037	2.037	606.183	77.895	7.782	15.850
11	0.134	0.275	2.048	2.048	504.337	81.903	6.158	12.609
13	0.119	0.250	2.097	2.097	461.800	85.395	5.408	11.340

The building plan comprises three-dimensional frames with a boxed shear wall (core) in the centre point. Six numerical simulations of reinforced concrete wall-frames systems were chosen; they are three-, five-, seven-, nine-, eleven-, and thirteen-story models with constant three-bays. The same analyses and parameters were used to run and show these systems.

4.3.1. Determination of R-factor parameters

The results of the pushover analysis, as well as the pushover curve for each configuration of the third system as graphed in Fig. 8.

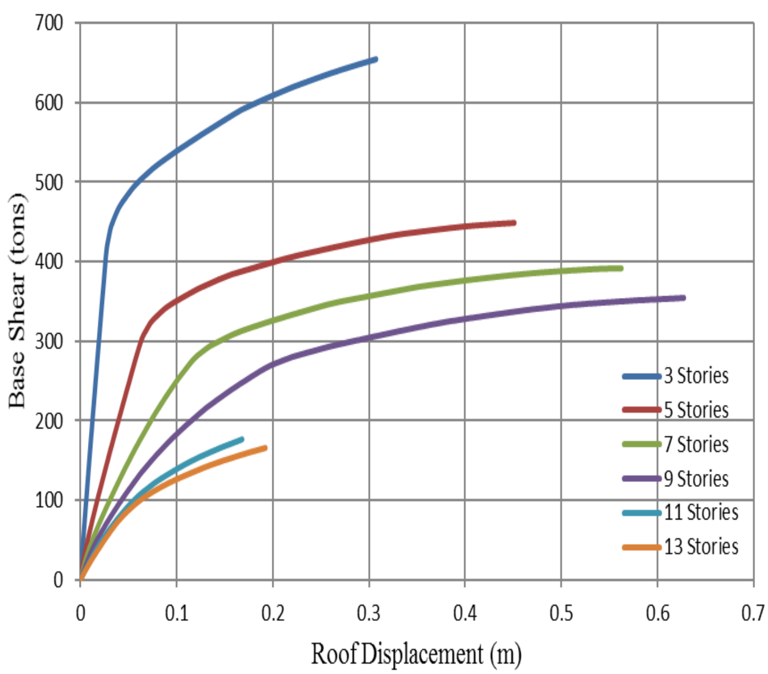


Fig. 8 Base shear versus roof displacement for the third system models

Table 3 compiles the pushover curve parameters for the various models of the third system for comparison.

Table 3. Response modification factor parameters versus the number of stories for the third system models

No. of stories	δ_y (m)	δ_u (m)	μ	R_μ	V_y (tons)	V_d (tons)	R_s	R
3	0.038	0.307	8.154	8.154	654.994	59.187	11.066	90.231
5	0.073	0.452	6.216	6.216	448.395	67.250	6.668	41.447
7	0.094	0.563	5.976	5.976	392.326	73.152	5.363	32.048
9	0.117	0.627	5.374	5.374	354.427	77.895	4.550	24.454
11	0.071	0.169	2.370	2.370	177.296	81.903	2.139	5.072
13	0.077	0.193	2.517	2.517	165.580	85.395	1.939	4.881

The ductility reduction factor is affected by the number of stories, with the ductility reduction factor decreasing by 31% for 3-, 5-, 7-, 9-, and 11-story buildings. The factor is then increased by 8% for structures with 13 stories. The overstrength factor is also influenced by the number of stories, with the overstrength factor decreasing by 59 per cent for 3-, 5-, 7-, and 9-story buildings. The factor is reduced by 9% for structures with 11 and 13 stories. For 3-, 5-, 7-, and 9-story buildings, the Response Modification factor is reduced by 71%. This factor is then reduced by 3% for structures with nine, eleven, and thirteen stories.

4.4. The results of the fourth system

This system comprises three-dimensional frames with rectangular shear walls at the external frames and a box shear wall (core) in the building plan's centre. Six mathematical models of reinforced concrete wall-frames systems were chosen; they are three-, five-, seven-, nine-, eleven-, and thirteen-story models with constant three-bays. The same analyses and parameters were used to run and show these systems.

4.4.1. Determination of R-factor parameters

The results of the pushover analysis, as well as the pushover curve for each configuration of the fourth system as graphed in Fig. 9.

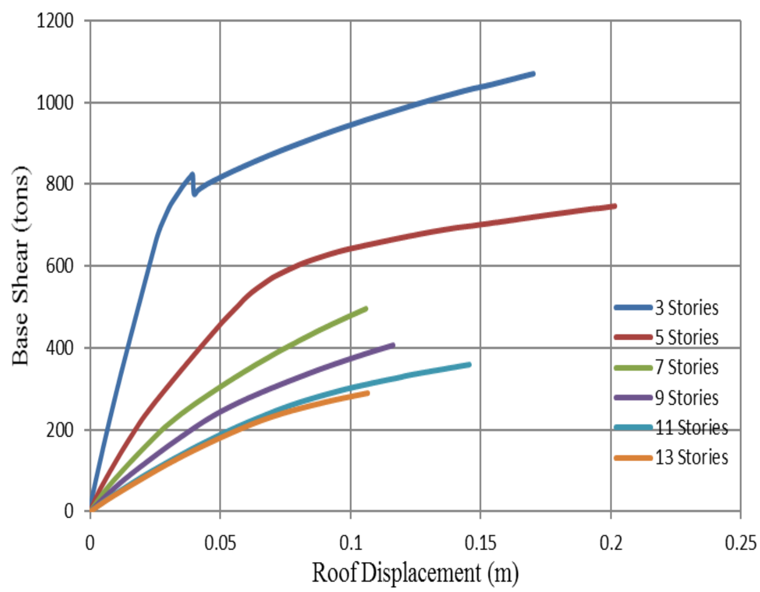


Fig. 9 Base shear versus roof displacement for the fourth system models

Table 4 gathers the pushover curve parameters for several models of the fourth system to be compared.

Table 4. Response modification factor parameters versus the number of stories for the fourth system models

No. of stories	δ_y (m)	δ_u (m)	μ	R_μ	V_y (tons)	V_d (tons)	R_s	R
3	0.031	0.122	3.911	3.378	1092.99	59.187	18.467	62.389
5	0.047	0.133	2.796	2.796	710.752	67.250	10.569	29.547
7	0.039	0.067	1.744	1.744	415.354	73.152	5.678	9.902
9	0.044	0.082	1.885	1.885	374.882	77.895	4.813	9.072
11	0.056	0.105	1.871	1.871	332.102	81.903	4.055	7.587
13	0.051	0.085	1.662	1.662	287.602	85.395	3.368	5.597

The number of stories affects the ductility reduction factor, which is reduced by 48 per cent for 3-, 5-, and 7-story buildings. The factor is then reduced by 12 per cent for buildings with nine, eleven, and thirteen stories. The number of stories affects the overstrength factor for 3-, 5-, 7-, and 9-story buildings, with the overstrength factor decreasing by 73%. The factor is reduced by 17% for buildings with 11 and 13 stories. The Response Modification factor for 3-, 5-, and 7-story buildings is reduced by 84%. Then, for structures with nine, eleven, and thirteen stories, this factor is reduced by 38%.

4.5. The Results of the Fifth System

This system comprises three-dimensional frames with L-shaped shear walls at the external frame corners and a boxed shear wall (core) in the building plan's centre point. Six mathematical models of reinforced concrete wall-frames systems were chosen; they are three-, five-, seven-, nine-, eleven-, and thirteen-story models with constant three-bays. The same analyses and parameters were used to run and show these systems.

4.5.1. Determination of R-factor parameters

The results of the pushover analysis, as well as the pushover curve for each configuration of the fifth system as graphed in Fig. 10.

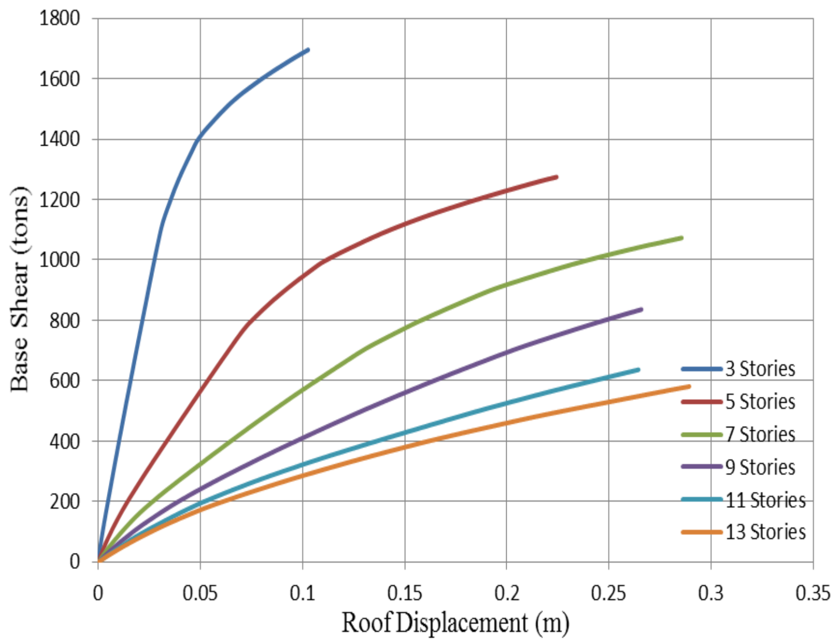


Fig. 10 Base shear versus roof displacement for the fifth system models

Table 5 presents the pushover curve parameters for several models of the fifth system to be compared.

Table 5. Response modification factor parameters versus the number of stories for the fifth system models

No. of stories	δ_y (m)	δ_u (m)	μ	R_μ	V_y (tons)	V_d (tons)	R_s	R
3	0.040	0.103	2.561	2.162	1695.38	59.187	28.644	61.937
5	0.084	0.224	2.658	2.658	1274.36	67.250	18.950	50.363
7	0.120	0.286	2.377	2.377	1072.90	73.152	14.667	34.860
9	0.136	0.266	1.953	1.953	836.075	77.895	10.733	20.961
11	0.135	0.264	1.956	1.956	636.551	81.903	7.772	15.204
13	0.137	0.289	2.110	2.110	581.135	85.395	6.805	14.360

The ductility reduction factor is affected by the number of stories in a building, with the ductility reduction factor decreasing by 27% for 3-, 5-, 7-, and 9-story buildings. The factor increases by 13% for structures with 11 and 13 stories. The overstrength factor too is influenced by the number of stories, with the overstrength factor decreasing by 73 per cent for 3-, 5-, 7-, and 9-story buildings. The factor is reduced by 16 per cent for structures with 11 and 13 stories. For the 3-, 5-, 7-, and 9-story buildings, the Response Modification factor is reduced by 76%. Then, for buildings with 11 and 13 stories, this factor is reduced by 13%.

5. Conclusion

The following are the conclusions reached as a result of this paper;

- The response modification factor is sensitive to both material strength and geometric configuration, with changing geometric configuration having a greater influence on the R-factor value.
- The R-factor stated in seismic provisions codes will not be identical to any structure with a known type of lateral load resisting system because the study has shown that the values of "R" are dependent on ductility reduction and overstrength calculated values and vary widely from one structure to another based on geometry and height.
- The structure's overstrength factor is inversely proportional to the number of stories.
- For low-rise structures, the overstrength factor value is greatly exaggerated because gravity loads govern the design process more so than lateral loads, resulting in the meaning of the term overstrength design.
- The R-factors for the first, third, and fourth systems in the 13-story models are 4.85, 4.88, and 5.60, respectively. The average R-factor for 13-story models for the second and fifth systems, on the other hand, is 11.34 and 14.36, respectively.
- The rectangular shear walls in the analyzed systems have a 12 per cent tendency to the ECP-201 (2012) stated values of "R." Systems having L-shaped shear walls, on the other hand, deviate from the stated code value of "R" by 70%.

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