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The Effect of Soil Structural Interaction on Evaluation of Seismic Response Reduction Factor of Multi-Story Concrete Buildings

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الملخص العربى :

لقد تم ملاحظة أن السلوك الزلزالي للمنشات الخرسانية يتأثر بشدة ليس فقط بالاستجابة الزلازالية للمنشأ ولكن أيضا بالاستجابة الزلازلية للتربة أسفل الأساسات. تم إجراء العديد من الابحاث للنظر في تحليل الاستجابة الزلازالية للمنشات الخرسانية بأكملها بما في ذلك النظام الانشائي للمنشأ والأساسات وتفاعل تربة التأسيس اسفل أساسات مع المنشأ. قد يمثل تفاعل تربة التأسيس مع المنشا أحد العوامل الحيوية التي تؤثر على العوامل الجيوتقنية التي يمكن أن تغير بشكل كبير الاستجابة الزلزالية للمنشات الخرسانية. ويساهم التحسين للعوامل الجيوتقنية في استجابة زلزالية صغيرة مع زيادة التردد الطبيعى للمنشا بالإضافة إلى التأثير على نسبة التخميد. يمكن تلخيص الأهداف الأساسية لهذه الدراسة البحثية على النحو التالي ؛ دراسة الابحاث السابقة حول تفاعل تربة مع المنشأ وتأثير ذلك على معامل تعديل ردود الافعال ، ودراسة آثار تفاعل تربة التأسيس مع المنشأ على الأداء الزلزالي للمباني متعددة الطوابق ، وكذلك دراسة تأثير أنواع التربة المختلفة ومعامل تفاعل الطبقة السفلية على تصميم العناصر الإنشائية المقاومة لأحمال الزلازل واقتراح القيم المنطقية الصحيحة لمعامل تقليل ردود الافعال للمباني متعددة الطوابق التي تم تحليلها مع أخذ التفاعل بين للتربة والمنشا فى الاعتبار ، بناء على تحليل نتائج الدراسة ، ليتم أخذها في الاعتبار في كود التصميم المصري.

الكلمات الدالة : تداخل تربة التاسيس , طيف التجاوب , قوي القص , معامل تقليل الاستجابة الزلازالية , معامل تفاعل الطبقة السفلية.

ABSTRACT

It has been observed that the seismic behavior of concrete structures is strongly influenced not only by the seismic response of the structure but also by the seismic response of the soil under the foundations. Several studies have been conducted to consider the analysis of the seismic response of the hollow concrete structure, including the structural system of the structure, foundations, and the soil structure interaction. The interaction of the foundation soil may represent one vital affecting geotechnical parameters that can significantly alter the seismic response of a structure. Studying the effect of foundation soil interaction affects the seismic response of a structure that helps in achieving a more relevant design. The smooth idealization of geotechnical parameters contributes to a small seismic response with the increased natural periods as well as

affecting the damping ratio. The fundamental objectives of this research study can be summarized as follows; investigating the previous studies on Soil Structural Interaction and its effect on Seismic Response Reduction Factor, Investigating the effects of a soil structural interaction on the seismic performance of multi-story buildings, Investigating the effect of different soil types and the modulus of subgrade reaction (k_s) on the seismic design of the structural elements resisting the lateral loads and Suggesting properly rational values of the response reduction factor for the analyzed multi-story buildings when taking the soil structural interaction, based on the analysis results, to be considered in the Egyptian Design Code.

KEYWORDS: soil structural interaction, Pushover analysis, Base shear, Response modification factor, Seismic Zones, Spectrum type.

INTRODUCTION

In recent years The determination of the seismic performance of buildings has gained very much interest, and today there are a greater number of specifications and regulations containing provisions on this issue (Nist 2012) [1]. The interaction between the three statically systems; (which are superstructure, foundation and soil medium surrounding the foundation system), plays an important role in the seismic behavior of a building (Council and Gcr 2009) [2]. Although in literatures there were studies suggesting the use of force-based computational approaches for the modeling of SSI(FEMA-440 2005) [3], the using of them has been so limited, and they have not found significant use in practice (NIST 2012) [1]. Particularly over the last two decades the widespread use of displacement-based methods, which include nonlinear calculations such as static pushover analysis, provide to investigate SSI beyond the elastic limits (FEMA-440 2005) [3], (ASCE 2013) [4]. Realistic estimations of both displacement capacities and seismic drift demands became possible by using nonlinear analysis methods. The damage observations and detailed structural analyses have shown that SSI could significantly alter both the capacity and demand-related structural parameters such as vibration period and drift capacity and hence the seismic performance of buildings (Mylonakis and Gazetas 2000) [5] and (Raheem, Ahmed, and Alazrak 2015) [6]. All these observations and findings have shown that SSI effects should be considered necessary for the design and assessment of buildings.
Academically studies considering the SSI have increased in the United States towards the end of the 2000s and some of them so it summarized in the FEMA-440 report [3]. In this report, regulations and expressions presented to explain how SSI could be considering in nonlinear static analyses. The findings of these studies were also included in US code specifications (ASCE 2013) [4]. However, the expressions in FEMA-440 and the (ASCE 2013) [4] regulations are not recommended for nonlinear time history analyses and therefore this situation required new studies on this subject. The results of subsequent studies related with the SSI in performance-based earthquake engineering were summarized in 2012 and the method, which can be used in non-linear time history analysis, was proposed(Nist 2012) [1]. This approach also used in this study during the

analyses of selected building models and SSI represented based on the expressions taken from these studies.

(Mylonakis and Gazetas 2000) [5] studied the seismic analysis and design of bridge piers and proposed simple expressions for the calculation of kinematic effects (Fatahi, Tabatabaiefar, and Samali 2011) [7] examined the seismic performance of empirical buildings with SSI and showed that the seismic performance of buildings varied significantly depending on the soil conditions. (Raheem, Ahmed, and Alazrak 2015) [6] Investigated the variations in SSI effects depending on the use of different demand calculation methods in multi-story buildings. Their research has shown that seismic performance evaluations are not within the reliable limits if the effects of SSI ignored .

A various study has aborted the SSI using Finite Element Method (Lin, Roesset, and Tassoulas 1987) [8]; (Matthees and Magiera 1982) [9], (Bolisetti and Whittaker 2011) [10] and (Roy, Bolourchi, and Eggers 2015) [11] investigating the SSI effect for nuclear structures. (Cacciola, Banjanac, and Tombari 2017) [12] Employed FE approach assuming linear behavior for the soil and structures and studied the impact of a vibration barrier on an existing masonry structure .

An increasing number of academic studies and engineering reports imply that in the next generation codes SSI modeling will necessarily be required and consideration of SSI effects in design will be mandatory. However, buildings constructed before these findings will still be the weak point of the cities that are prone to seismic risk.

Our study point has three branches, the first branch is the soil structural interaction (S.S.I), which had used instead of fixed support, the second branch is the response modification factor, which we will evaluate and the third branch is the pushover analysis method (P.O.A), which we used as a nonlinear seismic analysis for seismic performance evaluation.

The main objective of this study is to investigate the previous studies on Soil Structural Interaction and its effect on Seismic Response Reduction Factor, investigate the effects of a soil structural interaction on the seismic performance of multi-story buildings, investigate the effect of different soil types and the modulus of subgrade reaction (k_s) on the seismic design of the structural elements resisting the lateral loads and suggest properly rational values of the response reduction factor for the analyzed multi-story buildings when taking the soil structural interaction, based on the analysis results, to be considered in the Egyptian Design Code.

CONCEPT OF SOIL STRUCTURE INTERACTION (SSI)

(Reissner 1936)[13], in 1936, it was proposed a theory about the vibration of the foundation soil. (Veletsos and Meek 1974) [14], according to them, inertial interaction effects for buildings induce a lengthening of the natural period of the soil-structure system, because the structure is more flexible compared with the corresponding Fixed Base structure, and an increase of soil-structure system damping, due to dissipated energy and to radiated waves from the structure back into the soil. (Wolf and Obernhuber 1985) [15], proposes the direct approach for SSI analyses that solves the dynamic equilibrium equation of the soil-structure assembly, distinguishing the case of a Flexible foundation motion applied to a FB model.

(Federal Emergency Management Agency (FEMA) 2020)[16], in section 6.3 there are two methods about how to modelling the SSI. The first method is rigid foundation and flexible soil (see figure 1) in which the modelling of foundation depended on six formulas for six degree of freedom see table 2. The second method is flexible foundation and linear flexible soil (see figure 2) in which distributed springs representing the soil support as a discretized continuous medium, with a uniform value for the springs along the length of the footing. This method is best used when the flexibility of the structural elements of the foundation are modeled explicitly using formula 1 to determine the unit subgrade spring coefficient (K_s).

$$k_{sv} = \frac{1.3G}{B_f(1-\nu)} \tag{1}$$

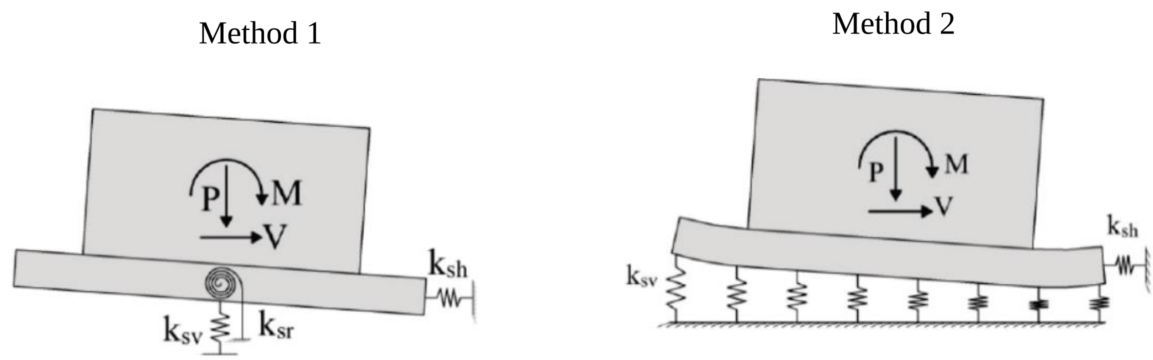


Figure 23: Two methods for foundation modeling approaches with vertical and rotational springs presented in FEMA (2020).

Determination of Subgrade Reaction (k_s)

The ratio between the pressure (q) at a known point and the settlement (δ) produced by load application at the same point this relation is the coefficient of subgrade reaction (k_s). So to determine this coefficient (k_s) more studies worked on it in two ways (experimental and theoretically). One of the most popular models in determining is the (E. Winkler 1867) [17] model in which the subgrade soil behaved like an infinite number, this infinite number is linear elastic springs which stiffness is named as the modules of subgrade reaction (K_s). The parameters such as type of foundation, depth, shape, and soil type are taking into consideration for this modulus. So in the next few words, we discuss the determining (k_s) by using Empirical Formula and experimental results.

Winkler (1867), Biot (1937), Terzaghi (1955), Vesic (1961), Meyerhof and Baike (1965), Selvadurai (1984) and Bowles(1998) have investigated the factors affect the determination of ks.There are some different formulas to calculate the modulus of subgrade reaction (k_s) look at the table1.

Table 1: Some different formulas to calculate the modulus of subgrade reaction, k_s.

no	Investigator	year	Suggested Formula
1	Winkler	1867	$k_s = \frac{q}{\delta}$
2	Biot	1937	$k_s = \frac{0.95E_s}{B(1-\upsilon_s^2)}\left[\frac{B^4E_s}{(1-\upsilon_s^2)EI}\right]^{0.108}$
3	Terzaghi	1955	$k_{sf} = k_{sp}\left(\frac{B+B_1}{2B}\right)$
4	Vesic	1961	$k_s = \frac{0.65E_s}{B(1-\upsilon_s^2)}\sqrt[12]{\frac{E_sB^4}{EI}}$
5	Meyerhof and Baike	1965	$k_s = \frac{E_s}{B(1-\upsilon_s^2)}$
6	Selvadurai	1984	$k_s = \frac{0.65}{B}\cdot\frac{E_s}{(1-\upsilon_s^2)}$
7	Bowles	1998	$k_s = \frac{E_s}{B_1(1-\upsilon_s^2)mI_sI_F}$

Where:

k_s = the coefficient of subgrade reaction. q = the pressure per unit of area. δ = the settlement produced by load application. B1 = side dimension of square base used in the plate load test. B = width of footing. k_{sp} = the value of subgrade reaction for 0.3 × 0.3 (1 ft wide) bearing plate. k_{sf} = value of modulus of subgrade reaction for the full-size foundation. E_s = modulus of elasticity. υ_s = Poisson's ratio. E_I = flexural rigidity of footing, m = takes 1, 2, and 4 for edges, sides, and center of footing, respectively. I_S and I_F = influence factors depend on the shape of footing.

(Elsamee 2013)[18], using the plate load tests in this experimental study, in which the settlement of sandy soil was measured under different stresses, each sample has been placed in an open box in the field and compacted in layers with different relative densities and different depths of foundations.

Subgrade reaction k_s of cohesionless soil increases with increasing footing depth as well as footing size. Subgrade reaction k_s of cohesionless soil under rectangular footing is higher than that under the square and that under circular one (at same equivalent area), Subgrade reaction k_s of cohesionless soil increases with increasing stress (q), look at figure 2.

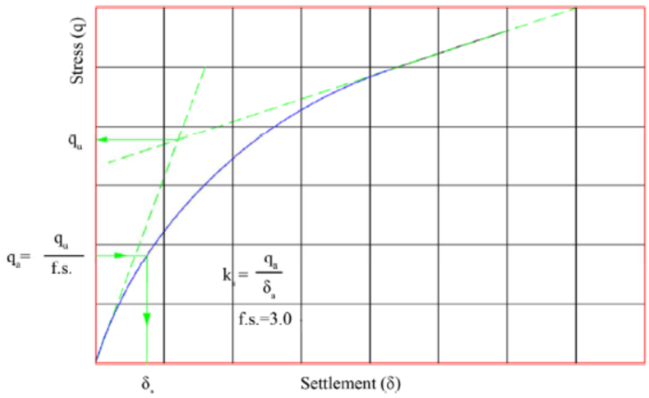
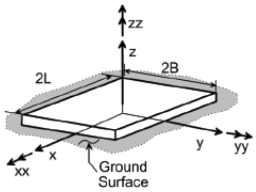


Figure 0: Determination of subgrade reaction (k_S).

Determination of Surface Stiffness

(FEMA 2020) [16], in section 6.3 there are three methods about how to modelling the SSI. The first method is rigid foundation and flexible soil (figure1) in which the modelling of foundation depended on six formulas for six degree of freedom table 2.

Table 2: the modelling of foundation depended on six formula of six degree of freedom.

Surface Stiffness (FEMA 2020) [16] under Title “ **A Practical Guide to Soil-Structure Interaction”** 2BX2L 
$k_{x,sur} = \frac{GB}{2-v}\left[6.8(\frac{L}{B})^{0.65} + 2.4\right]$
$k_{y,sur} = \frac{GB}{2-v}\left[6.8(\frac{L}{B})^{0.65} + 0.8\frac{L}{B} + 2.4\right]$
$k_{z,sur} = \frac{GB}{1-r}\left[3.1(\frac{L}{B})^{0.75} + 1.6\right]$
$k_{xx,sur} = \frac{GB^3}{1-r}\left[3.2(\frac{L}{B})^{1} + 0.8\right]$
$k_{yy,sur} = \frac{GB^3}{1-r}\left[3.73(\frac{L}{B})^{2.4} + 0.27\right]$
$k_{zz,sur} = GB^2\left[4.24(\frac{L}{B})^{2.45} + 4.08\right]$
where G = shear modulus, v = Poisson’s ratio for elastic half space, B and L = Half-width and half-length of rectangular foundation.

Determination of Response Modification factor (R)

Most earthquake design codes of the building reduced the forces caused by seismic by using a single factor. This factor in the Egyptian code (ECP 2020) is called the response modification factor (R-factor), in the Euro code [19] is the behavior factor, and in (ASCE 2013) [4] is the response modification coefficient. The factor accounts for the nonlinear response of a structure by taking advantage of the fact that the structures possess significant reserve strength and capacity to dissipate energy, called over strength and ductility, respectively, [ATC, (1995a), (Elnashai and Mwafy 2002) [20], (Asgarian and Shokrgozar 2009) [21] Thus, the response reduction factor (R) is:

$R = R_\mu \times \Omega$ (2)

$\Omega = V_u / V_d$ (3)

$\mu = \Delta_u / \Delta_y$ (4)

$R_\mu = 1.0$ for zero-period structures

$R_\mu = \sqrt{2\mu - 1}$ for short-period structure

$R_\mu = \mu$ for long-period structure

$R_\mu = 1 + (\mu - 1)\, T/0.70 \quad (0.70 < T < 0.30)$ (5)

Where:

R_μ is the ductility reduction factor V_u is the actual strength, V_d is the design strength. μ is the displacement ductility, Δ_u is the ultimate displacement and Δ_y is the yield displacement.

different formulations to determine the reduction factor (R) are in the next studies, (Newmark and Hall 1969)[22], (Uang 1991) [23], (Paulay and Priestley 1992) [24], (Miranda and Bertero 1994) [25], (Kappos 1997) [26], (Priestley 2000) [27], (FEMA 2000) [28], (Maheri and Akbari 2003) [29], (Elnashai and Mwafy 2002) [20], (Mondal, Ghosh, and Reddy 2013) [30], (Freeman 1990) [31], (Lee, Cho, and Ko 2005) [32], (Rodrigues et al. 2012) [33], (Varum 2003) [34].The relation between the base-shear of a structure and its roof displacement which can be calculated by a nonlinear static analysis has been illustrated in figure 3.

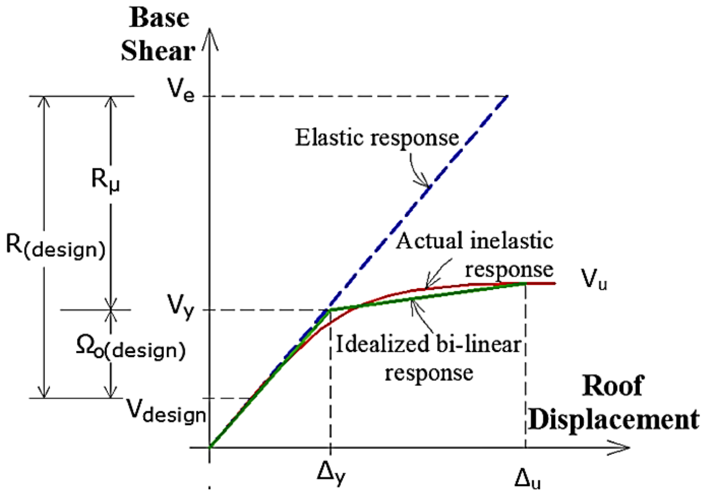


Figure 0: Relationship between force reduction factor (R), structural over-strength (Ω), and ductility reduction factor (R_μ).

PUSHOVER ANALYSIS (NONLINEAR STATIC ANALYSIS) AND PERFORMANCE LEVEL

In the current study to evaluate the global limit states of the RC-MRF (moment resistant Frames) in terms of drift and force level nonlinear static analysis, which is known as pushover analysis, was used. In this analysis, the increasing forcing function. Structures with predictable performance within established thresholds of risk and dependability are the result of performance-based engineering (FEMA 356 and ATC). The main goal is to keep the structure from collapsing completely. This indicates that the top-level can resist catastrophic collapse. (CP); the sub-level, which houses the critical structures, can be minimally damaged yet still be occupied immediately. (IO). Between the sub and upper levels, there is a Life-Safety. (LS) level condition. Nonlinear load-deformation relation must be defined according to FEMA's nonlinear procedures. Figure 4 depicts such a curve.

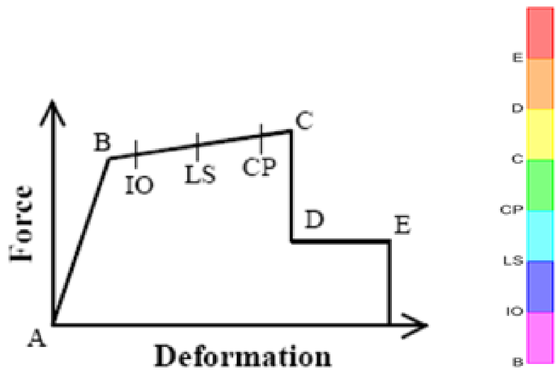


Figure 0: Typical load-deformation relation and target performance levels

According to FEMA, Points (A. B. C. D. and E) are utilized to describe the behavior of hinge. The hinge's approval requirements are defined by three additional ones: immediate-occupancy (IO), life-safety (LS), and collapse-prevention (CP). (ASCE, 2017b) lists many performance targets for various levels, including the seismic transition phases.

COMPARATIVE EXAMPLE

A one-span, two-story, flexure-critical reinforced concrete frame was tested by Vecchio and Emara (1992) to gain further insight into the magnitude and influence of shear deformations in flexure-critical frame structures and to assess the accuracy of analytical procedures developed. The frame was constructed with a center-to-center span of 3.5m and 2 m story height with 4.6 m total height. All elements were 30 cm in width and depth of 40 cm. The frame was constructed with a massive, extensively reinforced concrete base to provide a stable foundation as shown in Figure 6. Cylinder testing and steel coupon tests were used to assess material characteristics, as shown in Figure 5 (a) and Figure 5 (b). The model laboratory evaluated the frame by delivering a continuous axial load of 700 kN to each column while adding a lateral load to the second story beam in a monotonous manner until the frame's maximum capacity was attained. Two hydraulic jacks of 450 kN applied through two transverse beams in the force-controlled mode produced the column loads. In a displacement mode, a 1000 kN capacity actuator was placed laterally against a responding strong wall to supply the lateral load, and the Base shear Versus Displacement Curves was plotted.

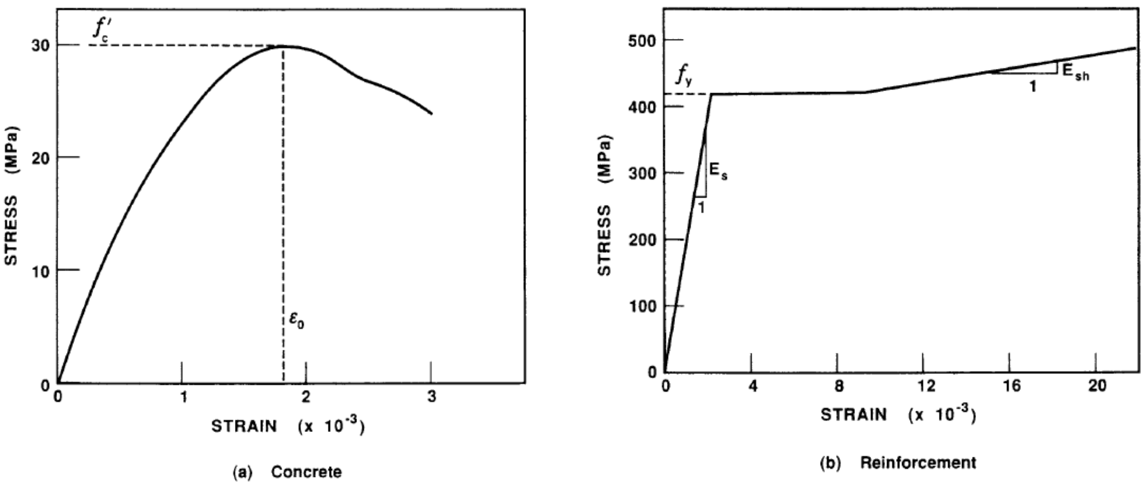


Figure 5: Material stress-strain properties

Pushover analysis is applied on the buildings and displacement control analysis is used with targeted monitored displacement at the top story about 4% from total building height. The results were compared with the experimental results and RUAUMOKO software results (Güner,2008) [35]. The results showed that the result from SAP 2000 software is almost the same as the experimental and other software results. Ultimate and yield steps base shear and displacement very close and almost identical for both.

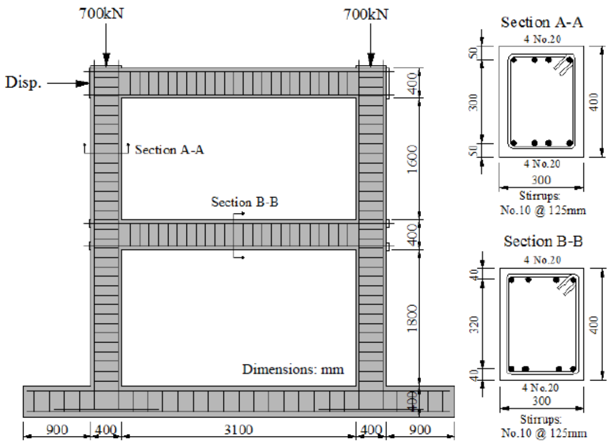


Figure 6: Details of Frame (Vecchio and Emara 1992)

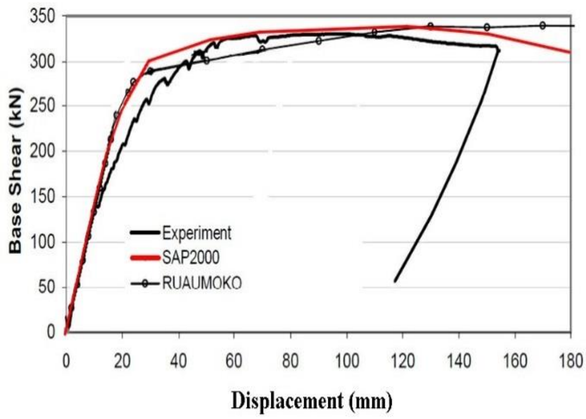
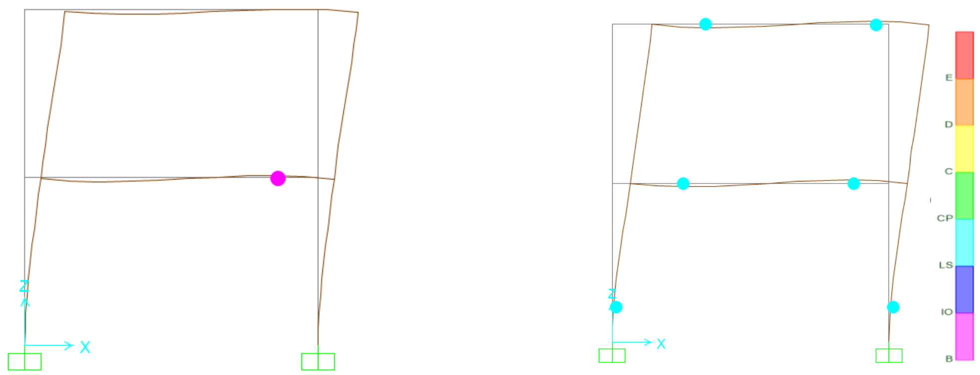


Figure 7: Base shear Vs. Displacement Curves.



V = 268.0 kN Δ= 23.2 mm V =342.0 kN

Δ= 101 mm

Figure 8: Condition of the hinges for Vecchio and Emara Frame using SAP 2000 program

From Figure 6 to Figure 8, it is obvious that experimental and numerical RC structures illustrate that finite element software Sap2000 can be used efficiently for predicting the nonlinear seismic performance of RC concrete structures. Ultimate and yield base shear, as well as displacement, are almost identical.

Table 3: Comparison of Analytical and Experimental Results for Vecchio and Emara Frame

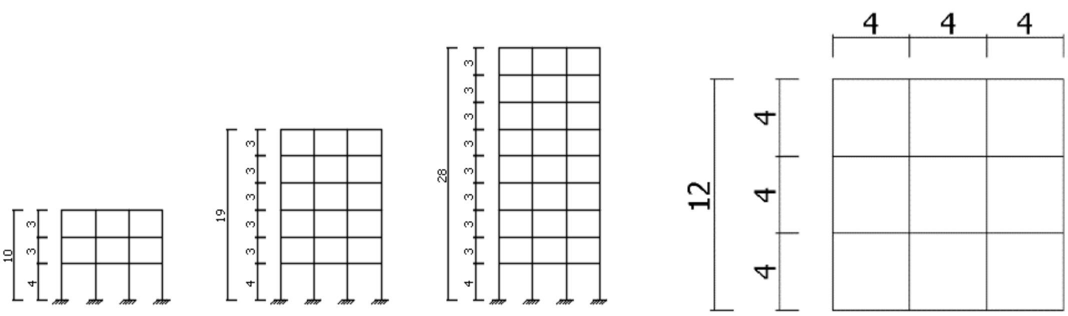
		Experiment (Vecchio and Emara (1992))	**RUAUMOKO** (Serhan Güner, 2008) [35]	**SAP2000** (Present work)
Yielding	Load (kN)	264	265	268
	Disp. (mm)	30.2	22.0	23.2
Ultimate	Load (kN)	332	339	342
	Max Disp. (mm)	97	180	101
	Failure Mode	6 Plastic Hinges	4 Plastic Hinges	6 Plastic Hinges

NUMERICAL STUDY FOR SEISMIC PERFORMANCE FOR MULTI-STORY BUILDING CONSIDERING SSI

The main purpose of this study is to discuss seismic performance for a multi-story building considering SSI. The buildings, which have been studied in this numerical study, are reinforced concrete framed buildings. These systems have been designed according to ECP-203 (2020) against gravity and seismic loads using ECP-201 (2012) (spectrum type 2). The pushover analysis method (P.O.A.) is used to assess the seismic design demands of structures so that the (P.O.A.) is statically nonlinear analysis method dependent on applying equivalent lateral loads due to shaking movement along with the height of the structure, and the structure is pushed along the side until a pre-characterized failure state is come to. The expected plastic hinges will appear at the main structural members under incrementally increasing loads, hence, we can draw a pushover curve representing the relationship between the base shear force and roof displacement for each structure to determine the 'R' factor. The Analysis has been calculated by the sap 2000 program which period of vibration (T) by its empirical equation so that we make a comparison between the design base shear for two seismic zone intensity 0.25g using ECP-201 (2012) [spectrum type 1 and 2]. The determination of plastic hinges status at yield and ultimate states have been performed by nonlinear pushover static analysis (P.O.A) then the calculation of the response modification factor 'R' for reinforced concrete framed buildings with 3, 6, and 9 stories. The discussions and suggestions are given in this field according to the results.

Description of Models

Reinforced Concrete multi-story 3 bays framed buildings with 3, 6 and 9 stories have been investigated utilizing SAP2000 (V20.1) auxiliary examination programming bundle (2016) The structures region unit displayed 3D framed structure using frame elements for columns, longitudinal beams, and transverse beams, using frame elements for columns with rigid floor diaphragms distribute uniformly the lateral loads on the vertical components. Figure 9 shows elevation and plane layout for buildings dimensions. Material properties for reinforced Concrete buildings are represented in Table 4. Stress-strain curves for concrete and, steel bars are illustrated in figure 10.

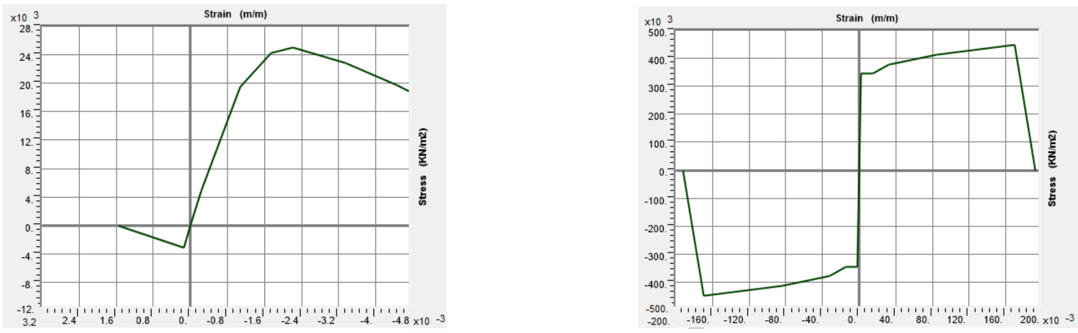


Elevation Plan

Figure 9: Layout of studied buildings (Framed buildings)

Table 4: Material Properties for Buildings

Fc	25000 kN/m²	concrete strength
Fy	345700 kN/m²	rebar yield strength
Ec	22000000 kN/m²	modulus of elasticity of concrete
Es	2.0E+8 kN/m²	modulus of elasticity of rebar
G	10356491 kN/m²	Shear modulus
Y	0.2	Poisson's ratio



(a) Stress-strain curve for concrete (b) Stress-strain curve for steel bare

Figure 10: Stress-strain curves introduced in SAP2000 (Computer & Structures Inc., 2018)[36]

The following loading assumptions have been considered:

Total Dead Load (D) is equal to DL+SDL+CL

Dead Load (DL) is equal to the self-weight of the members and slabs.

Super-imposed Dead Load (SDL) equals 3.0 kN/m². SDL includes partitions and ceiling weight.

Cladding Load (CL) is applied only on perimeter beams.

Live Load (L) equals 2.0 kN/m².

The studied buildings are subjected to different types of load combinations according to ECP 2012. These combinations are applied by the following terms:

$U = 1.40\ D + 1.60\ L$

$U = 1.12\ D + \alpha\ L \pm S$

Where D is the dead load, L is the live load; S is the seismic load and superposition factor of the structure's the residential buildings.

RC frame buildings with 3, 6 and 9 stories have been designed according to ECP-203 (2007) against gravity and seismic loads using ECP-201 (2012) (spectrum type 2). The analyses have been carried out using spectrum types 2 for the 0.25g zone. The soil is considered soil class C and the reduction factor limited ductility of the moment-resisting frame, R, is taken equal 5. Software (Sap2000 v20.1) [36] is utilized to create a 3-D finite element model, for computation of the ultimate straining actions on slabs, beams, and columns due to designed loads. The following points have been considered through the design process:

The moment resisting frame type is considered sway type (for calculating effective length factor).

The inter-story drift should not exceed 0.005 of the story height, h, to verify the damage limitation requirements.

The assumed steel ratio for the columns is varying from 1.0% to 1.6% relative to the cross-section area. In case the element capacity for axial load and biaxial bending does not satisfy the corresponding design value, the column section is increased keeping the same steel ratio.

In case the ultimate resistance force provided by shear reinforcement does not satisfy the demand design value for the shear force, the specified stirrups for the column are changed to satisfy such demand.

The base code used for column design in the software Sap2000 is BS8110. Modifications to some design parameters are implemented to be compatible with the design requirement of the Egyptian code.

The Design code of aids and examples, part 1 according to ECP-203 (2007) have been used to check the design of the column.

For RC multi-bay frame buildings with 3, 6, and 9 stories, tables 5, 6, and 7 summarize design column sections. In these tables, the design column sections are given for seismic zone intensitie 0.25g using spectrum type 2. The steel reinforcements of beams are given in the table 8 for each designed building. The capacity/demand ratios for most columns are in lower stories of all the studied buildings and within the range from 0.75 to 0.90.

Table 5: Column sections for 3 story buildings by seismic loads using ECP-201 (2012), Multi-bay frames

Design zone	Spectrum type	Story number	
		(1), (2), (3)	
		Interior column	Exterior column
0.25g	(I)	35x35 (8 ϕ16)	25x60 (10 ϕ16)
	(II)	40x40 (8 ϕ16)	25x75 (14 ϕ16)
0.15g	(I)	35x35 (8 ϕ14)	25x45 (6 ϕ16)
	(II)	35x35 (8 ϕ16)	25x50 (8 ϕ16)

Table 6: Column sections for 6 story buildings by seismic loads using ECP-201 (2012), Multi-bay frames

Design zone	Spectrum type	Story number			
		(1), (2), (3)		(4), (5), (6)	
		Interior column	Exterior column	Interior column	Exterior column
0.25g	(I)	45x45 (12 ϕ16)	25x70 (12 ϕ16)	40x40 (8 ϕ16)	25x60 (10 ϕ16)
	(II)	55x55 (12 ϕ18)	25x120 (22 ϕ18)	45x45 (12 ϕ16)	25x100 (12 ϕ16)
0.15g	(I)	45x45 (12 ϕ16)	25x65 (10 ϕ16)	35x35 (8 ϕ16)	25x50 (8 ϕ16)
	(II)	45x45 (8 ϕ18)	25x80 (14 ϕ16)	40x40 (8 ϕ16)	25x70 (10 ϕ16)

Table 7: Column sections for 9 story buildings by seismic loads using ECP-201 (2012), Multi-bay frames

Design zone	Spectrum type	Story number					
		(1), (2), (3)		(4), (5), (6)		(7), (8), (9)	
		Interior column	Exterior column	Interior column	Exterior column	Interior column	Exterior column
0.25g	(I)	50x50 (12 ϕ16)	25x100 (16 ϕ16)	40x40 (8 ϕ16)	25x85 (14 ϕ16)	35x35 (8 ϕ16)	25x70 (12 ϕ16)
	(II)	65x65 (16 ϕ18)	25x125 (16 ϕ18)	50x50 (14 ϕ18)	25x100 (16 ϕ18)	40x40 (8 ϕ16)	25x80 (12 ϕ18)
0.15g	(I)	50x50 (12 ϕ16)	25x90 (12 ϕ16)	40x40 (8 ϕ16)	25x70 (10 ϕ16)	35x35 (8 ϕ16)	25x50 (8 ϕ16)
	(II)	55x55 (12 ϕ18)	25x100 (12 ϕ18)	45x45 (8 ϕ16)	25x85 (12 ϕ16)	35x35 (8 ϕ16)	25x70 (10 ϕ16)

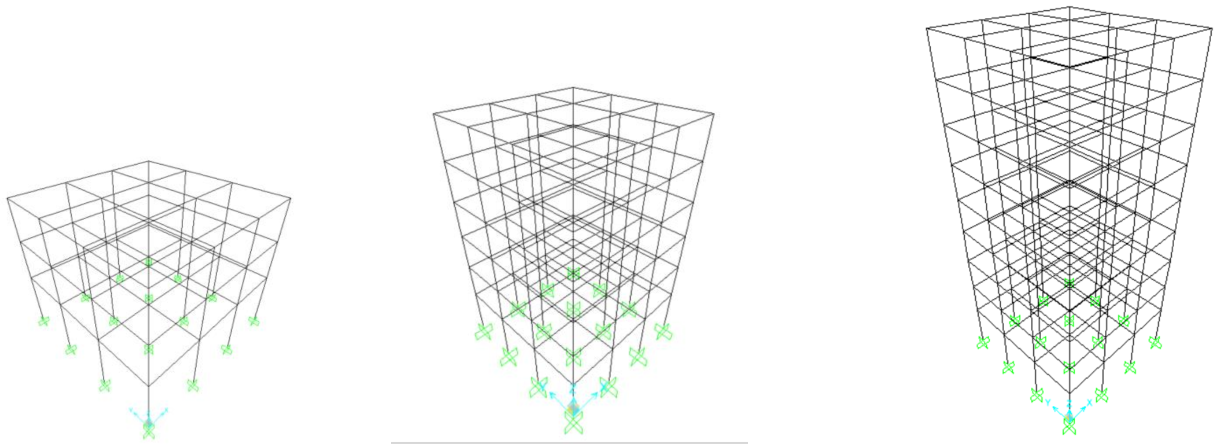
Table 8: Reinforcement of Beams by seismic loads using ECP-201 (2012), Multi-bay frames

Design zone	Spectrum type	3 story building		6 story building		9 story building	
		Upper	Lower	Upper	Lower	Upper	Lower
0.25g	(I)	4 ϕ16	3 ϕ16	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16
	(II)	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16	6 ϕ16	5 ϕ16
0.15g	(I)	3 ϕ16	2 ϕ16	3 ϕ16	2 ϕ16	2ϕ16+2 ϕ14	2ϕ16
	(II)	3 ϕ16	2 ϕ16	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16

After that, (the second step), the fixed supports changed by spring supports using equations in table 6 and Mechanical properties of selected soils in table 9 (See figures11, 12, and 13 for the cases of the study. In the third step, the fixed supports also changed by isolated footing support with (k_s) value are measured using formulas in table 2 and Mechanical properties of selected soils in table 9 for the only case of 3 stories from ECP 201 (2012) for (type 2 spectrum) and (seismic zone pressure 0.25 g).

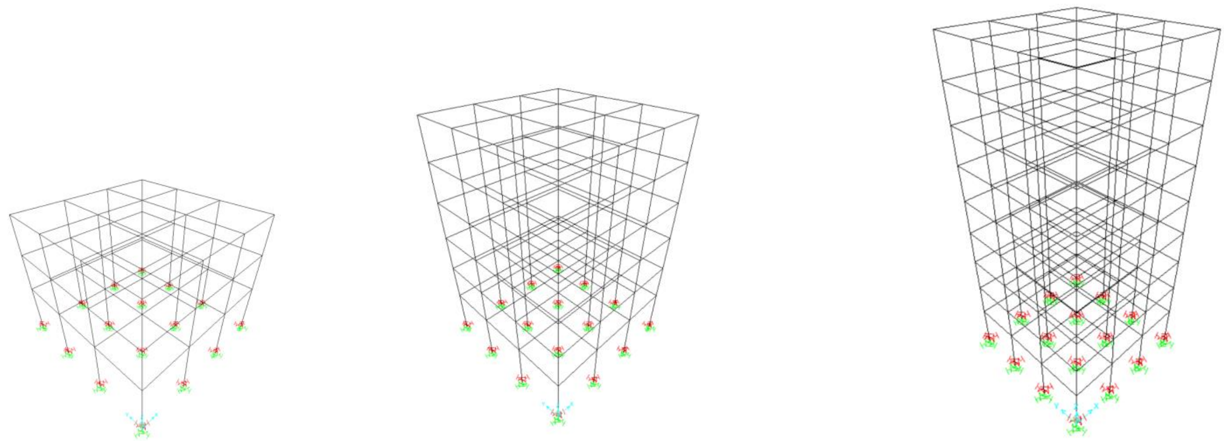
Table 9: Mechanical properties of selected soils

Soil type	Bulk unit weight (Y) (t/m3)	The angle of shearing resistance (φ)	Soil young`s modulus (E_s)	Poisson`s ratio	Mean shear wave velocity (m/s)
B (very dense sand)	1.9	45	200	0.3	540
C (medium dense sand)	1.9	40	80	0.3	290
D (loose soil)	1.9	30	45	0.3	150



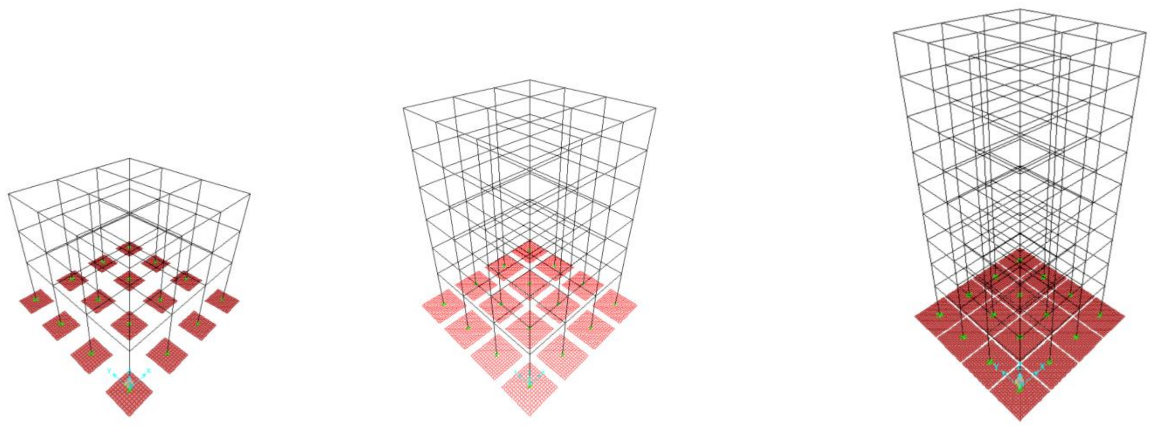
3 stories Fixed supports 6 stories Fixed supports 9 stories Fixed supports

Figure 11: Multistory buildings with Fixed supports



3 stories spring supports 6 stories spring supports 9 stories spring supports

Figure 12: Multistory buildings with spring supports



3 stories isolated footing with Ks 6 stories isolated footing with Ks 9 stories isolated footing with Ks

Figure 13: Multistory buildings with isolated footing with K_s.

The dimension of footing used in the previous models was designed by using ECP 201 (2012) by using the envelope reaction of the models. So the isolated footings for 3, 6, and 9 stories were taken as (2.2x2.2x0.60) m, (3.1x3.1x0.60) m, and (3.75x3.75x1.00) m respectively.

Cases Of Study

For RC framed structures with 3, 6, and 9 stories, the following cases were considered:
Compare the period of vibration (T) calculated by SAP 2000 program [36] for the fixed supports, spring supports, isolated footing with k_s modulus, and isolated footing with soil layers buildings.
Plot the (P.O.C.) for the fixed supports, spring supports, isolated footing with ks modulus, and isolated footing with soil layers buildings
Determination of the plastic hinge status by pushover analysis method (P.O.A.) for the fixed supports, spring supports, isolated footing with ks modulus, and isolated footing with soil layers buildings
Determine the response modification factor R for RC framed buildings with 3, 6, and 9 stories for the fixed supports, spring supports, isolated footing with k_s modulus, and isolated footing with soil layers buildings which have been designed according to ECP-203 (2007) against gravity and seismic loads using ECP-201 (2012) for seismic zone intensity 0.25g using (spectrum type 2) for ECP-201 (2012).

Results and Discussion

The results of the cases studies were grouped into the 3, 6, and 9 stories buildings groups for (type 2 spectra) and (seismic zone pressure 0.25 g) from ECP 201 (2012). In which were compared between different types of supports under structures. These supports were the fixed supports, spring supports, and isolated footing with ks modulus buildings.

Fundamental Natural Periods of The Structures

Assurance of the fundamental period of vibration (T) of a structure is essential in earthquake design. Standard design practices typically use code-recommended empirical equations to estimate the design base shear. The current code equations (ECP (2012) provide the formulas or the approximate period of moment-resisting frames (MRFs), which are only dependent on the height of the structures.
The Fundamental Natural period obtained from SAP2000 (v20.1) [36] is outlined in table 10 and figure 14 for the 3 stories buildings for (type 2 spectrum) and (seismic zone pressure 0.25 g) from ECP 201 (2012).

Table 10: Fundamental Natural periods for different types of supports of the study structure

0.25g-SP2	T		
model	3 stories	6 stories	9 stories
fixed support	0.476	0.778	1.094
Type C SPRING	0.52134	0.844912	1.179987
Type D SPRING	0.56625	0.896569	1.250244
Type C Isolated Footing	0.566801	0.874269	1.178802
Type D Isolated Footing	0.610728	0.914394	1.222637

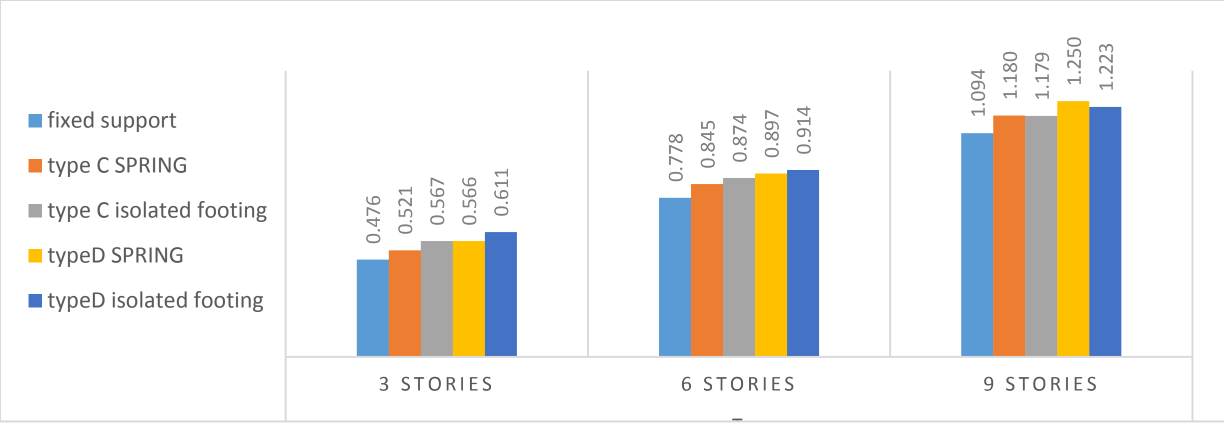
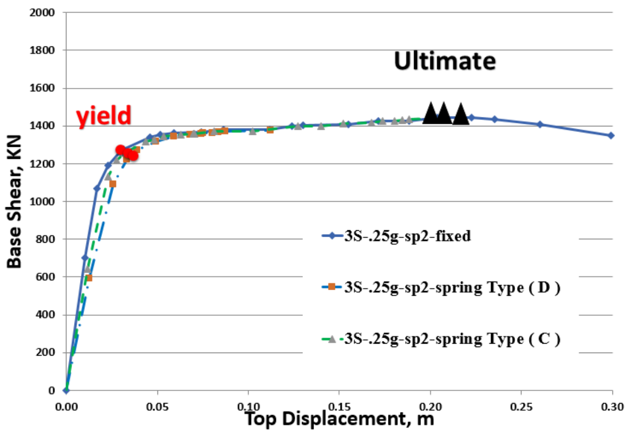
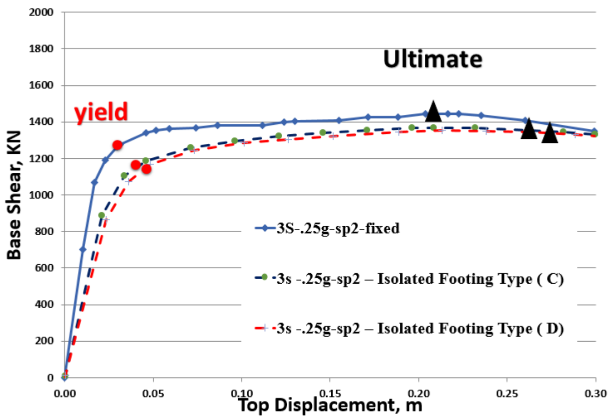


Figure 14: Fundamental Natural periods for different types of supports of the study structure

Plot The Pushover Curve (P.O.C.) For The Studies Building

In figure 15, the pushover curve plotted from sap 2000 v20.1 for the 3 stories buildings for (type 2 spectrum) and (seismic zone pressure 0.25 g) from ECP 201 (2012). The x-axis is the Top displacement by meter unit and the y axis is the base shear by KN unit. Figure 15 left shows the curves of the all stories type 2 spectrum zone0.25g with fixed support and spring support which is equal to type C and Type D. Figure 15 right shows the curves of all stories type 2 spectrum zone 0.25g with fixed support and Isolated footing which is equal to type (C) and (D).





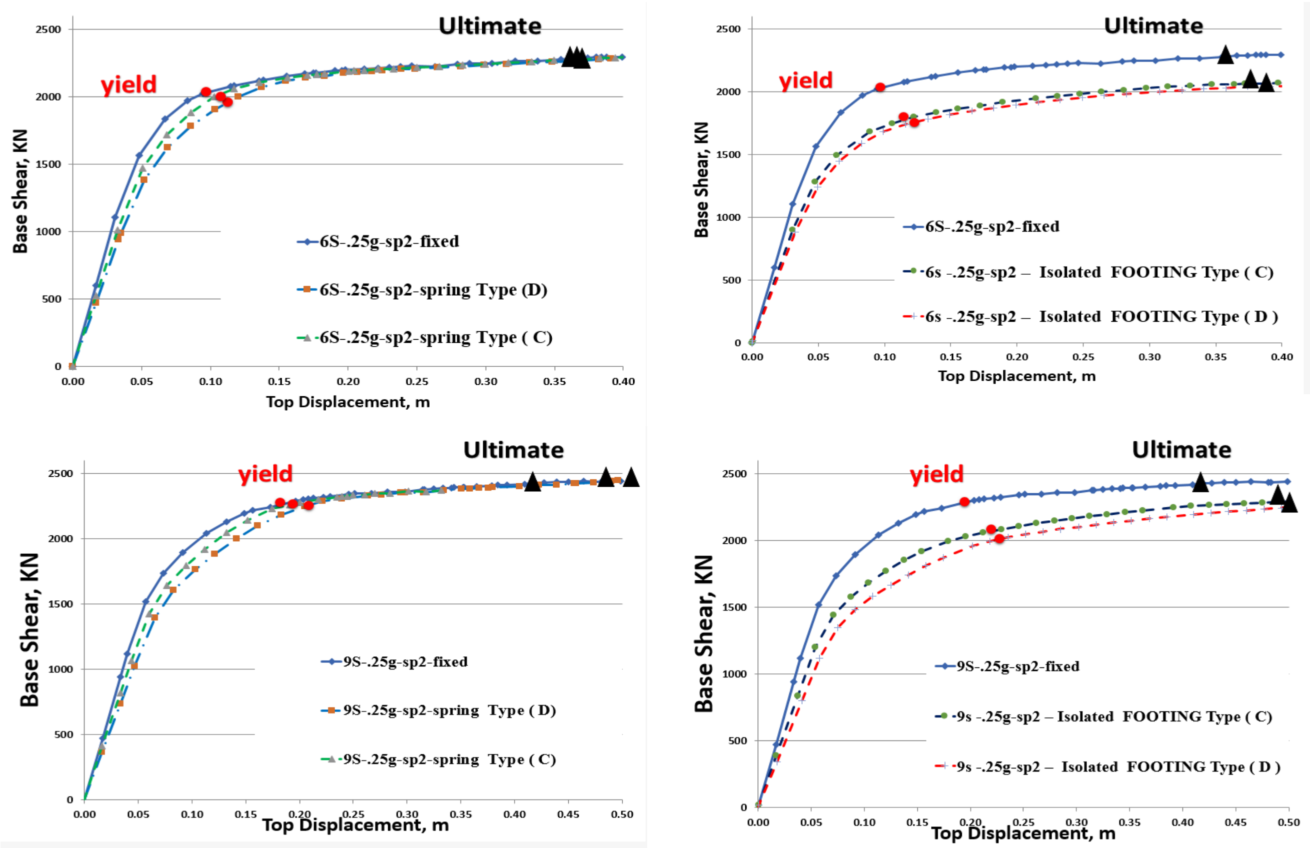


Figure 15: Plot the pushover curve (P.O.C.) for the spectrum type 2 and zone 0.25 g buildings

Calculate The Response Modification Factor R For RC Framed Buildings With 3, 6, And 9 Stories For Different Types Of Supports

Formulas (2), (3), (4), and (5) are used for estimating the “R” from pushover curve results for all the studied structures (three, six, and nine stories). Table 11 shows the values of three stories type 2 spectrum zone 0.25g with fixed support, spring supports and isolated footing which is equal to type (C) and (D). Table 12 shows the values of six stories type 2-spectrum zone 0.25g with fixed support, spring supports and isolated footing, which is equal to type (C) and (D). Table 13 shows the values of nine stories type 2-spectrum zone 0.25g with fixed support, spring supports and isolated footing which is equal to type (C) and (D). all this value are in figure 16.

Table 11: Response Modification factor(R) by using this equation ($R = R_u * R_s$) for 3 stories models

0.25g-SP2								
3 stories models.	Δu	Δy	μ	R_u	V_y	V_d	R_s	R
Fixed support	0.216	0.031	6.935	3.59	1276.114	647.666	1.970	**7.07**
Type C SPRING	0.215	0.032	6.686	3.52	1260.448	667.647	1.888	**6.64**
Type D SPRING	0.228	0.039	5.832	3.27	1272.34	667.647	1.906	**6.22**
Type C isolated footing	0.282	0.046	6.112	3.35	1184.209	667.647	1.774	**5.94**
Type D isolated footing	0.288	0.048	5.965	3.31	1168.375	656.088	1.781	**5.89**

Table 12: Response Modification factor(R) by using this equation ($R = R_u * Rs$) for 6 stories models

0.25g-SP2								
6 stories models.	Δu	Δy	μ	R_u	V_y	V_d	R_s	R
Fixed support	0.366	0.117	3.131	2.29	2085.532	1025.153	2.034	**4.67**
Type C SPRING	0.366	0.136	2.703	2.10	2106.286	1018.315	2.068	**4.34**
Type D SPRING	0.365	0.137	2.657	2.08	2073.664	959.644	2.161	**4.49**
Type C isolated footing	0.372	0.139	2.679	2.09	1834.985	984.122	1.865	**3.89**
Type D isolated footing	0.374	0.149	2.508	2.00	1820.231	940.937	1.934	**3.88**

Table 13: Response Modification factor(R) by using this equation ($R = R_u * R_s$) for 9 stories models.

0.25g-SP2								
9 stories models.	Δu	Δy	μ	R_u	V_y	V_d	R_s	R
Fixed support	0.430	0.198	2.176	2.18	2304	1213	1.899	**4.13**
Type C SPRING	0.500	0.209	2.393	1.95	2288.75	1153.581	1.984	**3.86**
Type D SPRING	0.512	0.213	2.404	1.95	2268.64	1088.004	2.085	**4.07**
Type C isolated footing	0.521	0.212	2.451	1.98	2059.761	1154.681	1.784	**3.52**
Type D isolated footing	0.527	0.219	2.411	1.96	1993.927	1113.168	1.791	**3.50**

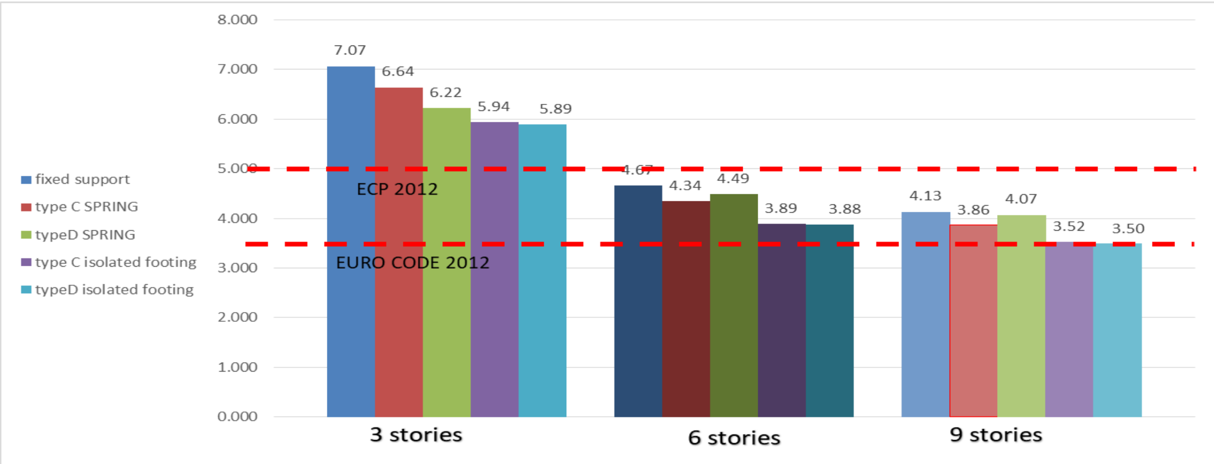


Figure 16: Response Modification factor(R) for 3, 6, and 9 stories models.

SUMMARY AND CONCLUSIONS

The impacts of SSI on the seismic performance of 3D RC moment-resisting frames:-
In terms of internal forces, the SSI affects the structure of internal forces such as bending moment and axial forces, which alters the structural element's plastic hinge development.
Changing in soil characteristics affect the seismic performance of buildings, with taller structures and softer soil profiles seeing alterations that are more substantial.
The SSI effects should be addressed while evaluating the performance of RC structures, particularly when they are built on soft soils. Furthermore, it is determined that estimating the total performance of buildings based just on the performance of some specific structural components may be misleading.

Seismic reactions of all structural elements should be evaluated, particularly for weak ground conditions, to more precisely predict the seismic performance of structures.

For the studied cases 3, 6, and 9 buildings, the maximum reduction in R because of taking into account SSI reached 16% (R fixed support vs R isolated footing type D) in loose soils. The percentage of reduction in V_u max because of taking into account SSI in loose soils is within 12% (V_u max fixed support vs V_u max isolated footing type D). The percentage of reduction in V_d as a result of taking into account SSI in loose soils is within 10% (V_d fixed support vs V_d isolated footing type D).

Response reduction factor (R) of RC limited ductility framed buildings:-

The response reduction factor is considerably affected by the seismic zone and fundamental natural period of the structure. It reduces as the seismic zone increases and increases as the fundamental time period increases.

The given value of R-factor at ECP-201(2012) equals 5.0 for limited ductility class of reinforced concrete moment frame structures is un-conservative value; as the accurate value of R-factor is less than the given value.

Recommended value of response reduction factor R for limited ductility class of limited ductility reinforced concrete moment frame structures in ECP-201(2012) is 3.9 for multi-story multi-bay frames.

It may be noted that Eurocode-8 (2004) specify values response reduction factor range between 3.0 and 3.9 for medium ductility reinforced concrete moment frames according to the frame configuration (multi-story multi-bay frames). UBC 97and IBC 2018 identify for RC ordinary frame buildings response reduction factor 3.5 and 3.0 respectively.

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