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DYNAMIC ANALYSIS USING THE SUPERPOSITION OF MODES AND HORIZONTAL EARTHQUAKE SPECTRA: COMPARISON WITH STATIC ANALYSIS IN THE EXAMPLE OF A 20-STOREY BUILDING

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Abstract

This paper applied the three earthquake analysis methods for analyzing the response of 20-storey building, the paper provides a comparison study between dynamic and static analysis methods to evaluate the level of accuracy when using the static method, where system and ground motion parameters addressed in the Iraqi code for the city of Baghdad are used in this study. The study compared the results of Equivalent Lateral Force Analysis (ELFA) and Response Spectrum Analysis (RSA) with Time History Analysis (THA). Therefore, the aim of the study is to compare the results of ELFA with THA applied by using superposition of modes and artificially earthquake accelerations scaled to the site design spectrum in order to compare the results of ELFA for the example of 20-storey building, the study concluded that static analysis is not directly sufficient for 20-storey building, because it has been showing inaccurate response results, where it shows a lower design base shear if compared with elastic base shear obtained from dynamic analysis further to inaccurate displacements results. Finally, dynamic analysis leads to more seismic resistant design than static analysis if used with a desired ductility and desired strength modifications factors.

Keywords: Earthquake, Static, Dynamic, Response, Baghdad, Iraq

MODLARIN VE YATAY DEPREM SPEKTRUMLARININ ÜST ÜSTÜNE GELMESİNİ KULLANAN DİNAMİK ANALİZ: STATİK ANALİZLE KARŞILAŞTIRMA 20 KATLI BİR BİNA ÖRNEĞİNDE

Öz

Bu makale, 20 katlı bir binanın davranışını analiz etmek için üç deprem analiz yöntemini uygulamıştır, bu makale, sistem ve yer hareketi parametrelerinin aşağıda ele alındığı statik yöntemi kullanırken doğruluk seviyesini değerlendirmek için dinamik ve statik analiz yöntemleri arasında bir karşılaştırma çalışması sunmaktadır. Bağdat şehri için Irak kodu bu çalışmada kullanılmıştır. Çalışma, Eşdeğer Yanal Kuvvet Analizi (ELFA) ve Tepki Spektrum Analizi (RSA) sonuçlarını Zaman Tanım Alanı Analizi (THA) ile karşılaştırdı. Bu nedenle çalışmanın amacı, ELFA sonuçlarını, 20 katlı bina örneğinde ELFA sonuçlarını karşılaştırmak için modların süperpozisyonu ve saha tasarım spektrumuna ölçeklenmiş yapay deprem ivmeleri kullanılarak uygulanan THA ile karşılaştırmaktır. çalışma, statik analizin 20 katlı bina için doğrudan yeterli olmadığı sonucuna varmıştır, çünkü dinamik analizden elde edilen elastik taban kesme kuvveti ile karşılaştırıldığında yanlış yer değiştirme sonuçlarına göre daha düşük bir tasarım taban kesmesi gösterdiği hatalı davranış sonuçları göstermektedir. Son olarak, dinamik analiz, istenen süneklik ve istenen dayanım modifikasyon faktörleriyle birlikte kullanılırsa, statik analizden daha sismik dirençli tasarıma yol açar.

Anahtar Kelimeler: Deprem, Statik, Dinamik, Müdahale, Bağdat, Irak

1. Introduction

Dynamic analysis considers important method because it produces structural designs are more seismic resistant (Wilson & Edward L., 2002). The paper provides a comparison study between dynamic and static analysis methods to evaluate the level of accuracy when using static method in the analysis, where system and ground motion parameters addressed in the Iraqi code are used in this study. The code parameters are defined in "Earthquake - Iraqi Building code-303" (ISC-303, 2017), where ELFA and RSA are specified in Chapter 3, THA considered a supplementary method in the same chapter should consist a mathematical model of the structure to find its response to acceleration history of a ground motion, where these accelerations must be equivalent to the site design spectrum, further to a constant Response Modification Factors (R) values are specified to be used in ELFA and RSA, where RSA results must be not lower than 85% of ELFA (ISC-303, 2017). The issue of this paper: due to use constant value of *R,* results obtained from RSA gives lower design base shear, despite this method considers dynamic analysis for evaluating the response of structural systems, where it depends on create a mathematical model of superposition of modes and seismic inputs. But the lower RSA strength results lead to scale it to ELFA results. Therefore, verifying ELFA results is the aim of this paper, where the verification needs to provide practical approach to simulating an earthquake motion. Fortunately, the numerical simulation using THA has been providing this approach. If conducting this dynamic analysis manually, several steps have explained in the textbooks are needed and summarized in this paper. Therefore, superposition of modes has been explained basically by (Rajasekaran, 2009), he explained the mathematical model required to obtain the eigenpairs which are depending on constructing lamp mass and stiffness matrices. (Taranath, 2017) has provided initial understanding of the structural response in a simple manner; this book has included a simplified procedures and formulas for constructing the mathematical model to determine mass and stiffness distribution for tall buildings. Hasgür, Z. provides very useful concepts about *"Structural Design of Tall Buildings,"* where principles of structural dynamics are presented in this course and Muto's method has used in stiffness calculations (Hasgür, 2022), therefore, applying Muto's method lead to results are harmonizing the finding of Cruz, E. F. & Chopra in their paper about *"Elastic Earthquake Response of Frames"* in which the assumption of columns and beams are affected by bending deformation with joint rotation would lead to reduce errors in dynamic calculations (Cruz, E. F. & Chopra, A. K., 1986). Pervious researches, such as: (Bleichner & Noah, 2020) conducted a study on seismic analysis methods based on ASCE 7 , applied on shear frames up to 10-storey in height, the authors recommended to apply ELFA with dynamic seismic analysis methods because RSA gives responses 30% to 50% lower than the ELFA. (Meleka, N., Hekal, G., & Rizk, 2016) present a comparison between static and dynamic analysis, the researchers used Egyptian code, EC8 and UBC1997, they concluded that ELFA method gives higher results than RSA when computing drifts and overturning moments. (Mahmoud & Abdallah, 2014) also provide a study related to Egyptian code for 14-storey building, this study compares the torsional irregularity ratio in both ELFA and RSA analysis, the study recommended RSA to predict seismic response. (Faizah, Soebandono & Sugeha , 2021) have provided a study about the level of acceptance static analysis for 15-storey building analyzed by SAP2000, they concluded if building designed using ELFA a potential damage may occur when they used Yogyakarta Earthquake (2006) data in the analysis**Error! Reference source not found.**. A simple example about the reduction of elastic base shear is presented by (Chopra, 2020) and illustrated in Figure 1, when reducing the spectral acceleration by the Response Modification Coefficient (R), the building will be designed for a smaller base shear if compared with the elastic base shear corresponding to the design response spectrum (Chopra, 2020).

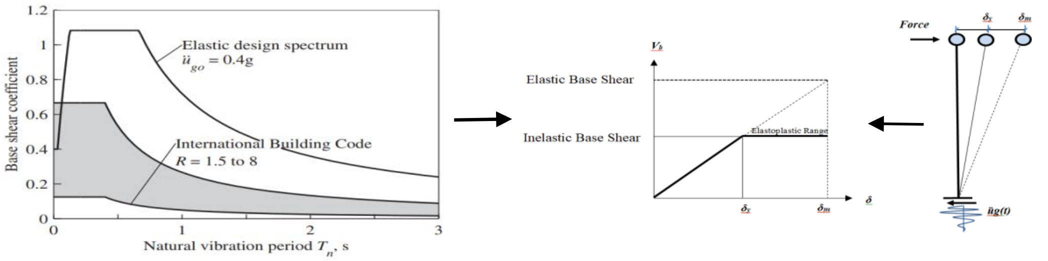


Figure 1: Reducing Base Shear from Elastic Response Spectrum, Yield Displacement $\boldsymbol{\delta_y}$ and Maximum Elastoplastic Displacement $\boldsymbol{\delta_m}$ (Chopra, 2020).

Base shear = *(A/g)w*, where *w* is the effective seismic weight of the structural system and *A* is the spectrum acceleration corresponding to the natural vibration period and damping of the system **Error! Reference source not found.**. The structure response interpreting by displacement indictor which is mainly depending on force magnitude acted on the structure; where the structure is designed to resist the force generated from a potential earthquake, if the design force greater or equal to the earthquake force, the system is within the elastic range. If not, then the system is entered to the elastoplastic rang, where design base shear performing the yield force which producing a yield displacement where this displacement indicates the system starts yield to elastoplastic range and it will be deformed at the end of earthquake. In the analysis process, the two indicators of yield displacement and maximum elastoplastic displacement are usually estimated depending on the ductility factor or Displacement Amplification Factor (C_d), this factor usually specified as a constant value less or equal to *R* depending on the procedure specified in some building codes and in which these factors are used in both ELFA and RSA. The situation is a bit different with THA, where Ductility Factor (μ) and Strength Reduction Factor (R_d) are assumed depending on the engineering judgment and based on the simulated earthquake spectra and codes conditions, where (Newmark & Hall, 1982) provides a useful method for estimating these factors. Simulating earthquake accelerations can be employed to conduct the analysis related to the linear or non-linear earthquake response for the structural system, actual or artificially earthquake motions can be used (Thomas & Priestley, 1992), where the response of the structure to these accelerations can be obtained, therefore, peak ground velocity and peak ground displacement are indicators may defined numerically by using linear interpolation of excitation which also refers as (time stepping method) every 0.02 sec (Chopra, 2020), this method lead to determine the Elastic Design Response Spectra (EDRS) based on the selected ground motion acceleration. While this design curve has been obtained, THA can be conducted using superposition of modes data corresponding the spectrum accelerations, where the results should be combined. Therefore, Complete Quadratic Combination (CQC) is preferable (Chopra, 2020).

2. Method

Procedures of methods have been used in the analysis are presented in this section. System and ground motion parameters used in the analysis are presented in accordance to ISC-303, these parameters are used to determine the maximum considered earthquake for Baghdad, this ultimate design representing 5% damped response spectral acceleration at short and 1 second periods, (Ss) and (S_1), in order to determine the base shear and displacements for the model analyzed statically using ELFA. Dynamic analysis using THA has been performed in this study is based on the principle of superposition of modes. Moreover, the procedure of using horizontal earthquake spectra for simulating earthquake accelerations, drawing steps of the artificially elastic design response spectra and matching site design spectrum are also described.

2.1. Analyzed Model

The preliminary model has been used in this study is dual system consists of special reinforced shear walls and special reinforced moment frames; the height of the building is up to 20-storey. The building will be used for office purposes with normal number of occupants in the potential event of occurring an earthquake. Therefore, the structure Risk Category is II with seismic importance factor, I_e= 1. The height of each story is 3.05m; in addition to, one meter height above the roof is the parapet along the perimeter 144m around the building, where, the dimensions of the building are (30 × 42 m). Plan of the building is shown in Figure 2, all stories are supposed to be used for office purposes, about 2% of the area for storage area to be used for light office equipment. The roof includes a penthouse to be constructed from braced steel in order to be used as mechanical room, however, it is not considered as a separated story. Two-way slab with beams has been selected for the floor system with thickness 150mm, where all columns and shear walls are assumed fixed at the base. Structural elements dimensions, loads details and strength proprieties are presented in Tables 1, 2 and 3.

Table 1: Strength Properties

Properties	***Data***
Modulus of Elasticity for Reinforced Concrete	29750 MPa
Concrete Class	C50
Yield Strength of Steel	420 MPa
Concrete Density	25 kN/m^3

Table 2: Structural elements dimensions for the 20-storey building.

Structural elements		***Dimensions (mm)***	
		b	***h***
Columns	C1	900	300
	C2	300	900
Shear walls	W1	2000	300
	W2	300	2000
Core	W1	2000	300
	W3	300	6000
Connection Beams	All beams	300	700

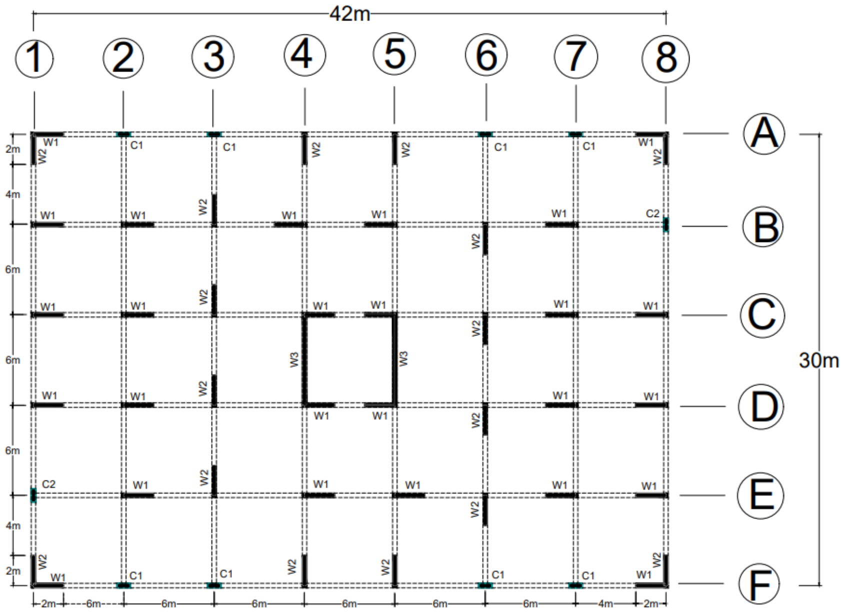


Figure 2: Plan of the 20-storey building

Table 3: Design live and dead loads acted on the building

Case	***Load Category***	
	Live Load	***Dead Load***
Flooring		1.5 kN/m^2
Ceiling and mechanical system		0.8 kN/m^2
Cladding		1.53 kN/m
Occupancy	4.8 kN/m^2	
Partitions	1 kN/m^2	
Storage	7.5 kN/m^2	
Penthouse		3 kN/m^2
Occupancy on roof	1 kN/m^2	
Single load on roof		9 kN
Elevator Machines		8 kN/m^2

Effective Seismic Weight (ESW) included all dead loads, further to 62.5% of the storage areas, 87.5% of partition and 50% of the rest of live loads. Therefore, determining ESW for each storey is obtained as the sum of all calculated weight, but it is important to distinguish between loads acted on roof and others loads acted on storey. Therefore, total ESW calculated in this study is shown below:

$$ESW_{storey} = (ESW_{beams} + ESW_{columns} + ESW_{slab} + ESW_{flooring} + ESW_{partitions} + ESW_{storage} + ESW_{ceiling} + ESW_{clad}) \quad (1)$$

$$ESW_{roof} = (ESW_{beams} + ESW_{slab} + ESW_{flooring} + ESW_{ceiling} + ESW_{penthouse} +$$

$$ESW_{elevator\ machine} + ESW_{parapet} + 0.5\ ESW_{clad}) \tag{2}$$

2.2. Static Analysis

The Analysis performed using simplified ELFA method has been conducted according to the steps described in (ISC-303, 2017):

Step-1- Determine the fundamental period of the structure (T):

1. Determine approximate fundamental period (T_a):

$$T_a = C_t \times h_n^x \tag{3}$$

where

C_t, x - Approximate period parameters

2. Determine upper limit period (T_u)

$$T_u = C_u\ T_a \tag{4}$$

where,

C_u - coefficient of upper limit on computed period equals 1.62 by interpolation.

If $T_u < T_a$, then T_u must be used.

Step-2- Determine the base shear (V), the designed base shear shall be determined by applying the equations below:

$$V = Cs\ W \tag{5}$$

where

V- base shear or the sum of the lateral shear forces

Cs - seismic response coefficient

W- effective seismic weight (kN)

$$C_s = \frac{S_{D1}}{T(R/I_e)} \tag{6}$$

But Cs shall not be less than:

$$Cs = 0.044\ \ S_{DS}\ \ I_e \tag{7}$$

Step-3- Distribute the base shear vertically, the distribution of base shear to storeys has been assumed depending on the response of the structure when it performs its first mode of vibration, the force F_x, is determined by the following equation:

$$F_i = C_v V \tag{8}$$

$$C_v = \frac{W_x h_x^z}{\sum W_i h_i^z} \tag{9}$$

where

W_x- effective seismic weight of a story (kN)

h_x – height of the story

W_i, h_i- mass and hight of all stories

z- parameter to approximate high mode effect

z is an exponent related to the structure period, it can be determined as follows:

interpolation is interpreted by the equation below **Error! Reference source not found.**:

$$z = 0.75 + 0.5\,T \tag{10}$$

Step-4- Determine the lateral deflection and drift, therefore, the yield displacement δ_y, or as known the deflection at any level, can be computed approximately by a simplified method illustrated in the following equations:

$$\Delta_i = \frac{Fi \times 10^4}{k} \tag{11}$$

$$\delta_{yi} = \sum_{i=1}^{i=x} \Delta\, i \tag{12}$$

where

Δ_i – drift of a storey i

k - considered stiffness in N-S directions calculated by Muto's method.

δ_{yi}- yield displacement at storey i

Then, the yield displacement should be amplified by the factor C_d:

$$\delta m = \frac{C_d\, \delta_{yi}}{I_e} \tag{13}$$

where

C_d - Deflection amplification factor

I_e- Importance factor

δm- Elastoplastic displacement calculated at level i

2.3. Apply Superposition of Modes

Develop a mathematical model of structure is required in order to determine a suitable mass and stiffness distribution, this model will lead to determine a set of mode shapes { $\O_m$ } corresponds to a period (T_m), then for each mode ($_m$) the participation mass factor and effective mass ratio have been determined in order to determine the number of modes to be considered in the analysis (Taranath, 2017), capture greater than 90 % of the total mass of the structure. Therefore, the analysis conducted manually to determine the base shear and displacements. The dynamic analysis has been summarized in the flowing steps:

Step-1- Construct lump mass matrix for effective seismic weight ($m=Wi/g$):

$$[M] = \begin{bmatrix} m_{20} & 0 & 0 & 0 & 0 \\ 0 & m_{19} & 0 & 0 & 0 \\ 0 & 0 & m_{18} & 0 & 0 \\ 0 & 0 & 0 & \ddots & \vdots \\ 0 & 0 & 0 & \dots & m_1 \end{bmatrix} \tag{14}$$

Step-2- Construct stiffness matrix under effect of earthquake load by using Muto's method for multi-storey frame. This method lead to calculate the stiffness of the building, in which columns and beams are effected by bending deformation with joint rotation (Cruz, E. F. & Chopra, A. K., 1986).

Therefore, Muto's method procedure is summarized by equations below (Hasgür Z., 2022):

$$k_{xi} = \frac{12\ E\ \Sigma\ D_{xi}}{L_i^2} = 12\ E \sum \left(\frac{I_{xi}}{L_i^3} a_{xi}\ n \right) \tag{15}$$

$$k_{yi} = \frac{12\ E\ \Sigma\ D_{yi}}{L_i^2} = 12\ E \sum \left(\frac{I_{yi}}{L_i^3} a_{yi}\ n \right) \tag{16}$$

where

$k_{xi,yi}$- stiffness of storey

E – modulus of elasticity of concrete (MPa)

D_i- shear rigidity of i^{th} column or shear wall (kN/m)

L_i – columns or shear wall height

$I_{xi,yi}$- gross moment of inertia around x or y direction.

$a_{xi,yi}$ - stiffness modification coefficient

$$D_i = k_{ci}\ a_i \times n \tag{17}$$

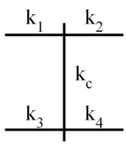
n- number of columns or shear walls

$$k_{cxi} = I_{xi}/L_i \tag{18}$$

$$k_{cyi} = I_{yi}/L_i \tag{19}$$

$k_{cxi,yi}$ – column relative rigidity

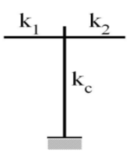
For story columns:



$$k^{-} = \frac{k_1 + k_2 + k_3 + k_4}{2\ k_c} \tag{20}$$

$$a = \frac{k^{-}}{2 + k^{-}} \tag{21}$$

For bottom story columns:



$$k^{-} = \frac{k_1 + k_2}{k_c} \tag{22}$$

$$a = \frac{0.5 + k^{-}}{2 + k^{-}} \tag{23}$$

$$E = 4700\sqrt{fc} \qquad \text{(for RC component)} \tag{24}$$

where,

k_c- column relative rigidity

k_1, k_2, k_3, k_4- beams relative rigidity

f_c- concrete strength (40 MPa)

Then, stiffness matrix constructed as follows **Error! Reference source not found.**:

$$[K] = \begin{bmatrix} k_{20} & -k_{20} & 0 & 0 & 0 \\ -k_{20} & k_{20}+k_{19} & -k_{19} & 0 & 0 \\ 0 & -k_{19} & k_{19}+k_{18} & \dots & 0 \\ 0 & 0 & -k_{18} & \ddots & -k_2 \\ 0 & 0 & 0 & -k_2 & k_1+k_2 \end{bmatrix} \tag{26}$$

Step-3- Determine natural frequencies (ω) and normalized mode shapes ($Ø$). The principle of "*Inverse Iteration, Stadola- Vianello Method*" (Craig & Kurdila, 2006), used as vector iteration with shift method for determining the eigenpairs, where the procedure of inverse vector iteration is used in this method basing on the concept of shifting the eigenvalue spectrum (Chopra, 2020), this procedure provides a convergence rate of the iteration method in order to obtain the eigenpairs (λ, $Ø$), this method provides a practical tool for computing many natural frequencies and mode shapes, the procedure of this method is summarized in the equations below**Error! Reference source not found.** (Chopra, 2020):

$$[\check{K}]\, Ø = \check{\lambda}\, [M]\, Ø \tag{27}$$

$$\check{K} = K - \check{\mu}\, M \tag{28}$$

$$\check{\lambda} = \lambda - \check{\mu} \tag{29}$$

where,

$\check{K}$- converged stiffness matrix

$\check{\mu}$- converge factor used to scale the mathematical model towards the preferred eigenvalue

$\check{\lambda}$- converged eigenvalue

M- matrix of mass

$Ø$- final eigenvector representing the normalized mode shape

After converged matrix has been constructed, then $x^{\wedge}$ determined by solving algebraic equation shown below:

$$x^{\wedge} = \frac{Mx_j}{\check{K}} \tag{30}$$

where

$x^{\wedge}$ – transfer displacement vector

x_j- initial displacement vector

Then, estimate the eigenvalue using Rayleigh's quotient evaluation:

$$\lambda = \breve{\lambda} + \mu = \frac{x^\wedge \, M \, x_j}{x^{\wedge T} \, M \, x^\wedge} + \mu \tag{31}$$

Then, we obtain the natural frequency and primary normalized mode shape:

$$\omega = \sqrt[2]{\lambda} \tag{32}$$

$$\text{Ø}_m = \frac{x^\wedge}{\sqrt[2]{x^{\wedge T} \, M \, x^\wedge}} \tag{33}$$

where

λ- final eigenvalue

ω- natural frequency

Ø_m – primary mode shape

$$If \;\; \text{Ø}_m{}^T M \, \text{Ø}_m = 1 \tag{34}$$

Then final normalized mode shape Ø_{mn} can be obtained which is used in displacement calculations and base shear distribution on storeys:

$$\text{Ø}_{mn} = \text{Ø}_{im} / (max(\text{Ø}_{im})) \tag{35}$$

This method may apply for several iterations for each mode, these iterations can be stopped when obtain a convergent eigenvalues, usually three to five iterations are enough to obtain desired λ.

Step-4- From each natural frequency the period of each mode ($T_{\text{Ø}m}$) has been obtained by applying the following equation:

$$T_{\text{Ø}m} = \frac{2\pi}{\sqrt{\lambda}} \tag{36}$$

Step-5- By using ($T_{\text{Ø}m}$) obtained from step-4 the spectral acceleration can be determined from designed response spectrum diagram illustrated in Figure 1.

Step-6- The base shear for each mode is determined from form the flowing equation (Taranath, 2017):

$$V_m = \alpha_m \, S_{am} \, W/g \tag{37}$$

where

V_m - base shear from m_{th} mode

α_m - effective mass ratio for m_{th} mode

S_{am} - spectral acceleration for m_{th} mode determined from the response spectrum

W/g – effective seismic weight of the building

$$\alpha_m = \frac{\left(\sum_{i=1}^{n} \frac{W_i}{g} \text{Ø}\right)^2}{\sum_{i=1}^{n} \frac{W_i}{g} \sum_{i=1}^{n} \frac{W_i}{g} \text{Ø}^2} \tag{39}$$

Step-7- The modal participation factor (PF_{xm}) for the m_{th} mode should be determined in order to calculate yield and maximum displacement, this factor can be determined for each mode from the equation shown below (Taranath, 2017):

$$PF_{xm} = \left(\frac{\sum_{i=1}^{n} \frac{W_i}{g} \text{Ø}_{im}}{\sum_{i=1}^{n} \frac{W_i}{g} \text{Ø}^2{}_{im}} \right) \text{Ø}_{xm} \tag{40}$$

where

PF_{xm} - participation factor at level x for the m_{th} mode

Wi/g - mass of level i

$Ø_{im}$- mode amplitude of the m_{th} mode at level i

$Ø_{xm}$ - amplitude of the m_{th} mode at level x

n- level of a story

Where effective mass M^* can be determining by the equation below:

$$M^* = 1^T M \text{Ø}_m PF_{xm} \tag{41}$$

Step-8- Determine the total dynamic base shear (V_b). Results of each mode have been combined by using the Complete Quadratic Combination (CQC). Therefore, the general formula for CQC methods are shown in equations below (Chopra, 2020):

$$R_{\max\_CQC} = \sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} f_{ij} R_i R_j} \tag{42}$$

where

R_{max} - maximum modal response

f_{ij} -modal correlation coefficient indicates the interactions between modes i and j

R_i, R_j – response quantity at modes i and j

$$f\text{ij} = \frac{8\zeta^2(1+r)r^{1.5}}{(1+r)^2 + 8\zeta^2 r(1+r)^2} \tag{43}$$

where

ζ- damping coefficient assumed 0.05

r – frequency ratio ω_j / ω_i.

Therefore, all frequency ratios can be representing in the matrix pattern shown below **Error! Reference source not found.**:

$$r_{ij} = \begin{bmatrix} \omega_1/\omega_1 & \omega_2/\omega_1 & \dots & \omega_n/\omega_1 \\ \omega_1/\omega_2 & \omega_2/\omega_2 & \dots & \omega_n/\omega_2 \\ \vdots & \vdots & \vdots & \vdots \\ \omega_1/\omega_n & \omega_2/\omega_n & \dots & \omega_n/\omega_n \end{bmatrix} \tag{44}$$

Then, combined V_b can be obtained from the equations below:

$$V_{b\_CQC} = \begin{bmatrix} V_1 & V_2 & \dots & V_n \end{bmatrix} \begin{bmatrix} f_{11} & f_{12} & \cdots & f_{1n} \\ f_{21} & f_{22} & \cdots & f_{2n} \\ \vdots & \vdots & \vdots & \vdots \\ f_{n1} & f_{n2} & \cdots & f_{nn} \end{bmatrix} \begin{bmatrix} V_1 \\ V_2 \\ \vdots \\ V_n \end{bmatrix} \tag{45}$$

Step-10- The final step is determining lateral yield displacement (δ_y) and maximum elastoplastic displacement (δ) for each level in each mode, these displacements are determined according to the equations shown below:

$$\delta_{y\_im} = PF_{xm} \frac{S_a g}{R_d} \emptyset_{mn} \frac{1}{\omega^2} \tag{46}$$

$$\delta_{im} = PF_{xm} \frac{S_a g}{R_d} \emptyset_{mn} \frac{1}{\omega^2} \mu \tag{47}$$

where

δ_{y_im}- yield lateral displacement

g- ground acceleration (9.81)

R_d- strength reduction factor

μ - ductility factor

the lateral displacement for each mode and each floor has calculated, also this displacement must be combined by using the CQC method.

2.4. Create Artificially Earthquake Motion

The artificially earthquake motions are created depending on three actual records to be scaled as simulated accelerations then analyse the response of the structure to these accelerations, the selected earthquakes are shown below:

E_1 -Imperial Valley Earthquake-El Centro (US) of May 18, 1940 (Chopra, 2020).

E_2- Fruili (Italy) Earthquake of May 6, 1976 (PEER, 2023).

E_3- The Kobe (Japan) Earthquake of January 16, 1995 (PEER, 2023).

The selected ground motions are scaled in order to match the site design response spectrum. Therefore, the targeted acceleration is the minimum acceleration corresponding with (T=0) in (Eq. 68) and scaling process conducted according to the formula below:

$$AGA = \frac{0.782(E_1) + 0.393\ (E_2) + 0.393\ (E_3)}{3} \tag{48}$$

where,

AGA- artificially generated accelerations

2.5. Define Velocity and Displacement for the Artificially Earthquake

Then, we need to obtain peak ground velocity and peak ground displacement, these indicators may defined numerically by linear interpolation of excitation using (time stepping method) every 0.02 sec and damping ζ=0.05 , the procedure of this method is summarized by the following steps (Chopra, 2020):

Step-1- assume a single degree of freedom with m=1 and T= earthquake period which is (40s) used in this paper, then ω=0.157 rad/sec, and $k = \omega^2$ ton/m.

Step-2- consider the displacement (u) at T=0 is u_0= 0, where the velocity ($\dot{u}$) at the same period is $\dot{u}0$=0.

Step-3- determine u_{i+1} by the recurrence formula shown below:

$$u_{i+1} = A_r u_i + B_r \dot{u}_i + C_r p_i + D_r p_{i+1} \tag{49}$$

$$A_r = e^{-\zeta\omega_n \Delta t}\left(\frac{\zeta}{\sqrt{1-\zeta^2}} sin\omega_D \Delta t + cos\omega_D \Delta t\right) \tag{50}$$

$$B_r = e^{-\zeta\omega_n \Delta t}\left(\frac{1}{\omega_D} sin\omega_D \Delta t\right) \tag{51}$$

$$C_r = \frac{1}{k}\left\{\frac{2\zeta}{\omega_n \Delta t} + e^{-\zeta\omega_n \Delta t}\begin{bmatrix}\left(\frac{1-\zeta^2}{\omega_D \Delta t} - \frac{\zeta}{\sqrt{1-\zeta^2}}\right) sin\omega_D \Delta t \\ -\left(1+\frac{2\zeta}{\omega_n \Delta t}\right) cos\omega_D \Delta t\end{bmatrix}\right\} \tag{52}$$

$$D_r = \frac{1}{k}\left[1 - \frac{2\zeta}{\omega_n \Delta t} + e^{-\zeta\omega_n \Delta t}\left(\frac{2\zeta^2-1}{\omega_D \Delta t} sin\omega_D \Delta t + \frac{2\zeta}{\omega_n \Delta t} cos\omega_D \Delta t\right)\right] \tag{53}$$

$$\dot{u}_{i+1} = \omega_n\left(-u_i sin\omega_n \Delta t + \frac{\dot{u}_i}{\omega_n} cos\omega_n \Delta t + \frac{p_i}{k} in\omega_n \Delta t + \frac{\Delta p_i}{k}\frac{1}{\omega_n \Delta t}[1 - cos\omega_n \Delta t]\right) \tag{56}$$

$$p_i = -m(AGA) \tag{57}$$

Where,

A_r, B_r, C_r, D_r- recurrence coefficients

P_i- step force (kN)

Step-4- repeating steps 1-3 for each Δ=0.02, in order to determine the peak ground velocity $\dot{u}_{go}$ and peak ground displacement $u_{go,}$:

$$\dot{u}_{go} = Max|(\dot{u}_{go}, T)| \tag{58}$$

$$u_{go} = Max|(u_{go}, T)| \tag{59}$$

Step-5- similarly determine peak ground acceleration $\ddot{u}_{go}$ which is the maximum value of AGA:

$$\ddot{u}_{go} = Max|(\ddot{u}_{go}, T)| \tag{60}$$

2.6. Create Artificially Elastic Design Response Spectra

Basing on the AGA, Artificially Elastic Design Response Spectra (AEDRS) has been constructed and matching the design response spectrum specified in ISC-303. While u_{go} and $\dot{u}_{go}$ have been obtained, then the elastic design spectra can be constructed by using Newmark's amplification factors - 84.1th percentile for damping = 0.05 (Newmark & Hall, 1982), (Chopra,2020) the procedure of this method is illustrated in the steps below:

Steps-1- select the value for acceleration, velocity and displacement amplification factors:

$$\alpha_A = 2.71,\ \alpha_V = 2.3,\ \alpha_D = 2.01$$

Step -2- Multiply $\ddot{u}_{go}$ by the amplification factor α_A to obtain the constant value of artificial-acceleration A.

Step -3- Multiply $\dot{u}_{go}$ by the amplification factor α_V to obtain the constant value of artificial –velocity V.

Step -4- Multiply u_{go} by the amplification factor α_D to obtain the constant value of artificial –displacement D.

Step -5- Determine the short period (T_a), intermediate period (T_b) and long period (T_c):

$$T_a = 1/8 \tag{61}$$

$$T_b = 2\boldsymbol{\pi}\,(V/A) \tag{62}$$

$$T_c = 2\boldsymbol{\pi}\,(D/V) \tag{63}$$

Step -6- Draw AEDRS depending on the constant values of A, D, V:

$$\text{For, } T_n = 0, \quad A(T_n)/g = \ddot{u}_{go} \tag{64}$$

$$\text{For, } T_a \le T_n \le T_b, \quad A(T_n)/g = A \tag{65}$$

$$\text{For, } T_b < T_n \le T_c, \quad A(T_n)/g = 2\boldsymbol{\pi}\,V/(g\,T_n) \tag{66}$$

$$\text{For, } T_c < T_n \le 6s \quad A(T_n)/g = (2\boldsymbol{\pi}/T_n)^2\ (D/g) \tag{67}$$

Step -7- Draw DRS depending on ISC-303 ground motion parameters:

$$\text{For } T < T_o \quad S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \tag{68}$$

$$\text{For } T_o \le T < T_s \quad S_a = S_{DS} \tag{69}$$

$$\text{For } T_s < T \le T_L \quad S_a = \frac{S_{D1}}{T} \tag{70}$$

Step-8- Matching AEDRS with DRS.

Therefore, inelastic design spectra can be conducted by using desired reduction factor R_d, this factor can be determined, as the recommendation of R_d is to be function of the system ductility μ for short period (T_a), intermediate period (T_b) and long period (T_c) (Newmark & Hall, 1982):

$$\text{For T} < T_a, \quad R_d = 1 \tag{71}$$

$$\text{For } T_a < \text{T} < T_b, \quad R_d = (2\,\mu - 1)^{0.5} \tag{72}$$

$$\text{For } T_b < \text{T}, \quad R_d = \mu \tag{73}$$

3. Results

In this section results obtained from dynamic and static analysis are compared, in which ELFA and RSA are depending on ISC-303 parameters, where Response Modification Factor (R) = 6.5 and Displacement Amplification Factor (C_d)= 6.5. Regarding to THA, different parameter have been used depending on desired Ductility (μ) and Strength Reduction (R_d) factors.

3.1. Effective seismic weight and calculated stiffness

Effective seismic weight (ESW) and stiffness for the building is summarized in Table 4. Therefore, mass and stiffness presented in Table 4 are used in static and dynamic analysis. The stiffness is representing the contribution of columns, beams and shear walls used in the system and calculated by applying Muto's method. Therefore, the results of dynamic analysis have obtained by using equivalent mass and stiffness matrices, where lump mass matrix is in (Ton) and stiffness matrix is in (kN/m).

Table 4: Effective Seismic Weight and Calculated Stiffness

Level	***ESW (Ton)***	***Stiffness- NS (kN/m)***
20	1322	1828736
2 to19	1740	1828736
1	1740	45850721
sum	34382	-

3.2. Results of Static Analysis

The results have presented in Figure 7 determined statically using ELFA, two design base shear illustrated in Table 5 are capturing 100% and 85% of static analysis results, δ_{yy} considered yield displacement which has obtained depending on forces acted on the storey and calculated stiffness, this displacement have amplified by the factor C_d =6.5 in order to obtain δ_m which is considered the maximum elastoplastic displacement.

Table 5: Initial Results of Static Analysis

Percentage %	***Base shear (kN)***	***δ_{yy} (mm)***	***δ_m (mm)***
100	6915	37.74	245.31
85	5878	32.08	208.52

3.3. Results of Response Spectrum Analysis

The purpose of conducting this dynamic analysis in order to illustrating the issue of the paper, due to use constant value of R, results obtained from RSA gives lower design base shear, despite this method considers a dynamic analysis for evaluating the response of structural systems, where it depends on create a mathematical model of superposition of modes and seismic inputs. But the lower RSA strength results lead to scale it to 85 % ELFA results. RSA has been conducted manually as described in methodology, where analysis steps are compatible with ISC-303 procedure, R *and* C_d has been used as same as to the ELFA. Table 6 and Table 7 have illustrated the combined results of elastic and inelastic base shear.

Table 6: Periods, frequency, spectral acceleration, participation factors, effective mass ratio and elastic base shear for selected modes in N-S obtained from RSA.

Ø	***1***	***2***	***3***	***4***	***5***	***6***	***V_{b_CQC}***

							(kN)
ω (rad/s)	2.638	7.8782	13.0964	18.1097	23.2119	32.696	
T (s)	2.381	0.797	0.480	0.347	0.271	0.202	
S_{am} g	0.067	0.201	0.312	0.312	0.312	0.312	
PF_m	1.269	-0.353	0.219	-0.274	0.125	0.076	
$α_m$	0.795	0.054	0.024	0.052	0.010	0.003	
***V_m* (kN)**	17983	3637.99	2467.21	5446.23	819.33	309.85	19491

Table 7: Inelastic base shear for selected modes in N-S obtained from RSA.

Ø	***1***	***2***	***3***	***4***	***5***	***6***	***Vb_CQC (kN)***
***V_m* (kN)**	2772	559.69	379.57	837	126.05	44.26	3096

Results are based on selected modes shape have presented in Figure 3, these modes capture greater than 93% of the total mass ratio ($α$), and used to estimate the base shear, distribution of the base shear to storeys, yield displacement, maximum elastoplastic displacement and overturning moments as combined results.

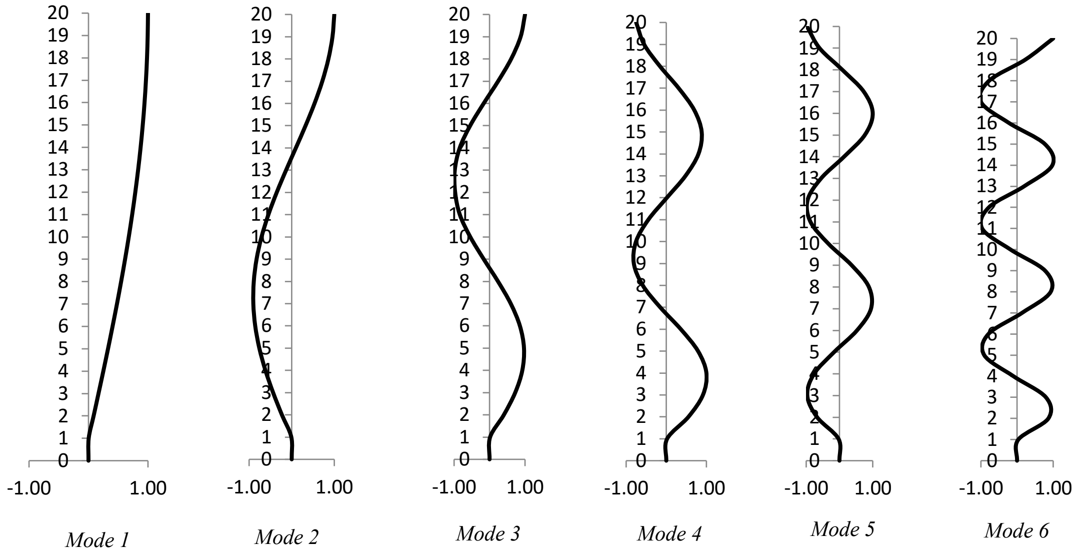


Figure 3: Selected modes capture greater than 93%.

Table 8: Initial Results of RSA

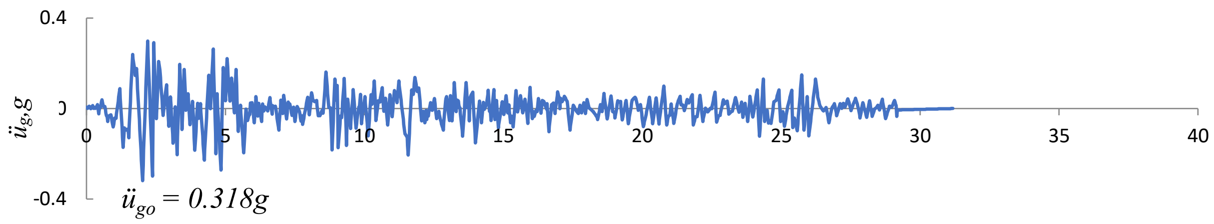
Base shear (kN)	***$δ_{yy}$* (mm)**	***$δ_m$* (mm)**
3096	18.64	121.18

3.4. Compare results of RSA with ELFA

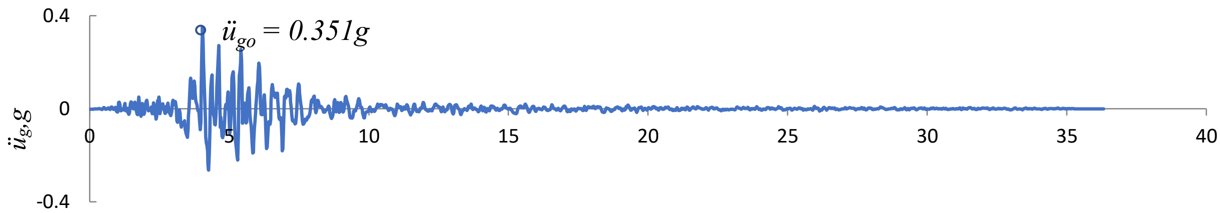
In order to illustrate the deference of results between the RSA and ELFA methods, this comparison is focused on the base shear results, where the inelastic base shear of RSA is obtained based on the design response spectrum by combining its value depending on the selected mode shapes. Despite both methods are using the same *R and* C_d, but Table 5 and Table 8 are presenting the contrast between inelastic base shear obtained from ELFA and RSA, where ELFA gives a significant inelastic base shear if compared with the base shear obtained from RSA. Therefore, with this results, ISC-303 required in section (8/10-3) to scaling forces and displacement results obtained from RSA to 85% of ELFA.

3.5. Time History Analysis Results

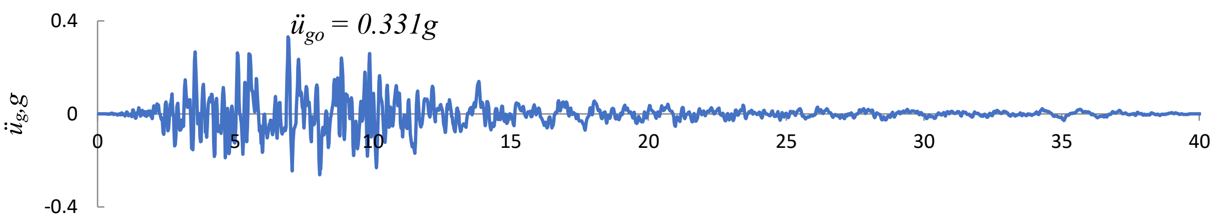
Basing on the simulated earthquake accelerations, Artificially Elastic Design Response Spectra (AEDRS) has been created equivalent with the design response spectrum specified in ISC-303. Therefore, three earthquake motions have been used in this study are shown in Figure 4. The scaling has been made in order to match the site design response spectrum related to Baghdad in periods and accelerations range. The final artificially generated accelerations are representing the average of the scaled ground motions, where the targeted acceleration is the minimum acceleration corresponding at T=0 (equation 68) in the site design response spectrum, the final simulated accelerations are shown in Figure 4-(d).



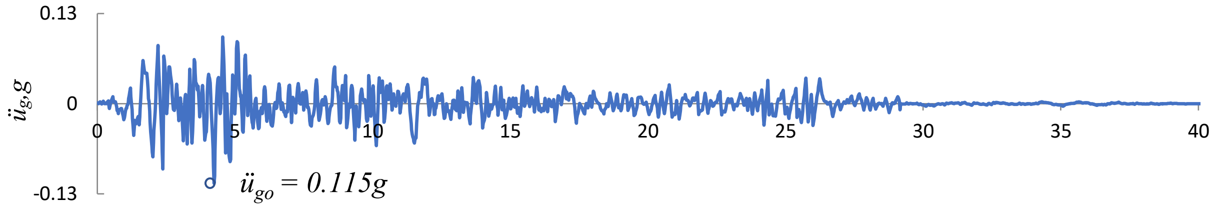
(a)



(b)



(c)



(d)

Figure 4: (a) Imperial Valley Earthquake-El Centro (US) of May 18, 1940 (Chopra, 2020). (b) Fruili (Italy) Earthquake of May 6, 1976 (PEER, 2023) . (c) The Kobe (Japan) Earthquake of January 16, 1995 (PEER, 2023). (d) final artificially generated accelerations

After obtaining the desired accelerations, the results of peak ground velocity and peak ground displacement defined numerically by using time stepping method (Chopra, 2020), every 0.02 sec and 0.05 damping ratio, results illustrated in Figure 5. Where the peak acceleration $\ddot{u}_{go} = 0.115g$, peak velocity $\dot{u}_{go} = 0.108$ *m/sec,* and finally peak displacement $u_{go} = 5.66$ *cm*.

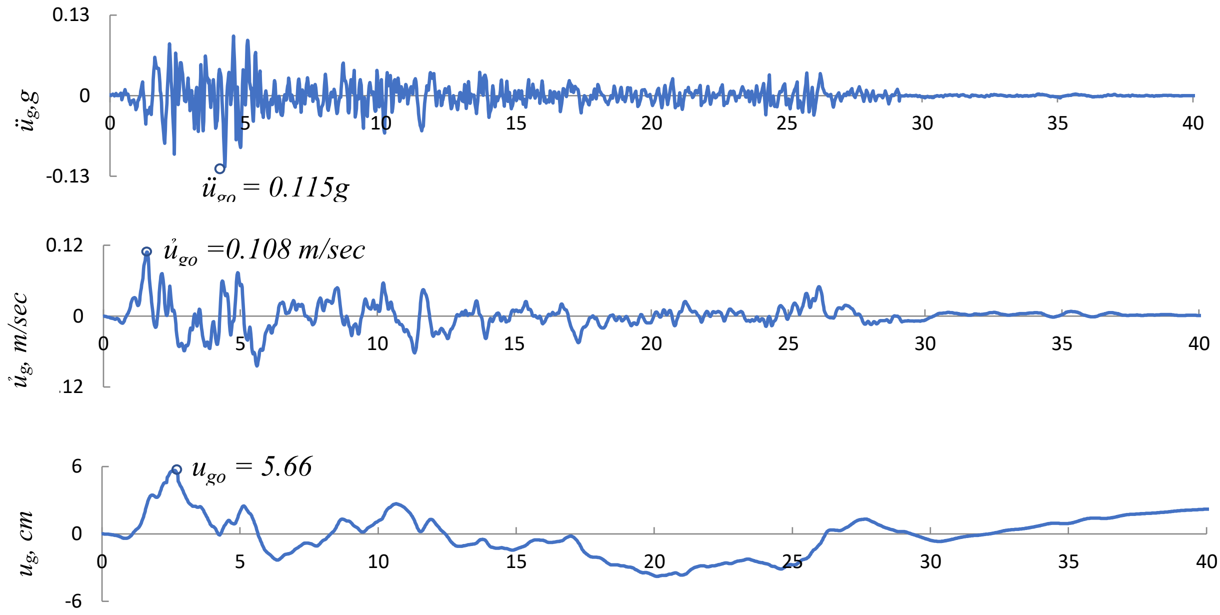


Figure 5: Artificial ground velocity and displacement computed by integrating the artificial ground acceleration using time stepping method

While three important components of peak ground motion have been obtained, then the elastic design spectra can be constructed by using Newmark's amplification factors - 84.1th percentile (Newmark & Hall, 1982). For example, when multiplying the peak ground acceleration 0.115g by its amplificatory factor α_A =2.71, the peak acceleration 0.3116g has been obtained which is very close to the peak design acceleration spectra permitted for Baghdad as shown in Figure 6-c.

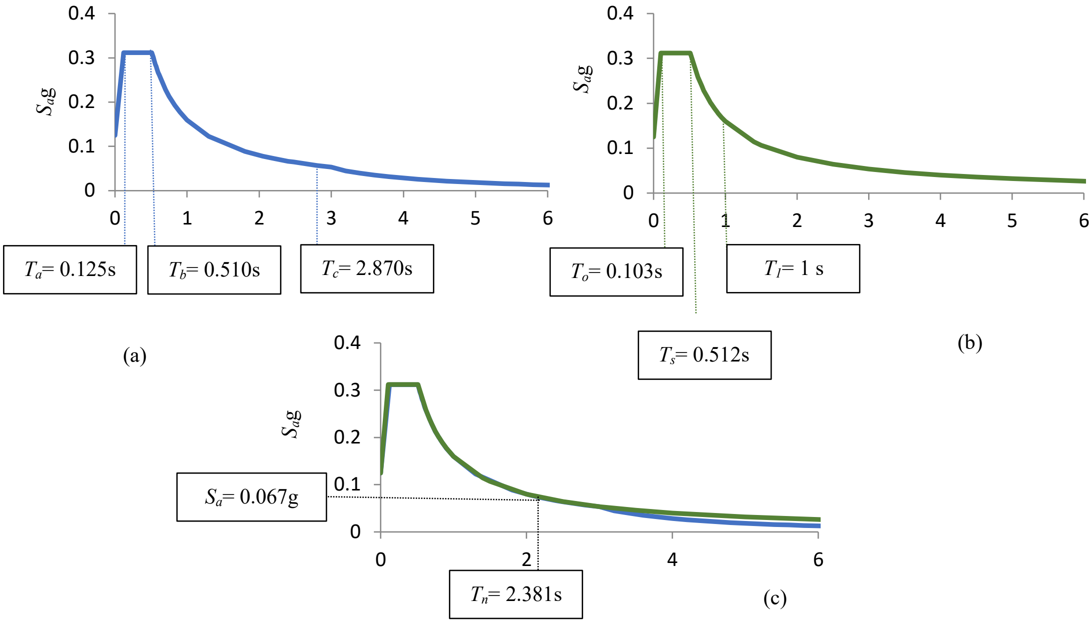


Figure 6: Response spectrum for artificially ground motion compared with the site elastic design response spectrum: (a) artificially elastic design response spectrum. (b) Site elastic design response spectrum for Baghdad. (c) Match the site design response spectrum

Therefore, the artificial accelerations are suitable to be dependable for THA analysis. For more verification let check the period T_b which is depending on the both amplificatory velocity α_V=2.3 and modified acceleration:

$$V=0.108\times2.3= 0.2484$$

$$T_b= 2\times3.14\times(0.2484/(0.3116\times9.81)= 0.510\ s \approx T_s\ (ok)$$

Ts- short period for code design response spectrum

Figure 6 illustrating the elastic design response spectrum generated from the artificially ground motion, where it is almost matching the site design spectra. It is clear now that the artificially design spectra is created depending on the amplification of the peak acceleration, velocity and displacement for the artificially ground motion, where, α_A =2.71, α_V =2.3 and α_D = 2.01. Therefore, each constant peak value has been multiplied by its amplificatory factor in order to obtain smooth design spectrum.

Therefore THA is used in order to obtain combined results of design base shear further to peak displacements response parameters related to three tests, these tests are representing the results of design base shear obtained from ELFA and RSA, as presented in Table 9.

Table 9: Three scenarios to be studied

Test I	***Test II***	***Test III***
V_{b1}= 6915 kN	V_{b2}= 5878 kN	V_{b3}= 3096 kN

The linear analysis have been performed in order to obtain the combined elastic base shear prior to conducting the analysis related to each test, therefore, the value of design elastic base shear obtained from each mode shape has been illustrated in Table 10.

Table 10: Results of THA - Elastic Base Shear

$Ø_m$	1	2	3	4	5	6	V_{e_CQC} (kN)
ω (rad/s)	2.638	7.8782	13.0964	18.1097	23.2119	32.696	
T (s)	2.381	0.797	0.480	0.347	0.271	0.202	
S_{am} g	0.067	0.200	0.3116	0.3116	0.3116	0.3116	
PF_m	1.269	-0.353	0.219	-0.274	0.125	0.076	
$α_m$	0.795	0.054	0.024	0.052	0.010	0.003	
V_{em} (kN)	17927.23	3624.96	2464.05	5439.25	818.28	309.45	**19434**

The results of the earthquake response for Test I indicate to 43.19 mm yield displacement, where the yield displacement continue in decreasing with increasing the ductility factor, it is obvious in the Test II and Test III, where yield displacement is 36.64 mm and 18.60 mm respectively.

Table 11: Results of three tests

Test	V_b (kN)	μ	$δ_{yy}$ (mm)	$δ_y$ (mm)
I	6915	2.8	43.19	120.82
II	5878	3.3	36.64	120.84
III	3096	6.5	18.60	120.92

The three tests have almost same maximum displacement which is the same maximum elastic displacement. The better performance from three analyses is Test I, because it performed the bigger yield displacement than Test II and Test III.

The next relative comparison is illustrating the differences between ELFA and THA results, and between RSA and THA, where THA is the benchmark of the evaluation. Therefore, the relative differences of computing yield displacement and maximum displacement are pointed in Table 12 and Table 13.

Table 12: Comparison of Selected Results between ELFA and THA

Test	Parameter	THA	ELFA	Difference %
I	$δ_{yy}$ (mm)	43.19	37.47	13
	δy (mm)	120.82	245.31	103
II	$δ_{yy}$ (mm)	36.64	35.32	10.72

***δy* (mm)**	120.84	208.52	72

Table 13: Comparison of Selected results between RSA and THA

Test	***Parameter***	***THA***	***RSA***	***Error %***
III	**δ_{yy} (mm)**	18.60	18.64	0.21
	***δy* (mm)**	120.92	121.18	0.21

Significant differences in the results of the maximum displacement obtained from ELFA, where the difference of estimating yield displacement are about 10-13% for Test I and Test II. On other hand, RSA gives results very close to THA, where errors of using RSA for estimating yield and maximum displacement are very slight if compared with THA, this result is due to matching the site design spectrum in this study. Therefore, RSA may give comprehensive results for entire analysis if used with the desired ductility factor.

It is clear that any base shear obtained ELFA is considered inelastic design base shear, which is the minimum required design base shear for a structural system and this indicator is a good benchmark to enhance the system strength. Therefore, In order to reach the desired inelastic design base shear, by using the elastic THA results, let assume the desired yield displacement would be half maximum elastoplastic displacement. Therefore, reduction factor Rd can be determined, where recommendation of factor Rd is to be function of the system ductility μ for short, intermediate and long fundamental period of structures as shown below (Newmark & Hall, 1982):

$$T < 0.2 \text{ sec}, \quad R_d = 1$$

$$0.2 \text{ sec} < T < 0.5 \text{ sec}, \quad R_d = (2\mu - 1)^{0.5}$$

$$0.5 \text{ sec} < T, \quad R_d = \mu$$

Therefore, $\mu = \boldsymbol{\delta_m / \delta_{yy}}$, then $\mu = 2$

$$\text{For } T_1, T_2, T_3, \quad R_d = \mu = 2$$

$$\text{For } T_4, T_5, T_6, \quad R_d = (4 - 1)^{0.5} = 1.73$$

Table 14: THA Results for Desired Ductility $\mu=2$

V_b (kN)	μ	**δ_{yy} (mm)**	**δ_y (mm)**
9860	2	60.28	120.56

As shown in Table 14, if the maximum considered earthquake happened and the structure designed for μ= 2 it will yield at the displacement 60.72mm, more details illustrated in Figure 7 which has presented the modal analysis summary. Therefore, it is important to be taken into account that the structure even designed with this ductility, it will be deformed at the end of the maximum considered earthquake, because it designed by a lower design base shear if compared with elastic base shear obtained from design response spectra.

3.6. Response Spectrum Analysis with Desired Ductility

RSA showed accurate results for maximum elastoplastic displacement if compared with THA, it also would give accurate yield displacement if used with the suitable ductility factor, and this paragraph has written to give an example about the possibility of using RSA analysis with the desired *R*. Therefore, it is clear that considering ELFA as benchmark to scale RSA because ELFA gave greater design base shear. The diagnosed reason for this measure is clear in this study, where RSA results is lower than ELFA results due to reduce the spectrum acceleration by higher value of *R*. Therefore, this study would suggest using RSA with different reductions factors used for static analysis in ISC-303. These factors can be determined depending on the iterations of assessing the structural response in design process, where same procedure used in THA can be used in RSA, where *R* can also be determined as a function of the system ductility C_d, and this conclusion is the result of applying the same procedure below in THA (Newmark & Hall, 1982):

$$\text{If, } T < 0.2 \text{ sec}, \quad R=1$$

$$\text{If, } 0.2 \text{ sec} < T < 0.5 \text{ sec}, \quad R= (2\,C_d - 1)^{0.5}$$

$$\text{If, } 0.5 \text{ sec} < T, \quad R= C_d$$

RSA* has used in this comparison with targeted factor C_d in order to obtain the same value of *R* used in THA, where Table 6 has shown the combined elastic base shear in RSA, where elastic base is a slightly higher in RSA than THA, because AEDRS has scaled to the site DRS with a bit lower spectral acceleration and this thing resulting of a bit lower elastic base shear in THA. Therefore, *R* has determined as shown below:

$$\text{For } T_1, T_2, T_3, \quad R= C_d = 2$$

$$\text{For } T_4, T_5, T_6, \quad R= (4 - 1)^{0.5} = 1.7$$

3.7. Comparison Summary between Results of Dynamic and Static analysis

In this section the comparison between dynamic and static analysis results has illustrated as modal summary for all storeys depending on ELFA, RSA and THA results, but RSA* has used in this comparison with targeted factor C_d =2 in order to compare THA design base shear and to illustrate an example about the possibility of using RSA without the need of scaling its results to ELFA. Figure 7-a illustrates that the yield displacement has been gradually increased when the design base shear has increased from 85% to 100% of ELFA results, and this displacement has more increased when using lower reduction factor *R* as happened with THA and RSA*. Regarding to maximum elastoplastic displacement, this displacement is overestimation whether in 100% or 85%ELFA results as shown in Figure 7-b, where this displacement would be much accurate if obtained from dynamic analysis as seen in THA and RSA*. Figure 7-c and 7-d illustrate that it possible to obtain the required strength design from RSA when using the desired *R*, if this method is not restricted by the constant factor *R* it would give an accurate results without the need of scaling it to ELFA, and this thing is clearly obvious in Figure 7, where THA and RSA* are almost giving very close results because the simulated earthquake accelerations has been scaled to site response spectra and this measure gives an advantage to RSA if used in a flexibility manner during inelastic analysis as happens with THA.

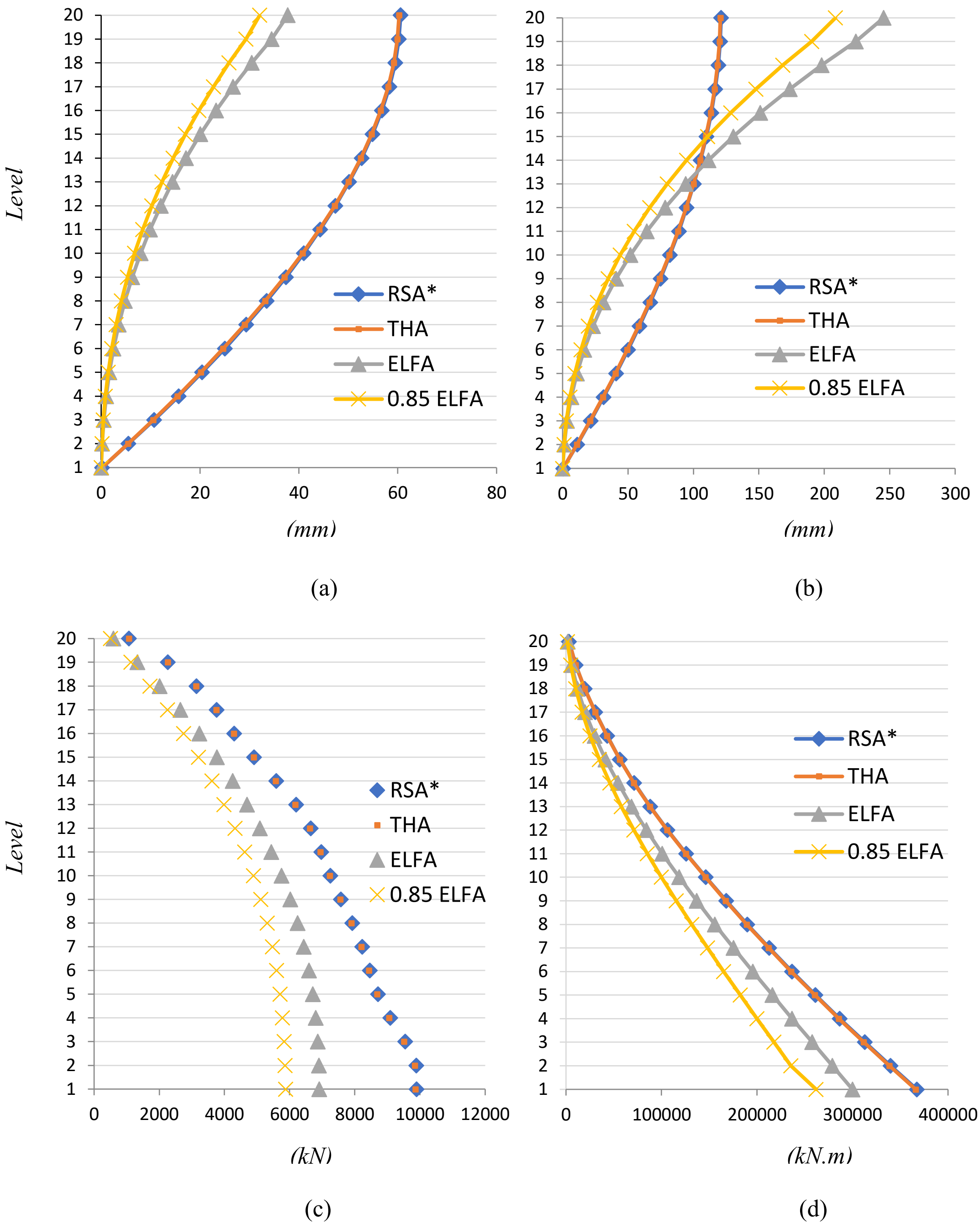


Figure 7: Modal summary of 20-storey building (a) Modal storey of yield displacement. (b) Modal storey of maximum elastoplastic displacement. (c) Modal storey shear (d) Modal storey of sum overturning moment

In summary, ELFA has compared with RSA and THA, testes have shown inaccurate response results are obtained from static analysis, where dynamic analysis gives reasonable results, and these results are mainly depended on two factors, first: accurate mathematical model for the structural system, second: use a suitable ductility factor to insure a sufficient structural resistant to the maximum considered earthquake.

4.DISCUSSION

The comparison between ELFA and THA has been conducted. The relative error with maximum displacement is significant if this value obtained from ELFA, where the error in computing yield displacement is between 10-13%. The design base shear obtained from ELFA is the minimum required strength which is lower than the elastic base shear. Therefore, it is considered a yield force if the potential maximum earthquake happened, RSA base shear is much lower than elastic base shear due to using higher constant reduction factor *R*. This constant *R* leads to decrease the yield displacement, and this measure would not give a clear view about the dynamic analysis. THA has been shown interesting results, where this analysis conducted based on simulated ground motion accelerations and artificially elastic design spectra, therefore, same RSA procedure of combination has been used in THA, the only differences; first, is related to create artificially design response spectra to be equivalent with site design spectrum, where it is already matched, then it leads to close results related to elastic base shear in both analysis, second, THA have been conducting in a flexibility manner where the designer may decide the desired ductility and strength reduction than using constant coefficients like C_d and *R*. Therefore, giving the flexibility of using desired ductility in the analysis would enhance the results of RSA, it is worth to mention that the design response spectrum which created by the code seems as a simulation tool for an earthquake data and it would give a logical estimation for the required elastic base shear to resist the maximum considered earthquake.

5.CONCLUSION

ELFA, which refers the static analysis part in this paper, the design base shear obtained from this method is the result of applying approximate ISC-303 procedure in order to obtain a higher level of results related to strength design, where yield displacements is in adequate level of calculation, as this displacement obtained depending on the stiffness calculation using Muto's method, on the other hand the maximum elastoplastic displacements are much higher than targeted displacement, where targeted maximum displacement assumed in this study is about 0.002 of the building's height.

RSA, which refers the first part of dynamic analysis has been conducted using superposition of modes method and design site response spectrum with ISC-303 coefficients, therefore, six modes shapes is selected to capture greater than 93% as effective mass in N-S directions, combined RSA results are shown the base shear and yield displacement are in lower level, but the maximum elastoplastic displacement is close to the targeted displacement.

THA, which refers the second part of dynamic analysis in this study, also have been conducted with same mode shapes used in RSA, but with artificially ground motion acceleration in order to create artificially design response spectra equivalent with the site design response spectrum related to Baghdad, where ductility and strength reduction factors are assumed depending on the desired value in order to evaluate the results of RSA and ELFA, the evaluation results shows RSA gives much accurate maximum displacement than ELFA.

Static analysis is approximate method to establish the initial minimum required strength design, it is not directly sufficient for 20-storey building, because its displacement results are associated with relative errors if compared with dynamic analysis, furthermore, it shows a lower design base shear if compared with the elastic base shear obtained from dynamic analysis. Therefore, results of static analysis would always be verified by dynamic analysis, on the other hand, using non-constant reduction factor R would defiantly recommended to enhance the results of Response Spectrum Analysis if used with accurate superposition of modes method, this study would set some recommendations may assist to summarize this purpose:

1. Create appropriate mass distribution of the structure under consideration to be used in dynamic analysis, once the total building mass has obtained, then initial design base shear would be obtained from static analysis as minimum required design base shear depending on system and ground motion parameters, it is recommended to consider 100% of the static design base shear at the first emphasis.
2. Create appropriate stiffness distribution, Muto's method is recommended to conduct this step due to its simplified procedure and satisfactory results can be employed effectively with the mass distribution for calculating the natural frequencies and considered mode shapes.
3. The elastic base shear would be calculated from the selected mode shapes and response spectrum inputs then compare it with the required design base shear to obtain the targeted R, and according to obtained R the procedure of Newmark's method for inelastic design spectra can be applied to determine C_d.
4. Static design base shear should be evaluated by assessing the performance of the structure depending on yield displacement and maximum elastoplastic displacement, to ensure the structure meets the desired performance objectives and according to this evaluation modification factors may rearrange again to obtain the targeted performance for resisting seismic forces.

Finally, response assessment would help of taking suitable design decisions to improve the structural performance, this measure may include modifying the structural elements size or adding dampers to improve the structure's ability to resist earthquakes forces (Plevris, V., Georgia K., & Yasin F., 2017), where dynamic analysis leads to more resistant design than static analysis because it is dealing with the variation of earthquake loads in the structural design.

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CONFLICT OF INTEREST

The authors declare that they have not any conflict of interest.

AUTHOR STATEMENT

The authors have to declare that if there is any ethical approval, consent to participate, consent for publication, availability of data and material, and code availability etc.

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