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Analyzing the nonlinear dynamics of the three degree of freedom frame structure based on the NewMark-β method and the effect of stiffness on the displacement of the structure

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ABSTRACT

The analytical method of the simplified model of a structure has provided a basis for the analysis under the effect of earthquake (dynamic) load, which has an important importance in the seismic analysis of the structure. In this article, three degree of freedom steel frame structure is simulated and analyzed in MATLAB based on the NewMark-β method, taking into account the effect of harmonic loads. The vibration response of the steel frame structure has been analyzed considering the stiffness. The results show that the NewMark-β method is a new idea for earthquake response. The construction of the steel frame according to the range of changes of its elastic modulus through dynamic analysis makes the seismic analysis of the frame structures more practical, as well as the analysis of the SS model, which provides a basis for the size of the stiffness coefficient. By applying force on the top and bottom floors, different structural responses are observed according to the stiffness of the structure. The structure of the SS frame and the effect of the force on it when the force is applied to its floors, according to the height of the floors and the position of applying the force with the change of time, the binding stiffness of the amount of displacement will also change and the stability of the structure will be greatly reduced.

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1. Introduction

The ever-increasing progress in the construction industry is particularly prominent. So that every day we see a general change in building frames, especially in tall buildings, which in addition to increasing the height, the vertical design and shape which is more and more complex, the structural system also becomes increasingly diverse, which causes more prerequisites for The design of the vibrations of building structures. The purpose of seismic design is to make the building have the corresponding resistance ability to earthquakes of different frequencies

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and intensity during the service life [1]. The distribution of structural stiffness is accompanied by certain unevenness. Therefore, to a large extent, it will lead to the overall deformation of the building, and in serious cases, it can cause local cracking of the building. Therefore, the bottom frame structure is difficult to be used effectively in some areas with high fortification intensity. To further improve the seismic design value of building structures, the influence of structural stiffness must be considered [2].

Unlike linear analysis, incremental iterative formulas for structural nonlinear analysis have not been developed to the extent that every researcher follows the same logical steps [3]. In most cases, only the Newton-Raphson method or other algorithms are used as tools for incremental iterative analysis [4]. In structural dynamics, a multi degree of freedom (MDOF) structure is often equalized by a single degree of freedom (SDOF) model [30]. Such a SDOF system is based on the dynamic properties adopted from the MDOF system. Structural responses of this SDOF model to earthquake is determined by conducting a nonlinear time history analysis of the model, subjected to a set of ground motion records. Accordingly, the numerical analysis of SDOF systems is of high importance in this field of structural engineering. [31]. However, there are many varieties in each stage of the calculation structure, especially in the calculation of the element node forces (referred to as the correction stage), which will affect the accuracy of the calculation results and the convergence of the iterative process [5]. For the NewMark-β method, when the control parameters $\alpha = 1/2$ and $\beta = 1/4$, the mean constant acceleration method has second-order accuracy (to meet the engineering requirements). NewMark-β is unconditionally stable and is widely used. By controlling the NewMark-β, the results can be more accurate and converging, and the velocity and acceleration responses can be calculated more easily than other methods. used the nonstationary Kanai Tajimi model to record the ground acceleration in MATLAB, simulating the idealized model of the frame structure. investigated the seismic input single-degree of freedom (SDOF) structure and found that the acceleration changed linearly with the step. The response spectrum is used to analyze the response of the structure linear system and seismic input, without considering the nonlinear factors. Three constitutive models for nonlinear analysis of structures were proposed, as follows: linear elastic-elasticity, complete plasticity, and Armstrong Frederick cyclic hardening plasticity. calculated the dynamic response multi-degree-of-freedom nonclassical damping linear system by (MDOF) based on idealized shear stiffness matrix construction and assuming that all floors have the quality. It provides the seismic acceleration time history of the shock response spectrum program, which can be used to calculate peak ground acceleration and velocity based on the known shock response spectrum of the acceleration time history of the earthquake [6]. investigated the response of linear SDOF earthquakes to ground motion through the NewMark method. The response of linear single degree of freedom to seismic ground motion provides a simplified calculation method, which is easy to implement by MATLAB program. The non-linear (or linear) response of a single degree of freedom damped mass-spring system under external forces was predicted. The dynamic analysis of multi-structure underground excitation was presented. However, the idealized model based on the foregoing does not consider the effect of structural stiffness reduction and it is not used by considering the RCS structure. Geometrical nonlinearity of materials is an important characteristic of RCS structures. The design of (RCS) structures is generally based on elastic theory. However, the stiffness coefficient of RCS structure was not constant due to the plastic development of RCS. Due to the effect of seismic force, the response of structural stiffness reduction became more and more obvious [7]. investigated the influence of nanomaterials on reinforcement plasticity, which provided a reference for the research on structural stiffness [8]. investigated the influence of nano-strengthening on the properties and microstructure of recycled concrete, providing another idea for the study of concrete stiffness. The review of the existing literature showed that the scholars have provided different assumptions on the seismic modes and structural mode shapes. He conducted reasonable numerical simulations based on these assumptions and proposed different ideas for structural seismic modal analysis. However, none of the above researches considered time factors caused by reinforced concrete (RCS) structural stiffness reduction. Therefore, the objective of this study is to introduce the stiffness reduction according to the empirical formula, compare the displacement, velocity, and acceleration responses before and after the stiffness reduction is applied at different positions, and obtain the functional relationship between the stiffness reduction factor and the acceleration of the third floor of the RCS frame structure.

Only when the constitutive models of RCS structures of different materials are well-mastered can the stiffness coefficient be analyzed effectively. It is of great significance for different scholars to study the material characteristics through experiments [9,10]. According to the different theoretical basis of mechanics, the existing constitutive models can be roughly divided into the following types: linear elastic and nonlinear elastic constitutive models based on elastic theory; elastoplastic and elastoplastic hardening constitutive models based on the classical plasticity theory; and the constitutive model of concrete described by inner time theory. Later Ni et al. also established a series of constitutive models under different conditions through numerical simulation and experiments

[11,12]. analyzed the stiffness and deflection of RCS members after cracking and established constitutive models at different stages [13]. conducted shear and tension compression experiments on RCS plates and obtained the stress-strain relationship of the corresponding RCS plates [14]. used the hypothesis of strain coordination and strength equivalence to construct the RCS structure plastic damage model. This model can be used in the ABAQUS finite element damage plasticity numerical analysis of RCS walls structures and provides another way of thinking for the analysis of the RCS wall structure seismic damage model [15]. used ABAQUS to conduct finite element simulation in mass concrete and obtained the nonlinear constitutive relation and failure criterion for concrete. proposed a three-dimensional constitutive model for concrete and obtained good failure characteristics of RCS structures by the model through experiments [16]. used the William Wanke plastic constitutive model to describe the nonlinear behavior of concrete, taking into account the interaction between steel bars and concrete and verifying it through experiments [17,18]. classified the RCS structure constitutive models, applying the elastoplastic model of the flow law, considered the softening behavior, and used the damage model to correct it. Zhang et al. conducted a numerical simulation on RCS beams, taking into account the influence of stiffness degradation during the loading and unloading of RCS structures, and verified its constitutive model. simulated the hysteretic relationship of RCS columns based on the uniaxial tension and compression of concrete provided by the China Code for Design of Concrete Structures [19]. These constitutive relations can well reflect the variation trend of elastic modulus in the loading process so that the change rate of stiffness can be analyzed. The analysis of the constitutive model provides a reasonable idea for the reduction of the stiffness coefficient and a reference for the reasonable value of the stiffness coefficient. The process of flowchart is shown in the graphical abstract.

2. NewMark-β method

The newMark-β method is a method to unify the linear acceleration method, being widely used in finite element analysis. Acceleration is modified by the NewMark-β method, as:

$$\dot{y}(t+\Delta t)=\dot{y}(t)+[(1-\alpha)\ddot{y}(t)+\alpha\ddot{y}(t+\Delta t)]\Delta t \tag{1}$$

$$y(t+\Delta t)=y(t)+\dot{y}(t)\Delta t+[(\frac{1}{2}-\beta)\ddot{y}(t)+\beta\ddot{y}(t+\Delta t)]\Delta t^2 \tag{2}$$

If α = 1/2, only keep β, which is the NewMark-β method. If α = 1/2, β = 1/6, that is equivalent to the acceleration varying linearly over Δt, which is the Wilson-θ method with θ = 1; if the acceleration is constant in Δt, it is the average acceleration method.

It can be obtained from Equations (1) and (2) [6]:

$$\ddot{y}(t+\Delta t)=\frac{1}{\beta\Delta t^2}[y(t+\Delta t)-y(t)]-\frac{1}{\beta\Delta t}\dot{y}(t)-(\frac{1}{2\beta}-1)\ddot{y}(t) \tag{3}$$

$$\dot{y}(t+\Delta t)=\frac{1}{\beta\Delta t}[y(t+\Delta t)-y(t)]-(\frac{\alpha}{\beta}-1)\dot{y}(t)-(\frac{1}{2\beta}-1)\Delta t\ddot{y}(t) \tag{4}$$

Substitute the Equations (3) and (4) into the dynamic equation at t + Δt:

$$\bar{K}(t+\Delta t)y(t+\Delta t)=\bar{F}(t+\Delta t) \tag{5}$$

Now we paste:

$$\bar{K}(t+\Delta t)=\frac{1}{\beta\Delta t^2}M+\frac{\alpha}{\beta\Delta t}C(t+\Delta t)+K(t+\Delta t) \tag{6}$$

$$\bar{F}(t+\Delta t)=M[\frac{1}{\beta\Delta t^2}y(t)+\frac{1}{\beta\Delta t}\dot{y}(t)+\left(\frac{1}{2\beta}-1\right)\ddot{y}(t)] \tag{7}$$

The accuracy of the NewMark-β calculation results is mainly determined by the time step Δt. The determination of Δt needs to consider the load change and the length of the natural vibration period T. In general, it is required that Δt be less than 1/7 of the natural vibration period of the minimum structure that has an important effect on the response. When β ≥ 0.25, the NewMark-β method is unconditionally stable; when β < 0.25, it is conditionally stable [20].

$$\{\ddot{u}\}_{t+\Delta t}=\frac{1}{\gamma\Delta t^2}(\{u\}_{t+\Delta t}-\{u\}_t)-\frac{1}{\gamma\Delta t}\{\dot{u}\}_t-(\frac{1}{2\gamma}-1)\{\ddot{u}\}_t \tag{8}$$

$$\{\ddot{u}\}_{t+\Delta t}=\frac{\beta}{\gamma\Delta t}(\{u\}_{t+\Delta t}-\{u\}_t)-(1-\frac{\beta}{\gamma})\{\dot{u}\}_t-(1-\frac{\beta}{2\gamma})\Delta t\{\ddot{u}\}_t \tag{9}$$

Considering the vibration differential equation at time t + Δt, it can be obtained as:

$$[M]\{\ddot{u}\}_{t+\Delta t}+[C]\{\dot{U}\}_{t+\Delta t}+[K]\{u\}_{t+\Delta t}=\{R\}_{t+\Delta t} \tag{10}$$

Solve for available {u}t+Δt and $\{\dot{u}\}$t+Δt first, then solve $\{\ddot{u}\}$t+Δt

The calculation steps of NewMark-β method are as follows [21]:

2.1. Initial calculation:

Forming stiffness matrix [K], Mass matrix [M], and the damping matrix [C];

Setting initial value {u}0, $\{\dot{u}\}$0, and $\{\ddot{u}\}$0;

Select the integral step size t, parameter α and β, and calculate the integral constant:

$$\alpha_0 = \frac{1}{\gamma \Delta t^2} \quad \alpha_1 = \frac{\beta}{\gamma \Delta t} \quad \alpha_2 = \frac{1}{\gamma \Delta t} \quad \alpha_3 = \frac{1}{2\gamma} - 1 \tag{11}$$

$$\alpha_4 = \frac{\beta}{\gamma} - 1 \quad \alpha_5 = \frac{\Delta t}{2}(\frac{\beta}{\beta} - 2) \quad \alpha_6 = \Delta t(1 - \beta) \quad \alpha_7 = \beta \Delta t$$

Forming effective stiffness matrix:

$$[\bar{K}] = [K] + \alpha_0[M] + \alpha_1[C] \tag{12}$$

2.2. Calculating the time step:

Calculating the effective load at time t + Δt:

$$\{\bar{F}\}_{t+\Delta t} = \{F\}_{t+\Delta t} + [M](\alpha_0\{u\}_t + \alpha_2\{\dot{u}\}_t + \alpha_3\{\ddot{u}\}_t) + [C](\alpha_1\{u\}_t + \alpha_4\{\dot{u}\}_t + \alpha_5\{\ddot{u}\}_t) \tag{13}$$

Getting the displacement at time t + Δt:

$$[\bar{K}]\{u\}_{t+\Delta t} = \{\bar{F}\}_{t+\Delta t} \tag{14}$$

Calculating the velocity and acceleration at time t + Δt [9]:

$$\{\ddot{u}\}_{t+\Delta t} = \alpha_0(\{u\}_{t+\Delta t} - \{u\}_t) - \alpha_2\{\dot{u}\}_t - \alpha_3\{\ddot{u}\}_t \tag{15}$$

$$\{\dot{u}\}_{t+\Delta t} = \{\dot{u}\}_t + \alpha_6\{\ddot{u}\}_t + \alpha_7\{\ddot{u}\}_{t+\Delta t} \tag{16}$$

3. Analysis and discussion

Taking the multilayer frame structures as an analysis example, the structural calculation model is shown in Figure1 and the stiffness information of each floor is shown in Table 1and 2.

3.1. Structural stiffness reduction of RCS structure

Structure and affect the resistance and dynamic characteristics of the structure, resulting in local or overall, per-performance loss [22]. The experiment of high-strength steel provides an idea of reinforcing steel, and the combination effect with concrete is more obvious [23]. Scholars have proposed some models, that well-reflected the force characteristics of stiffness degrading components, and obtained some hysteretic cycle rules, such as the IMK hysteretic cycle rule. The MODLMK bending moment Angle model proposed through the OpenSees simulation experiment well-reflected the relationship between column top force and displacement, which validates the effectiveness and stability of the model [6].

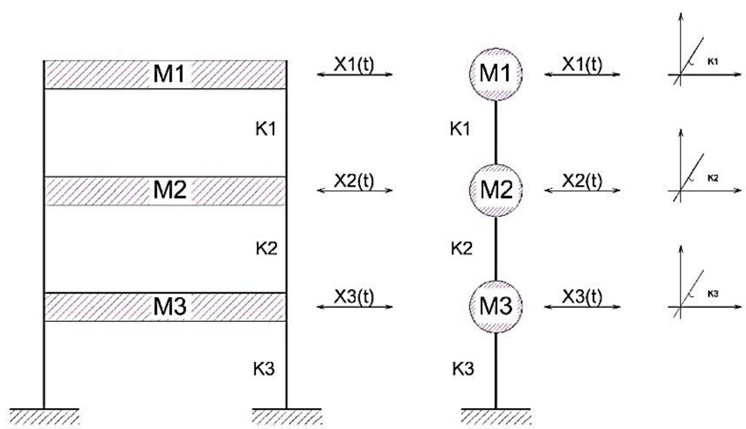


Figure 1: The structure of the computational model of the three-story frame

Table 1:

Characteristics of the mass and stiffness of the structure with a floor height of 3 meters

Floor	Mass (Ton)	Column cross-section	Section moment of inertia (mm4)	Hardness (N.mm)
1	200	IPB 200	$5.7*10^7$	$1.06 * 10^9$
2	0	IPB 240	$1126*10^5$	$2.1 * 10^{10}$
3	0	IPB 280	$1927*10^5$	$3.6 * 10^{10}$

Table 2:

Characteristics of the mass and stiffness of the structure with a floor height of 6 meters

Floor	Mass (Ton)	Column cross-section	Section moment of inertia (mm4)	Hardness (N.mm)
1	200	IPB 200	$5.7*10^7$	$1.33 * 10^8$
2	0	IPB 240	$1126*10^5$	$2.63 * 10^9$
3	0	IPB 280	$1927*10^5$	$4.5 * 10^9$

The stiffness reduction coefficient of the scaffold column took into account an overall comprehensive stiffness reduction coefficient. In the elastic second-order analysis, the overall displacement of the structure and the horizontal displacement between layers were approximately the same as the Stiffness degradation that occurs in plastic hinge areas of RCS frame structures under repeated loads. The accumulation of structural fatigue will increase the plastic zone of the results of the nonlinear analysis [24]. In the American (ACI 318-14) code [25], the structural effect was calculated, the reduction coefficient of the beam member was 0.35, and the column member was 0.7. The cracked wall was 0.35 and the plate was 0.25 In the New Zealand (NZS3101) code, the reduction coefficient of beam members was different due to different sections. The rectangular section was 0.4, and the T-type and L-type components were 0.35. Column components are valued according to different axial compression ratios. China Code for Design of Concrete Structures GB50010-2010 [26] also considered the influence of stiffness reduction, and the reduction coefficient of beam members was 0.4, column members were 0.6, and wall members were 0.45. In this paper, a harmonic load calculation example was applied. The calculation example is considered as the stiffness coefficient is 0.75. During the nonlinear analysis, a stiffness reduction coefficient of 0.75 was introduced to reduce the elastic stiffness K0 to K0 * 0.75, so that under the original load level of the structure, the elastic analysis and nonlinear analysis could produce approximately the same structural response [6]. The internal forces and deformations of the structure under nonlinear conditions were simulated by a unified stiffness reduction factor, rather than giving different reduction factors for different stiffness degradation sections. The unified reduction factor was called the comprehensive stiffness reduction factor. The analysis of the constitutive model provides a reasonable way to reduce the stiffness coefficient [27]. The stiffness reduction coefficient was introduced to analyze the structural acceleration response with different reduction coefficients of stiffness α in the ranges of (0.5, 1). By analyzing the experimental results made by Zhan et al [15]. the constitutive relation of RCS is derived, and the variation trend of elastic modulus in the loading process is obtained by fitting the constitutive relation of the RCS beam, as shown in Figures 2 and 3. It is found that the variation of the stiffness coefficient keeps the descending section and has obvious polynomial characteristics with the gradient of variation at 0.05 So under the original load level of the structure, the elastic analysis and nonlinear analysis produced approximately the same structural response.

Using this principle, the same nonlinear constitutive model S in the structure of different input levels corresponded to different reduction factors, but with the engineering practicability, through a universally applicable reduction factor. However, given engineering practicability, through a universally applicable reduction factor, it was used in the elastic second-order analysis method. The elastic second-order method and the nonlinear finite element method are used to calculate the interlayer displacement Angle and interlayer displacement. The results were equivalent, instead of just constraining the stiffness reduction coefficient α of a certain state [28]. In addition, carbon nanotube cement mortar also improves the workability of concrete and has important research value in mitigating stiffness degradation response [29].

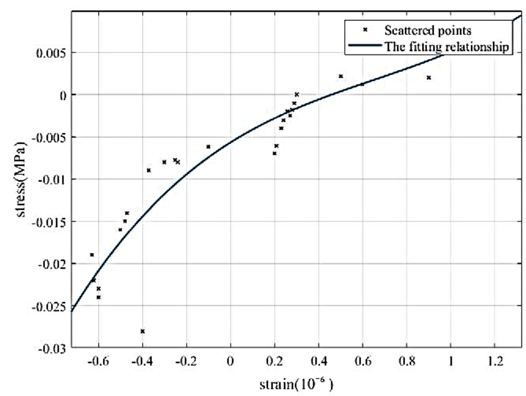


Figure 2: Stress–strain relationship fitting made [18]

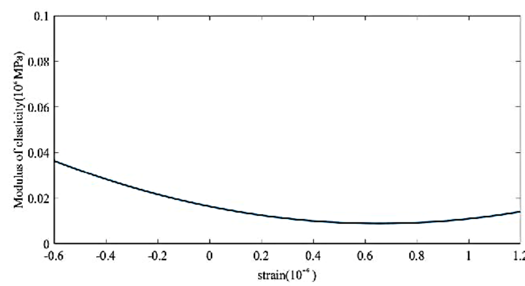


Figure 3: Variation of elastic modulus

3.2. Analysis results

The three degree of freedom frame structure is three-story exposed to a harmonic load once at the top or bottom of the structure for 0.2 seconds, and the response time is 3 seconds with time steps of 0.01 seconds and damping at the value of k * 0.005 is formed The steps of the damping matrix are made based on the Rayleigh method. Damping is used to compare the loads applied in different positions before and after reducing the stiffness, for vibration analysis, MATLAB simulation is used. The forms in different conditions of the stiffness matrix of the structure and the three-story frame element are made according to Figure 1 and analyzed in the local coordinate system.

Material parameters including elastic modulus Mpa E = $2.1*10^5$, frame height in structure "A" H = 3 @ 3 meters, frame height in structure "B" H = 6 @ 3 meters, and other specifications based on tables 1 and 2 were made and the stiffness matrix of the frame element is formed in the local coordinate system as shown in Figure 1 so that the weight force is located only in the M1 level.

The mass matrix of the three-story frame:

$$M = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \Rightarrow M = 2 * 10^6 \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

The stiffness matrix of the three-story frame A:

$$K = \begin{bmatrix} k_1 & -k_1 & 0 \\ -k_1 & k_1 + k_2 & -k_2 \\ 0 & -k_2 & k_2 + k_3 \end{bmatrix} \Rightarrow$$

$$K = 10^{10} \begin{bmatrix} 0.106 & -0.106 & 0 \\ -0.106 & 2.21 & -2.1 \\ 0 & -2.1 & 5.7 \end{bmatrix}$$

The stiffness matrix of the three-story frame B:

$$K = \begin{bmatrix} k_1 & -k_1 & 0 \\ -k_1 & k_1 + k_2 & -k_2 \\ 0 & -k_2 & k_2 + k_3 \end{bmatrix} \Rightarrow$$

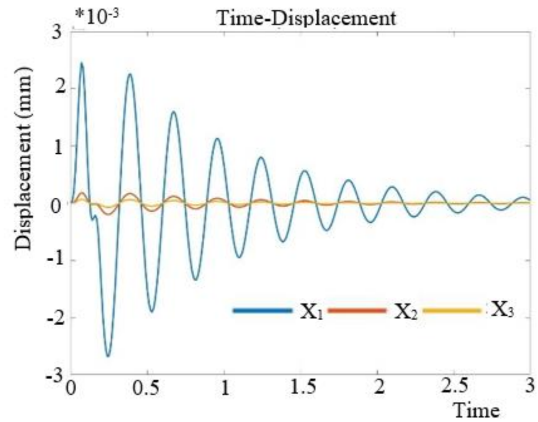
$$K = 10^{9} \begin{bmatrix} 0.133 & -0.133 & 0 \\ -0.133 & 2.67 & -2.63 \\ 0 & -2.63 & 7.12 \end{bmatrix}$$

Harmonic load F = 500 sin (20 * π * t) at the top or bottom of the structure, with a force action time of 0.2 seconds, a response time of 3 seconds, and a step size of 0.01 was made based on Rayleigh damping matrix method.

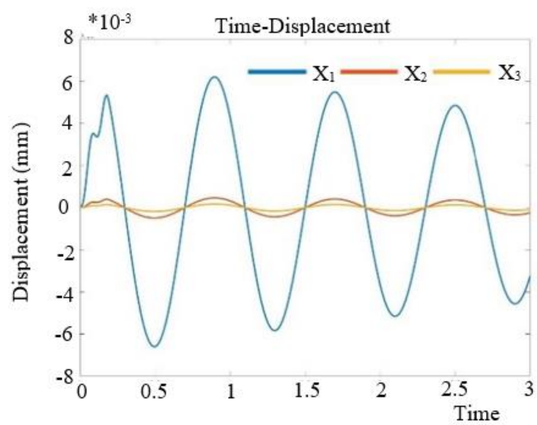
A harmonic load F = 500 sin (20*π*t) is applied to the top of the frame structure, and the structural response is shown in Figure 4.

As can be seen in Figures 4 and 5, the displacement responses were compared before and after reducing the stiffness under the applied harmonic load conditions. In Figure 4, the top change of the three-story frame structure is in the range of m1 alignment, which in case A was in the time limit of about 0.25 seconds, around $-2.8 * 10^{-7}$ to $2.5*10^{-7}$. These displacements in the level of m2 and m3 are very small and they move again and it was approximately with the change of the harmonic function. It reached its peak in the change of about 0.2 seconds and then its displacement fluctuation was gradually leveled with the stop of load application in the change of about 4 seconds. In case B, by changing the case that can be applied in changing the stiffness reduction factor of 0.875 due to the increase in the height of each floor of the structure from 3 meters to 6 meters, the displacement range that can be changed and expands with time and in changing the range $(-6.4 * 10^{-7}\ to\ 5.7 * 10^{-7})$, and the maximum absolute displacement value was lower than before. So that it can reach its peak in the change of about 0.5 seconds and then its displacement fluctuation is gradually leveled by stopping the application of load in the change of about 11 seconds. The reduction of displacement difference between different classes also increased.

(2) The harmonic load F = 500 sin (20*π*t) is applied to the bottom of the frame structure, and the structural response is shown in Figure 5.



(a)

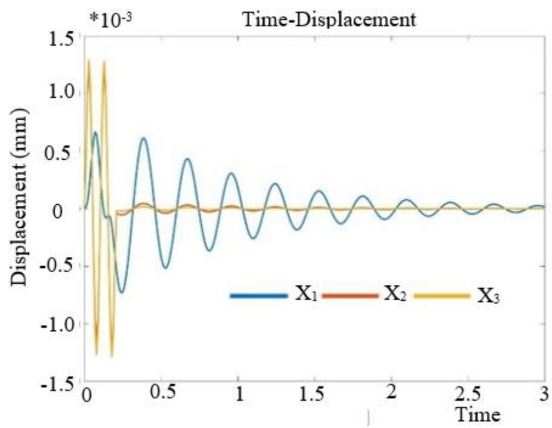


(b)

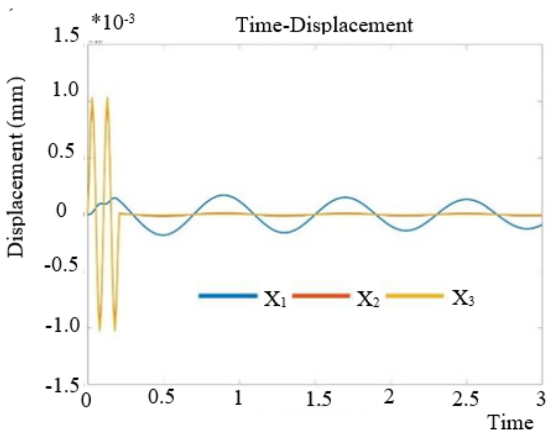
Figure 4 - The amount of displacement of the structure when the force is applied to the maximum level of the floor - (A, the height of the structure is 3 meters - B, the height of the structure is 6 meters)

When downward load was applied, displacement images were compared before and after reduction. The curve was very different from the harmonic curve. In case A, the displacement process of m2 and m3 floors were the same and close to each other before the reduction. After reduction, the displacement difference of each layer increased and the absolute value of its maximum displacement has changed a lot. The displacement responses, before and after reduction, were compared under the applied harmonic load conditions. At the top of the three-story frame structure, the range of displacement change before reduction with time was in the range

$(-1.33 * 10^{-8}\, To\; 1.3\; * 10^{-8})$ The displacement of the first, second and third floors act in different directions and move with time, and it is approximately with the change of the harmonic function and reached its peak in about 0.1 second, and then its displacement fluctuation gradually leveled off with the stop of applying the load in about 4.4 seconds. became. In case B, by changing the case that can be applied in the reduction factor, the stiffness of 0.875 due to the increase in the height of each floor of the structure from 3 meters to 6 meters, the range of displacement that can be expanded with time and in the range of $(-1.04 * 10^{-7}\; to\; 1.02\; * 10^{-7})$ and the maximum absolute value of displacement was higher than before. So that it can reach its peak in the change of about 0.1 second, and then its displacement fluctuation is gradually leveled by stopping the application of load in about 12 seconds. The decrease, the difference in displacement between different classes also increased.



(a)



(b)

Figure 5- The amount of displacement of the structure when the force is applied to the lowest level of the floor - (A, the height of the structure is 3 meters - B, the height of the structure is 6 meters)

In summary, under low load operators, the displacement and velocity over time had a large change due to the reduction in stiffness, it is clear that while it can be in the process of changing the acceleration, it almost keeps the original shape. Reducing the stiffness also has an obvious effect on the stability of the structure.

3.3. Effects of reducing stiffness in the structure

At the top level of the frame structure, the force F = 500 sin (20* π * t) harmonic load, once at the top level of the frame structure and once at the bottom level of the frame structure and comparing the displacement of the structure according to the stiffness coefficient, in diagrams A of Figures 4 and 5 It is clear. The more rigid the structure is and the shorter the height, the more regular the height of the floors will be, compared to situations where the structure has more height and less stiffness. Once again, once at the top level of the frame structure and once at the bottom level of the frame structure, the comparison of the displacement of the structure according to the stiffness coefficient is clear in diagrams B of Figures 4 and 5. When the force is applied at the highest level, the more rigid the structure is, and the shorter its height (Figure A), the shorter the period compared to the case where the structure is less rigid and higher (Figure B) Meanwhile, the main mass is at the level of m1 and its value is zero at the levels of m2 and m3, and if the force is applied at the lower level, the value of the period is almost equal. Considering that the mass is in the m1 level and the m2 and m3 levels have no mass, the maximum displacement is in the m1 position when the force is in the upper level, and the maximum displacement is in the m2 and m3 level when the force is in the lower level.

4. Conclusion

- The NewMark-β method provides a new idea for seismography and the response of the steel frame structure is more practical for the analysis and analysis of the frame structures caused by harmonic load vibrations.
- Through the analysis of the built pattern of the steel frame structure, the range of changes of the elastic modulus is obtained, which can be provided as a basis for the value of the stiffness coefficient.
- The application of top load and bottom load have different structural responses to the steel frame structure, and the effect of the load on the structure is less favorable when it can act on the load at the bottom.

- The higher the height of the structure and the lower its stiffness, the displacement of the upper floor in the case where the force can be applied to the lowest level has a lower displacement than the case where the height of the structure is lower and its stiffness is higher.

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