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The Effect of Non-implementation of the Beam Roof of the Last Floor Block on the Irregularity of the Analysis and Design of the Structure

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Abstract

Knowledge about the earthquake phenomenon is increasing day by day, and building regulations are evolving due to these developments. Previously, all the structures were designed in the elastic range and depending on the intensity of seismicity of the region and the importance of the structure, but now the regulations consider the structures to withstand significant inelastic deformations under the forces resulting from earthquake movements. In general, in earthquake-prone areas, it is not economically appropriate to design common buildings in such a way that these structures remain in the elastic range during severe earthquakes. The development and progress of the knowledge of the dynamics of structures, on the one hand, and the increase of information obtained from recorded earthquakes, on the other hand, show that various factors are effective in the amount of earthquake force. Some of these factors are related to the dynamic properties of structures, such as periodicity, damping, mode shape, structure malleability, etc. In addition, other factors, such as the type of soil and the level of seismicity of the place, are also effective in determining the strength of the earthquake. In fact, the building is not rigid, and it changes shape and vibrates during an earthquake so that the displacement and acceleration created in it gradually increase from the first floor upwards. Also, the natural period of its vibration will be longer, and the acceleration caused by the earthquake will be smaller. In other words, the taller the building, the smaller the acceleration of the earthquake.

Keywords: Earthquake, Regional seismicity, Structure ductility, Acceleration.

1 | Introduction

Since a long time ago, much scientific research has been conducted in the field of preventing financial and life losses caused by earthquakes. This research has been widely carried out in different countries, especially

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earthquake-prone countries such as Japan and America. Based on Newton's second law, in 1993, the first earthquake code was presented in America, in which the force of the earthquake is calculated as the product of the earthquake coefficient, C, in the weight of the building [1]. Although, in traditional methods, structure design is based on their elastic resistance; The new regulations rely on design methods based on the performance level of the structure [2].

Performance-based design was first introduced into the American codes in 1959 by the California Society of Structural Engineers. Due to the unpredictability of earthquake intensity, location, and time, seismic design codes have always specified certain behaviours for different levels of earthquakes. They expect the structure. On the other hand, the non-linear behaviour of materials causes an increase in the period and damping of the structure, which causes a decrease in the force of the earthquake. In the following, first, the seismic design objectives of some regulations are examined. Then, while examining the history of the development of earthquake regulations, the seismic criteria of some design regulations and types of analysis are examined.

2 | Objectives of Seismic Design

Before determining the amount of earthquake load, design method, etc., the accepted design criteria of the structure against the earthquake load must be defined first. Most of the world's seismic regulations have common seismic planning objectives. However, they differ in some details. For the first time in 1976, the principles of seismic design were stated in the SEAOC regulations as follows: In low-intensity earthquakes that may happen repeatedly during the life of the structure, damage to non-structural components should be prevented. As a result, structural components should not be damaged. As a result of moderate earthquakes, damage to structural components should be prevented, and non-structural damage should be minimized [3]. In severe earthquakes, which will rarely happen, the collapse of the building should be prevented. In Iran Standard 2800, the third edition, the minimum criteria and regulations for the design and implementation of buildings against the effects caused by earthquakes are such that it is expected by observing them:

I. By maintaining the stability of the building in severe earthquakes (Design Earthquake (DE2)), the casualties should be minimized. A planned earthquake is an earthquake whose probability of occurrence in the 50 years of the building's useful life is less than 10%.

II. Buildings of great importance during the occurrence of severe earthquakes, without major structural damage, maintain their ability to operate without interruption.

III. The building should be able to withstand mild and moderate earthquakes (earthquakes of the operational level) without incurring major structural damage. An earthquake of the utilization level is an earthquake whose probability of occurrence in the 50 years of the useful life of the building is more than 99.5%.

These criteria are stated in the fourth edition of the 2800 standard as follows:

I. Buildings should not suffer major structural or non-structural damage in severe earthquakes, and the casualties should be minimal. A severe earthquake is an earthquake whose probability of occurrence is less than 50% in the 50 years of the useful life of the building.

II. Buildings of great importance during the occurrence of severe earthquakes, without major structural damage, maintain their ability to operate without interruption.

III. As a result of very strong earthquakes, the building should maintain its stability and not collapse. A very severe earthquake is an earthquake whose probability of occurrence in the 50 years of the useful life of the building is less than 10%.

IV. All buildings taller than 50 meters, as well as all buildings of high and very high importance, should not be damaged by moderate earthquakes and should maintain their operability [4].

In ATC-40, three earthquake levels are defined:

I. Service Earthquake (SE) level is an earthquake whose probability of occurrence in 50 years of the useful life of the building is less than 50%.

II. Design Earthquake (DE2) is an earthquake whose probability is less than 10% in the 50 years of the useful life of the building.

III. Maximum Earthquake (ME1) is an earthquake whose probability is less than 5% in the 50 years of the useful life of the building.

3 | Linear Analyses

The meaning of linear analysis is the analysis of the structure considering the linear elastic behaviour of its components. In general, linear analysis methods are suitable when the behaviour of structural components is within the linear range during an earthquake or when a small number of components are out of the linear range. If the ratio of the forces caused by the earthquake to the bearing capacity of the components is less than 2, the effect of nonlinear behaviour is not significant and linear analysis methods can be used [5].

In linear analysis, only the main members are modelled, and the non-main members are only controlled to change the shapes resulting from the analysis because the non-main members usually have a significant reduction in stiffness and strength under reciprocating loads and are quickly removed from the lateral load system. They turn In linear analysis, only the main members are modelled, and the non-main members are only controlled to change the shapes resulting from the analysis because the non-main members usually have a significant reduction in stiffness and strength under reciprocating loads and are quickly removed from the lateral load system. They turn If P- or cracking of concrete or masonry components is considered, these effects are included in the linear analysis in a simplified form. For example, "P-effect" in linear static analysis is introduced in the form of lateral overload and cracking effect simply by reducing the characteristics of the members' sections in the model [6].

3.1 | Linear Dynamic Analysis

In the linear dynamic analysis method, the forces and deformations caused by an earthquake are determined using the dynamic equilibrium relations governing the elastic model of the structure. Since in this method, the dynamic characteristics of the structure are included in the analysis, the results obtained are more accurate than the linear static analysis method, but in any case the nonlinear behavior of the model materials is not considered. Linear dynamic analysis can be performed by two methods: spectral and time history [7].

In the spectral method, the spectrum used should be the linear elastic spectrum without correction for nonlinear deformations. The results obtained from the linear dynamic analysis are close to reality for structures whose behavior remains linear during an earthquake.

In time history analysis, the response of the structure is calculated using dynamic relations at short time steps. In this method, the response of the structure model under the excitation of ground acceleration must be calculated based on at least three acceleration maps [8]. The specific assumptions of this method in the range of linear behavior are:

I. The behavior of the structure can be calculated as a linear combination of the states of the different vibration modes of the structure that are independent of each other.

II. The period of vibration of the structure in each mode is constant during the earthquake.

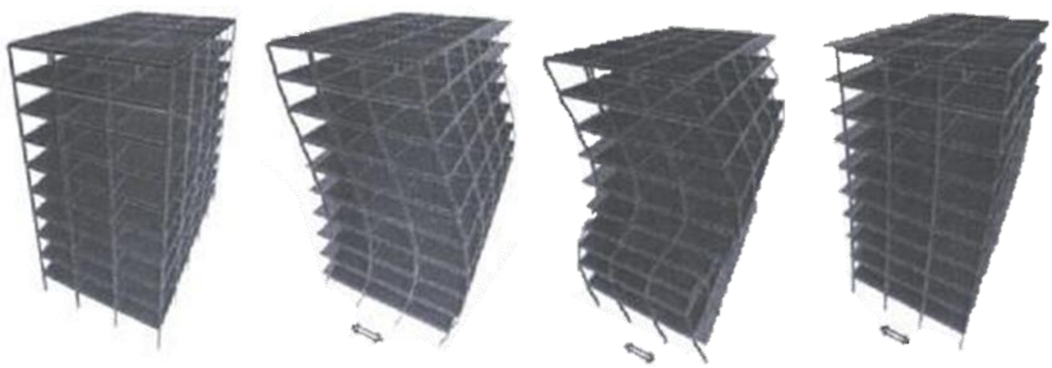
3.2 | Linear Quasi-Dynamic or Spectral Analysis

In static analysis, only the stiffness parameter of the structure is important in the force distribution and analysis results, but in dynamic analysis, in addition to stiffness, mass is also very important. The seismic performance of the structure also depends greatly on the strength and ductility of the structural members.

The purpose of dynamic analysis is to determine more accurately the base shear and its actual distribution in the structural members. According to the 2800 code, the equivalent static method is acceptable for regular structures with a height of less than 50 meters and irregular structures with a height of less than 18 meters. Considering the codes of other countries, it can be said that these criteria are somewhat simplistic [9].

With the development of computers, linear dynamic analysis was used for structures higher than 50 meters after 1971. Since parts of the structural members enter the nonlinear region during an earthquake, nonlinear dynamic analysis must be performed to examine the precise behavior of the structure in an earthquake. However, for this to be possible, a large number of properties of the materials and sections of the members and input records must be available, which is often difficult or impossible. In addition, the use of the results requires a lot of experience and knowledge. As a result, some building codes recommend the use of nonlinear dynamic analysis only for important structures with very high irregularities. It should be noted that linear dynamic analysis is more accurate in the distribution of forces than the equivalent static method and is also much simpler than nonlinear dynamic analysis.

Today, the nonlinear static method is widely used in the process of strengthening existing structures. The structure can vibrate in different modes, according to the figure. In most short structures, the first mode usually accounts for about 90% to 95% of the vibration of the structure. As the number of floors increases, the contribution of higher modes also increases. Therefore, for tall buildings (usually higher than 10 stories), even if the building is regular, the contribution of higher modes is still significant. For example, the first three modes of a 10-story building are shown in the figure below. In practice, for many reasons, irregularities occur in the building due to the asymmetric distribution of mass and stiffness. As shown in *Fig. 1*, the effect of higher modes is also greater. In the equivalent static method, such effects cannot be considered in the behavior of the structure [10].



a. b. c. d.

Fig. 1. Modal polyhedron of a 10-story building; a. mode 1, b. mode 2, c. mode 3, d. mode4.

The basic assumption in the equivalent static method is that only the first mode of the building has a major contribution to the vibration, and the contribution of higher modes is not significant. Also, the first mode is simply proportional to the height and mass of the floors (and not their stiffness) and is assumed to be approximately inverted triangular. As a result, the equivalent static method cannot be used for buildings in which the contribution of higher modes is of great importance.

Because in some members, the design force is greatly underestimated, and in others it is greatly overestimated. The time-history dynamic method for determining earthquake-induced displacements and forces in structures is laborious and time-consuming and usually has to be carried out by electronic calculators. If, instead of the displacement history, only the maximum values due to different modes are taken, the dynamic analysis of structures is considerably simplified. Since the maxima of different modes do not occur at the same time and do not necessarily have the same sign, the maximum values cannot be added together. The best that can be done in a quasi-dynamic or spectral analysis is to combine the maximum solutions obtained from different modes based on probability theory. Various approximate formulas are used to combine the maxima, the most common of which is the root-sum-of-squares formula. Most of the energy from an earthquake is absorbed in the first few modes. Therefore, for structures with very high degrees of freedom, it is usually sufficient to combine the first 3 to 6 modes, thus achieving a significant saving in calculations. In this method, dynamic analysis is performed assuming the linear elastic behavior of the structure and using the maximum reflection of all oscillatory modes of the structure that have a significant effect on the reflection of the entire structure. The maximum dynamic reflections of the structure, such as internal forces of members, displacements, floor forces, floor shears and foundation reaction in each mode, should be determined by known statistical methods such as the root-sum-of-squares method (SRSS) or the perfect combination of squares of the maximum reflections of each mode (CQC). Combining the effects of maximum modes in buildings that are irregular in plan or in cases where the period times are When two or more structural modes are close to each other, it should be done only by methods that take into account the interaction of vibrational modes, such as the perfect square combination method [10].

3.3 | Linear Dynamic Time History Analysis

The dynamic analysis method (moment-by-moment calculation of building reflections under the influence of real earthquake accelerometers) can be used for all buildings. In general, for completely regular buildings or buildings that are regular in height, if this method is used, it can be performed separately in two orthogonal directions of the building. However, if the building is so irregular in plan that its oscillation in some or all modes mainly occurs simultaneously in two orthogonal directions, i.e., the building has oscillatory modes [8]. In which the modes are motion in one direction along with motion in a direction perpendicular to it. To take into account the effects of these combined motions, the building must be calculated by dynamic analysis using a three-dimensional model. In this method, the reflections of the structure at each time point during the earthquake are determined by applying the accelerations caused by ground motion (Gasht acceleration) to the base level of the building and performing the relevant dynamic calculations. This analysis method can be used in linear elastic analysis or nonlinear analysis of existing structures. Comparison between the results of the elastic analysis of the structure using the standard design spectrum or the site-specific design spectrum or that obtained from linear time history analysis is mandatory, and the possible reasons between them should be justified in a comprehensive technical report [11].

3.4 | Nonlinear Analyses

Nonlinear analysis refers to the analysis of a structure by considering the nonlinear behavior of its components due to the nonlinear behavior of materials, cracking, and nonlinear geometric effects. In nonlinear analysis methods, plastic joints are predicted at the points of maximum moment due to gravity loads, and the structural model is analyzed accordingly. After the analysis, using the results obtained, the bending moment diagram of the member should be redrawn and the location of the plastic joints should be controlled. For this purpose, similar to linear methods, the moment diagram is obtained from the sum of the moment diagram of gravity loads and the moment obtained from the analysis under lateral earthquake load (unlike linear methods where the moment corresponds to the expected capacity of the member was placed at both ends) and should be compared with the expected capacity of the member along the entire length. If the predicted position for the plastic joint is not correct, it is necessary to reanalyze the structure and correct the position of the plastic joint. The figure below shows the difference between the two linear and nonlinear methods. The curved line represents the actual behavior of the material or the behavior of a part of the structure, and the straight line represents the assumed linear behavior. In the range indicated by the letter a, there is no difference between the linear and nonlinear methods, but in the range b, where the deformations obtained from the linear analysis are similar to those obtained from the nonlinear analysis, it is necessary to increase the lateral force [8].

In this way, the deformations are calculated with the desired accuracy, but it is necessary to correct them appropriately before using the internal forces of the members for control or design.

In the nonlinear analysis, all the main and non-main members are modeled, and the effect of reducing the strength and stiffness of the components (deceleration) is included in the model [8].

4 | Linear Static Analysis

One hundred years ago, after the December 28, 1908 earthquake in Messina (Italy), a scientific committee formed by the Italian government made simple recommendations for the design of some structures against earthquakes. For the first time in a technical report, it was mentioned that the nature of earthquake loads is dynamic and should be considered as such. Of course, at the same time, it was stated that due to the insufficient progress in the field of dynamic analysis, the simplified equivalent method is used. This method was called the equivalent static method [12].

4.1 | Building Analysis and Design Parameters

Each of the 5 and 7-story buildings was analyzed and designed using the equivalent static method using the Iranian Standard 2800 reflectance spectrum. The analysis and design of the buildings were performed using ETABS-Ver 9.7 software. For static and dynamic analysis and design, the structural specifications were considered as follows: the structure is residential and constructed with a medium-sized concrete flexural frame system in an area with high seismic risk (Chalus City) and a location with type soil. The damping ratio of the structure is considered to be 5%, the structural importance factor is I=1, and the design base acceleration is A=0.3.

In the design of the members, the behavior coefficient (R) was used in accordance with the 2800 Earthquake Code, which is given below. The loading of the frames was carried out based on the sixth topic of the National Building Code and the 2800 Standard, fourth edition. The design of the concrete members (beams and columns) was carried out based on the ninth topic of the National Building Code.

The reference for the design of steel frames is the American Steel Code ACI 318-99. For the design of concrete members (beams, columns and floor ceilings), the 28-day strength of a cylindrical concrete specimen is 280 kg/cm2, the modulus of elasticity of concrete is 2.18e5 kg/cm2, the yield stress of the flexural reinforcement and shear reinforcement are 4000 kg/cm2 and 3000 kg/cm2, respectively, and Poisson's ratio is 0.2. For the design of steel members (rebars), ST37 steel with a minimum yield stress of 2400 kg/cm2, an ultimate stress of 4000 kg/cm2, and a modulus of elasticity of steel of 2.18e6 kg/cm2 and a Poisson's ratio of 0.3 was used. In the design of beams, square concrete sections have been used, and columns have been used in order to approach their ultimate capacity. The closer the designed sections are to their ultimate capacity, the better the behavior of the structure will be.

5 | Inspection of a 5-Story Building

In order to compare the two considered cases, two models of the 5-story building, namely 5T and 5TS, were analyzed separately, and their results were compared. As it was stated, the co-basing of the shear in the regular structure after the initial analysis, the reflection values should be multiplied by 85% of the ratio of the static base shear equivalent to the base shear obtained from the spectral analysis. Also, the cumulative mass absorption coefficient should reach 90%. According to *Table 1*, *Table 2* and *Fig. 2*, this condition is satisfied in both models.

Table 1. Section of classes resulting from the preliminary analysis of the 5T model.

STORY	LOAD	LOC	VX	VY	T	MX	MY
STORY3	SPY	BOTTOM	0	163919.1	2048989	1109365	0
STORY2	EPX	TOP	-235424	0	3237082	0	-1494365
STORY2	EPX	BOTTOM	-235424	0	3237082	0	-2260316
STORY2	ENX	TOP	-235424	0	2648521	0	-1494365
STORY2	ENX	BOTTOM	-235424	0	2648521	0	-2260316
STORY2	EPY	TOP	0	-235424	-3237082	1494365	0
STORY2	EPY	BOTTOM	0	-235424	-3237082	2260316	0
STORY2	ENY	TOP	0	-235424	-2648521	1494365	0

Table 1. Continued.

STORY	LOAD	LOC	VX	VY	T	MX	MY
STORY2	ENY	BOTTOM	0	-235424	-2648521	2260316	0
STORY2	SPX	TOP	192778.7	0	2409733	0	1109365
STORY2	SPX	BOTTOM	192778.7	0	2409733	0	1708217
STORY2	SPY	TOP	0	192778.7	2409733	1109365	0
STORY2	SPY	BOTTOM	0	192778.7	2409733	1708217	0
STORY1	EPX	TOP	-235424	0	3237082	0	-2260316
STORY1	EPX	BOTTOM	-235424	0	3237082	0	-3009013

Table 2. 5T model mass absorption coefficient control.

Mode	Period	UX	UY	SUMUX	SUMUY	SUMRX	SUMRY	SUMRZ
1	1.284428	80.1018	0	80.10181	0	0	99.6275	0
2	1.284428	0	80.1018	80.10181	80.1018	99.6275	99.6275	0
3	1.17625	0	0	80.10181	80.1018	99.6275	99.6275	80.2813
4	0.391971	0	11.3269	80.10181	91.4288	99.6546	99.6275	80.2813
5	0.391971	11.3269	0	91.4288	91.4288	99.6546	99.6546	80.2813
6	0.36334	0	0	91.4288	91.4288	99.6546	99.6546	91.5519
7	0.202005	0	4.8895	91.4288	96.3182	99.9622	99.6546	91.5519
8	0.202005	4.8895	0	96.3182	96.3182	99.9622	99.9622	91.5519
9	0.188806	0	0	96.3182	96.3182	99.9622	99.9622	96.367
10	0.128051	0	2.4221	96.3182	98.7404	99.9744	99.9622	96.367
11	0.128051	2.4221	0	98.7404	98.7404	99.9744	99.9744	96.367
12	0.12074	0	0	98.7404	98.7404	99.9744	99.9744	98.7561
13	0.08993	0	1.2596	98.7404	100	100	99.9744	98.7561
14	0.08993	1.2596	0	100	100	100	100	98.7561
15	0.08591	0	0	100	100	100	100	100

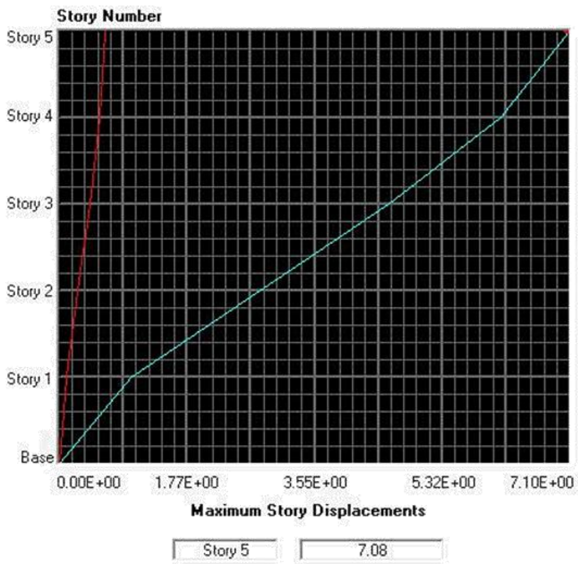


Fig. 2. Maximum displacement of floors (in centimetres) under load - EPX model 5T [13].

6 | Conclusion

I. According to the obtained results, the spectral dynamic analysis is equivalent to the static analysis.

II. The maximum floor displacement in a 5-story building, the case where the last roof is a slab (5TS), is greater than the case where all the roofs are block beams (5T) under earthquake load (equivalent static).

III. The maximum displacement of floors in a 5-story building in the case where the last roof is a slab (5TS) is higher than in the case where all the roofs are block beams (5T) under dynamic load.

IV. The shearing of the base of the floors in a 5-story building, the case where the last roof is a slab (5TS), is greater than the case where all the roofs are block beams (5T) under earthquake load (equivalent static).

V. The maximum displacement of floors in a 5-story building in the case where the last roof is a slab (5TS) is higher than in the case where all the roofs are block beams (5T) under dynamic load.

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