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# Seismic Response of Nonseismically Designed Reinforced Concrete Low Rise Buildings

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## ABSTRACT

In this paper, the time-history responses of a square plan two-story reinforced concrete prototype building, considering the elastic and inelastic behavior of the materials, were studied numerically. ABAQUS software was used in three-dimensional (3D) nonlinear dynamic analysis to predict the inelastic response of the buildings. Concrete Damage Plasticity Model (CDPM) has been used to model the inelastic behavior of the reinforced concrete building under seismic excitation. The input data included geometric information, material properties, and the ground motion. The building structure was designed only for gravity load according to ACI 318 with non-seismically detailing requirements. The prototype building was subjected to El Centro 1940 NS earthquake at different amplitudes (PGA=0.05g, PGA=0.15g, and PGA=0.32g). The elastic and inelastic responses of the 3D numerical model of the same building were evaluated. The differences between the elastic and inelastic displacements and base shear forces were analyzed. It was found from the results that base shear responses are significantly more sensitive to the numerical model of analysis than displacement responses. The evaluation showed that the base shear force and displacement responses of a two-story R.C. building subjected to severe earthquake excitation are very sensitive to the numerical model used whether it is elastic or inelastic.

**Keywords:** Elastic and Inelastic Dynamic Responses, 3D Finite Element, CDPM, Earthquake Excitation.

# الاستجابة الزلزالية للابنية الخرسانية المسلحة واطئة الارتفاع وغير المصممة لتأثير الزلازل

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## الخلاصة

في هذا البحث تمت دراسة عددية لاستجابات سجل الزمن لبناية خرسانية مسلحة مربعة المسقط ذي طابقين ماخوذ بعين الاعتبار التصرف المرن واللامرن للمواد. تم استخدام برنامج ABAQUS في التحليل اللاخطي ثلاثي الابعاد لتوقع الاستجابة اللامرنة لمنشآت الابنية. استخدم انموذج تضرر الخرسانة اللدن (CDPM) لنمذجة الاستجابة اللامرنة للبناية الخرسانية المسلحة المعرضة لاهتياج زلزالي. احتوت المدخلات على ابعاد البناية وخواص المواد وحركة الارض. تم تصميم منشأ البناية لاحمال الجاذبية طبقا لمدونة المعهد الامريكي للخرسانة (ACI 318) لمتطلب التفاصيل غير الزلزالية. تم تعريض النماذج الاصلية للمنشأ لهزة ارضية نوع (El Centro 1940 NS) بشدات مختلفة ( PGA=0.05g, PGA=0.15g and PGA=0.32g). تم حساب الاستجابات المرنة واللامرنة للانموذج ثلاثي الابعاد ولنفس البناية. تم تحليل الفروقات بين الازاحات وقوى القص الناتجة من الحالة المرنة واللامرنة. وجد من النتائج بأن استجابات قوى القص هي

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اكثر حساسية بشكل ملحوظ للانموذج العددي المستخدم في التحليل اكثر من استجابات الازاحة. اظهرت الحسابات بأن استجابات قوة قص القاعدة والازاحة لبناية خرسانية مسلحة ذي طابقين معرضة لاهتياج زلزالي عنيف حساسة للغاية للانموذج العددي المستخدم في التحليل.

**الكلمات الرئيسية:** الاستجابة الحركية المرنة واللامرنة، العناصر المحددة ثلاثية الابعاد، انموذج تضرر الخرسانة اللدن، اهتياج زلزالي.

## 1. INTRODUCTION

In recent years, advanced methods of structural dynamic showed that much more work in both experimental and theoretical needs is to be done to develop a proper understanding of the inelastic behavior of buildings under dynamic loads. The nature of the inelastic response in a structural system might be changed significantly under moderate to severe dynamic loadings. The partial and complete collapse or damage of reinforced concrete buildings under seismic loadings has demonstrated the need to design structural members, like beams, columns, etc., to be able to withstand the complex nature of seismic excitations. For reinforced concrete members, inelastic deformation does not concentrate in a critical location, but rather spreads along the member, **Otani, 1980**. A various number of numerical models have been proposed to represent the distribution of stiffness within a reinforced concrete member. On the other hand, the changes in member capacities and in structural response demand caused by moderate to severe seismic loadings, as compared to minor loadings, are not fully understood.

Nonlinear dynamic analysis of a reinforced concrete structure requires two types of mathematical modeling, **Otani, 1980**. The first is for the distribution of stiffness along reinforced concrete members, and the other is the force-deformation relationship under stress reversals.

**Moehle** and **Alarcon, 1986**, studied the response of reinforced concrete structures having irregular vertical configurations to uniaxial strong base motions. Two frame-wall multi-story reinforced concrete structures were constructed in small-scale and subjected to uniaxial earthquake simulations on a shaking table.
The responses of the two buildings were computed by using the following analytical methods:

1. Inelastic dynamic response analysis.
2. Inelastic static analysis.
3. Elastic modal spectral analysis.
4. Elastic static analysis.

All analyses were based on one analytical model. It comprised a frame, a wall, and lumped masses constrained to have the same lateral deflections at floor levels. The experimental responses of the multi-story reinforced concrete structures were compared with responses computed by the above methods. Moehle and Alarcon concluded that the main advantage of the dynamic methods was that they were capable of estimating maximum displacement responses, i.e., they provided an indication of the maximum displacement response, whereas the static methods alone are generally incapable of indicating displacement amplitudes for a given seismic event. They also concluded that the computed inelastic response was sensitive to small changes in modeling assumptions.
**Ermiao** and **PANKAJ, 2004** examined the influence of material modeling on the dynamic responses for reinforced concrete frames. The study showed that the influence of strain rate on the seismic analysis of reinforced concrete structures is small. They also showed that the inclusion of a small value of hardening parameter has only a small influence

In **2010**, the optimal combination of hysteresis-modeling and damping parameters was identified by **Lepage, et al., 2010**. The identification was used in the practical nonlinear dynamic analysis to obtain satisfactory correlations between the calculated and measured the seismic response of reinforced concrete frames in both amplitude and waveform. They characterized frame members by five modeling parameters, which are:

1. Initial stiffness.
2. Bond–slip rotations.
3. Post-yield stiffness.
4. Unloading stiffness.
5. Viscous damping.

Experimental data were measured from three small-scale shake table for multistory test structures and a seven-story instrumented building. By using the Frequency Domain Error (FDE) index, the goodness-of-fit of the computed response to the recorded experimental data was measured. It was concluded that, in general, the derived models have satisfactory accuracy when representing the global measured response of the test structures. Also, it was found that the measured response can be tracked successfully by the calculated roof displacement, base shear, and overturning moment histories.
The seismic response of existing reinforced concrete buildings can be evaluated by four different analysis approaches as follows:

1. Linear Static Analysis.
2. Linear Dynamic Analysis.
3. Nonlinear Static Analysis.
4. Nonlinear Dynamic Analysis.

In order to compare efficiency and differences of above approaches, **Caprili, et al., 2012** had executed a thorough investigation on a reinforced concrete existing building whose dynamic behavior was evaluated by the interpretation of experimental dynamic results. The four approaches are characterized by increasing complexity and, at the same time, increasing capability to describe the effective structural behavior. Based on a quantitative comparison between the performed seismic assessment procedures in term of percentage of damaged beams and columns for bending and shear failure. It was observed that, in general, for linear dynamic analysis the total number of damaged beams and columns (values of internal forces) is higher in comparison to nonlinear static analysis and nonlinear dynamic analysis. It is worthwhile to mention that the average percentage of damaged beams in shear was varied from 18% in the linear dynamic analysis to 4% in the nonlinear dynamic analysis.

## 1. RESEARCH OBJECTIVE

The main objective of this research is to study the effect of peak ground acceleration (PGA) on the response of a two-story prototype non-seismically designed reinforced concrete building with two types of numerical modeling: elastic modeling and inelastic modeling.

## 1. NUMERICAL MODELING OF A TWO-STORY SQUARE PLAN PROTOTYPE BUILDING

### 1.1. Design Criteria and Material Properties

The design of the two-story prototype reinforced concrete office building is presented in this section, **Fig.1** The building is considered to be representative of low-rise buildings constructed in Iraq. Since wind loads seldom govern for low-rise buildings, the building is designed primarily to carry only gravity loads. Thus no considerations are made for seismic resistance and the general non-seismic detailing provisions of the ACI 318-11 (ACI Committee 318 2011) code are used for the design of the first model building, **Figure 2.** The basic material strengths assumed for the design of the structure are ASTM 615 Grade 60 steel ($\mathrm{fy} = 420\,\mathrm{MPa}$) and ordinary Portland cement concrete with a specified 28-day strength ($\mathrm{f}_\mathrm{c}' = 35\,\mathrm{MPa}$). The structure is assumed to be built on stiff soil/rock conditions such that no soil-interaction or differential settlements need to be considered.

### 1.2. Structural Description of the Building

The office building was designed for a superimposed dead load of 2 kPa and live load of 2.4 kPa. The gravity load combination of $1.4\,D_L + 1.7\,L_L$ (ASCE/SEI 7–10 2010) is the only design loading used to achieve the most adverse stresses in the members of the structure. It is a moment resisting framed structure consisting of one bay having spacing 3.6 m c/c in both directions as represented in **Figure 1**. All columns are 300mm × 300mm, and all beams are 300mm × 480mm. A scaled-down mode (1:6) has been proposed for the purpose of experimental and a prototype model has been modeled for numerical investigation, **Figure 2**. The nonseismically reinforcement details are shown in **Figure 2**.

### 1.3. Earthquake Ground Acceleration

Ground Motion for El Centro 1940 NS Accelerogram Component,

**Figure** 3 was applied at different amplitudes to evaluate the model structure performance under seismic excitation. To accomplish the objective of the research, i.e., study the effect of peak ground acceleration (PGA) on the elastic and inelastic responses of a two-story prototype reinforced concrete building under the earthquake chosen, the amplitude of the ground motion, El Centro 1940 NS, was scaled such that the peak acceleration for the earthquakes are 0.05 g, 0.15 g, and 0.32 g. These three earthquakes are representative of minor, moderate and severe ground motions, respectively in terms of ensuring different types of structural behaviors.

### 1.4. Finite Element Modeling

In this research, a finite element (FE) model is established and the numerical solutions are correlated with the experimental results obtained by **Al-Baghdadi, 2014** in order to check the adequacy of the model. The time-history of the story displacements during run El Centro 0.15g for both experimental and analytical results are shown in **Figure 4.** It can be seen from the figure that there is a good agreement between the numerical and experimental results.
The FE models are created using the finite element (FE) code ABAQUS/CAE 6.12-1 (Abaqus/CAE 6.12-1 2012). The models have the same geometry, dimensions, and boundary conditions of the tested frame building.
Three dimensional (3D) first order reduced integration continuum elements (C3D8R - Brick) are used to model the concrete members while the steel reinforcements are modeled by using (B32 – 3D Beam) element. These elements can be used in models for simple linear analysis or for nonlinear analyses involving contact, plasticity and large deformations. A typical mesh discretization of the concrete and steel rebar is used in the analyses.

Concrete Damage Plasticity Model (CDPM) is one of the possible constitutive models. In this paper, CDPM has been used to model the inelastic behavior of the reinforced concrete building under seismic excitation. The model is a continuum, plasticity-based, damage model for concrete.

## 2. NUMERICAL RESULTS AND DISCUSSION

In order to determine the elastic characteristics of the two-story R.C. building, a free vibration analysis was performed. **Table 1** summarizes the natural frequencies of the first three mode shapes. **Figure 5** shows the mode shapes of the prototype building.

The displacement-time histories obtained from elastic and inelastic analyses of the prototype building subjected to PGA=0.05g (minor) and PGA=0.15g (moderate) earthquake excitations are shown in **Figure 6** and **Figure 7**, respectively. It is shown from the figures that the responses for the elastic and inelastic behaviors were correlated and were in phase during the whole time histories. For the same amplitude intensities of excitation, the base shear-time histories for elastic and inelastic showed the same trend in behavior, **Figure 9a** and **Figure 9b**. On the other hand, for sever excitation (PGA=0.32g) it can be seen from **Figure 8** and **Figure 9c** that the displacement- and base shear-time history for elastic behavior is significantly different from the behavior of inelastic one. Moreover, the time-histories for both were not correlated and were not in phase.

It is useful in engineering practice, to get the maximum displacement amplitude of motion and the maximum base shear. **Table 2** summarizes maximum displacement and base shear for both inelastic and elastic behaviors of the prototype building. The relation between the maximum displacement for each story and ground acceleration is shown in **Figure 10.** It was found that under severe excitation, adopting elastic models in the analysis may give unconservative displacement results and will affect serviceability requirements in the design.

On the other hand, the variation of the maximum base shear with ground acceleration for both inelastic and elastic behaviors **Figure 11** showed that the response is not only significantly sensitive for severe base excitation but it is also significantly sensitive for moderate base excitation, and in both cases, the assumption of elastic models give base shear conservative results and will affect the economic requirements in the design.

Finally, **Figure 12** illustrates the trend of the ratio between the inelastic and elastic response versus ground motion of excitation for both displacement and shear force. It is seen that the trend is nonlinear in nature and depends mainly on the amplitude of the excitation. For a two-story building, it can be seen that for sever earthquake excitation the elastic numerical model gives differences of about 87% and 60% than of inelastic models for maximum displacement and maximum base shear, respectively and gives uncorrelated and not in phase time-response histories. Then the elastic models may be unacceptable in the modeling of R.C. buildings.

## 3. SUMMARY AND CONCLUSIONS

From the discussions carried out in the previous sections and depending upon the results obtained from the numerical analysis, the following main conclusions are drawn.

- Base shear responses are significantly more sensitive to the numerical model (elastic or inelastic models) of analysis than displacement responses.

- Base shear force and displacement responses of a two-story R.C. building subjected to severe earthquake excitation are very sensitive to the numerical model used (elastic or inelastic models).
- Depending on an inelastic numerical model CDPM, there is a good agreement between the numerical and experimental results.
- The linear elastic analysis method is not recommended to be used in the analysis of reinforced concrete buildings subjected to moderate and severe seismic loadings.
- The inelastic behavior of the R.C. building under sever excitation is significantly different from those corresponding to the elastic ones.

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## NOMENCLATURE

| Symbol | Meaning |
|---|---|
| ACI | american Concrete Institute |
| ASTM | american Society for Testing and Materials |
| FE | finite Element |
| MDOF | multi-Degree-of-Freedom |
| PGA | peak Ground Acceleration |
| 3D | three Dimensional |
| R.C. | reinforced Concrete |
| NS | north South |
| CDPM | concrete Damage Plasticity Model |

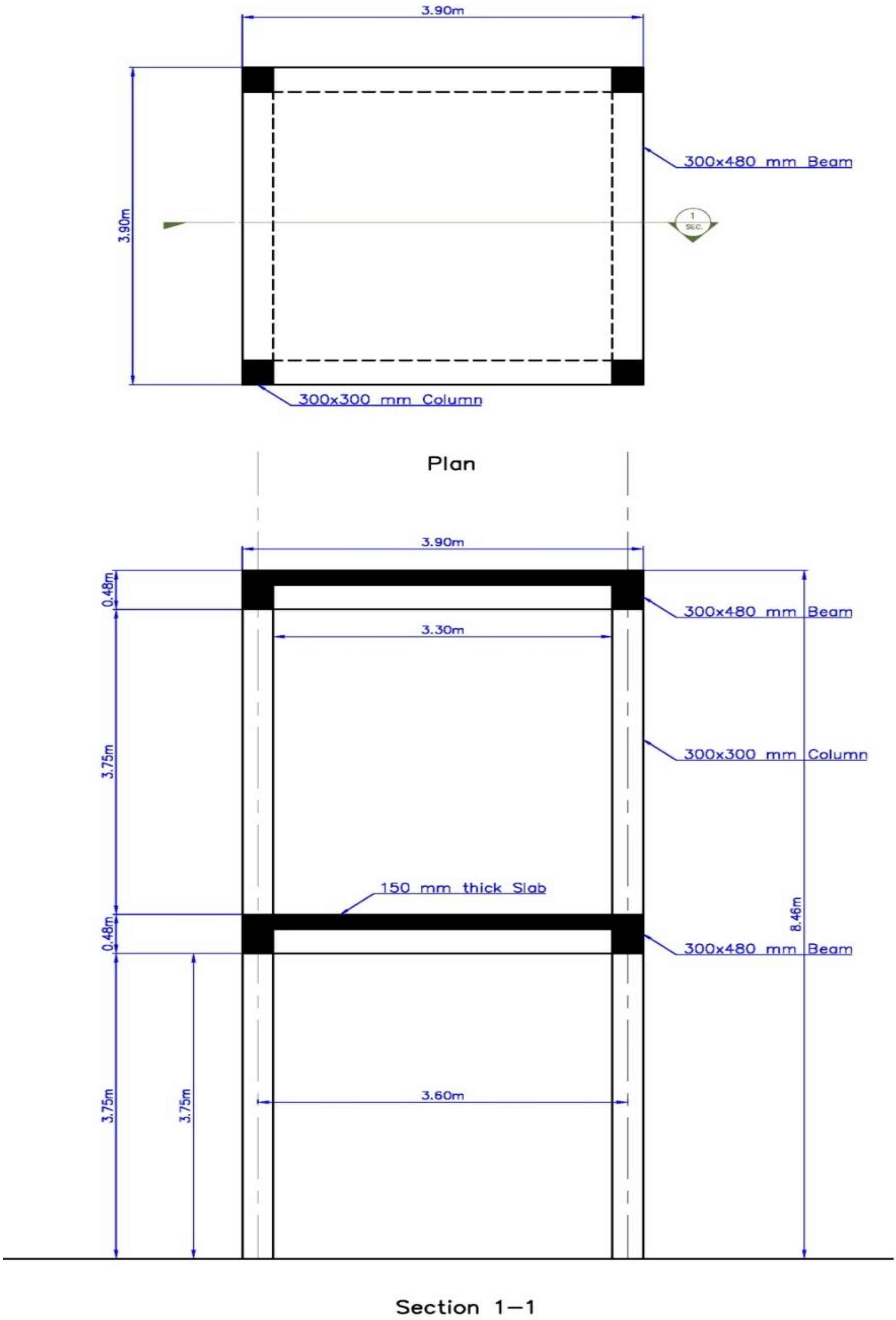



**Figure 1.** General Layout and Section of the Idealized Prototype Building.

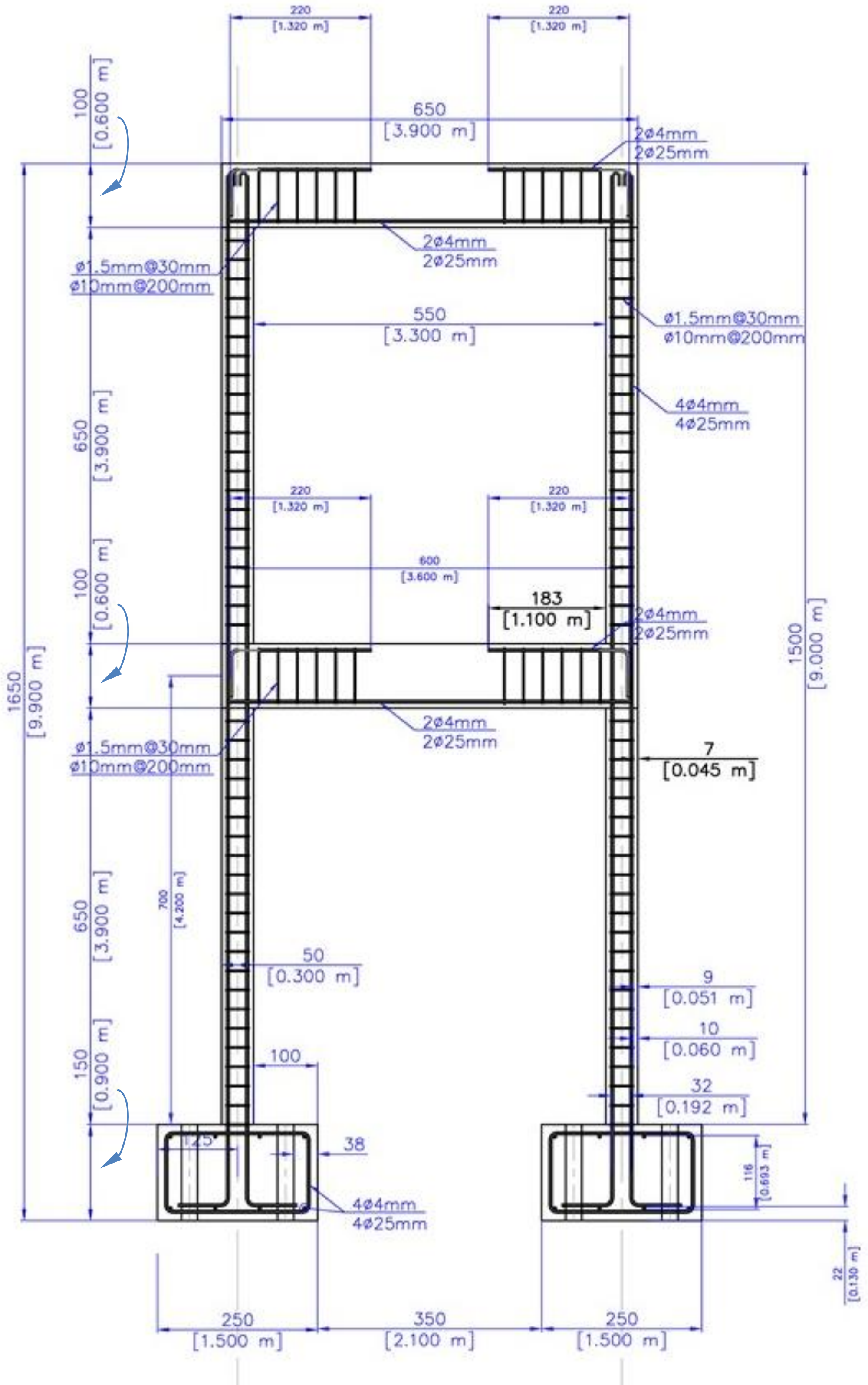



**Figure 2.** Details of Two-Story 1:6 Scale Model [Prototype] Building.

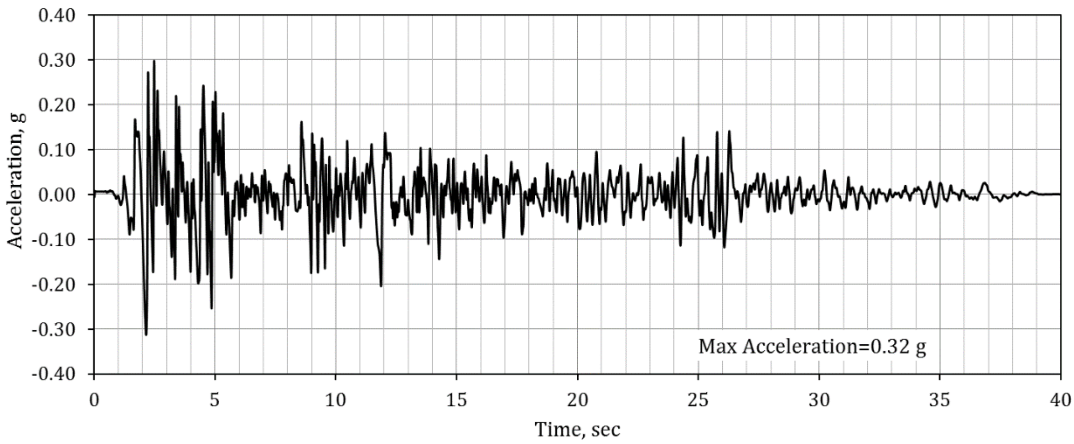
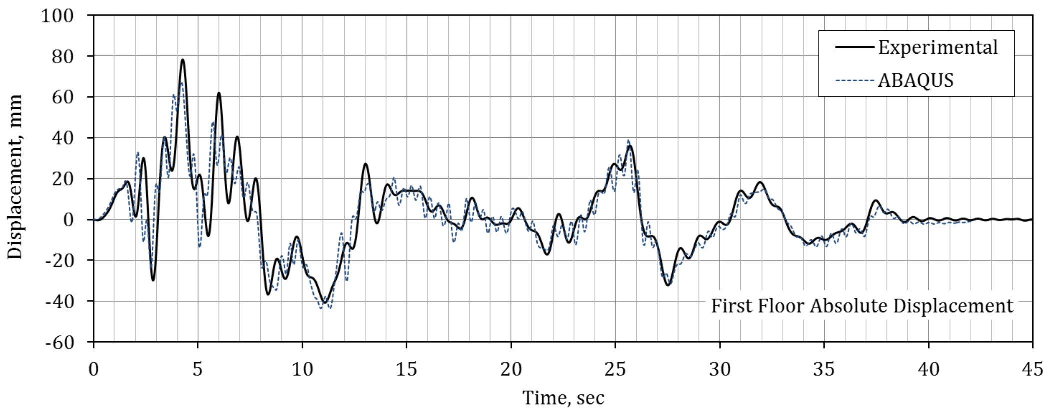
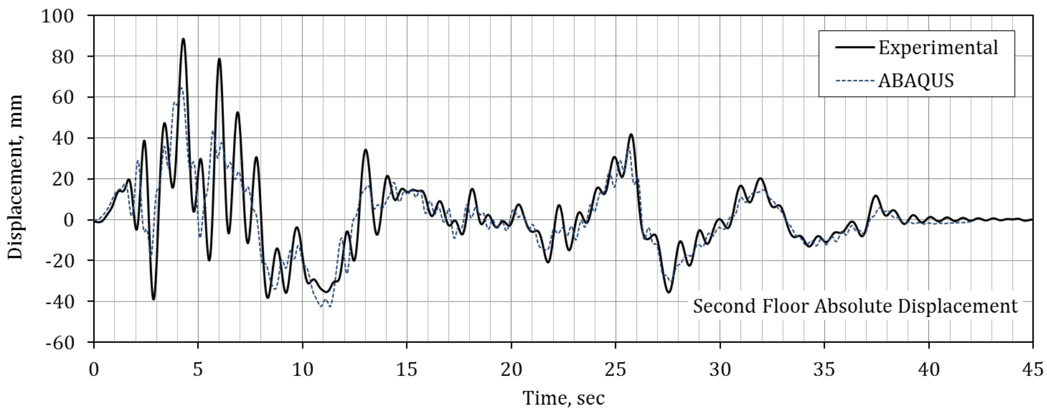



**Figure 3.** Ground Motion for El Centro 1940 NS Accelerogram Component.




(a) First Floor

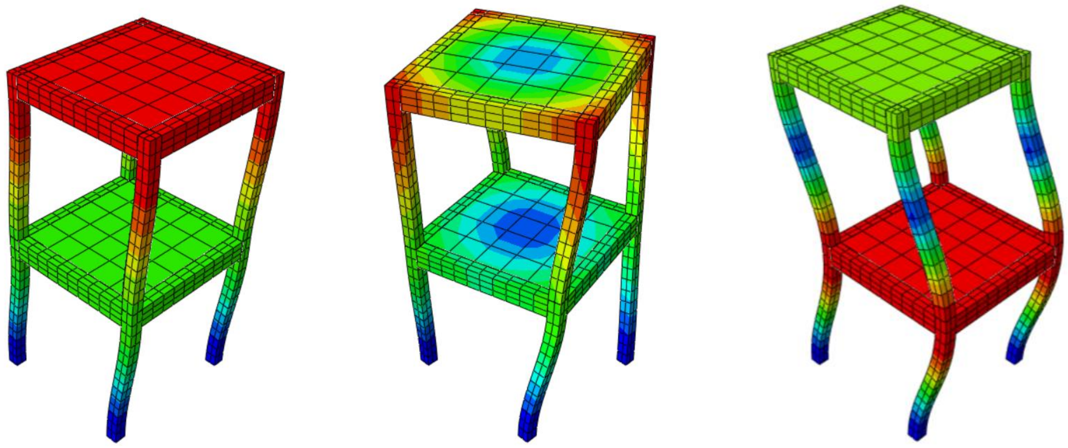



(b) Second Floor

**Figure 4.** Displacement-Time Response for Inelastic Behavior of Model Structure and Prototype Building, PGA=$\mathbf{0.15g}$ (Experimental Results after Al-Baghdadi, H. A., 2014).

**Table 1.** Elastic Free Vibration Analysis result of the Prototype Building.

| Mode | Frequency, Hz |
|---|---|
| First natural frequency | 2.476 |
| Second natural frequency | 3.792 |
| Third natural frequency | 6.733 |

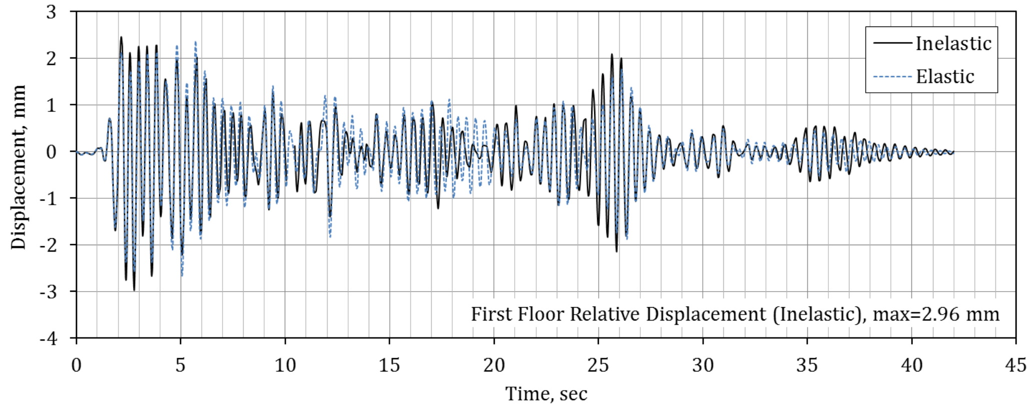


(First) (Second) (Third)

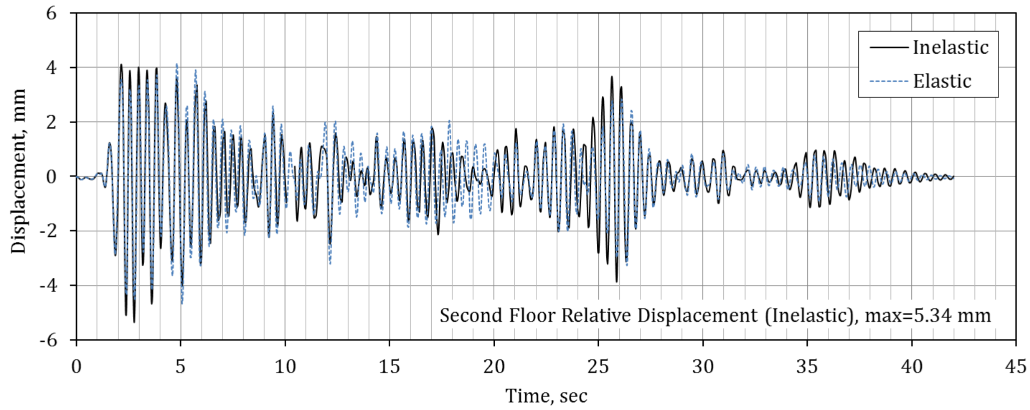
**Figure 5.** Mode Shapes of the Prototype Building.

**Table 2.** Maximum Displacement and Base Shear for Inelastic and Elastic Behaviors of the Prototype Building.

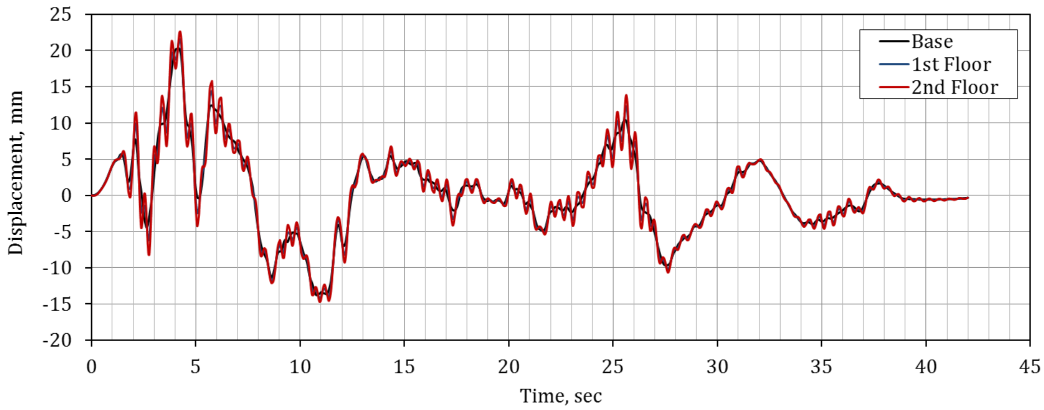
| Location | Elastic Response | | | Inelastic Response | | | Inelastic/Elastic Response (Unitless) | | |
|---|---|---|---|---|---|---|---|---|---|
| | $0.05g$ | $0.15g$ | $0.32g$ | $0.05g$ | $0.15g$ | $0.32g$ | $0.05g$ | $0.15g$ | $0.32g$ |
| First Story Displacement (mm) | 2.66 | 8.00 | 17.00 | 2.96 | 8.15 | 31.70 | 1.11 | 1.02 | 1.87 |
| Second Story Displacement (mm) | 4.66 | 14.00 | 30.00 | 5.34 | 14.25 | 38.10 | 1.15 | 1.02 | 1.27 |
| Base Shear (kN) | 30 $kN$ | 90 $kN$ | 191 $kN$ | 31 $kN$ | 83 $kN$ | 76 $kN$ | 1.03 | 0.92 | 0.40 |




(a) First Floor

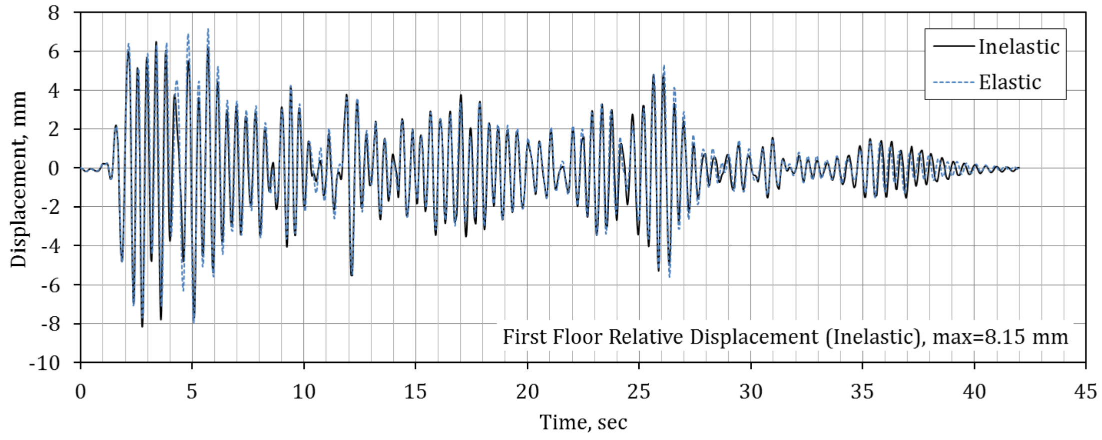



(b) Second Floor

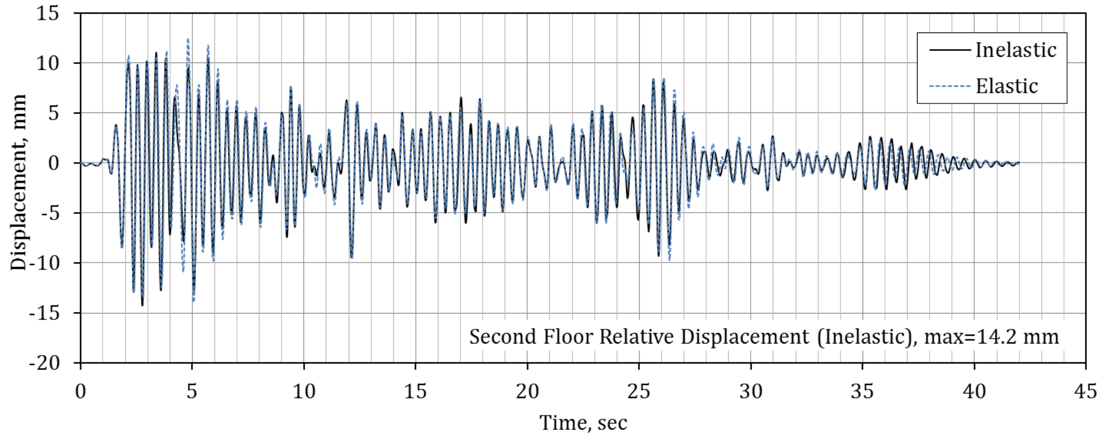



(c) Absolute Displacement

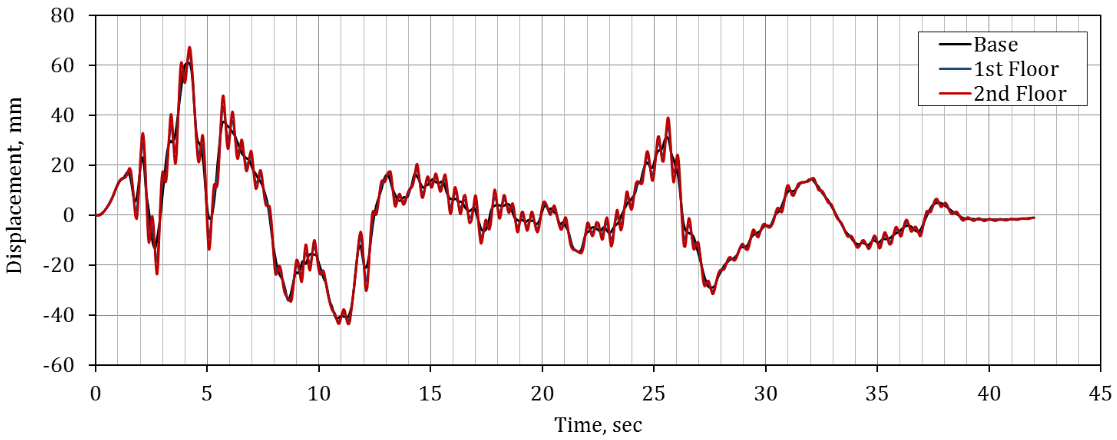
**Figure 6.** Displacement-Time History for Elastic and Inelastic Behaviors of the Prototype Building, PGA=$\mathbf{0.05g}$.




(a) First Floor

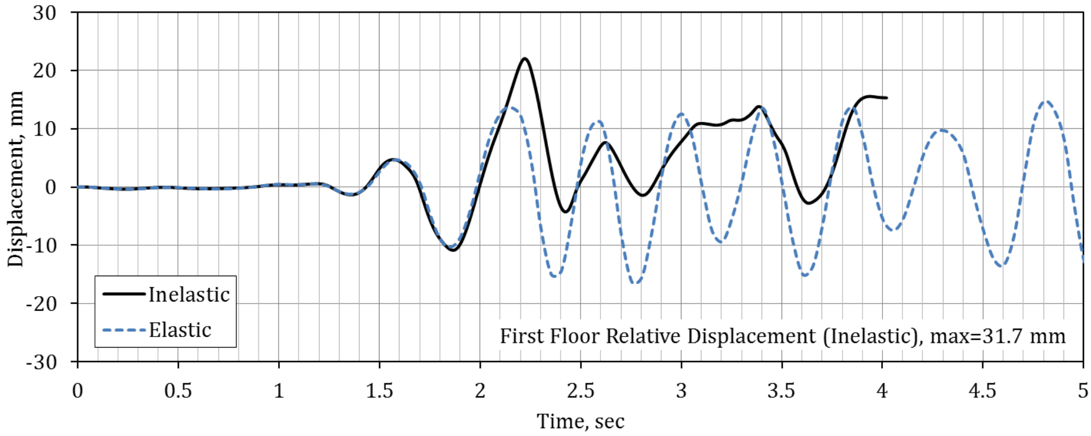



(b) Second Floor

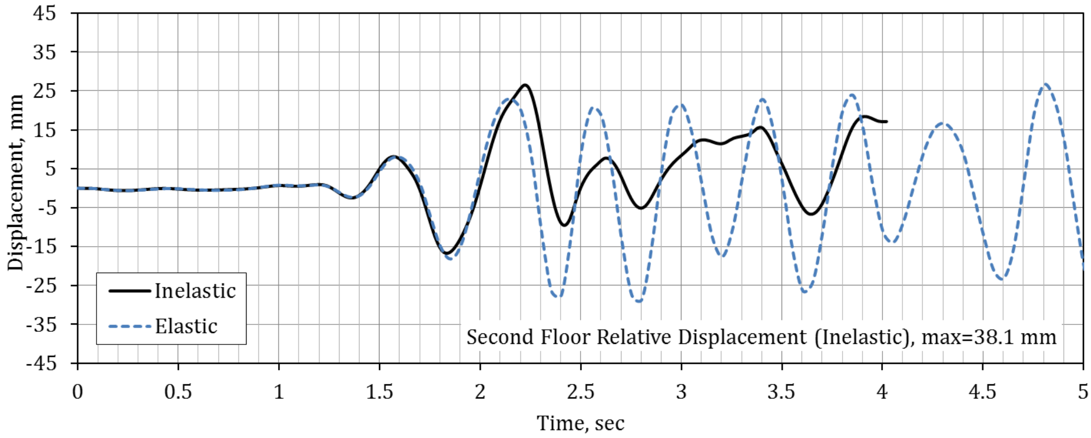



(c) Absolute Displacement

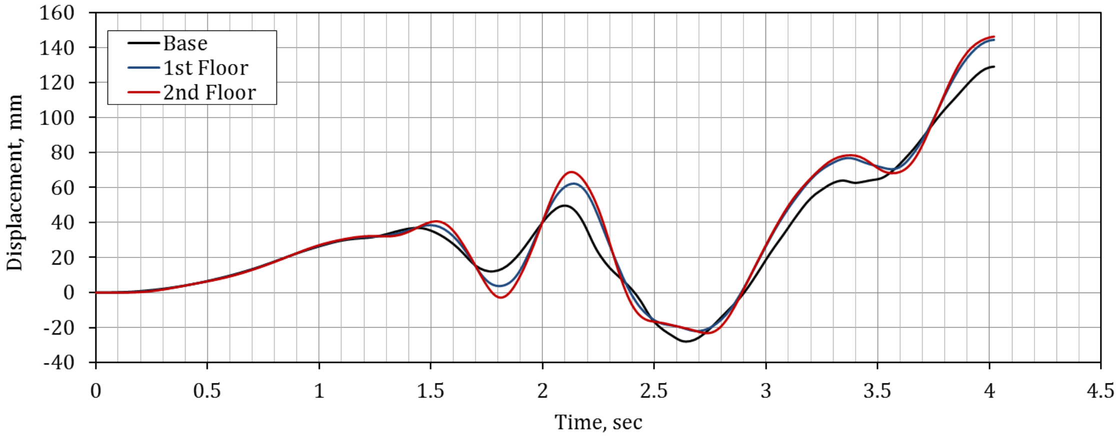
**Figure 7.** Displacement-Time History for Elastic and Inelastic Behaviors of the Prototype Building, PGA=$\mathbf{0.15g}$.




(a) First Floor

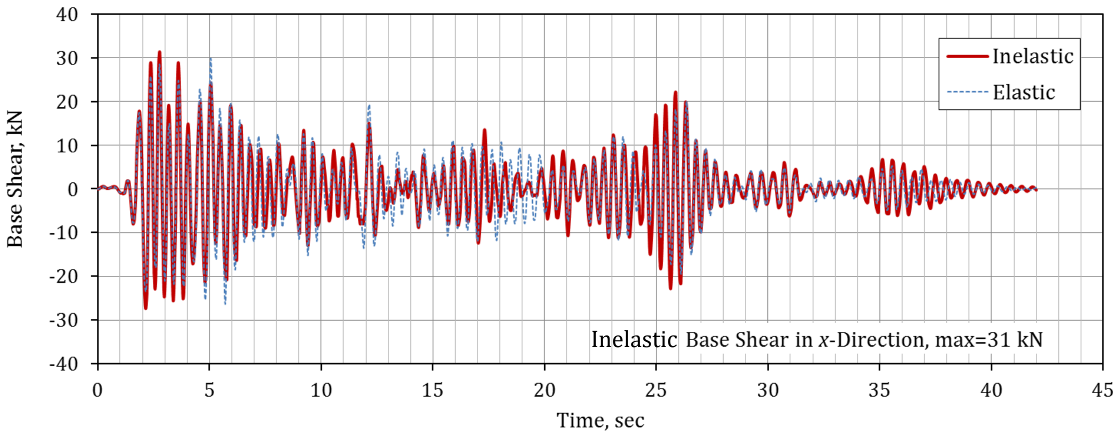



(b) Second Floor

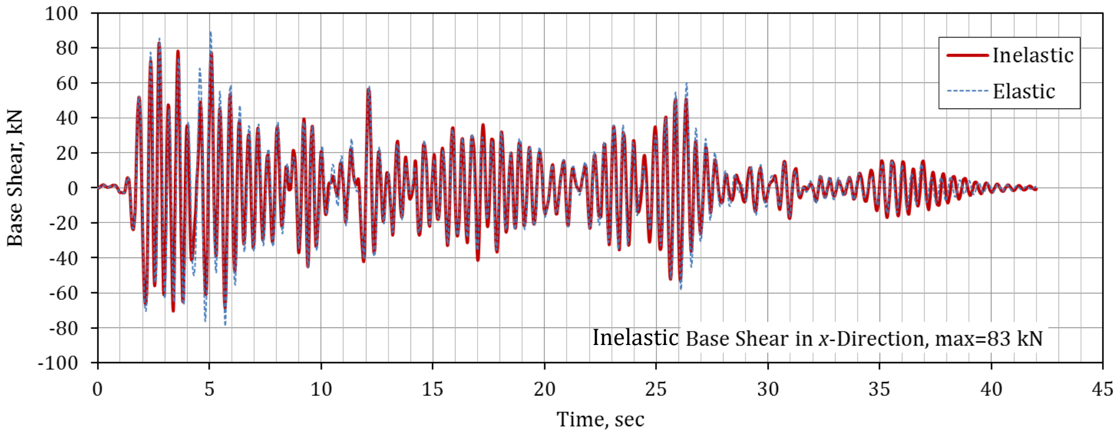



(c) Absolute Displacement

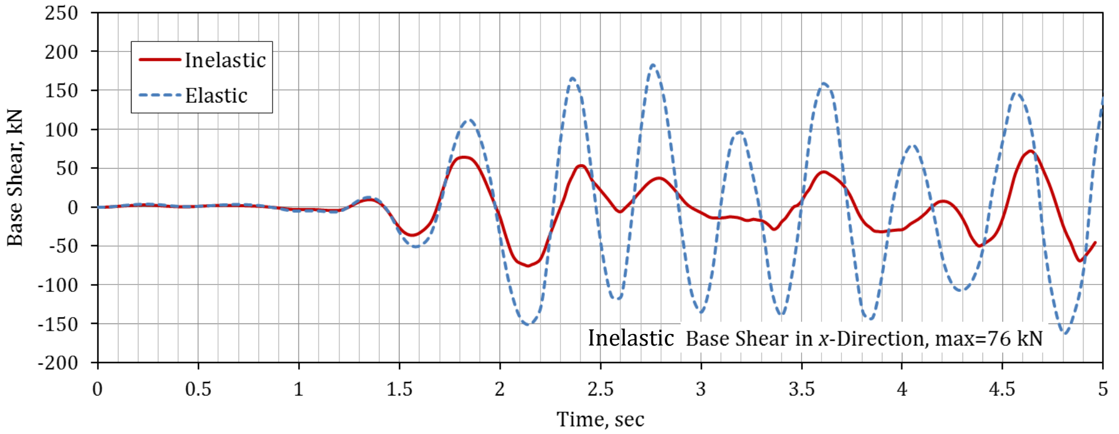
**Figure 8.** Displacement-Time History for Elastic and Inelastic Behaviors of the Prototype Building, PGA=$\mathbf{0.32g}$.




(a) PGA=0.05g.




(b) PGA=0.15g.




(c) PGA=0.32g.

**Figure 9.** Base Shear Time History for Inelastic and Elastic Behaviors of Prototype Building.

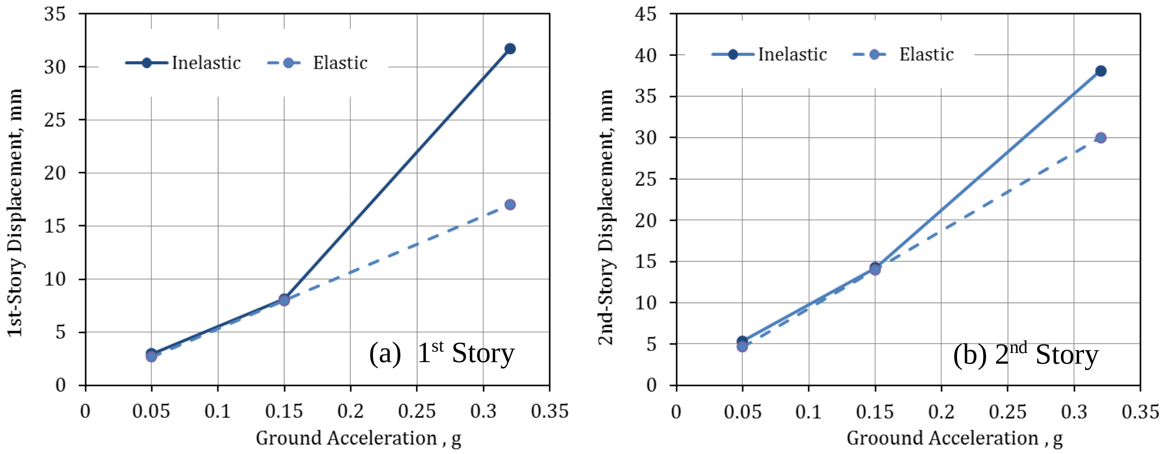



**Figure 10.** Variation of the Maximum Displacement with Ground Acceleration for both Inelastic and Elastic Behaviors of the Prototype Building (a) 1st Story, (b) 2nd Story.

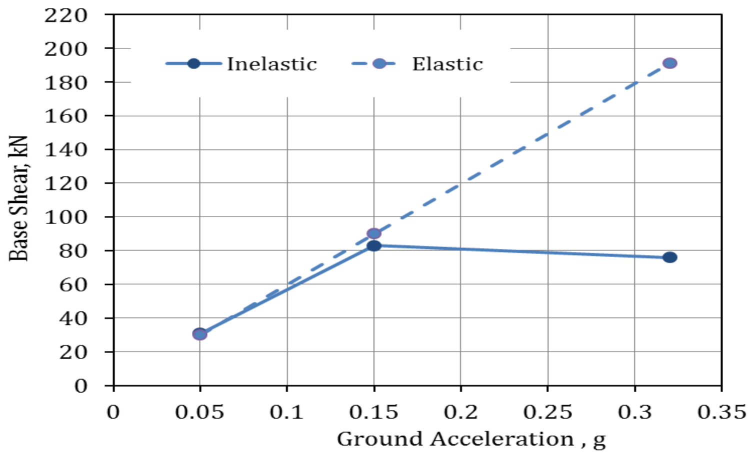



**Figure 11.** Variation of the Maximum Base Shear with Ground Acceleration for both Inelastic and Elastic Behaviors of the Prototype Building.

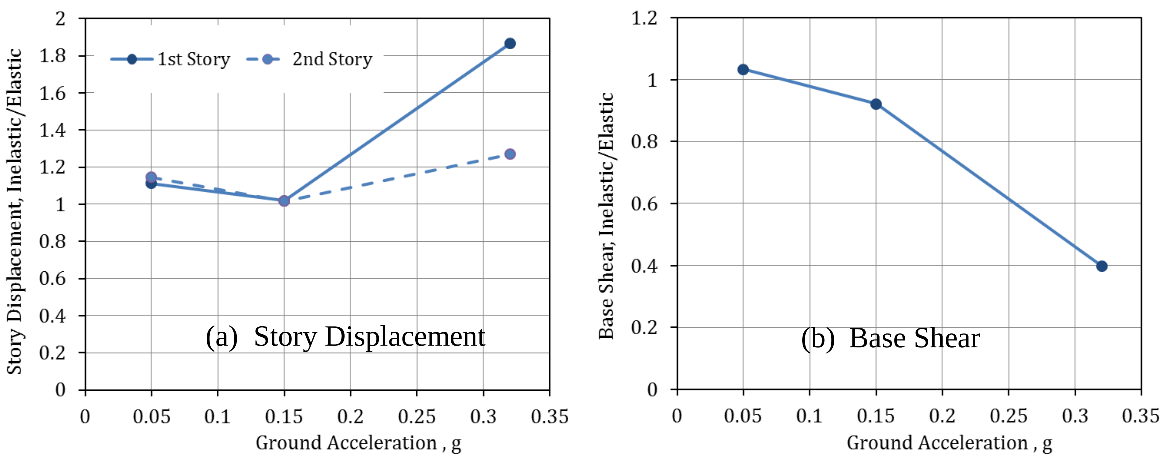



**Figure 12.** Variation of the Maximum Inelastic/Elastic Response with Ground Acceleration of the Prototype Building, (a) Story Displacement, (b) Base Shear.