Seismic Retrofiting of a Ten-Story Steel Framed Hospital

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ABSTRACT: *With the knowledge of constant threat of major earthquakes and especially the latest dramatic and catastrophic earthquake of northern Iran in 1990 and recent quakes of 1997, a multi-disciplinary program was launched in Iran to assess seismic vulnerability of important buildings and possibly offer cost-effective strengthening solutions for the ones in need. As a result of the latest awakening toward upgrading the existing buildings in the country, especially capital city of Tehran, this paper will concentrate on the upgrade design of a ten story steel framed hospital for both vertical and lateral loads, providing information on the strengthening procedures and considerations used to achieve the goals of the project. The existing structure under investigation is a ten story steel framed skeleton with four story completed and under use, constructed to old seismic code in 1985 and left alone due to economical reasons until 1997 the beginning of this project. Four different schemes were considered for this particular building which will be discussed herein with main considerations being economical and easy construction. Static equivalent procedure was utilized in the design of the upgrade system and then, behavior and design was controlled using non-linear dynamic analysis utilizing DRAIN-2D program. Details of this project will be discussed in this paper.*

Keywords: Seismic retrofit; Steel buildings; Dynamic analysis, Strengthening; Seismic vulnerability, Iranian seismic code

1. Introduction

The devastating 1990 earthquake of northern Iran with Ms=7.7 and PGA=0.65g was the worst seismic event of this century inflected on densely populated areas. It killed more than 40,000 people, injured 100,000 and left more than half a million homeless causing the worst economical losses in the history of this nation [1, 2]. Considering this very recent dramatic experience and the return period of 158 years for a strong earthquake with Ms > 7 for the capital city of Tehran with more than 10 million people living, seismic safety has become a high priority.

Numerous studies have shown that the greatest current threat to life safety arising from a large earthquake in Iran is posed by existing hazardous structures which was graphically illustrated by the experience in northern provinces. Therefore, there is now a considerable effort underway in the country to develop guidelines for evaluation and rehabilitation of the existing buildings. However, in the absence of such guidelines, structural engineers with the help of researchers are developing and implementing effective ways to upgrade structures [3, 4, 5, 6, 7, 8].

This paper is the result of one such study which was tailor designed based on requirements of the owners and tempered by the engineers and researchers to address the "weak links' of a ten story steel framed hospital. This building was designed to old seismic code and lacked sufficient lateral resistance. Luckily, after constructing the main structural frame, only first four floors were completed for use and due to economical reasons the next six floors were left alone as shown in Figure (1). This paper focuses on analytical evaluation of the vulnerability of the existing structure and also discusses the development of strengthening schemes used for achieving the objectives of the project.

2. Description of the Building

The existing structure under investigation is a ten story steel framed skeleton with four story completed and under use, constructed to old seismic code in 1985 and left alone due to economical reasons until 1997 the beginning of this project. In order to complete the building in 1997, the owners had to upgrade the building to new and more restricted standards and seismic code sanctioned by the government in 1991 after the devastating earthquake of northern Iran in 1990.

The building is 22m wide and 45.4m long with a floor to

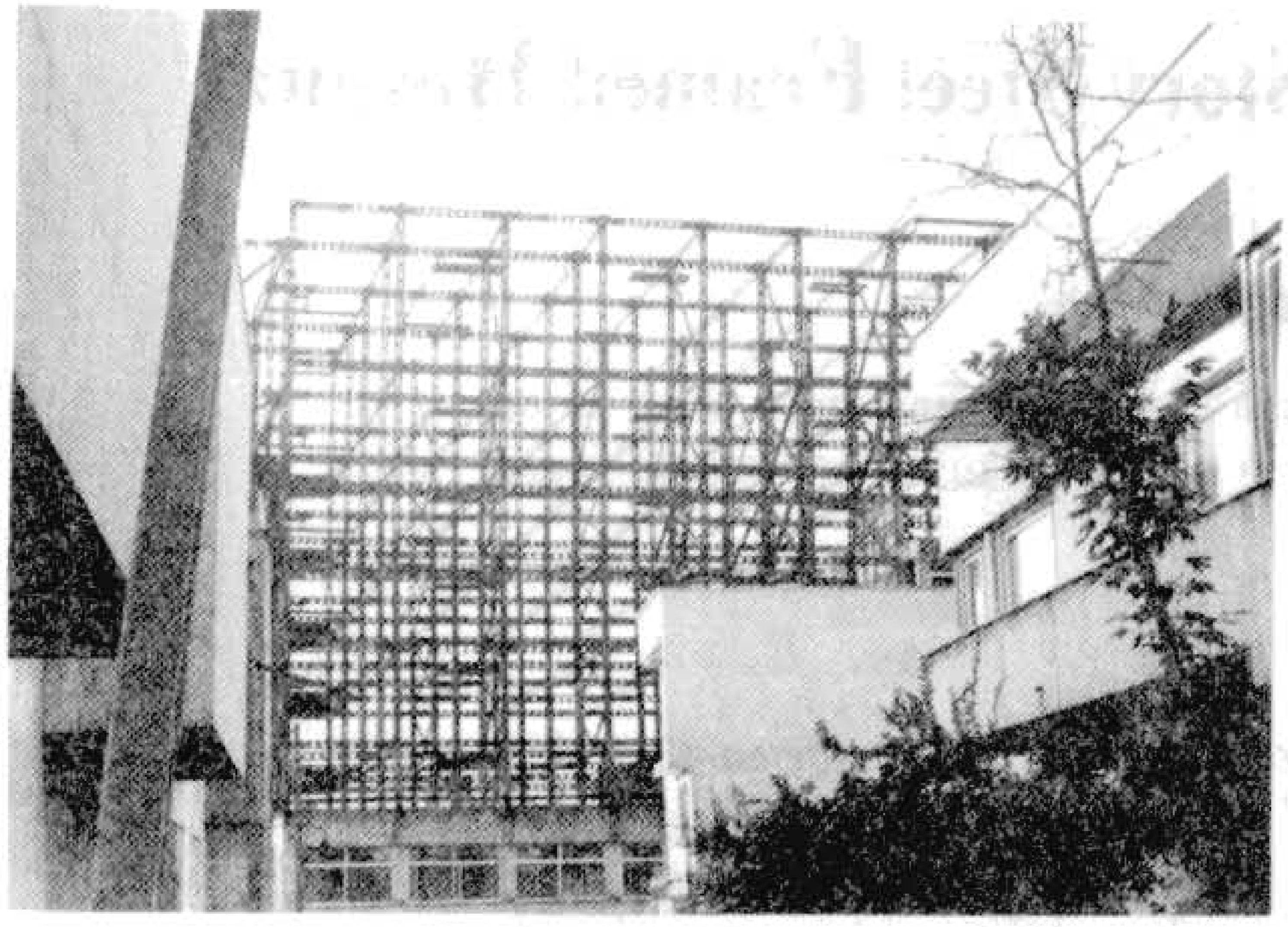

Figure 1. Photo of the existing structure.

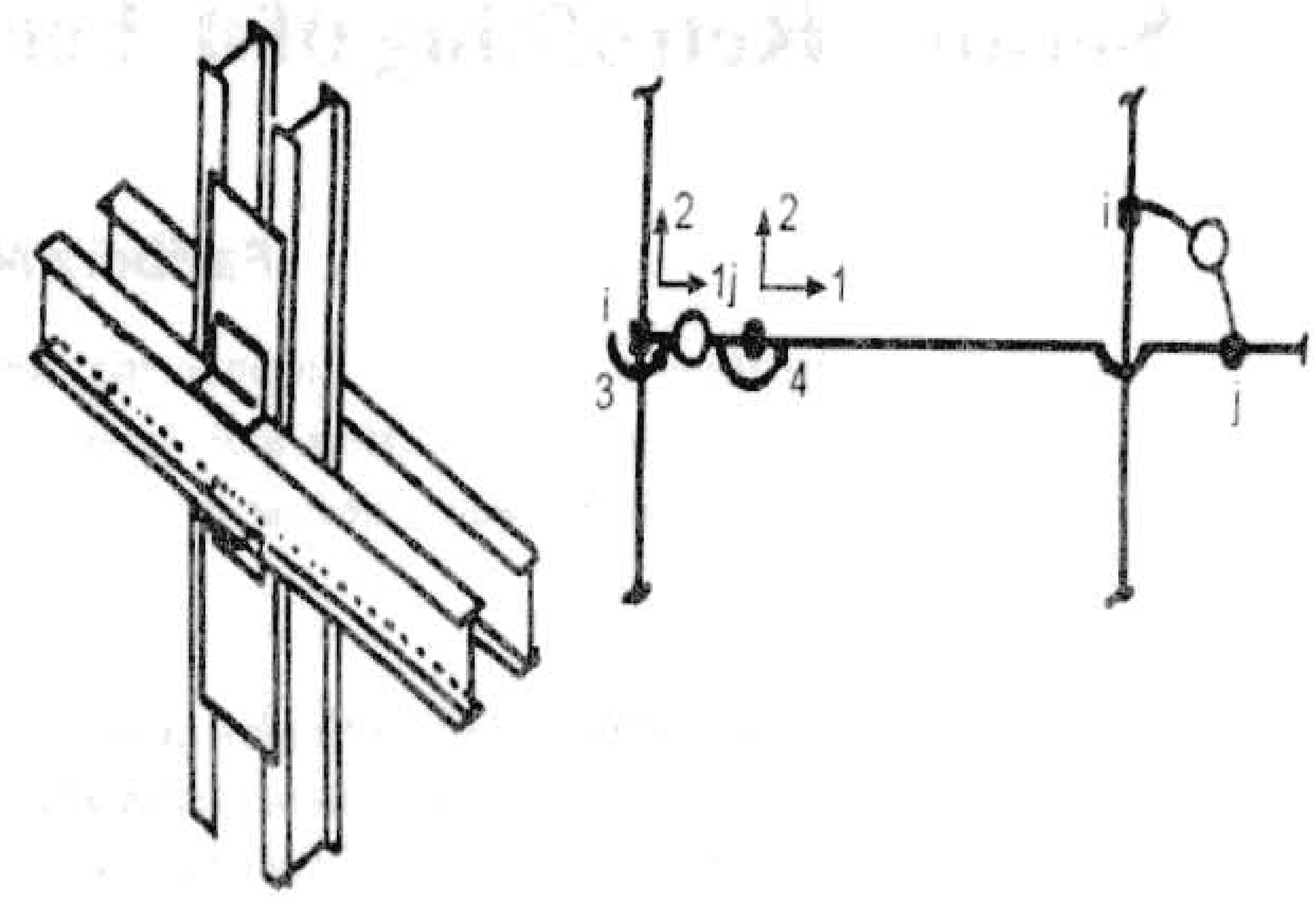

Figure 3. Typical beam-to-column connection.

floor height of 3 meters. Figure (2) shows a plan view of a typical floor. Its structural system, a simple steel frame with braces, consists of 5cm thick concrete topping on top of joist and block slabs. Beams are cast steel sections, 28cm deep and columns are 3INP 180+2PL 300x30 at ground floor in which dimensions vary from one story to another. Connections are simply supported and unique in terms of attachment to the columns. Schematic of a typical beam-to-column connection is shown in Figure (3). Foundation consists of single footings joined together by tie beams.

3. Building Inspection

Project began by inspection of the building. Luckily the old drawing were available. In order to make sure of the correct position of the braces in the existing structure, it was decided to open some sections of the walls and check the braces. Unfortunately, in some cases, it was observed that the original braces were not placed in the designated axis as indicated by the old drawings. Also inspection of the walls in first four floors indicated thick and heavy walls of approximately 42cm of heavy brick work. Welding quality was checked. Welds were not of the best quality, however acceptable. Foundation was exposed in few places and satisfactory results, meaning, placement in accordance to the original drawings were obtained. After inspecting the whole structure, a set of built drawings were drawn for the further analysis which was somewhat different than the original drawings especially at the locations of the braces.

4. Analytical Evaluation of the Existing Structure and Upgrade Schemes Considered

For this project, an analytical evaluation was carried out of the primary structural system considering the original ten story design. The purpose was to identify the "weak" members of the structural system and hence to justify the decisions relevant to the strengthening of the building.

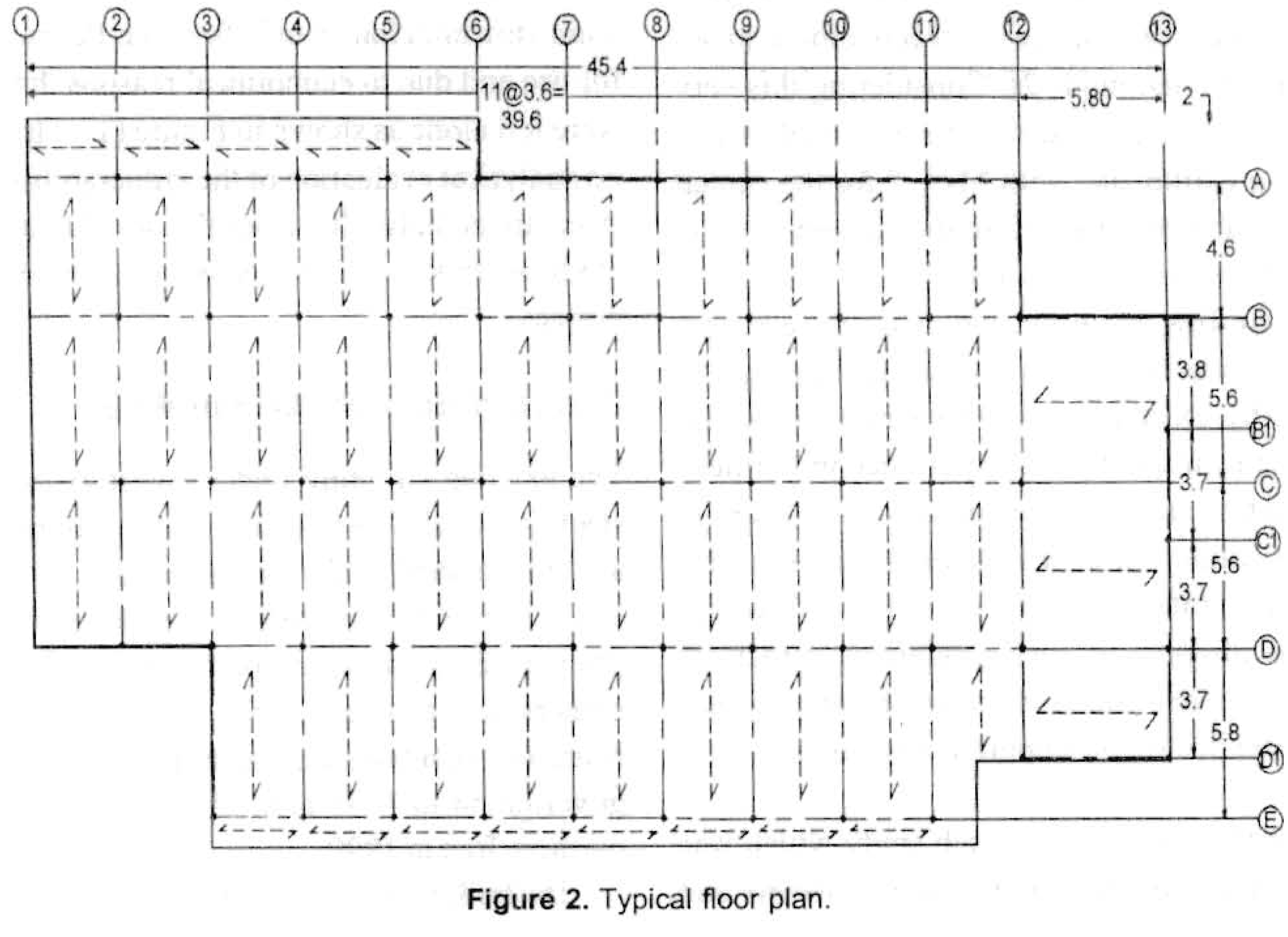

Figure 2. Typical floor plan.

All calculations led to the same conclusions that in order to seismically strengthen the building, both vertical and lateral force resistant systems needed upgrading. Low resistance was governed by the compression strength of the columns. Another deficiency of the structure was its excessive flexibility. Drifts under lateral forces exceeded considerably the values currently accepted by the seismic codes.

Different schemes were considered for retrofiting this particular building such as jacketing and turning simple frames into moment resisting frames, adding new R/C shear walls and new braces, and using infilled walls. In final analysis, considering economical, architectural and construction limitations, addition of new braces plus strengthening the old ones seemed more reasonable.

5. Analytical Evaluation of the Strengthened Structures and Details Used

In final analysis the structure was modeled as a 3D space frame with diagonal braces added. The effect of masonry infilled walls were ignored due to the fact that the top six floors had no walls. And to reduce the weight of the building, heavy brick walls of the first four floors were also going to be removed and replaced by light gypsum walls. This model was analyzed using loading conditions mandated by the Iranian Code 519, 1970.

Seismic loads were calculated using new Iranian standard [9]. For this building, static equivalent procedure was used governed by the following equations:

$$V = CW, \quad C = \frac{ABI}{R}$$

where W = Weight of the building; V = Base shear; C = Base shear coef.; A = Design base acceleration coef. = 0.35; B = Response coef. $= 2(T_0/T)^{2/3}$; T_0 = Period of soil = 0.3, 0.4, 0.5, 0.7; T = Fundamental natural period = $0.09H/D^{1/2}$; H = Height of the building = 35.4m; D = Dimension; D_x = 45.4m; D_y = 22.0m; I = Importance factor = 1.2; and R = Behavior coef. which is 7 for braced system.

Period of the building was calculated using conditions mandated by the static equivalent procedure resulting in T_x = 0.4728 and T_y = 0.6792sec. Dynamic analysis of structure resulted in periods of T_x = 0.92sec and T_y = 1.3sec. Using the actual period of the building, the calculated static periods were increased by a factor of 1.25. When static periods are less than dynamic periods, this increase is allowed. Using increased periods, base shear was calculated as V_x = 0.0925W and V_y = 0.0726W. Total weight of the building considering reduced wall weights was calculated at 9145 tons. In order to omit or minimize foundation strengthening, a series of new braces in both directions were introduced as given in Table (1). The new arrangements caused a reasonable distribution of forces amongst members and eliminated uplifting of foundations, therefore eliminating much expensive foundation work.

Table 1. New position of braces. First three stories are 2UNP 140, second three stories, 2UNP 120 and last four 2UNP 100.

Y-Direction		X-Direction	
1	[A-B & C-D]	A	[6-12]
4,6	[A-B & D-E]	B	[1-11, 12-13]
2	[C-D]	E	[3-11]
3	[A-B]		
5	[D-E]		
7, 8, 9	[A-B & D-E]		
10, 11			
13	[B-B1-C1-D-D1]		

Result of analysis indicated that beams on axis 12 needed strengthening which was achieved by doubling the existing beams. Columns were adequate, however load analysis of connections indicated the need for using stiffener plates as shown in Figure (4). Stiffener plates of 2x1/2PL 110x110x8mm were welded under seat angles. Cantilever portions of the building were controlled by using tension members as shown in Figure (5). Typical bracing systems used are also shown in Figures (6) and (7). Figure (8) also indicates the results of gravity and uplift force at

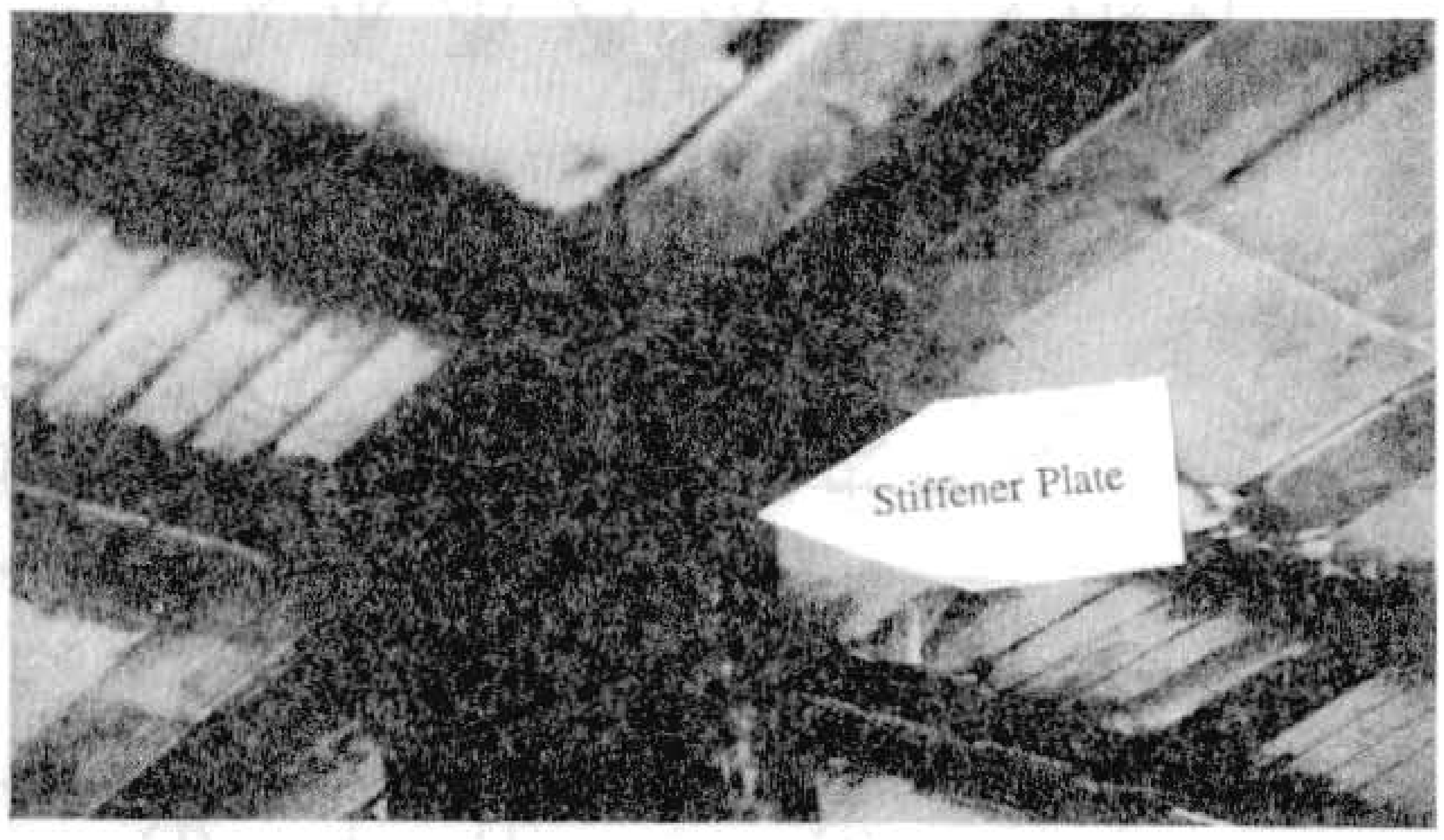

Figure 4. Stiffener plates for strengthening connections.

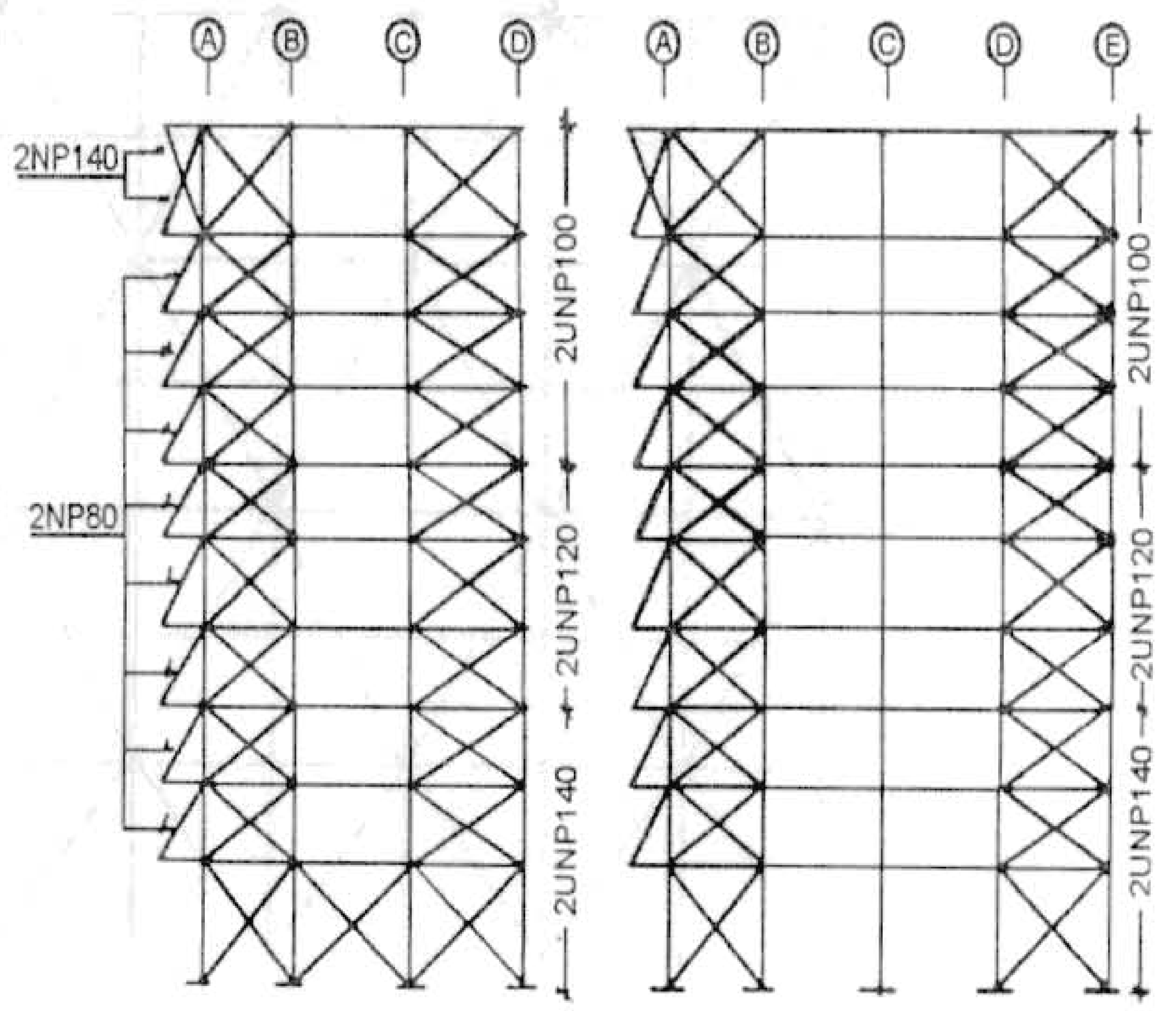

Figure 5. Bracing cantilever portions of the structure.

the foundation level. As seen from this figure, it is apparent that only at Axis-A-12 and Axis 1-D, there exists some minor uplift conditions that could be easily controlled. Figure (9) presents the behavior of the building after the strengthening which clearly shows the obtained satisfactory results in controlling the drift.

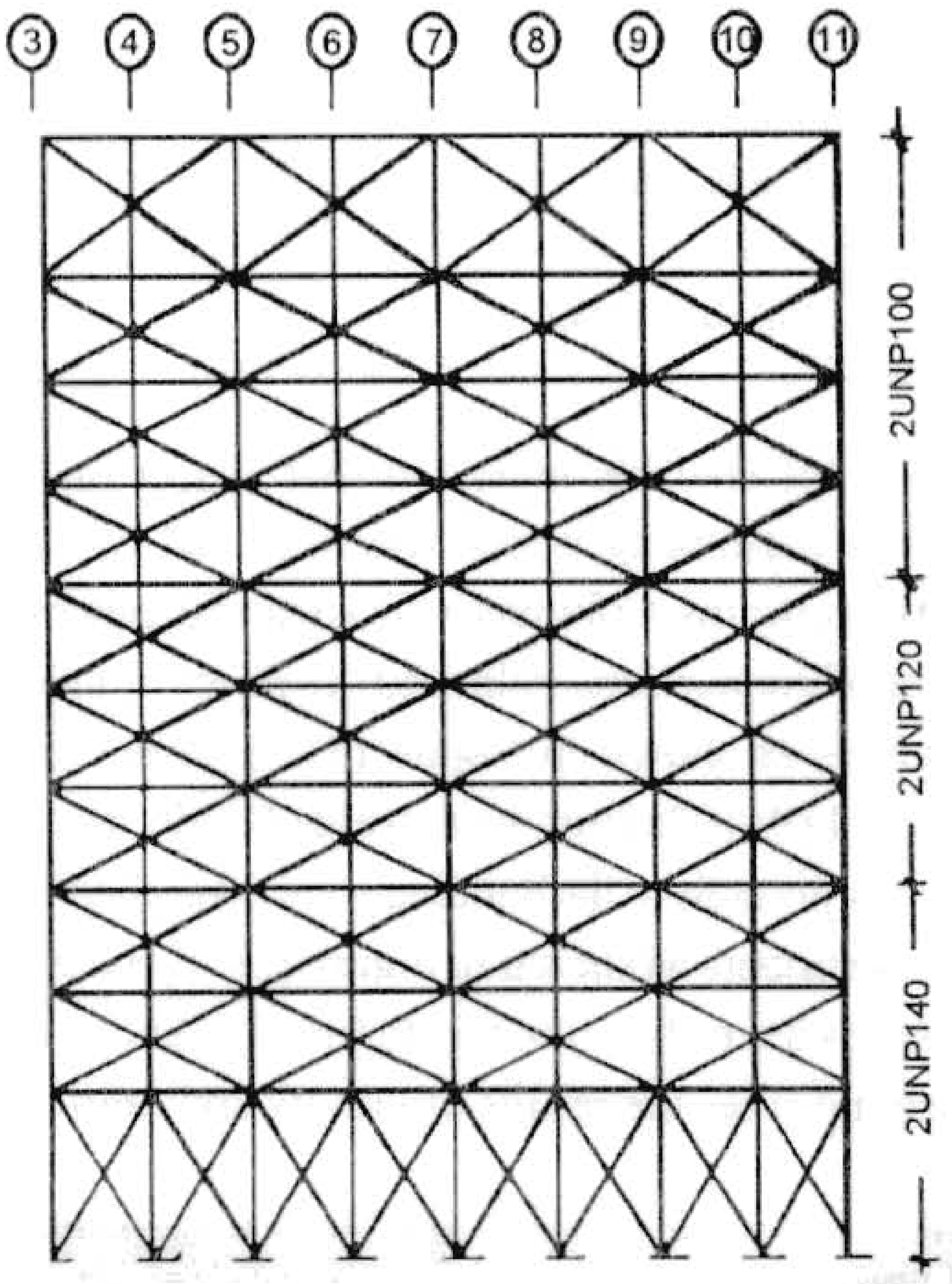

Figure 6. Typical new bracing used; Axis E.

6. Dynamic Behavior and Control of the Strengthened Structure

In order to check the behavior of the strengthened structure, it was decided to control the structure using time-history analysis. A non-linear dynamic analysis using DRAIN-2D was performed on the existing braced frames. Structure was strengthened with three types of bracing configurations which were similar to the ones in axis 1, 13, and B. In accordance to the Iranian Seismic Code, following two records were used:

Tabas; 1978-Iran, PGA = 0.933g

Naghan; 1977-Iran, PGA = 0.723g

For comparison purposes, 1940 El Centro Earthquake with PGA = 0.319 was also applied.

PGA's were scaled down to the recommended local EPA of 0.35g. The effects of all three records on all three frames were studied, but for presentation purposes, only the results obtained for frame at axis 1 with the Tabas record is presented herein.

Figures (10) and (11) indicate the displacement response of the top five stories. Maximum displacement of the structures occurs at about 15.35sec at the amount of 13.847cm which is below the allowable limit. Figure (11) shows the response with $P-\Delta$ effect. As seen, very little is contributed by the $P-\Delta$. It is due to the very stiff structure.

Figure (12) shows the maximum displacement of the

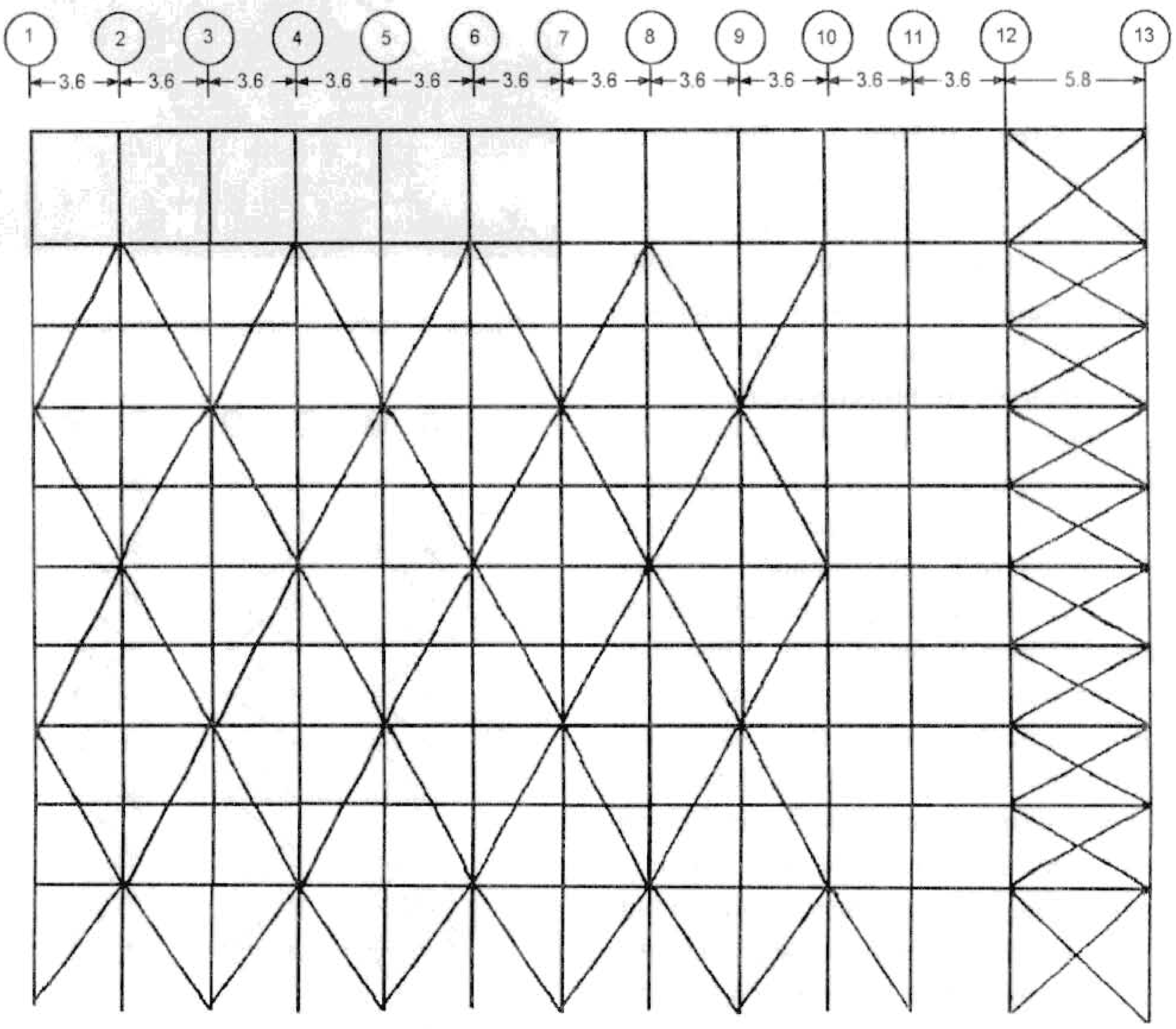

Figure 7. Typical new braces used; Axis B.

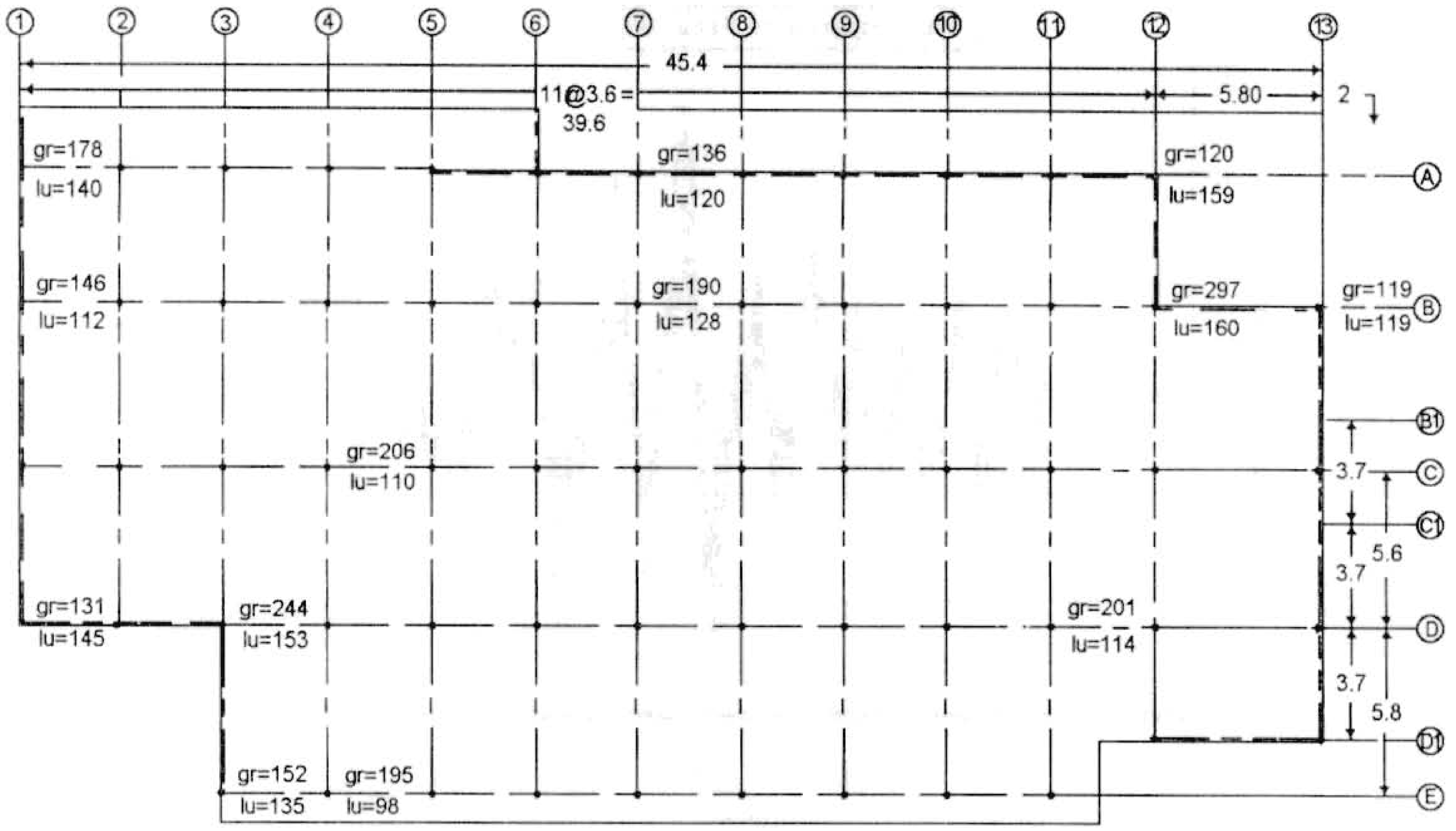

Figure 8. Gravity and liftup conditions at foundation level.

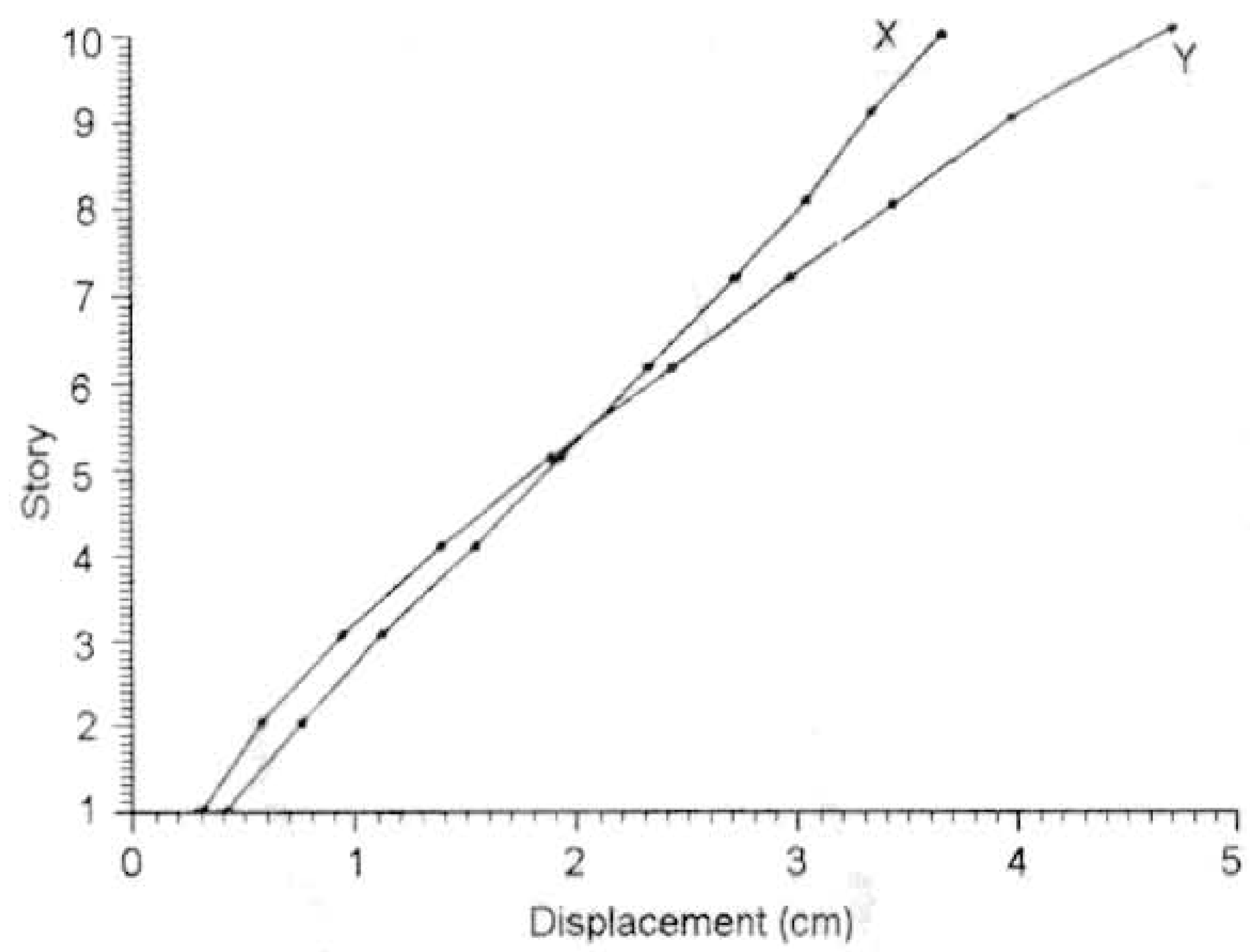

Figure 9. Behavior of structure after strengthening.

strengthened structures. As seen, interstory drift between floors 7 and 8 is a large amount about 3.3cm. This is due to the fact that at this level the column sizes have been changed and therefore caused reduction in inertia. Same thing is seen in 8 and 9th floors as well.

Ductility demand for braces were also studied as shown in Figure (13). This figure shows the maximum required ductility needed in axis A-B and C-D. Hysteretic behavior of the braces were also determined. An example is shown in Figure (14). As seen, this brace has the capacity of 18 tons in compression and 65 tons in tension. However, the energy absorption capacity is not very good as seen by the area under the curve. Column ductility

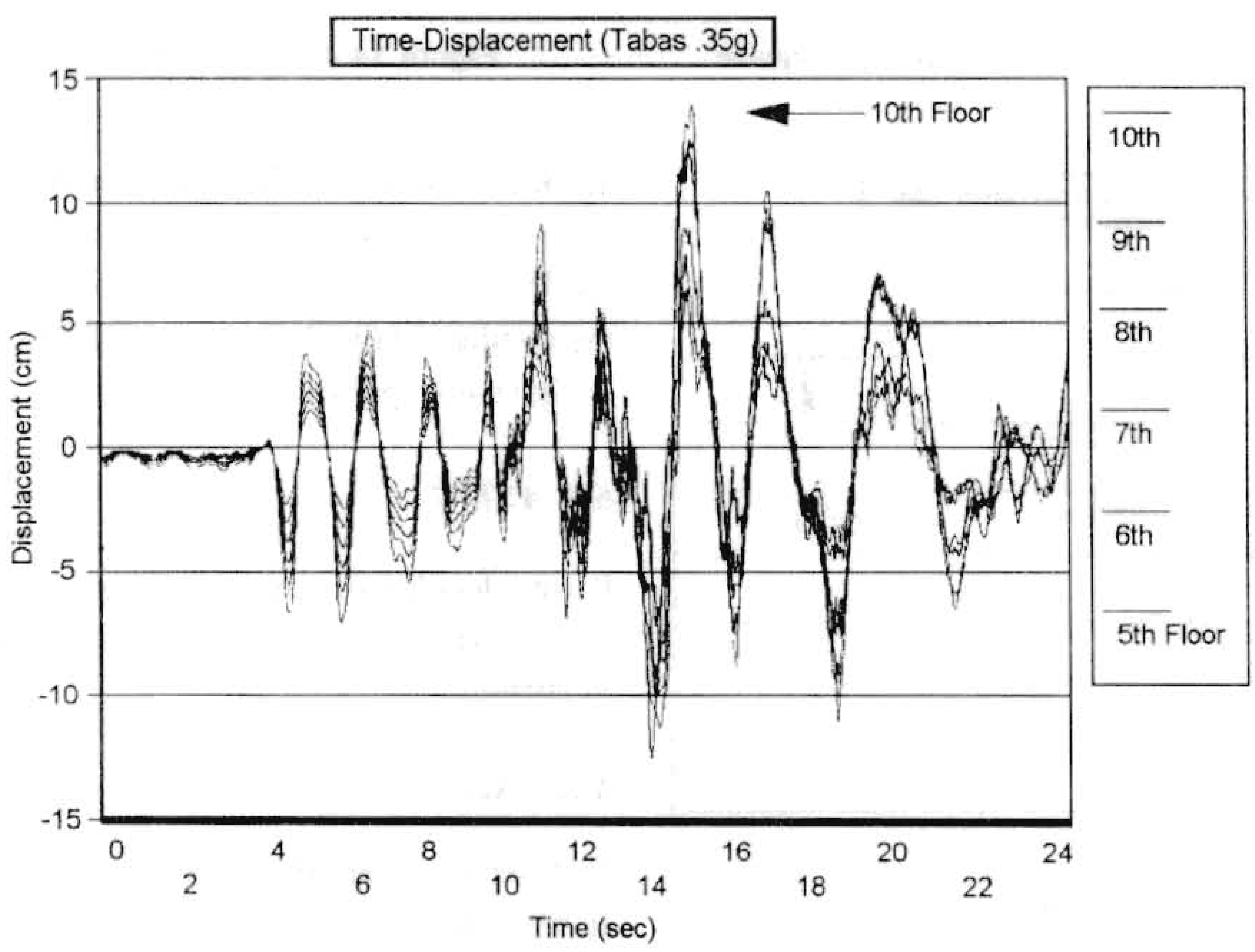

Figure 10. Time-displacement response of brace-1 in the top five stories.

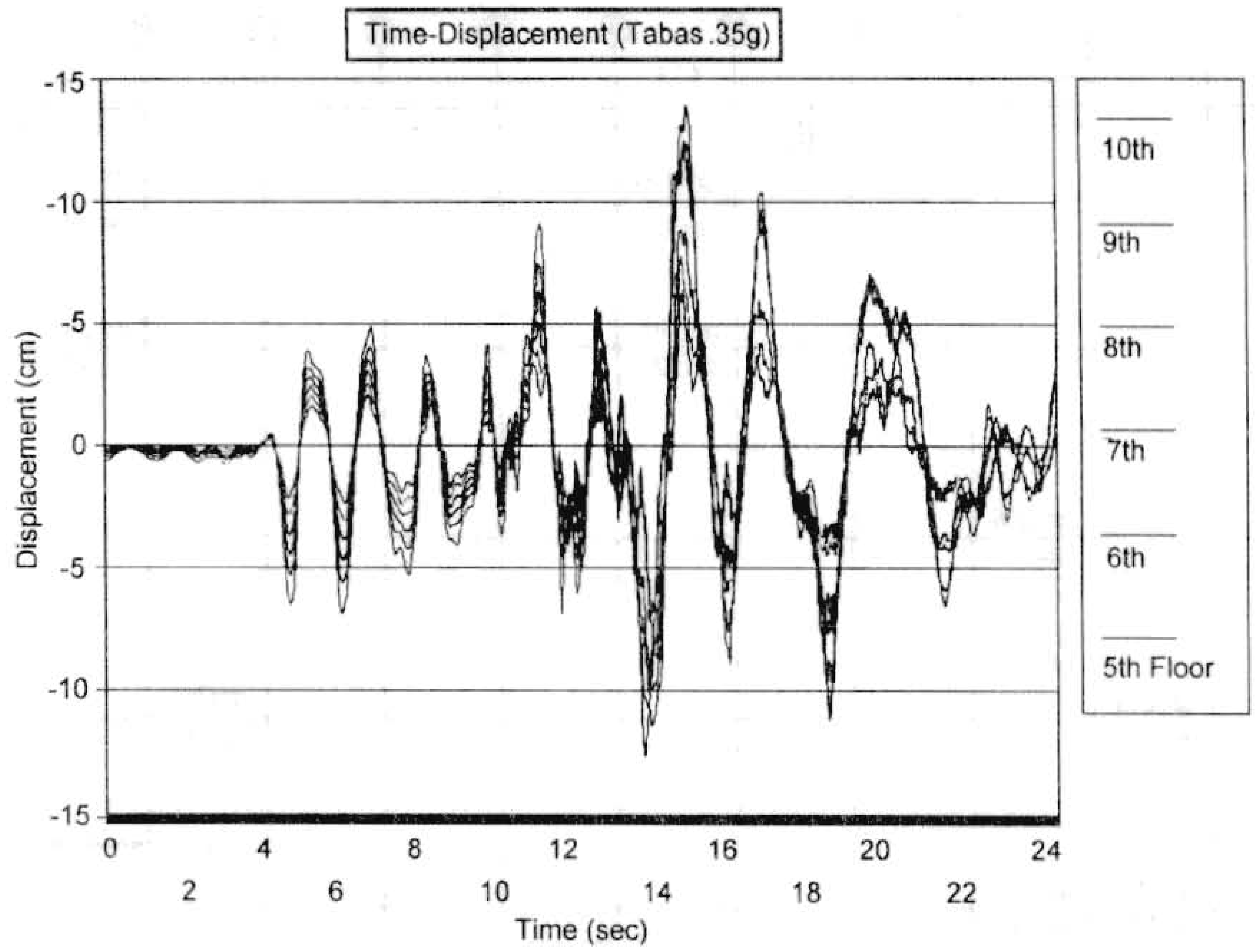

Figure 11. Time-displacement response of brace-1, including p-Δ effect in the top five stories.

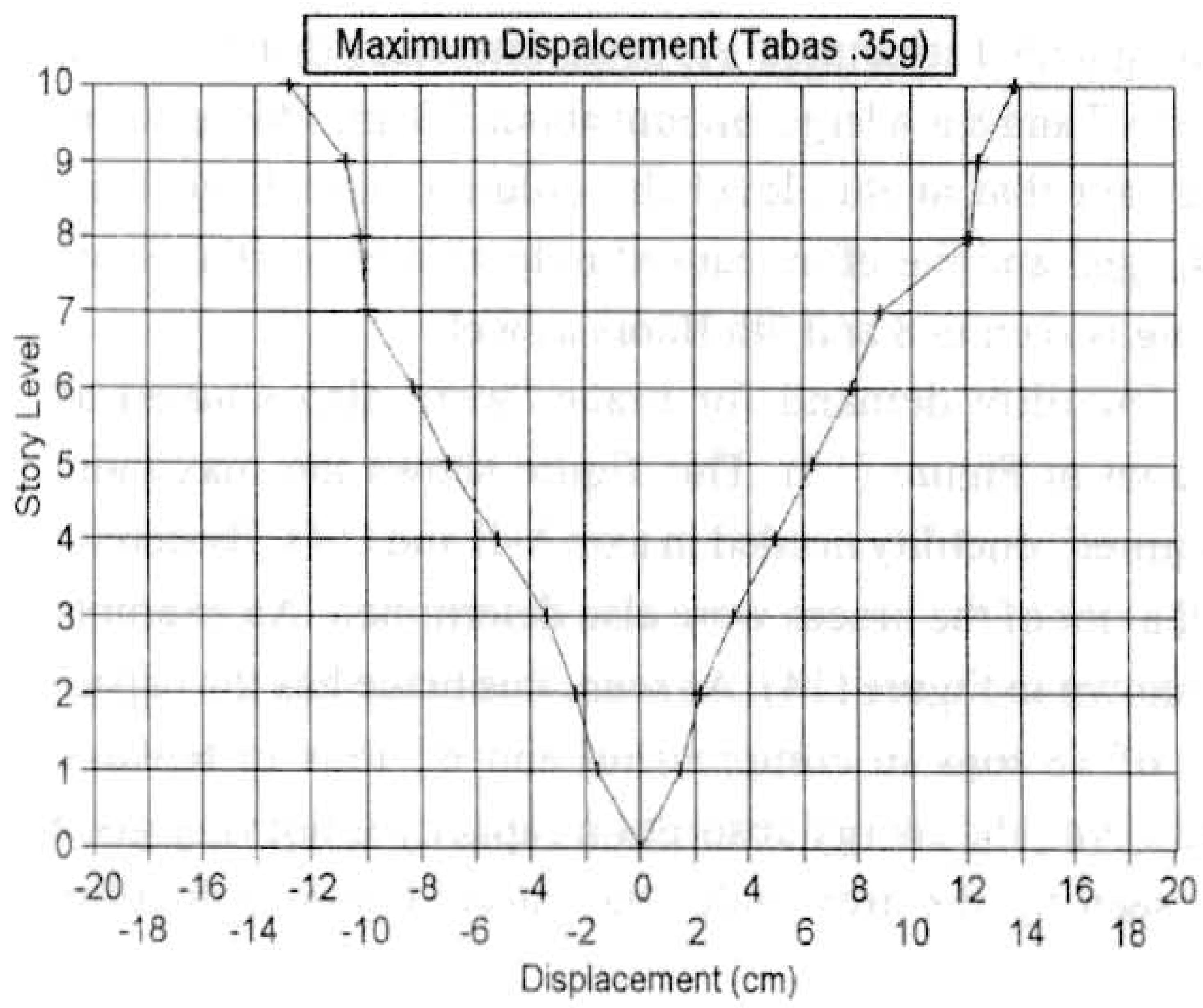

Figure 12. Maximum displacements for Tabas earthquake.

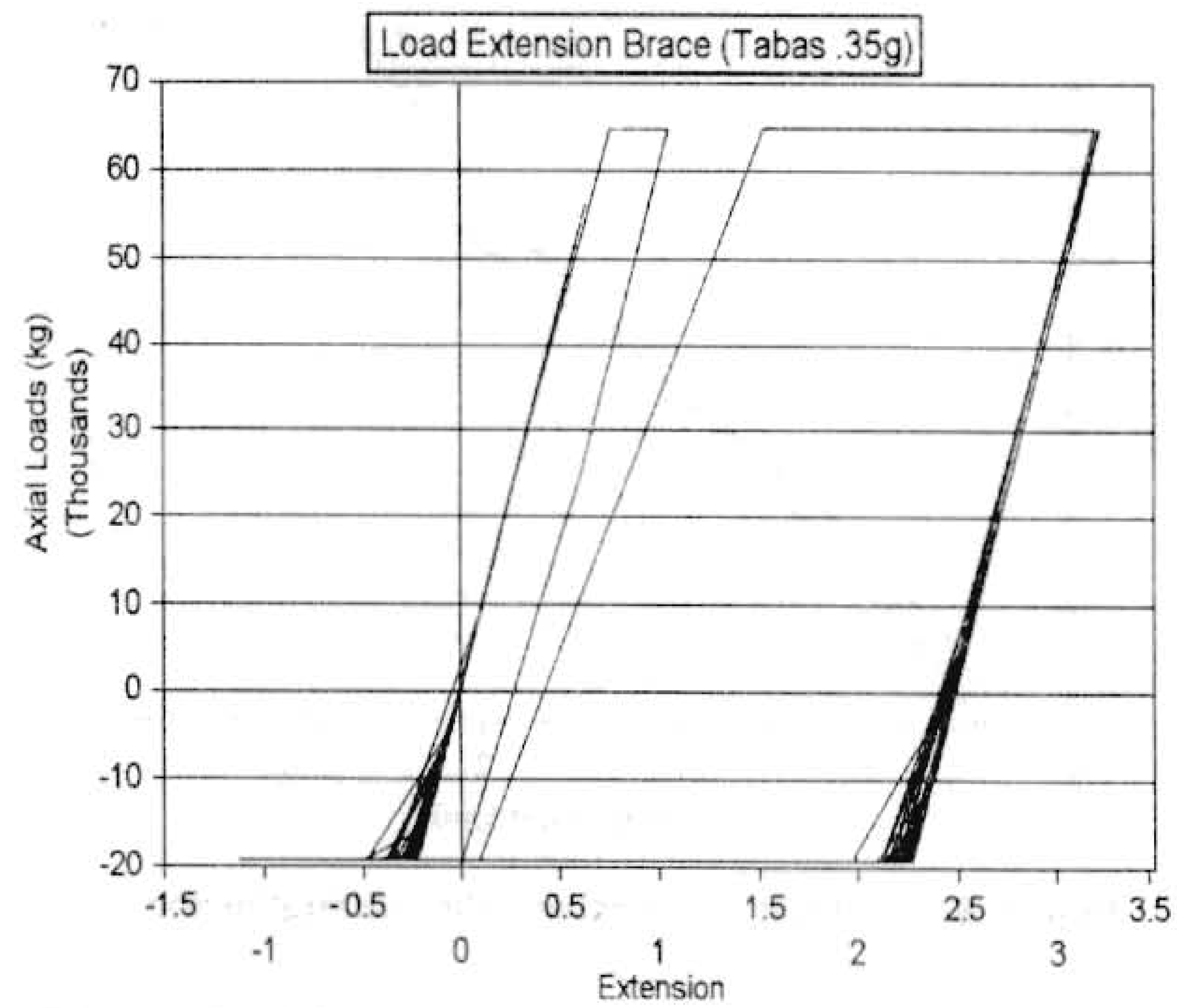

Figure 14. Hysteretic behavior of the braces.

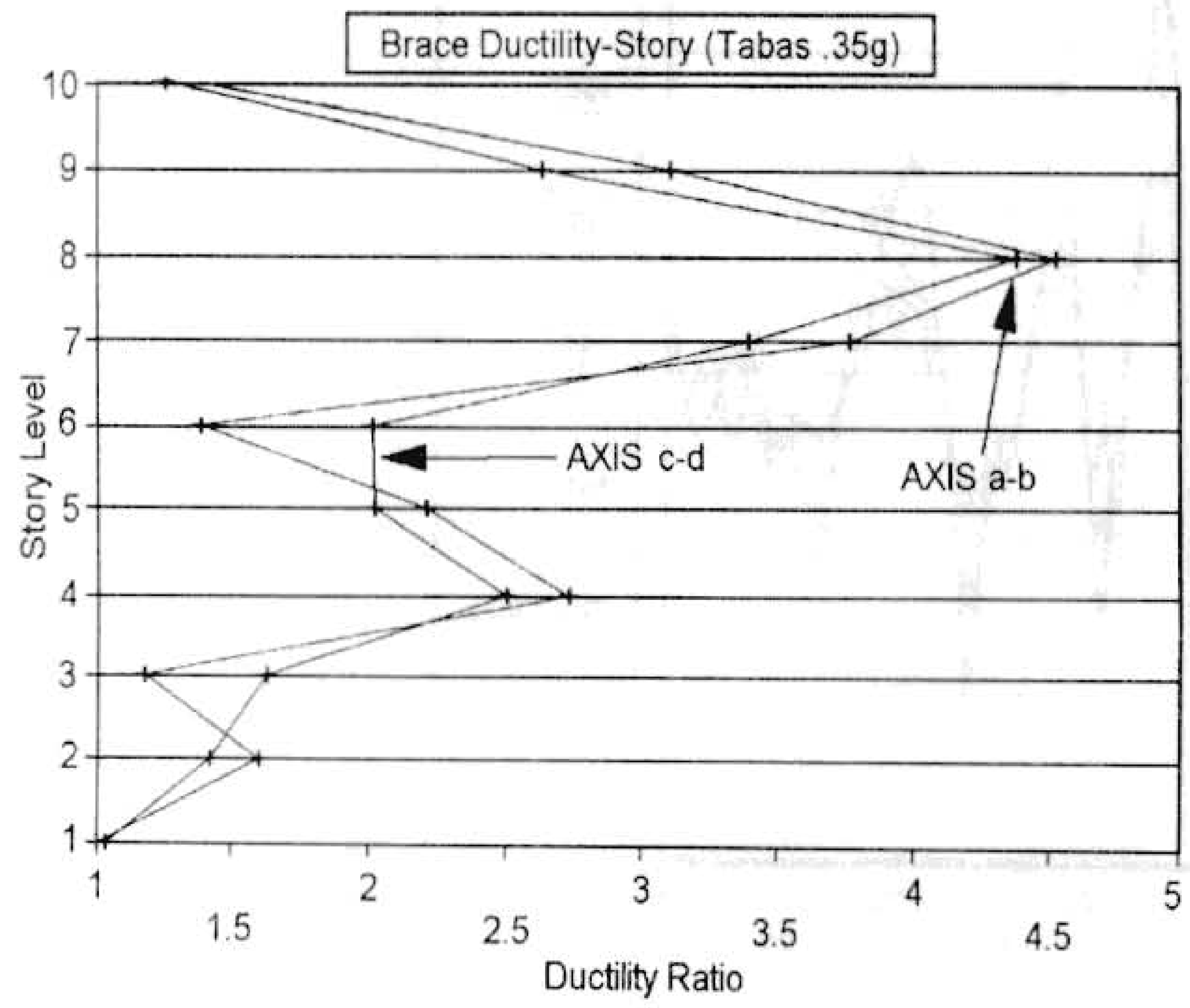

Figure 13. Ductility demand for braces.

ratios and ductility demands are also shown in Figures (15-17). As stated, this was done for all records which Table (2) indicates the summary of results obtained. Using the results obtained, some modifications on the upgrade design were applied as follows:

6.1. AXIS 1

It was decided to continue the plates at columns of top three stories in order to control the relatively large change in stiffness.

6.2. AXIS 13

In order to control the uplift forces at this bracing system, it was decided to change the single footings to the continuous footing as shown in Figure (18).

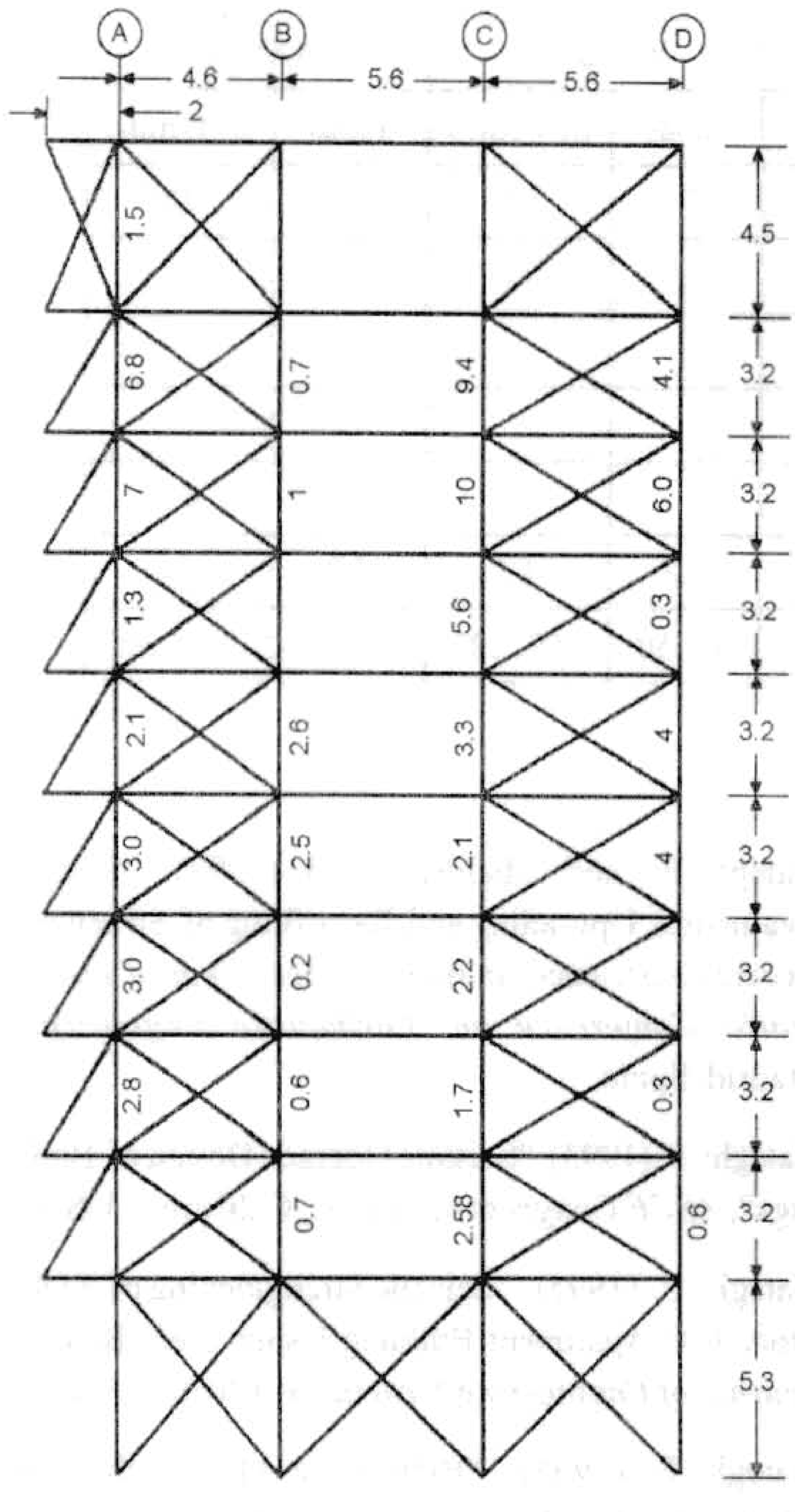

Figure 15. Column ductility ratio <AX1 Tabas>.

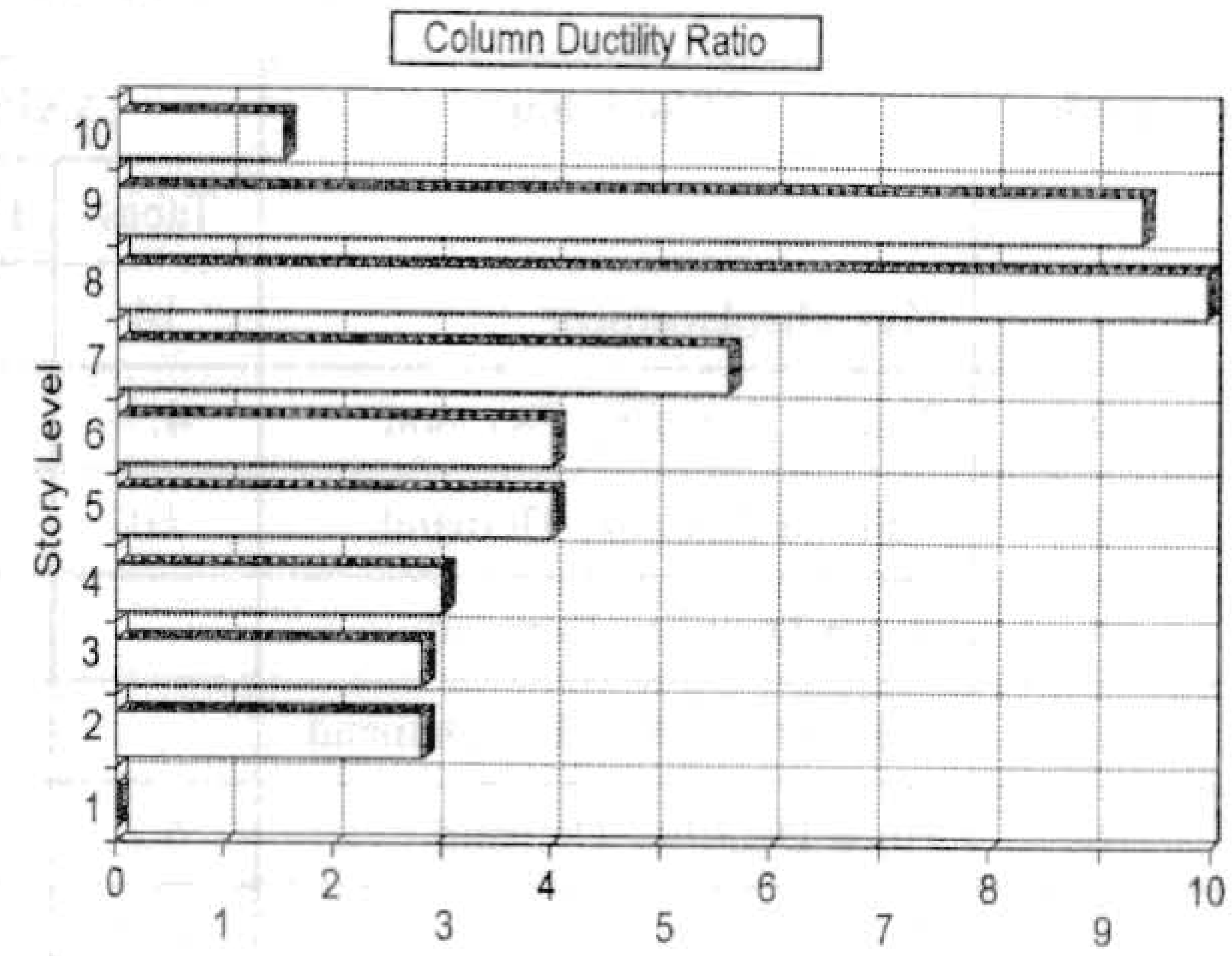

Figure 16. Ductility ratio <AX1 Tabas>.

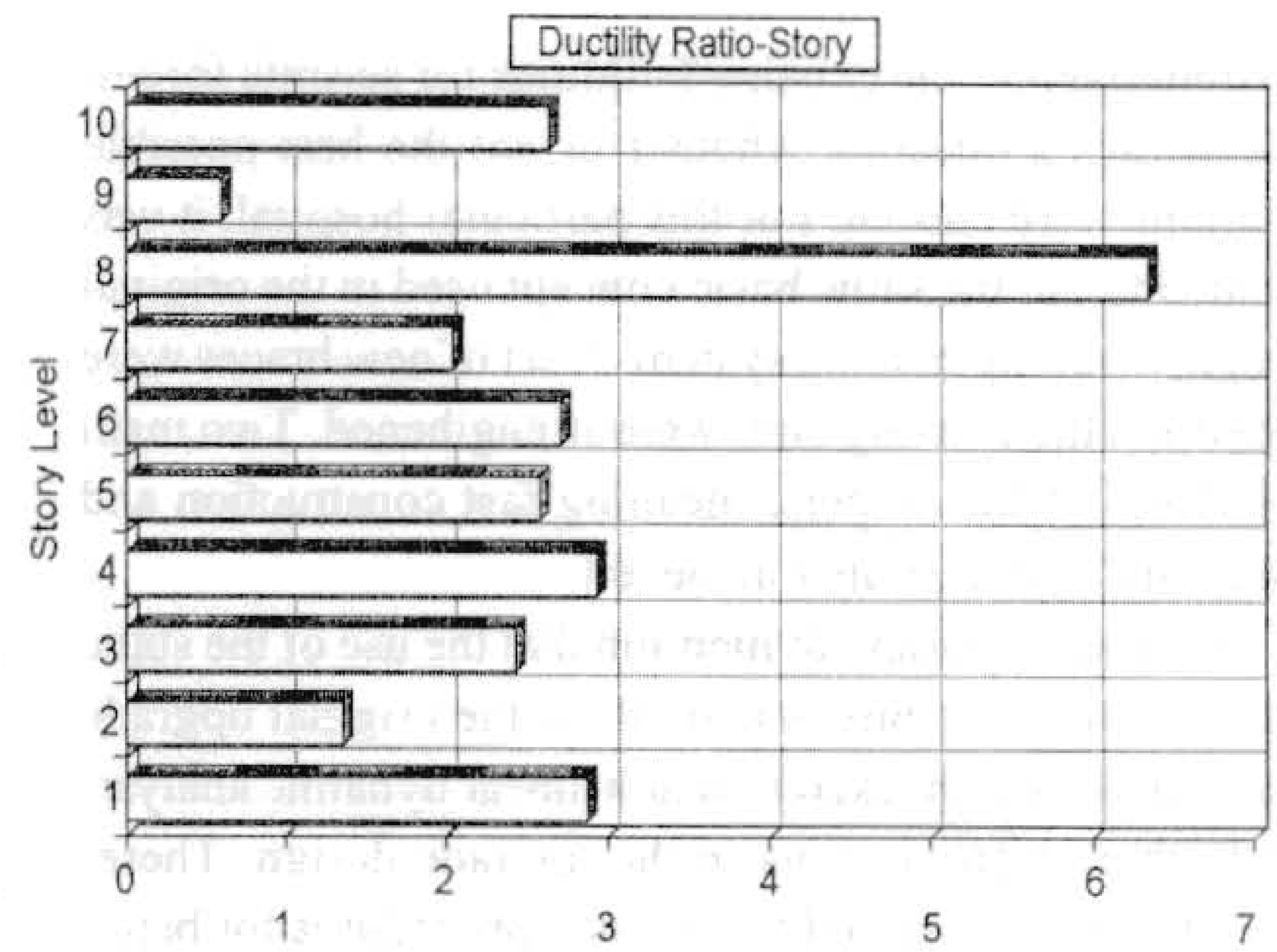

Figure 17. Ductility ratio <AX1 Tabas>.

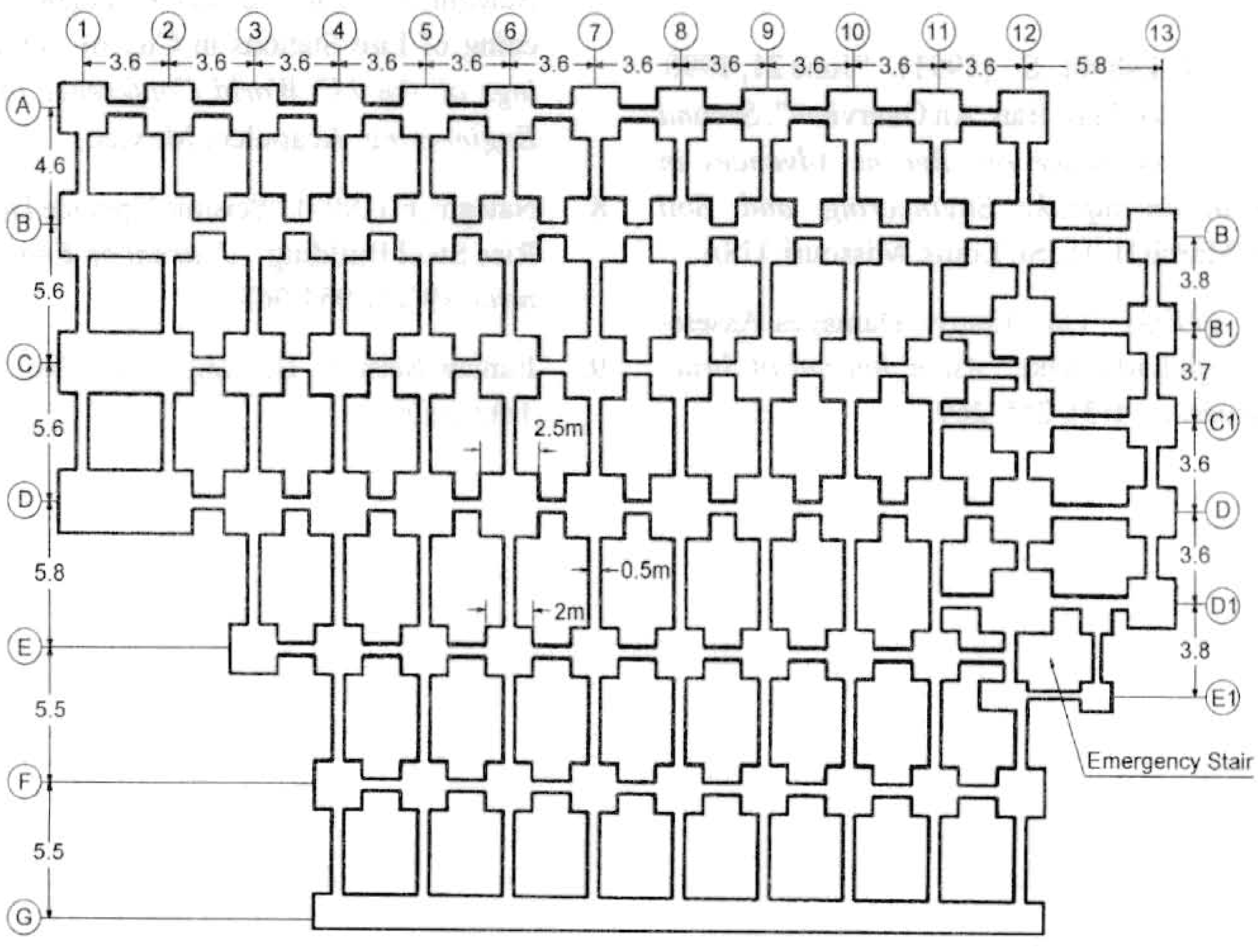

Figure 18. Existing foundation and changes made.

Table 2. Comparison of results obtained.

Item	Description	AXIS 1		AXIS 13		AXIS B	
		Tabas	El Centro	Tabas	El Centro	Tabas	El Centro
1	Max Displacement	14	20	9	21	13	22
2	Bracing Ductility Demand	4.5	3.4	2.4	5.8	2.7	4.7
3	Column Ductility Demand	10	7.2				
4	Beam Ductility Demand						
5	Connection Ductility Demand				4.2	2.4	4.5
6	Floor Ductility Demand	6.2	4.8	4.2	4.7	4.2	6.6
7	Uplift Force	32, 0	84, 60	60, 50	150, 120	33, 0	190, 36

7. Conclusions

In strengthening the existing buildings for seismic forces, it is always a question whether or not the best possible solutions were chosen. For this particular hospital, it was decided to use the same basic concept used in the original design, meaning bracing system. A set of new braces were added and the existing ones were strengthened. Two main objectives of the owners, meaning fast construction and minimum cost were also achieved.

It is also important to mention that the use of the static equilibrium procedure was used for the original upgrade design and later checked by a non-linear dynamic analysis for further improvements in the upgrade design. Therefore, it is recommended to use such procedures for better design purposes.

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