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Two-level vs. one-level seismic design of buildings

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ABSTRACT

This paper evaluates the merits and shortcomings of the one-level design described in the NEHRP Provisions and the two-level design specified in the Tri-Services Guidelines. In this study, a four-story reinforced concrete hospital building is designed according to the NEHRP Provisions and the Tri-Services Guidelines, separately. Then, the capacities of these two structures at the stages of first-yielding and collapse are evaluated and compared. The advantages and shortcomings of these two design approaches are discussed.

INTRODUCTION

It is not economical to design structures to resist seismic forces induced by large earthquakes within the elastic limits, because large earthquakes have a low probability of occurrence during the expected life of the structure. To deal effectively with the combination of extreme loading and low probability, the philosophy of seismic-resistant design in model building codes is that a building designed according to the seismic provisions will (1) to resist a moderate earthquake without structural damage, and (2) to resist a large earthquake without collapse so that life safety can be maintained. This philosophy leads naturally to the concept of two-level seismic design as prescribed in the Seismic Design Guidelines for Essential Buildings (Tri-Services Guidelines 1986). However, as the current design practice in the United States, most structures are designed for one level of earthquake that usually has a 10% probability of being exceeded in 50 years as specified in the NEHRP Provisions (1988). For both design approaches, structure members are allowed to go into inelastic range during a large earthquake. The NEHRP Provisions uses the

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response modification factor to reduce the base shear from an elastic force level to an inelastic force level. On the other hand, the Tri-Services Guidelines uses the inelastic demand ratios applied to individual members. Thus, even though the design philosophy in the NEHRP provisions and the Tri-Services Guidelines is the same, the approaches used in these two documents are quite different. This paper evaluates the merits and limitations of the one-level seismic design as described in the NEHRP Provisions and the two-level seismic design as specified in the Tri-Services Guidelines. In this study, a four-story reinforced concrete hospital building is designed according to the 1988 NEHRP Provisions and the 1986 Tri-Services Guidelines, separately. Then, the capacities at the stages of first yielding and collapse are evaluated and compared.

DESIGN USING THE NEHRP PROVISIONS

A four-story hospital building is assumed to be located in a moderate seismic zone with the coefficient of effective peak-velocity related acceleration A_V equal to 0.2. According to the NEHRP Provisions, the Seismic Hazard Exposure Group for a hospital building is III. From these two conditions, the Seismic Performance Category is determined as E. Thus, the special moment-resisting (SMR) frame is required to be used to resist gravity loads and earthquake forces. A typical floor plan and an elevation of the four-story hospital building are shown in Figs. 1 and 2, respectively. The beam size is 12 inches by 20 inches and column size is 14 inches by 14 inches throughout the building. This study focuses on the design of a typical interior frame in the north-south direction. The seismic design base shear V is calculated using the following formula:

$$V = C_S W = \frac{1.2\, A_V S}{R T^{2/3}} W \tag{1}$$

C_S is the seismic base shear coefficient and has $2.5\, A_a/R$ as the upper bound. Since A_V is equal to 0.2, the seismic coefficient of peak acceleration A_a is also taken as 0.2. The soil condition of the site is assumed to be classified as S_2; thus the soil factor S is equal to 1.2. For an SMR frame, the response modification factor R is set equal to 8 in accordance with the NEHRP Provisions. The fundamental period of the building is determined as 0.547 second and the total dead load W of the building is 3394.9 kips (Ch'ng 1990). From Eq. 1, the design base shear V is calculated as 182.70 kips. Using the member forces caused by the base shear and gravity loads, the SMR frame is designed according to ACI code 318-89 (1989) as shown in Ch'ng (1990). The arrangement of flexural reinforcement is shown in Fig. 3.

DESIGN USING THE TRI-SERVICES GUIDELINES

According to the Tri-Services Guidelines, an essential building shall be designed to resist the following two levels of earthquakes EQ-I and EQ-II. EQ-I is the maximum probable earthquake, likely to occur during the life of a building, and is defined as an earthquake with a 50% probability of being exceeded in 50 years (a 72-year earthquake). On the other hand, EQ-II is the maximum theoretical earthquake that can occur at a site, but has a low probability of occurrence during the life of a building. EQ-II is defined as an earthquake with a 10% probability of being exceeded in 100 years (a 950-year earthquake). The four-story hospital building to be designed according to the Tri-Services Guidelines has the same lay-out as shown in Figs. 1 and 2. The size of beams is 15 inches by 24 inches and columns is 20 inches by 20 inches throughout the building. The design response spectrum is defined as

$$S_a = \frac{1.22\, A_v\, S_i\, D_f}{T} \tag{2}$$

and S_a has the following upper bound:

$$S_a \leq 2.5\, A_a \tag{3}$$

For the building located in a seismic zone with A_v and A_a equal to 0.2 as shown in the ATC3-06 (1984) contour map, A_v and A_a of EQ-I are equal to 0.08. For the S_2 soil condition, the soil factor S_i is equal to 1.2. According to the Tri-Services Guidelines, the damping adjustment factor D_f is 1.00 for the reinforced concrete structure with a damping ratio of 5%. Substituting A_v, S_i, D_f, and A_a into Eqs. 2 and 3, the design response spectrum of EQ-I is determined. In accordance with the Tri-Services Guidelines, an essential building such as a hospital needs to be analyzed dynamically even though the building is a regular building. In this study, the four-story hospital building is idealized as a multi-degree-of-freedom stick model with a fixed base. For low-rise buildings, say up to about 5 stories, the modal analysis can generally be performed by using only the fundamental mode. Thus, the story lateral forces of the fundamental mode are used to obtain the member forces. From the free vibration analysis, the fundamental period, the corresponding mode shapes and the modal participation factor PF_{j1} at level j can be determined. The story lateral forces at level j are computed as

$$F_j = PF_{j1}\, S_{a1}\, w_j \tag{4}$$

where S_{a1} is the spectral acceleration at the fundamental period (0.427 sec) and is equal to 0.2 g as determined from the EQ-I response spectrum. w_j is the weight assigned at the level j. From these story lateral forces, the member forces caused by EQ-I can be determined and combined with those from the gravity loads. The combined member forces are used to design structural members in accordance with the design procedure specified in the ACI code 318-89.

In level-two design, the structure is analyzed to determine its ability to resist the forces and deformations caused by EQ-II. For A_v and A_a equal to 0.2 in the ATC3-06 contour map, A_v and A_a of EQ-II are equal to 0.25. For a reinforced concrete structure, a 10% damping ratio is used for post-elastic analyses; thus, the damping adjustment factor D_f is equal to 0.80. Using Eqs. 2 and 3, the design response spectrum of EQ-II is determined. Structural member forces are calculated by means of modal analysis using EQ-II response spectrum. The member forces from gravity loads and EQ-II computed by means of the elastic analysis procedure are allowed to be larger than the design ultimate capacity. However, the inelastic demand ratio (IDR) is implemented to control the overstress within an acceptable limit for each member. Since the shear capacity is greater than the flexural capacity for a special moment-resisting frame, the flexural capacities of beams and columns are used to represent the capacities of structural members. Hence, IDR is defined as the ratio of the elastic demand moment M_D to the design ultimate moment capacity M_C. For essential buildings, the Tri-Services Guidelines specifies that the IDR of beams shall not be greater than 2, and the IDR of columns shall not be greater than 1.25. The smaller IDR of column is to ensure strong column-weak beam behavior, that is, to ensure that hinging will take place in beam rather than in column. The maximum IDRs of all the members are determined and shown in Fig. 4. It can be seen that IDRs of several columns and beams exceed the allowable limit. Thus, the deficiencies must be corrected by redesign of these critical members. By a trial-and-error process, IDRs of all members are within the allowable limits set forth in the Tri-Services Guidelines. Thus, the frame structure has the capacity to resist both EQ-I and EQ-II. The flexural reinforcement arrangement of beams and columns is shown in Fig. 5.

LIMIT STATES AND STRUCTURAL CAPACITY

Two types of limit states are used in this study: first yielding and collapse of structure. For a frame structure, the first yielding is defined as the formation of first plastic hinge anywhere in the structure. If a structure subject to earthquakes does not reach the first yielding, then the structural response remains in the elastic range and the structure does not sustain any structural damage. The first-yielding limit state can be considered as a serviceability limit state. The collapse of structure is defined as the formation of a failure mechanism. Thus, the collapse limit state represents a strength limit state.

In this paper, the capacity spectrum method proposed by Freeman (1978) is used to construct the capacity curves for these two frame structures. The capacity curve displays seismic capacity in terms of the spectral acceleration versus the fundamental period. The capacity curves of the two frame structures designed according to the NEHRP Provisions and the Tri-Services Guidelines, are plotted in Fig. 6. It can be seen that these two curves are not in the same period range. To compare the capacities of these two structures, it is desirable to convert the spectral accelerations at the first yielding and collapse to the corresponding peak ground acceleration (PGA). To determine the PGA value corresponding to the first yielding of the frame, a response spectrum with a 5% damping ratio as specified in the NEHRP Provisions or the Tri-Services Guidelines is passing through the first-yielding point of the capacity curves. Then, the PGA values are obtained from the upper bound of the spectral accelerations divided by 2.5. The PGA values corresponding to the collapse of structures are determined in a similar manner. These PGA values are summarized in Table 1.

Table 1. Structural capacities in terms of PGA

Stage	NEHRP Provisions	Tri-Services Guidelines
First yielding	0.10 g	0.15 g
Collapse	0.31 g	0.47 g

CONCLUSIONS

From the design and evaluation of the two frame structures designed according to the 1988 NEHRP Provisions and the 1986 Tri-Services Guidelines, respectively, the advantages and short-comings of these two different approaches are discussed as follows:

1. The one-level seismic design, such as the approach used in the NEHRP Provisions, is easy to use by designers. However, the design earthquake as currently defined represents neither a

moderate earthquake nor a large earthquake. In addition, the designer is not required to evaluate the overall capacity of structure to resist a large earthquake. Therefore, there is some uncertainty regarding to the building performance in the event of a large earthquake.

2. The two-level seismic design, the approach employed in the Tri-Services Guidelines, follows naturally the current earthquake-resistant design philosophy. Two-level seismic design requires designers to evaluate the building performance and to discover possible weak spots that are susceptible to large earthquakes. Thus, the two-level seismic design can give designers confidence in the design of structures and can reduce the risk of catastrophic damage of buildings in the event of a large earthquake.

3. The building frame designed according to the Tri-Services Guidelines is much stronger than the one designed according to the NEHRP Provisions. The essential building designed according to the NEHRP Provisions seems to be unconservative based on the potential seismic hazard at the site.

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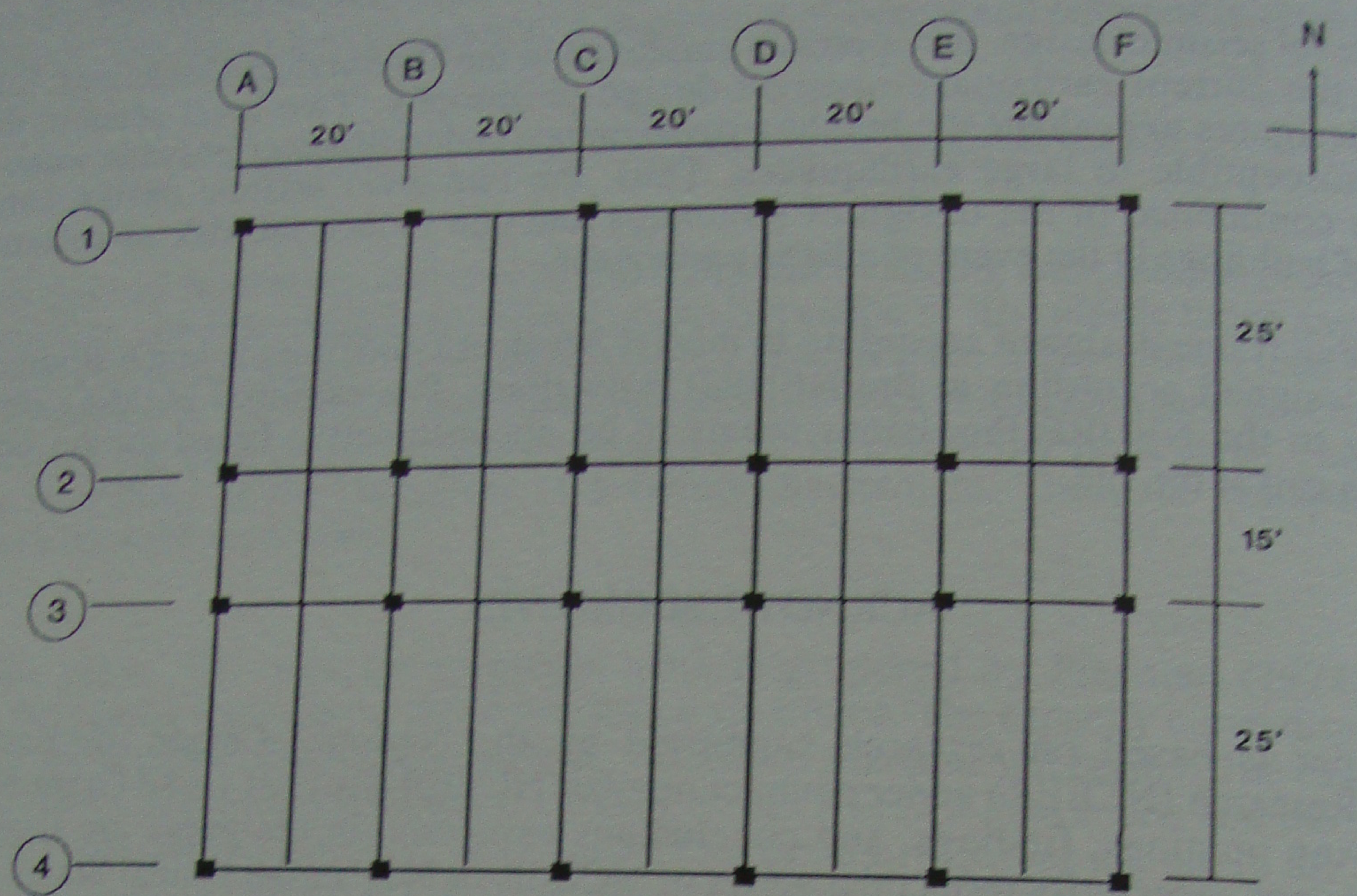

Figure 1. Typical floor plan of the four-story hospital building

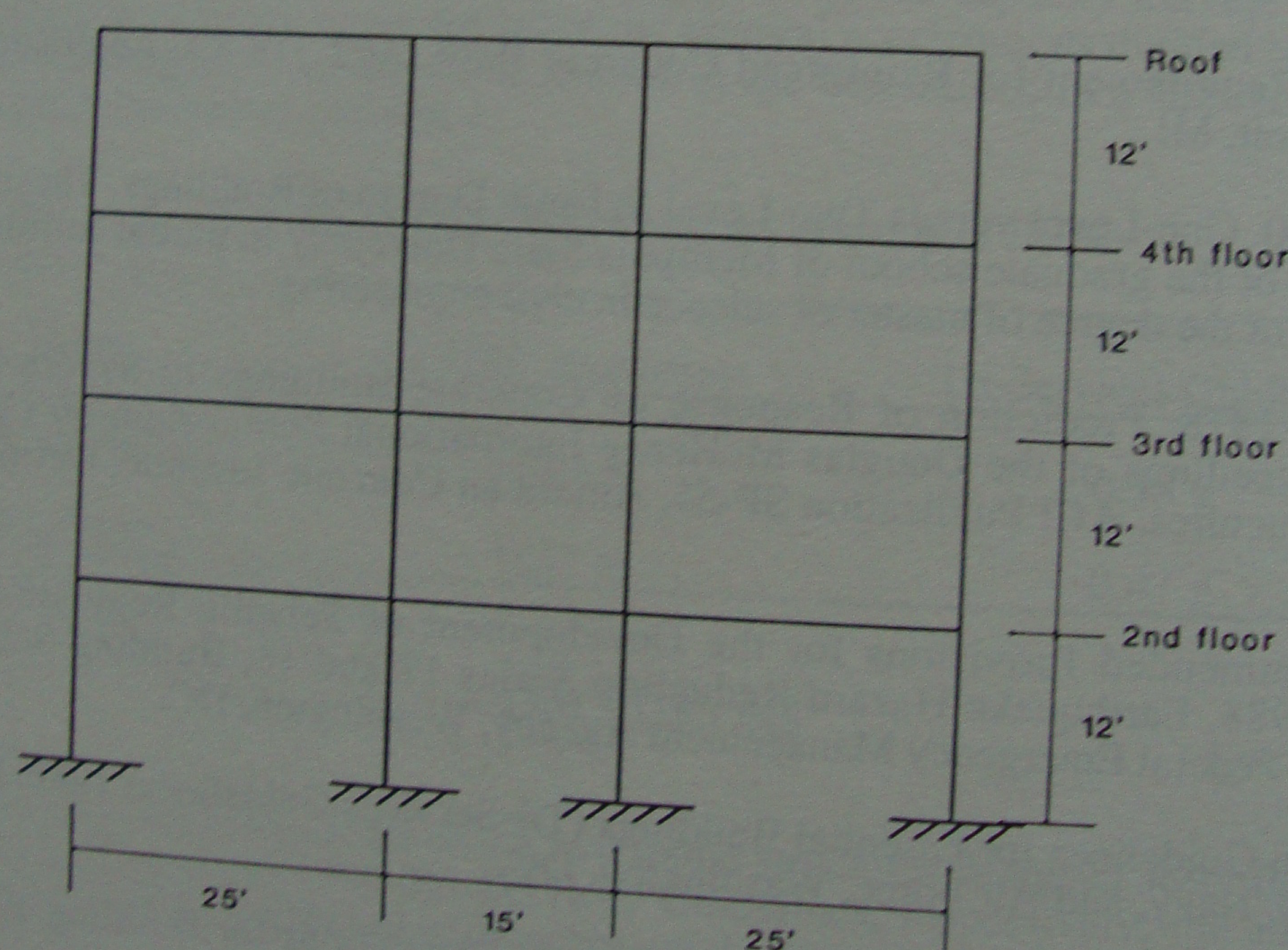

Figure 2. Elevation of an interior frame (N-S direction)

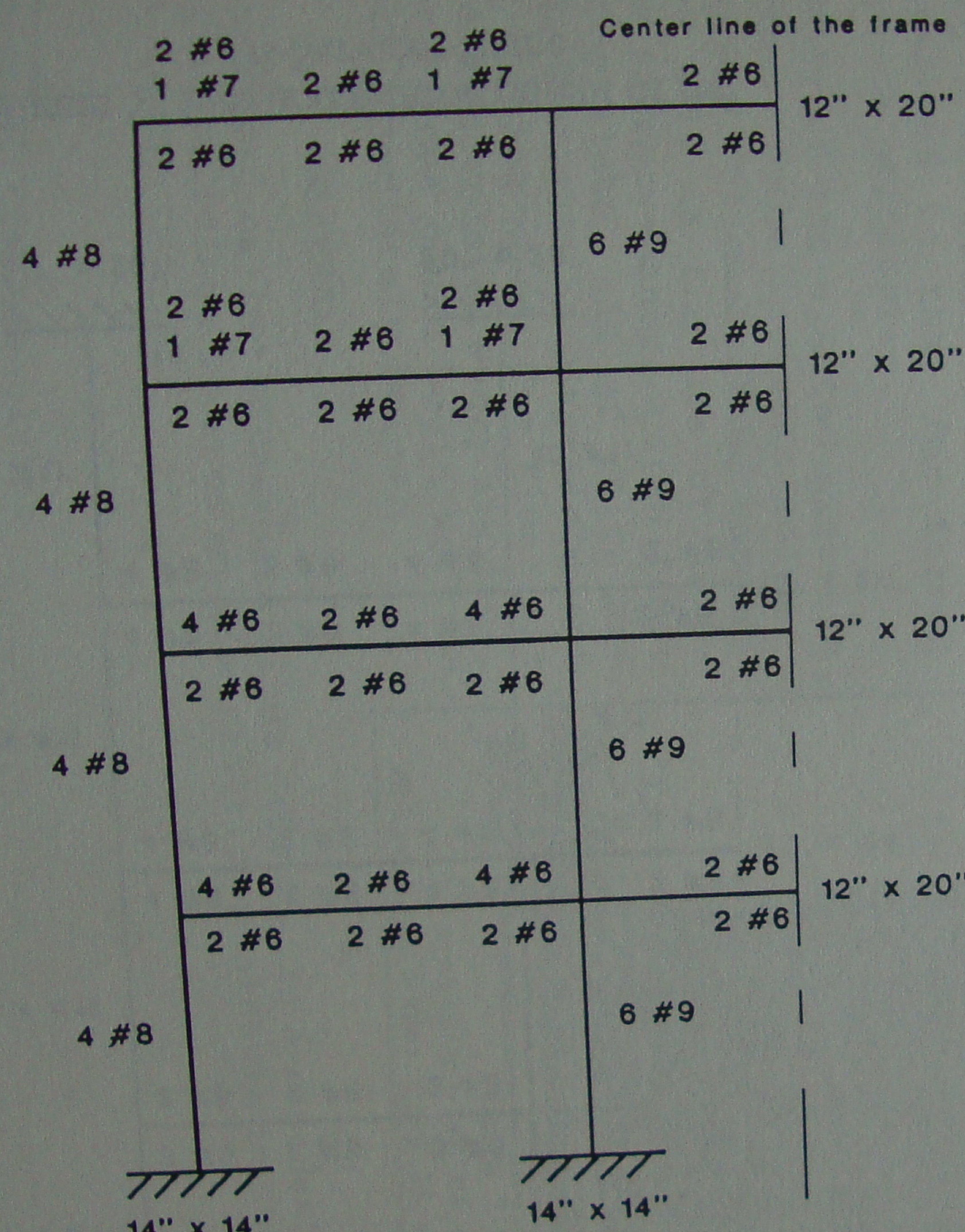

Figure 3. Flexural reinforcement of the NEHRP frame

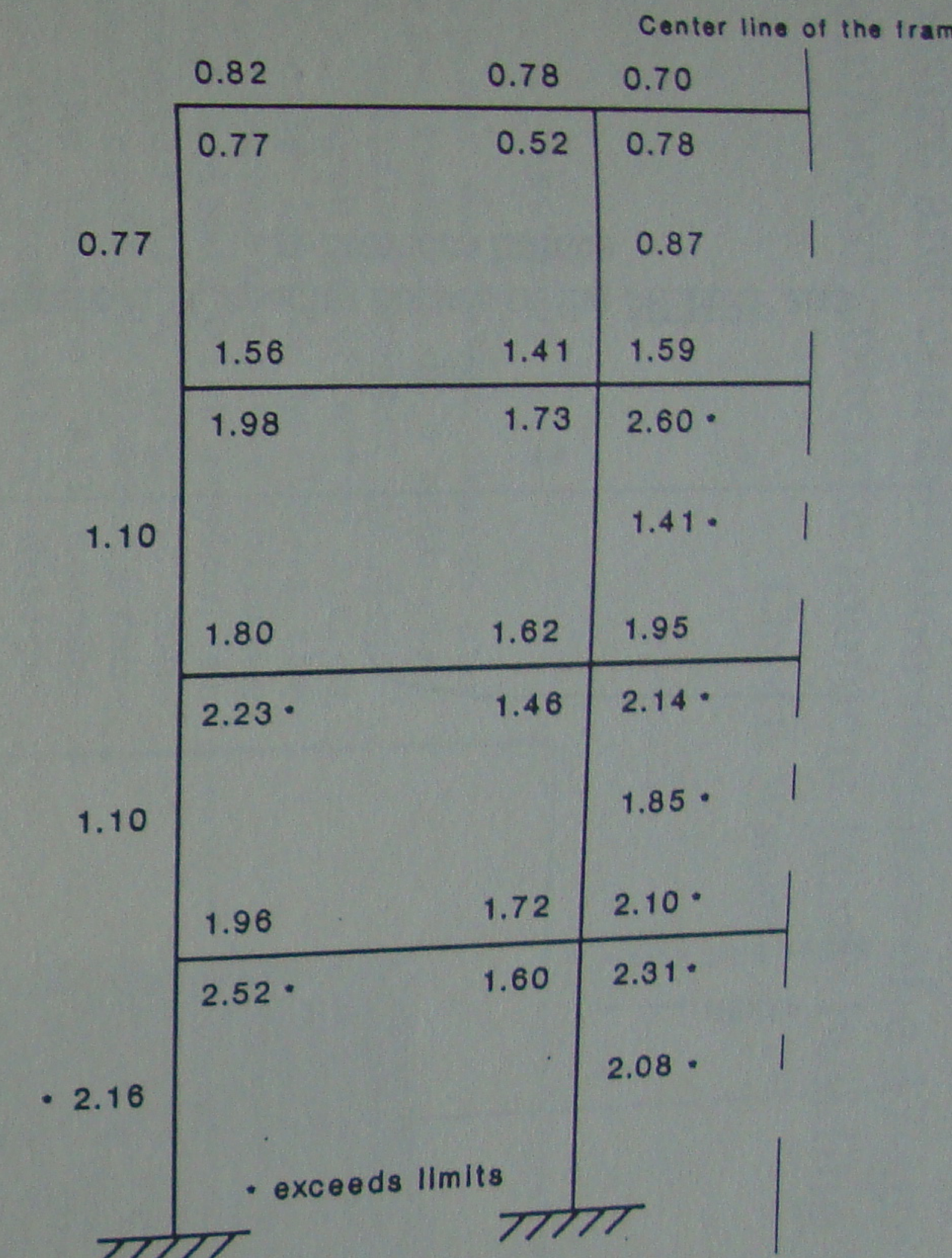

Figure 4. Inelastic demand ratios of the Tri-Services frame

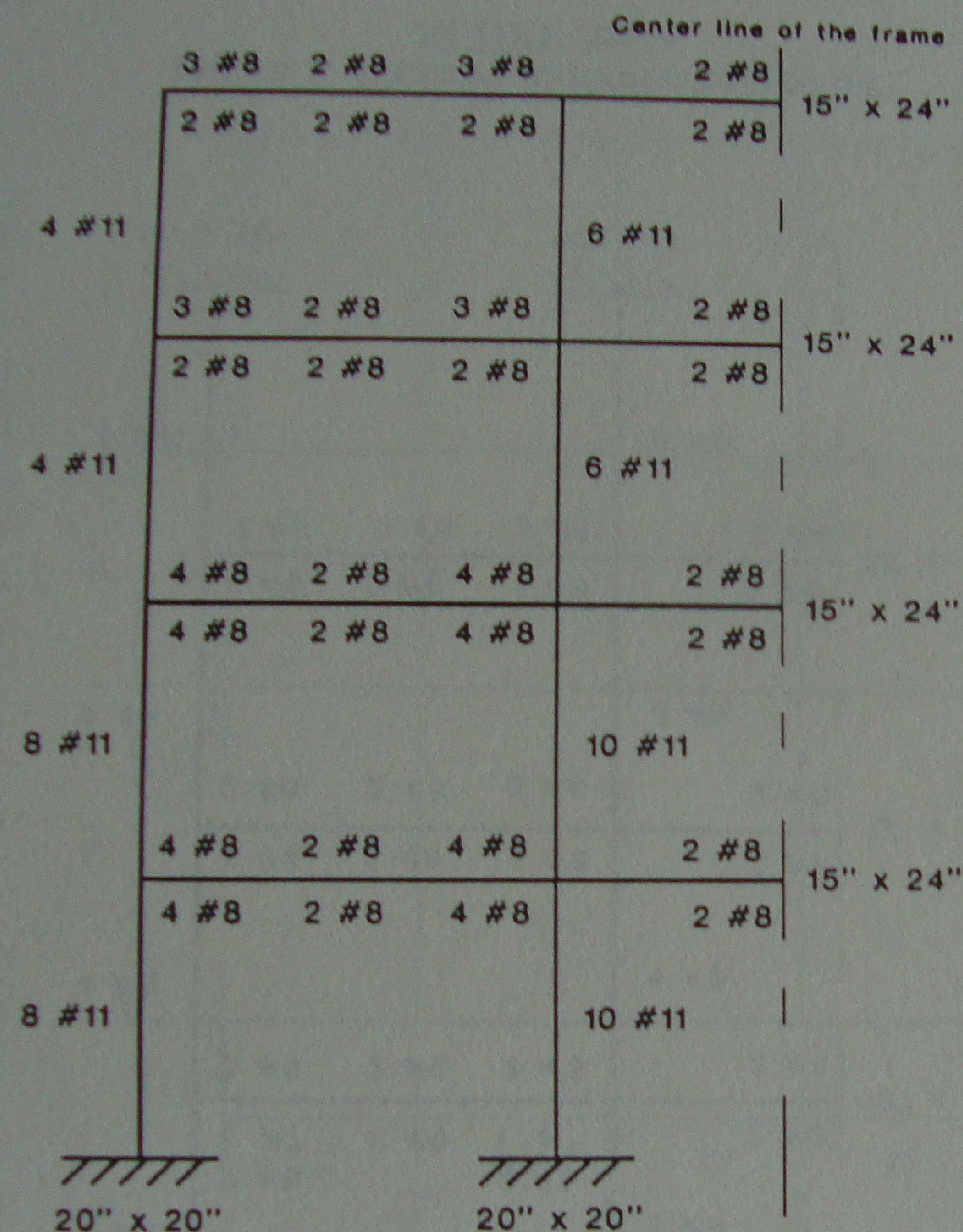

Figure 5. Flexural reinforcement of the Tri-Services frame

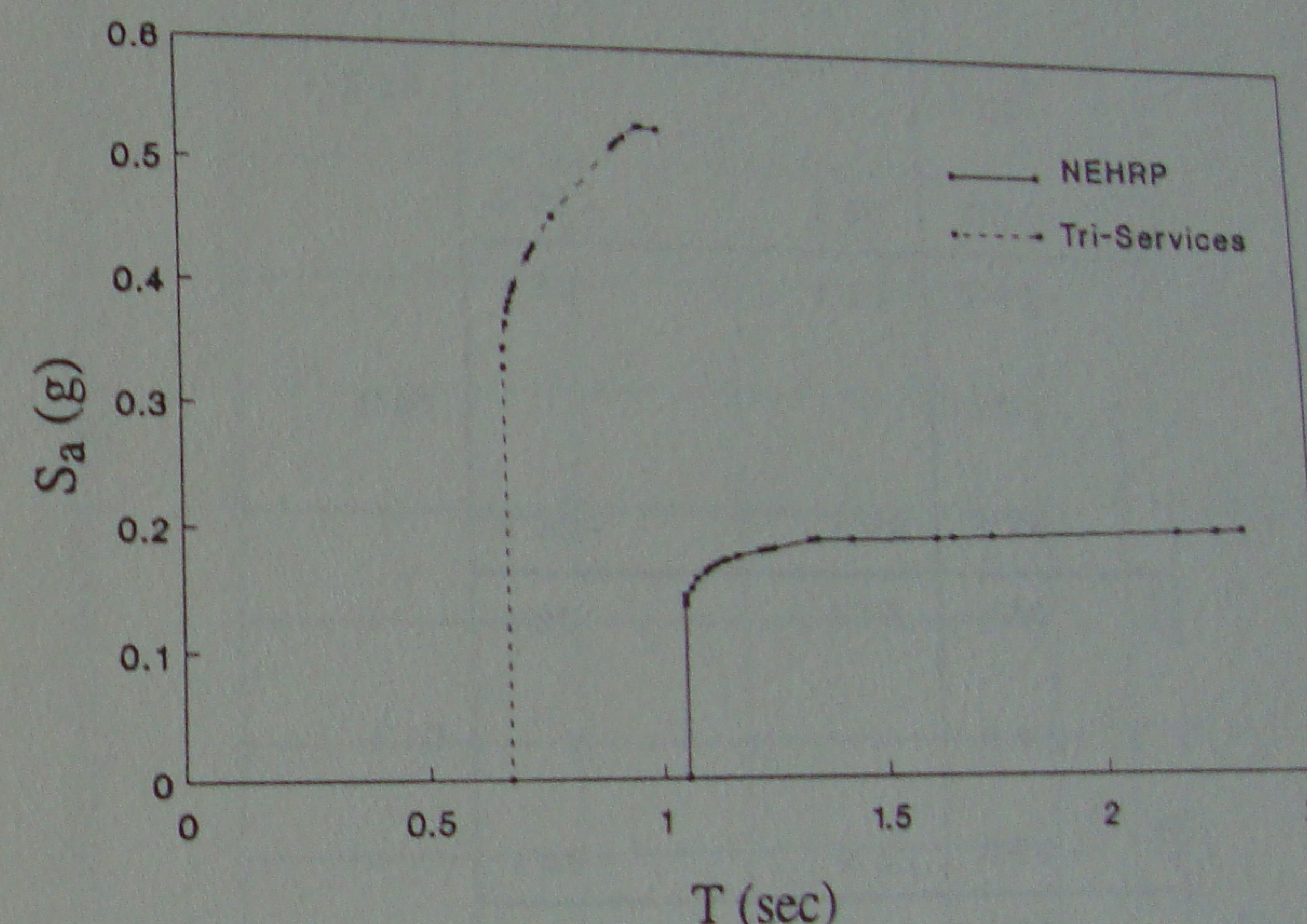

Figure 6. Capacity curves of the NEHRP and Tri-Services frames