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EVALUATION OF THE EMPIRICAL DESIGN OF LOW–RISE MASONRY STRUCTURES

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ABSTRACT

Many masonry low–rise buildings were designed by following a set of empirical rules without much, if any, consideration of the expected loads. These empirical design provisions were simple to apply and provided a cost effective design solution for low–rise masonry structures. Although the empirical design provisions were believed to be conservative, many modern masonry structures have exhibited poor structural performance during recent natural disasters. Older unreinforced masonry buildings have recently demonstrated a high degree of vulnerability to collapse even under moderate levels of ground shaking, as seen in the Northridge (California) Earthquake on January 17, 1994. In this research, the adequacy of empirically designed structures was evaluated by comparing the available resistance to the required capacity determined from a rigorous load analysis of seismic effects. A typical modern low–rise masonry building, located at a site with moderate seismicity near Memphis, Tennessee, was investigated. The building was designed in accordance with both the empirical and engineered design provisions given in the *Building Code Requirements for Masonry Structures* (ACI 530–92/ASCE 5–92/TMS 402–92). It was determined that the empirical design provisions provide insufficient capacities to meet the minimum load requirements of the seismic provisions of the *Minimum Design Loads for Buildings and Other Structures* (ASCE 7–93). Based on the findings of this research, recommendations are given to limit the applicability of the empirical design provisions for masonry structures.

KEYWORDS

Masonry, low–rise buildings, empirical design, seismic loads, seismic resistance.

INTRODUCTION

Empirical design provisions (EmpDPs), or "rules–of–thumb", are based on the past performance of masonry buildings. Engineered design provisions (EngDPs), or "rational design rules", are based on detailed structural engineering, which includes an assessment of loads and available resistance in order to achieve an appropriate design. The EmpDPs provide a quick and simple procedure to design (proportion) a building using masonry and are thought to be a conservative by requiring more material than necessary to resist the assumed loads. The poor performance of many empirically designed structures during recent earthquakes is not that surprising given that the EmpDPs were based on past experience with masonry buildings that are quite different in form from buildings being constructed today.

In order to determine the adequacy of the empirical design provisions, a typical low–rise masonry building (hotel) was considered in this research. The building was designed for a site near Memphis, Tennessee, representing the extreme loading conditions allowed by the EmpDPs. The capacities provided by both approaches, EmpDPs and EngDPs, were compared to determine what elements of an empirically designed building are overdesigned or, more importantly, underdesigned.

BUILDING DESIGN

The hotel considered in this research was a three–story building with a total height of 9.14 m and plan dimensions of 45.70 m by 18.29 m. The structural system of the building consisted of exterior and interior load–bearing masonry walls acting as shear walls. The layout is shown in Figure 1. The masonry walls were constructed of 254 mm × 203 mm × 406 mm hollow concrete masonry units with an unit compressive strength of 13.79 MPa. The masonry walls had grouted cores 406 mm on–center to meet the empirical provisions limiting the compressive stress in masonry. The floors and the roof were designed using 203 mm by 508 mm prestressed concrete slabs without topping and spanned between the walls in the N–S direction. The joints between the slabs were grouted, as prescribed by the manufacturer, to ensure diaphragm action.

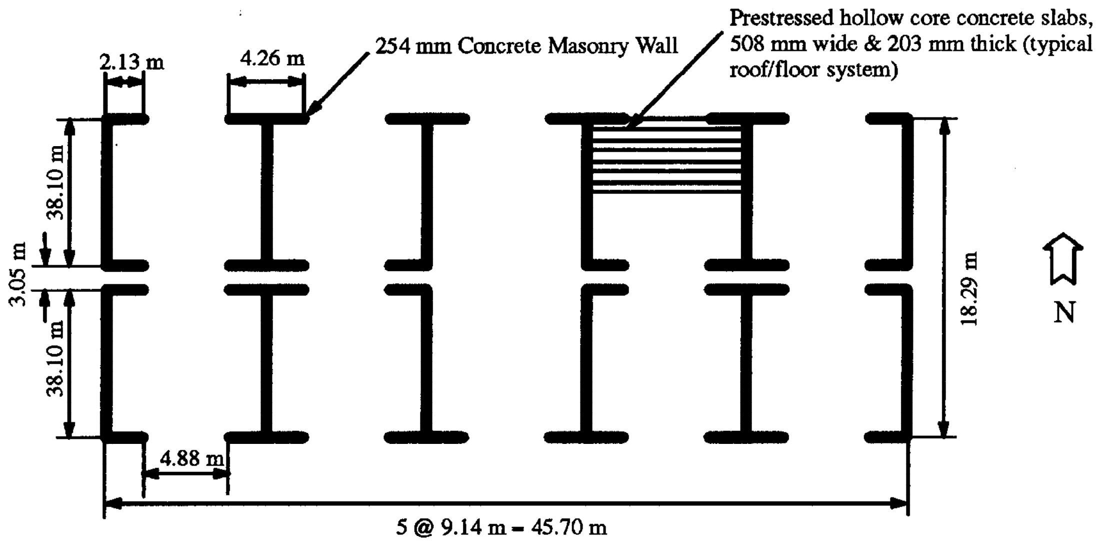

Figure 1. Hotel Building Plan View

Building Site and Design Loads

Empirical design of masonry is used in areas with significant variations in the magnitudes of seismic and wind loads. Provided that no other horizontal loads act on the structure, the EmpDPs for masonry buildings can be used in areas located in Seismic Zones 0, 1, and 2, and with a basic wind pressure less than or equal to 1.2 kPa.

The hotel building was assigned to Seismic Hazard Exposure Group I, since it was neither considered an essential facility nor represented a substantial public hazard. For such buildings, Seismic Zones 0, 1, and 2 corresponded to ASCE 7–93 Seismic Performance Categories A, B, and C, respectively. The basic wind pressure in ACI 530–92 was understood to be the reference wind pressure, which is only a function of wind speed and air density. Therefore, a reference pressure of 1.2 kPa is created by a 159 km/h wind speed.

The site selected was in a suburban area outside Memphis, Tennessee, with an effective peak velocity–related coefficient, A_v, equal to 0.19 (maximum permitted by EmpDPs), and with a basic design wind speed of 113 km/h. Based on the effective peak velocity–related coefficient and the Seismic Hazard Exposure Group, the Seismic Performance Category C was assigned to the hotel building. Two sets of seismic lateral loads were calculated in this study: one set reflected the loads for an unreinforced masonry (URM) building; the other set was for a reinforced masonry (RM) building. The seismic design loads for an unreinforced masonry building were larger, since the inherent ductility and corresponding R–factor were smaller.

All design loads, the out–of–plane wall pressure, the shear at the bottom of the walls, and the shear at the top of the walls for the hotel building are shown in Table 1. While laterally supporting the walls, the roof and floor diaphragms were loaded internally by uniformly distributed loads acting in their planes and were supported by the anchors which provided the connection between the diaphragms and the shear walls. The anchors, transferring the lateral load from the horizontal diaphragms to the shear walls, were loaded less in shear at the roof level than at the lower levels because there was less wall mass and area supported by them at the roof level.

Table 1. Design Loads for Hotel Building

Design Loads for Members and Connections	Design Loads	
	Seismic RM	Seismic URM
Out–of–Plane Wall Pressure[1]		
3rd Floor	1.7 kPa	
2nd Floor		
1st Floor		
Shear at Top of N–S Wall[2]		
Roof	4.5 kN/m	12.6 kN/m
2nd Floor	4.8 kN/m	13.5 kN/m
1st Floor		
Shear at Top of E–W Wall[2]		
Roof	6.2 kN/m	17.5 kN/m
2nd Floor	6.9 kN/m	19.3 kN/m
1st Floor		
Shear at Bottom of N–S Walls[3]		
3rd Floor	8.4 kN/m	23.6 kN/m
2nd Floor	15.2 kN/m	42.4 kN/m
1st Floor	18.5 kN/m	51.7 kN/m
Shear at Bottom of E–W Walls[3]		
3rd Floor	11.3 kN/m	31.5 kN/m
2nd Floor	20.2 kN/m	56.5 kN/m
1st Floor	24.7 kN/m	69.0 kN/m

[1]The equivalent pressure of the seismic lateral force for a wall considered to be an architectural component. Pressure was used in the design of the out–of–plane bending capacity of the walls.

[2]Uniform distributed shear load acting at the top of a shear wall along its length. Connections between roof and top of the walls must resist this load.

[3]Uniform distributed base shear load along the length of a wall subjected to in–plane lateral loads. Load was used in the design of the shear wall capacity of the walls.

All design gravity loads were determined according to ASCE 7–93. The average weight of the masonry wall was estimated to be 3.4 kPa. The roof dead load was calculated to be 3.5 kPa and the floor dead load to be 3.4 kPa. The minimum design live load was 1.0 kPa for the roof, 1.9 kPa for the floors and 4.8 kPa for the corridor section. The flat roof snow load was less than 1.4 kPa and therefore was only considered in the gravity load analysis and not in the seismic analysis of the building.

Design Procedure for EmpDPs and EngDPs

Empirical design is a procedure of sizing and proportioning masonry elements rather than a detailed design analysis. These provisions address minimum wall thickness, maximum building height, shear wall minimum cumulative length and maximum spacing, and maximum wall height or length to thickness ratio. The hotel building was proportioned in accordance with the EmpDPs and then designed in accordance with the EngDPs. A summary of selected requirements for the masonry walls of the hotel building is given in Table 2.

The EmpDPs permitted the building with the dimensions already described to be designed as unreinforced masonry. According to the additional seismic provisions of ACI 530–92, minimum reinforcement was required. This reinforcement consisted of one bar with a cross–sectional area of 129 mm^2 along the edges of each wall, around the perimeter of openings and additional horizontal bars in the bed joints spaced not more than 3.05 m apart vertically.

Table 2. Bar Reinforcement for the Hotel Building[1]

Shear Walls		Empirical Design	Engineered Design
Shear Walls in N–S Direction			
3rd Floor (N–S)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	1 bar @ ends A = 200 mm^2
	Horizontal Shear Reinforcement	n.r.	n.r.
	Vertical Shear Reinforcement	n.r.	n.r.
2nd Floor (N–S)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	2 bars @ ends A = 200 mm^2
	Horizontal Shear Reinforcement	n.r.	n.r.
	Vertical Shear Reinforcement	n.r.	n.r.
1st Floor (N–S)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	2 bars @ ends A = 284 mm^2
	Horizontal Shear Reinforcement	n.r.	n.r.
	Vertical Shear Reinforcement	n.r.	n.r.
Shear Walls in E–W Direction			
3rd Floor (E–W)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	1 bar @ ends A = 284 mm^2
	Horizontal Shear Reinforcement	n.r.	n.r.
	Vertical Shear Reinforcement	n.r.	n.r.
2nd Floor (E–W)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	2 bars @ ends A = 284 mm^2
	Horizontal Shear Reinforcement	n.r.	1 bar @ 0.61 m A = 129 mm^2
	Vertical Shear Reinforcement	n.r.	1 bar @ 1.07 m A = 129 mm^2
1st Floor (E–W)	Flexural Tension Reinforcement	1 bar @ ends A = 129 mm^2	2 bars @ ends A = 387 mm^2
	Horizontal Shear Reinforcement	n.r.	1 bar @ 0.61 m A = 200 mm^2
	Vertical Shear Reinforcement	n.r.	1 bar @ 1.07 m A = 129 mm^2

[1] Reinforcement required for in–plane flexural tension.

The EmpDPs for masonry do not have a special provision addressing the roof and floor anchoring when the roof and the floors are constructed from precast hollow core concrete slabs, although this type of structural system is permitted by ACI 530. The minimum construction requirements, specified by the manufacturer in terms of anchor size and spacing, were used for the empirical design of the diaphragm–wall connections and they were proven to be enough to resist all the design forces.

When the EngDPs were used, structural members and connections were designed to resist all designed loads and the most unfavorable effect of the load combinations, as required by ASCE 7–93. As permitted by ACI 530, the allowable stresses were increased by one–third when the load combinations including seismic loads were considered. Since the basis for the earthquake loads is a strength limit state beyond first yield of the structure, two sets of modifying factors were used to amplify the conventional allowable stresses in order to approximate the equivalent yield strength: one was a stress increase factor (1.7 for steel and 2.5 for masonry) and the second was a resistance, or strength reduction factor, that varies depending on the type of the stress resultant (0.85 for shear, 0.9 for flexure with axial compression, and 0.7 for flexure with axial tension). In other words, an allowable design stress is converted to a limit design stress.

The masonry walls were designed to resist out–of–plane and in–plane lateral loads in addition to the vertical loads. Wall flanges were included in the calculation of the shear wall resistance for the walls in the N–S direction. Effective flange widths not exceeding six times the wall thickness on either side of the web, according to ASCE 7–93, were used. For the E–W direction, the cross–walls were not included in the calculation of the stiffness because they did not add significantly to the moments of inertia and did not affect the shear deformations.

There was no reinforcement required for out–of–plane resistance of the walls. With the properties they had, the walls were capable of transferring all out–of–plane lateral loads to the horizontal diaphragms at the roof and floor levels without developing flexural stresses greater than the allowable stress. All the reinforcement was needed to provide the wall resistance to in–plane lateral loads. The cumulative length of the N–S shear walls was longer than that of the E–W walls. Therefore, the N–S wall sections were loaded with smaller in–plane lateral forces and thus required less reinforcement. The reinforcement was needed to resist the flexural tension developed in all the walls and the shear developed in the E–W walls at the first and second stories, as the amount of shear and flexural tension reinforcement increased for the walls in the lower story levels.

ASSESSMENT OF THE EMPIRICAL DESIGN PROVISIONS

A rational (engineered) analysis was used to evaluate the available resistance of the masonry walls satisfying the EmpDPs for masonry. The comparison of the empirical and engineer design approaches revealed that the area where the EmpDPs obviously showed insufficiency was the shear wall resistance capacity to in–plane lateral loads as shown in Figure 2.

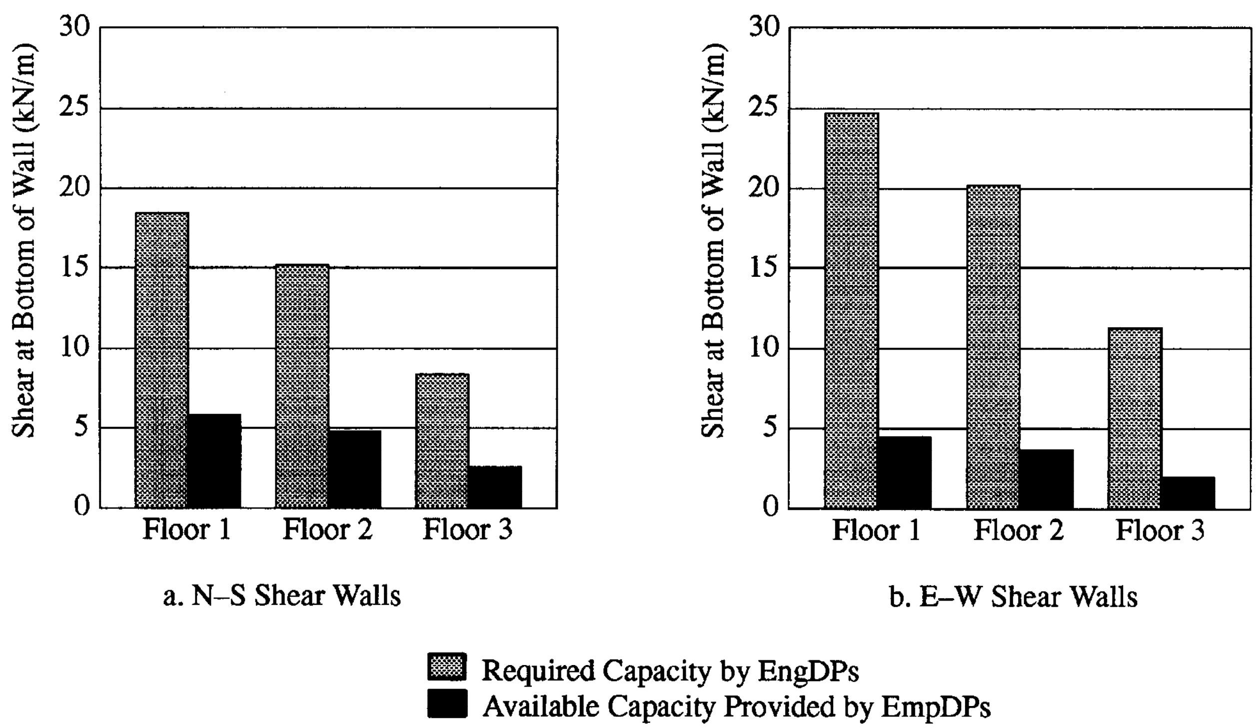

Figure 2. Comparison of In–Plane Resistance of the Shear Walls

According to the EmpDPs and the additional seismic provisions for masonry, the walls had one bar with cross–sectional area of 129 mm^2 at each wall end and, therefore, were analyzed as reinforced masonry for the loading condition of in–plane lateral load. When a seismic design of the building was performed, the different shear wall capacities of the unreinforced and reinforced masonry corresponded to different sets of design loads for unreinforced and reinforced masonry, based on the higher ductility of the reinforced masonry. Shear wall capacity provided by the EmpDPs was 18% and 31% of the minimum required by the seismic provisions of ASCE 7–95 for the walls in E–W and N–S directions, respectively.

The resistance of the masonry walls to out–of–plane lateral loads provided by the EmpDPs, was sufficient for all three stories, according to the rational analysis. The capacity of the walls required by the EngDPs to resist the minimum out–of–plane seismic loads was 94% of the capacity provided by the EmpDPs, as shown in Figure 3.

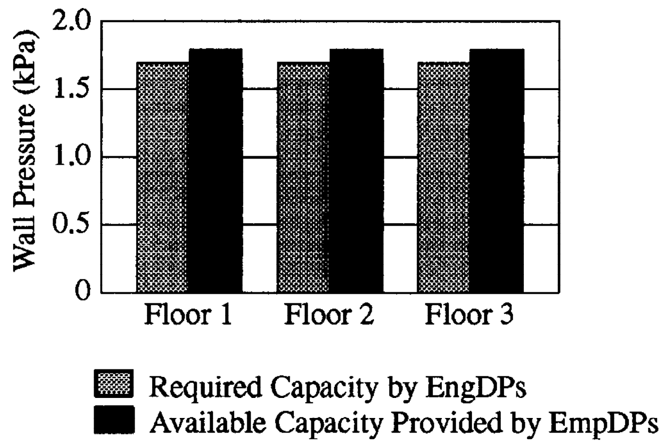

Figure 3. Comparison of Out–of–Plane Wall Pressure

Another issue of the building design investigated in this research was the lateral support of the walls spanning in the vertical direction between the floors, the floor and the roof, and the floor and the ground, as well as the lateral support of the roof and the floors to the shear walls, which was critical for the design of the wall–roof/floor connections. The demand for lateral support of the roof/floors to the E–W walls was greater than to the N–S walls due to the fact that there was less cumulative shear wall length and, therefore, fewer anchors transferring the seismic force at the floor/roof level to the shear walls in E–W direction. For the same reason, the lateral support capacity provided by the EmpDPs for the roof/floors to the E–W walls was less than to the N–S walls. The result of the comparison between the EngDPs and the EmpDPs showed that the lateral support of the roof/floors to the shear walls was satisfactory. The required capacity for lateral support of the roof/floors to the E–W and the N–S walls was only about 80% and 40%, respectively, of that provided by the EmpDPs, as shown in Figure 4.

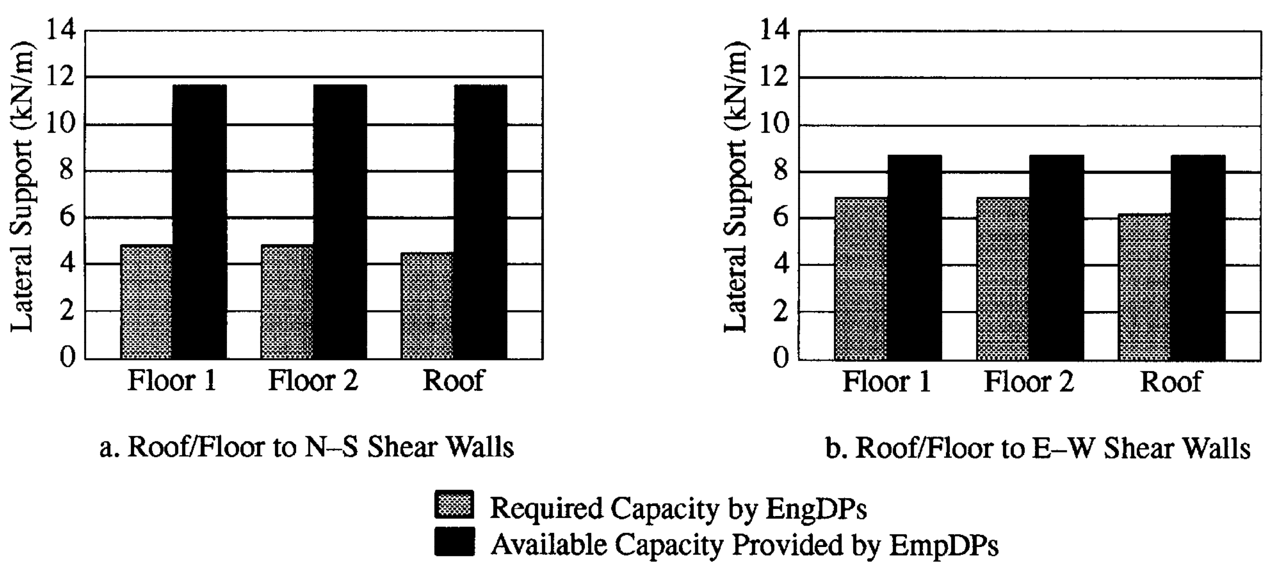

Figure 4. Comparison of Roof/Floor to Wall Lateral Support

CONCLUSIONS AND RECOMMENDATIONS

The design of the hotel reported in this paper was chosen such that it approached the design limits of the EmpDPs. The allowable compressive stress permitted by the EmpDPs was obtained in the building walls acting as load–bearing masonry walls. In addition, the building was designed for a location which was assigned the extreme load conditions allowed by the empirical design of masonry.

It is obvious that the design case discussed here was critical for the evaluation of the applicability of the EmpDPs. Although only one building was considered in this research, it can be expected that the empirical design of buildings with similar configurations will be inadequate. If the building height is reduced to two stories, the problem with the capacity of the empirically designed shear walls will be less likely to occur, and for a single story building it might not be a problem at all. The serious problems found in the seismic resistance of the hotel building reveal that the EmpDPs do not provide adequate protection for property and life safety for design and near design events. The EmpDPs need to be changed, and the issues that should be addressed are discussed here.

The EmpDPs for masonry were considered to be a simple and conservative way to design a building. As discussed before, the EmpDPs were initially derived from experience with small masonry structures. The typical low–rise masonry buildings built today differ from the buildings used to establish the EmpDPs mainly because of their larger scale and their location since the population has moved toward areas with more severe design events. As a result, the behavior of low–rise masonry buildings when subjected to lateral seismic loads has changed, and the EmpDPs for masonry do not result in a conservative or acceptable design for many of these structures.

The hotel building is an example of a building where having more interior walls was the reason for the insufficient shear wall resistance provided by the EmpDPs. A parallel analysis was performed for the same building located near Tampa, Florida, with low seismicity and high wind hazard. Although it seemed unusual to be concerned with the seismic provisions in this area, the seismic provisions did govern the design for some of the masonry shear walls because of the relatively high story–weight to shear–wall–length ratio.

Based on the results from the evaluation of the empirical design of masonry buildings, the following three recommendations are made:

1. The scope of the EmpDPs should be restricted to areas of minimum earthquake hazard so the building capacity provided by the EmpDPs will be enough to resist the minimum design loads. A step in this direction is made in the 1995 edition of ACI 530, where the empirical design is permitted for Seismic Performance Category A buildings only. However, the empirical design of the hotel building near Tampa, Florida, did not provide the required seismic resistance and, therefore, this step alone is not enough to guarantee the adequacy of the EmpDPs.

2. The EmpDPs can be improved by limiting the proportions of the buildings allowed to be designed empirically. In order to reduce the minimum design loads acting on the buildings, maximum building height and plan dimensions should be restricted, as well as the maximum ratio of story weight to the cumulative shear wall length .

3. The EmpDPs can also be improved by requiring the use of reinforced masonry.

If not adopting all three of these recommendation, the EmpDPs may be restricted either to being applied in very few and small areas, or to being permitted for small masonry buildings not typically built today. The combination of them, however, is the optimum solution of the problem since it will result in EmpDPs permitted for a wide variety of masonry buildings located in economically reasonable areas.

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