

# **Reinforced Concrete Frames with Unreinforced Masonry Infills: Design for Lateral Strength**

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## **SUMMARY**

The paper includes a brief review of the "equivalent strut" and the "braced frame" methods and compares their relative merits and limitations. Both the lateral strength and lateral stiffness predicted by each procedure are compared for a typical infilled frame component.

Following this a simplified design methodology is outlined and discussed. Although the suggested approach is cautious and conservative, it is expected that notable economy will be achieved in construction, particularly in countries or regions where the availability of low cost labor makes brick masonry less expensive than concrete or steel.

The procedure is recommended only for structures of less than 15 stories in height.

## Introduction

In the design of framed structures, masonry panel infills have traditionally been treated as nonstructural components. The ability of unreinforced non-load-bearing masonry panels to withstand horizontal forces is indeed low. However, it was shown over two decades ago<sup>(1)\*</sup> that bounding a masonry panel with a steel or reinforced concrete frame significantly improves its strength against racking loads. Considerable effort has since been made to study the nature of the wall-frame interaction which, as is now widely recognized, could result in appreciable economy in design.

The concept of an 'equivalent strut' was originally employed by Holmes<sup>(2,3)</sup> as a means of calculating the ultimate lateral strength of the infill-frame system. This concept was further refined by Stafford Smith<sup>(4,5)</sup> and a method of analysis for infilled frames was proposed<sup>(6)</sup>. More recently, the concept of a 'braced-frame mechanism' has been employed by Lefter<sup>(7)</sup> to assess the strength and stiffness of the system after the infill cracks. Lefter has suggested a detailed procedure which also covers masonry infill panels with openings. There have been other attempts employing stress functions and finite element techniques to compute the lateral strength of infilled frames but, owing to significant cracking of the panels, these techniques necessarily involve complex and complicated computation work and are not considered to be of practical interest at present.

Even though the present knowledge of the behavior of infilled frames is inadequate in several aspects, it is believed that the time has come to selectively employ this knowledge to achieve greater structural economy. This paper endeavors to outline a procedure that could be used to design low to medium rise reinforced concrete frames for lateral strength taking into account the contribution of infill masonry.

## Lateral Strength

It will be useful to start with a brief review of the two methods proposed by Stafford Smith and Carter<sup>(6)</sup> and Lefter<sup>(7)</sup> and to compare their relative merits and limitations.

**Equivalent Frame Method** - The equivalent frame concept is now well developed and has been substantiated by a fair amount of experimental work with infills without openings, assuming that the infill and the frame, while closely in contact, are not deliberately bonded together. Under the application of horizontal loads, the infill separates from the frame over a large part of the length of each side and the contact that remains suggests that the infill acts as a diagonal strut to brace the frame and contribute to its strength and stiffness. The 'effective width' of this 'equivalent' diagonal strut depends not only on the dimensions and physical properties of the infill material but also on its length of contact with the surrounding frame. This length of contact is in turn dependent on the relative stiffness of the infill to that of the frame which is approximately defined by the relation

$$\alpha/h = \pi/2\lambda h \quad (1)$$

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\*Refer to References

in which  $\alpha$  is the length of contact,  $h$  is the height of the frame panel between centerlines of beams and  $\lambda h$  is a non dimensional parameter expressing the relative stiffness of the frame to the infill. The term,  $\lambda$ , is analogous to the characteristic used in beams on elastic foundations and has the value:

$$\lambda = 4 \sqrt{\frac{E_1 t \sin 2 \theta}{4 E I h'}} \quad (2)$$

where  $E_1$ ,  $t$ , and  $h'$  are the modulus of elasticity, thickness and height of infill respectively;  $E$  and  $I$  are the elastic modulus and moment of inertia of the column; and  $\theta$  is the slope of the infill diagonal to the horizontal.

Experimental investigation has indicated that variations in beam stiffness have little effect on the behavior of the system. The relative stiffness parameter  $\lambda h$ , which is thus independent of the beam stiffness, assumes an important position in the equivalent frame concept. All important properties of the system, viz., the length of contact, the equivalent strut width and the strength of the system against various modes of failure have been expressed graphically in terms of  $\lambda h$  in reference (6).

Assuming that the frame has sufficient capacity to 'bound' the infill, that is, the windward column is safe with respect to tensile failure, the leeward column has adequate capacity in compression and, anchorage failure of reinforcing steel will not occur, the failure of the infill masonry can theoretically take place in one of the following modes: (a) shear cracking along the brick-mortar interface; (b) tension cracking through the mortar joints; (c) crushing in one of the compression corners of the infill. Because of the inherent weakness of masonry in shear and tension, the compression failure mode may be disregarded since one of the other two modes of failure will always be attained well before the compressive stresses reach the capacity limit.

The method recommended by Lefter (7) for calculating the shear capacity of an uncracked filler wall is simple and straightforward and is independent of any parameter such as  $\lambda$ . For panels without openings the horizontal area of infill multiplied by the shearing bond strength of the panel masonry gives the system capacity against lateral forces. A similar method to assess the lateral strength for panels with openings is used, although the capacity is restricted to the shear that will cause flexural cracking of any pier in the panel.

A braced frame mechanism is employed to assess the strength of the system after the infill cracks. The concept assumes a particular pattern of cracking which may not always be achieved in practice although it is claimed to give a conservative estimate of the post cracking lateral strength of the frame.

Comparing the two methods, it would appear that Lefter's approach is an oversimplification of a complex problem and that such simplification would lead to assessments of strength which may be far from realistic. A closer study, however, reveals that for most practical cases involving reinforced concrete frames and masonry infills, this simple approach has much to commend it. In fact the working strength of the system as determined by the two methods is in fair agreement.

Lefter's assumption that the precracking strength of the infill is governed only by the panel shear capacity is in fact corroborated by the curves given by Stafford Smith and Carter in Reference (6). Figure 1 gives a graphic relationship between the ratio of panel tensile strength to shearing strength (coefficient of internal friction  $\mu = 0.6$ ) and the parameter  $\lambda h$  for various aspect ratios (based on Figures 8 and 9 in Reference 6). It is seen that even for the lowest practical values of  $\ell/h$ , tensile failure of the panel could precede shear failure only if the infill masonry tensile strength is less than 70% of its shear strength. (From Ref. (9), shear bond strength of concrete masonry specimen with type M mortar is around 60% of the corresponding tensile bond strength.)

A typical single bay frame having 305 mm square concrete columns ( $f'_c = 20.5 \text{ N/mm}^2$ ) reinforced with 4 - #6 grade 50 bars, filler masonry of brick ( $f'_m = 5.5 \text{ N/mm}^2$ ) and a frame height of 3.7 m was chosen to compare the variation in lateral strengths (based on allowable working stresses) given by the two methods for aspect ratios ranging from 0.8 to 2.5 and infill thicknesses varying from 115 mm to 230 mm. The results of the comparison are graphically presented in Fig. 2. Also, two extreme cases, within the range of possibility in actual practice, were picked to represent the limiting values of  $\lambda h$  for the type of structures under consideration. The limiting values of  $\lambda h$  were found to be 7.6 for a flexible column and stiff infill and 1.8 for a stiff column and relatively flexible infill, represented by cases A & B respectively in Fig. 3. The maximum variation in lateral strength computed by the panel shear strength adopted by Lefter is -35% to +18% compared to the corresponding values obtained by the equivalent frame method. This variation is for extreme cases; the usual range is between 10 to 20% and the panel shear strength gives conservative values in most cases. Considering the statistical nature of materials, such a variation is of little import and the two methods agree in the results of computation of lateral strength.

Figures 2 and 3 also indicate the lateral strength of the 'braced frame mechanism' of the system (which is independent of  $\ell/h$  ratio) after the failure of infill masonry (values are converted to working strength assuming a load factor of 1.4). It will be seen that in most cases the system will have passed the peak of its strength when the infill cracks and that in general little reserve strength is available against total collapse after the infill fails. This will be particularly true if the design is based on the contribution of the infill to lateral strength which will necessarily result in smaller columns with lower moment carrying capacity. The important role played by the infill in resisting lateral forces is thus clearly illustrated.

#### Lateral Stiffness

In the assessment of lateral stiffness the two methods have different approaches. The equivalent frame method envisages the replacement of the infill by a diagonal strut resulting in an equivalent truss which may be solved for forces and displacements. Computations of displacements will involve evaluation of the familiar expression  $FUL/AE$  for each member of the truss. The 'area' of the diagonal will be based on the 'effective' width of the equivalent strut. The equivalent strut width is a function of the length of contact between the frame and the infill and is thus dependant on the lateral load applied to the frame and reduces as the lateral load increases.

Lefter likens the uncracked system to a cantilever beam and estimates the lateral stiffness of the frame wall as the reciprocal of the sum of deflections

due to bending and shear, using transformed sections:

$$K = \frac{1}{\frac{h^3}{12 EI_t} + \frac{1.2 hw}{A_w \cdot G_w}} \quad (3)$$

where  $K$  = initial stiffness of frame wall system,  $h$  = distance to mid height of beam,  $hw$  = height of infill wall,  $E$  = modulus of elasticity of concrete,  $I_t$  = moment of inertia of total frame wall system (transformed sections),  $A_w$  = area of wall (transformed sections),  $G_w$  = modulus of rigidity of wall material.

It has been suggested that the relative stiffness of a multi-bay system can be estimated from the above equation considering only the exterior columns and the total area of infill walls and inner columns. While simple methods to account for the effect of openings in the above procedure are available, it is to be noted that the influence of moderate size openings on the stiffness of the overall system is minor and can be neglected.

A comparison of the two methods indicates that neither may be close enough to the true stiffness of the system. It has been shown that unbonded infills subjected to lateral loads separate from the bounding frame for a part of the length of both columns and beams. The system would therefore not be as stiff as the cantilever beam analogy would indicate. On the other hand the assumption of pin connections in the equivalent frame, ignoring the stiffness introduced by the monolithic construction of reinforced concrete frames, would make the system appear more flexible than it really may be. The latter method also suffers from the severe handicap of its inability to handle infills with appreciable size of openings and the complications that would result in considering a multibay frame with structural infills existing only in selected panels.

Fortunately, there are two reasons indicating that the simpler method suggested by Lefter could be applied to practical design problems:

- (a) So long as the design is based on working strength and the allowable panel shear capacity governs the design for lateral strength, the separation between the bounding frame and the infill wall, at working loads, be minimal;
- (b) Since the frame stiffness values are required only for comparison between various frames in a particular structure, shortcomings of the computation method are by and large neutralized.

Considerable thought has been given to the effect of openings in the panel infills upon the lateral stiffness and strength of the system. Suggestions based on experimental verification have been made<sup>(8)</sup> for positioning of openings to minimize the reduction in strength and stiffness of an infilled frame even though no satisfactory method of estimating either has yet been evolved that could be used without reservations in the practical design of a structure. This lack of knowledge will inhibit the designer in exploiting the full potential of the infilled frame strength where openings are present. A selective approach is recommended so that the best possible structural infills are identified and determined at the initial planning stage of the structure. The openings in such panels have to be minimized and judiciously located so as to least interfere with the structural functions of the infill.



## The Design Procedure

The object of suggesting the design procedure that follows is to exploit the potential for lateral strength of the infill-frame system so that economy of construction could be achieved. Notwithstanding the fact that the approach is cautious and conservative, it is expected that notable economy will be achieved particularly in countries and regions where materials like brick masonry are far cheaper than steel or concrete. It is to be noted that the procedure suggested has in view reinforced concrete frames with unbonded (i.e., without shear connectors) brick or block masonry panel infills of say up to 15 stories in height. It could, however, be possible to extend the scope of these suggestions to cover other types of framed structures and for a greater number of stories.

At the stage the building plans are formulated, location of structural infill panels should be decided jointly by the architect and the engineer. A rough assessment of the total story shear (due to wind or earthquake) can be made to judge the quantity of wall area required to withstand the lateral loads of each story. This should help in locating the most suitable set of panels that could be used for structural infills and planned as such in the architectural arrangement so that openings are excluded or minimized. As the required structural infill area increases (in the lower stories), the engineer has a choice of increasing the number, thickness or strength (or any combination of these) of the infill but it is not essential that a structural infill in an upper story panel should be repeated in the lower stories also. Since the gravity loads are carried by the columns and beams of the frame and as long as the floor system acts as a rigid diaphragm in distributing lateral forces amongst various resisting frame-infill systems, there is a choice of altering the structural infill locations floorwise or panelwise. This flexibility is expected to offset the constraints placed on the location and size of openings in the infills which must be eliminated or minimized.

Having decided upon the location of the structural infill panels, the relative lateral stiffness of the individual frames in either direction can be worked out in the manner suggested by Lefter. If the arrangement varies from floor to floor, the relative stiffness has to be calculated at each floor. At this stage only structural infills should be considered in the stiffness calculations even though the architectural arrangements may include construction of other masonry infills which, because of sizeable openings, are not counted towards structural strength of the system.

The next step will be to determine the required strength of the infills to withstand the lateral forces. The allowable shear strength of infill masonry should be based on the relevant specifications (BIA, NCMA or ACI). Lefter's method will be simpler to use but it is suggested that for aspect ratios greater than 1.75, Stafford Smith and Carter's curves be used, where applicable, for comparison and the lower value be adopted for design. Alternatively, an arbitrary reduction factor may be applied in situations where the equivalent frame method cannot be conveniently used. A value of 0.8 is suggested for infills which have  $\lambda h$  value higher than 5 and 0.9 for  $\lambda h$  values between 3 and 5. No reduction need be applied to frames with  $\lambda h$  values lower than 3. No reduction would be required for aspect ratios lower than 1.75.

The ability of the floor system to act as a diaphragm for distribution of lateral forces could be checked with conventional methods. No special treatment of this issue is needed.

It has to be ensured that the frames are strong enough not only to support the vertical loading but also to resist the overturning forces induced in the columns by the lateral forces on the system. Windward columns need to be checked for tension while the leeward columns will have additional compression due to the overturning effect. Besides, the columns should be strong enough to withstand the panel shear at the base of the compression column at its junction with the equivalent strut. Loads given in Fig. 4, as suggested by Lefter, may be used to determine the overturning forces in the frame. Finally, in order to complete the requirements of the frame ability to 'bound' the infill effectively, it has been recommended that reinforcing bars should be anchored so as to develop 1.25 times the yield stress in the bar. The length of anchorage for full development, confirming to this recommendation may be taken as

$$l\alpha = \frac{A_b (1.25) (.04) f_y}{f_c'} \quad (4)$$

where

$l\alpha$  = development length of reinforcing bar  
 $f_y$  = yield stress of reinforcing bar  
 $A_b$  = area of reinforcing bar  
 $f_c'$  = concrete strength.

The treatment of panel walls with openings which are not considered to contribute to the design strength of the systems could be done in two ways:

(a) adopt methods of construction to ensure that such infills do not contribute to the stiffness of the frame.

(b) Use some or all of them to provide reserve lateral strength against collapse if the design forces are for some reason exceeded. This could be done on the basis of relative stiffnesses taking into account such walls and following the method suggested by Lefter. The effect of lateral forces attracted by these walls on columns shall have to be considered and provided for. This may not, however, involve any sizeable extra costs.

It is in fact recommended that wherever masonry infills are specified, these should be used either as structural infills or for developing reserve strength if the latter can be done without much extra cost.

### Conclusion

It is time not merely to recognize the potential for economy in framed structure design if panel infills are treated as structural elements contributing to the lateral strength of the system, but to put into practice the existing state of knowledge and derive benefits from it. A conservative approach is suggested because any economy that involves risk to the safety and stability of the structure is not acceptable. Even so, considerable saving in framing costs is expected and the overall economy will be appreciable particularly in countries/regions where availability of cheap labour makes masonry a much less expensive material than concrete or steel. The infilled frame structure has the added advantage of greater flexibility of floor space utilization as variation in floorwise location of infills is possible. No special techniques are

involved. The only requirement is that there should be contact between the frame and the infill masonry which should not be difficult to achieve in practice.

The procedure could also be used to check the lateral strength of existing framed structures and to devise measures to strengthen existing structures where needed.

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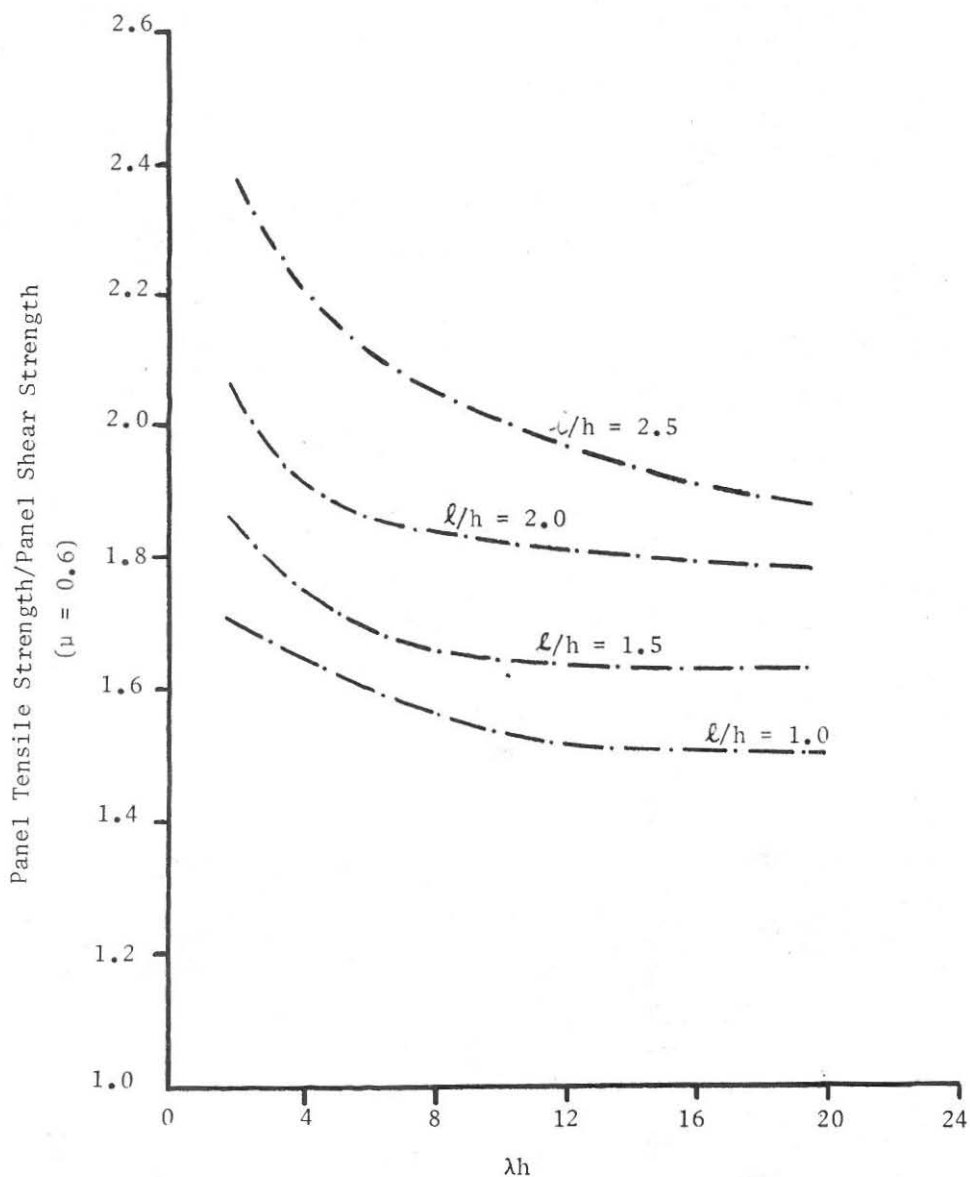


Figure 1 Comparison of Tensile and Shear Strength of Panels



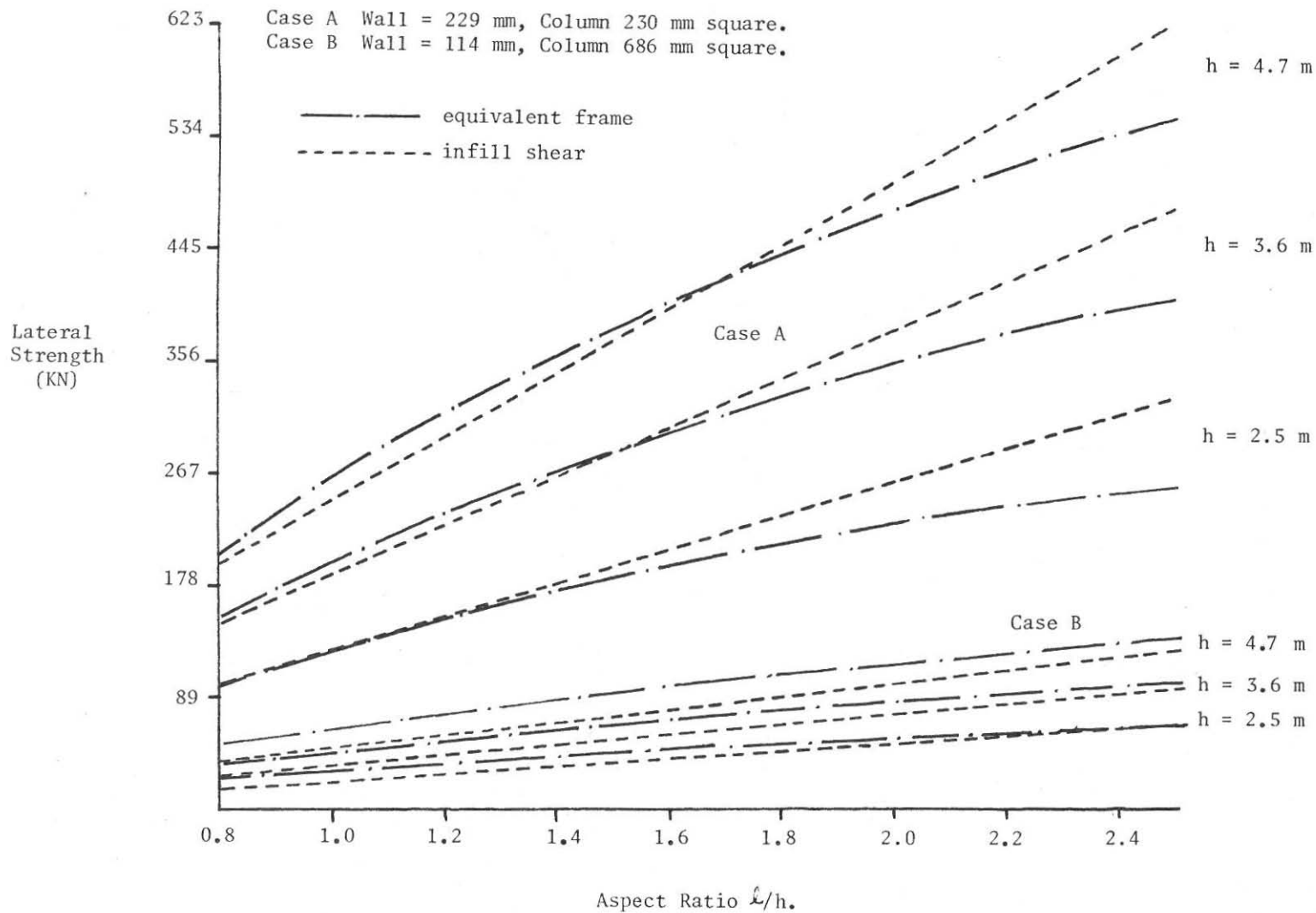


Figure 3 Comparison of Analysis Methods for Extreme Cases

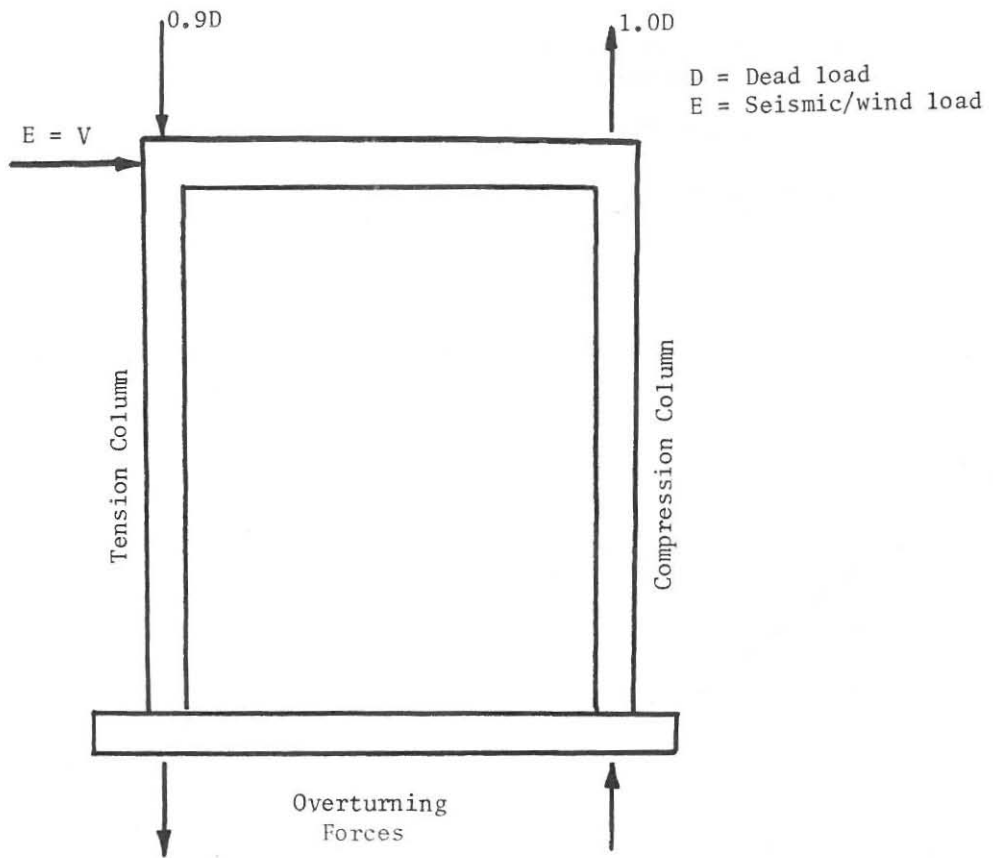


Figure 4 Check For Overturning