STRENGTH OF WALL / FRAME PANELS SUBJECTED TO VERTICAL LOADING

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ABSTRACT

This paper presents simple approximate methods for predicting cracking and ultimate strengths of masonry wall inside frames subjected to vertical loading. The approaches are based on some assumptions taking into consideration both stiffness of the frame and the properties of the masonry wall. Curves in terms of dimensionless parameters are prepared for design purposes.

Notations:

a	: Horizontal distance from point load to the column (shear span).
Cc,Ct,Cb	: Lengths of contact along column, top beam, and bottom beam respectively.
d	: Inclined distance from point load to the support.
E, E_w	: Modulus of elasticity of frame and wall respectively.
f	: Coefficient of friction between mortar and masonry unit.
F_m	: Crushing strength of masonry normal to the bed joints.
F_{mi}	: Compressive strength of the masonry in the inclined direction.
h, L	: Height and length of the panel respectively.
N	: Average normal stress at center of the wall at (d/2)
I_b , I_c	: Second moment of area of beam and column of the frame.
P,	: Load carried by the wall.
P_{cr},P_u	: Cracking and ultimate load of the wall.
R	: Inclined load
r	: Masonry strength reduction factor.
τ	: Average shear stress at center of the wall at (d/2).
t	: Thickness of the wall.
U	: Shear bond strength of the masonry.
ω	: Effective width of the inclined compressive strut.
θ	: Inclined angle = $\tan^{-1}$ (a/h)
λ_c h, λ_b L	: Dimensionless parameters.

INTRODUCTION

Masonry walls in concrete or steel frame buildings are not generally considered as structural elements although they share the frame system in resisting stresses due to wind and earthquake forces. Due to their high in-plane stiffness they can support considerable lateral shears in side the frame-infill system. Upon cracking, most of the large panel shear force is transferred to the frame members, which are usually subjected to a different kind of load distribution.

Researches on the behavior and analysis of infilled frames under lateral loading are many (1,2,3,4,5). Strength predictions and stiffness analysis have been made based on the concept of equivalent pinned frames assuming the infill to act as a diagonal compression member. Analysis based on linear or nonlinear finite element method has also been undertaken [6,7,8,9]. Plastic methods of analysis have also been suggested. [10,11] However no design recommendation has been given in building codes regarding how to consider masonry walls inside frames as structural elements.

Researches have been also carried out on reinforced masonry walls where flexural reinforcement is provided within a thin layer of concrete [12,13]. The only research available which may be related to this subject (vertical loading) is the small scale tests with concrete infilling carried out by Holmes [14]. The tests were carried out on one storey height infilled frames under the effect of combined horizontal and vertical loading. The infill length varied from 3 ft to 5 ft with the ratio of sides (l/h) varied between 1.0 to 1.5. The main objective was to study the lateral strength and stiffness. The tests showed that the specimen under the combined loading showed a diagonal tension crack at a higher lateral load , but failed in diagonal compression at a lower load than the specimen under lateral loading only. A semi-empirical method to predict the deformation and strength of the infill was proposed assuming the infill to act as a diagonal strut with an assumed equivalent width equal to one third of the diagonal length of the infill.

No investigation has been carried out on the strengthening effect of walls inside frames under vertical loading alone, i.e. wall / frame acting as a deep beam although the subject has been mentioned in general while discussing the effect of infill on the frame and its behavior. This may be because the vertical loading is usually small compared to the high lateral forces that the infill will be subjected due to wind or earthquake loading especially for multi storey buildings. Or when a high vertical loading is applied to a wall supported on two columns the designer would prefer an integral wall frame system forming a deep beam for which experimental works and researches are many [15,16]. Despite these in many situations a high vertical loading may be applied through flexible frame bounding masonry walls as shown in Figure (1). This type of wall / frame could satisfy both architectural requirement and economy.

The aim of this study is to present a method of strength prediction for non bonded masonry wall panels inside a frame under the action of vertical loading.

Theoretical Strength Prediction:

The problems and difficulties facing the analysis and strength prediction of wall/frame panels under vertical loading are the same as that for lateral loading. As mentioned earlier the analysis of this type of structure under lateral loading had been attempted using theory of elasticity and finite element analysis, however the uncertain boundary conditions between the masonry and the frame suggest that an approximate solution would be appropriate. For the purpose of strength prediction the following assumptions have been made :

1. The material is homogeneous . This assumption is reasonably accurate and has been adopted by others also.(1 to 7)
2. During loading the wall remains in contact with the frame only at certain contact lengths along its boundary (non bonded wall/frame system) .
3. The wall is replaced by an equivalent inclined compression strut.
4. The frame is strong enough to produce failure of the wall.

For an analysis to be carried out, the essential problems to be defined are :

(1) determination of lengths of contact between the frame members and the wall once separation occurred.

(2) finding an effective wall width for the equivalent compressive strut which replaces the wall in compression .

(2) to establish mode of failure and strength of the masonry.

Modes of Failure:

Based on the behavior of one storey masonry infilled frame subjected to lateral loading [1,2] which is similar to half of our structure under consideration, modes of failure could be stipulated.

A wall / frame subjected to vertical loading as shown in figure (1) may fail in several modes. At the initial stages of application of the vertical load, if good bonding exists at the interfaces between frame and the wall, there may be full composite action between the two components. However, cracks will develop between the wall and the frame members except in the region of the two supporting corners and at the load application interface where the wall remains in contact with the frame and there will be transmission of compressive forces into the wall as shown in figure (2). This action continues until the shear resistance of the masonry wall is exceeded and an inclined crack at the interface between the masonry unit and the mortar stepping between the loaded area and the supported corners is developed, with further increase in load parallel cracks of this type may develop .

The extent of cracking and spalling depends on the relative stiffness of the frame with respect to the panel size and material. The shear cracking of the wall does not define complete failure of the panel because of the restraining and confining influence of the bounding frame , it is possible to increase the load to produce eventually a

compressive failure of the wall material, and failure finally result from local crushing of the masonry in the region of concentrated stresses.
In some cases the strength of the structure may be limited by the strength of the frame members or joints. A reinforced concrete frame may fail by flexure or shear failure of the beams or columns due to the forces exerted upon them, the tensile failure of the bottom beam which will act as a tie member in addition to its bending action, or the formation of plastic hinges in steel frames at sections of maximum moments. The frame is usually made strong enough so that no frame failure would occur prior to wall cracking failure, but wall crushing may not be attained due to premature frame failure.

Length of contact :

The non-dimensional parameters $\lambda_c h$ and $\lambda_b L$ have been adopted to express the relative stiffness of the wall to the flexural stiffness of the columns and beams of the frame where :

$$\lambda_c = [E_w \ t \ \sin 2\theta / 4 E \ I_c \ h]^{1/4} \quad .....(1)$$
$$\lambda_b = [E_w \ t \ \sin 2\theta / 4 E \ I_b \ L]^{1/4} \quad ......(2)$$

The parameter λ was first adopted by Smith [17] for masonry infilled frames under lateral loading. The terms λ_c and λ_b are similar to the parameter whose reciprocal is known as the characteristic length in the general solution of the equation governing the analogous behavior of a beam on elastic foundation [18] $\lambda = (k / 4EI)^{1/4}$, where (k) is the foundation modulus. The length of contact between the frame and the wall can be estimated if the frame / wall interaction is assumed to be analogous to a beam on elastic foundation as shown in figure (3).
When a point load is applied to a beam of finite length the characteristic length is defined when $\lambda_b \ C_t = \pi$ in the general solution of a beam on elastic foundation, this represent the length of contact under the loaded top beam $C_t = \pi / \lambda_b$, for the supporting lower beam the length of contact is defined by $C_b = \pi / 2\lambda_b$ and for the columns at the supporting corners the length of contact is defined by $C_c = \pi / \lambda_c$.

Cracking strength:

As the initial crack occurs along the interface between the masonry unit and mortar, then the shear strength of the masonry depends on shear – bond strength which in turn depends on the amount of normal pre-compression transmitted from the frame to the wall.
Shear strength of the masonry may be taken as :

$$\tau = U + f \ N \quad ....(3)$$

The normal stress N applied at the bed joints depends on the length of contact and interface forces between the wall and the frame.
Figure (4) shows the forces transmitted by the frame to the wall through the lengths of contact . Stresses are non linear but a triangular stress distribution can be assumed for the purpose of analysis. The panel will act as a strut carrying the compressive force

(R) transmitting the load to the supporting corners. Stress at the center of this strut may be approximately computed assuming uniform distribution of stresses.

Average vertical shear stress at center of the strut

$\tau = (P/2) / (h.t)$ ....(4)

Total horizontal force transmitted from frame to the wall = R $\sin\theta$

$= (P/2) \tan\theta = (P/2)(a/h)$

Average horizontal normal stress at the center of the strut

$N = [(P/2)(a/h)/(h.t)](C_c/h)$...(5)

The relationship C_c/h has been suggested following the work by Seddon [19] on partially loaded concrete walls.

Substituting equations (4) and (5) in equation (3) , simplifying and putting the length of contact ratio $C_c / h = \pi / \lambda_c . h$, and the load on the wall at the point of cracking $P = P_{cr}$ We get:

$P_{cr} = [2Uh.t] / [1 - (fa/h)(\pi/\lambda_c.h)]$..(6)

And in the form of dimensionless parameters:

$P_{cr} / Uht = 2 / [1 - (fa/h)(\pi/\lambda_c h)]$.. (7)

The total load P carried by the wall / frame system may be taken as:

P total = P frame + P wall

At this stage P wall = P_{cr} , and the contribution of the frame is very small at the first cracking stage and may be neglected.

Crushing Strength:

Following initial cracking , the masonry wall will resist increased load transmitted from the frame by friction, wedging and arching action within the frame.

The second mode of failure defines the ultimate carrying capacity of a masonry panel inside a frame which is crushing of the masonry material near and at the supported compression corners and at the load application area. Compression stress concentration is high at the corners and directly under the load , failure should originate from these points , however in composite material such as masonry this basic criterion is altered due to the successive cracking of the panel. Crack propagation being more likely to cause spalling of the masonry units.

It is not possible to apply exact methods of analysis to the structure in this state, but as an approximation it might be assumed that crushing failure of the inclined band takes place over an effective width.

Effective Width:

A value for the effective width of the equivalent compressive strut can be obtained on the basis that the compression band is defined by the lengths Ct and Cc as shown in figure (5) .The distribution of compressive stress between the two limiting points will not be uniform and on the assumption that it is triangular with a maximum on the inclined line joining the loaded point with the supporting joint, the effective width may be taken as half the distance between points of contacts: considering top half of the loaded area .

$\omega = \frac{1}{2} [(C_t/2)^2 + (C_c)^2]^{1/2}$, or considering the bottom supported corners.

$\omega = ½ [(C_b)^2 + (C_c)^2]^{1/2}$

The author suggests to take the lowest value of the two equations as an effective width of the strut which will be conservative.

Based on above, an approximate estimate of the ultimate load carried by the wall panel can be made assuming the crushing failure of the equivalent strut takes place over the effective width (ω), therefore:

$R_u = F_{mi} .t .\omega$ or

$P_u = 2 R_u \cos\theta = 2 F_{mi} . t. \omega . h / d$ ....(8)

Since (F_{mi}) is the compressive strength of the masonry in the inclined direction, a reduction factor (r) must be applied to the ultimate compressive strength of the masonry (F_m) where the applied load is perpendicular to the mortar beds, i.e. $F_{mi} = r F_m$, then :

$P_u = 2 r F_m .t . \omega. h / d$ ....(9)

Substituting for (ω), $\pi / 2\lambda_b$ and π / λ_c for C_b and C_c respectively and putting the equation in the form of dimensionless parameters we get:

$P_u / r F_m t .h = (\pi h / 2 d) (1 / \lambda_c h) [(L/h)^{1/2} + 4]^{1/2}$ ....(10)

Since no experimental result is available to the author , (r) values may be taken between 0.6 to 0.9 depending upon the h/l ratio of the panel and the frame stiffness.

Design Curves:

Design curves for cracking load are prepared from equation (7) and shown in figure (6) for different panel dimension ratios h/l , the curves are shown in terms of the dimensionless parameters $P_{cr} / U.h. t$ and λ_c h , Coefficient of friction (f) has been taken as 0.75 .

The value of (f) may be found for the type of masonry units and mortar used for the wall through experiments. Similar design curves are prepared from equation (10) and shown in figure (7) for the ultimate load in terms of the dimensionless parameters $P_u / r F_m.t .h$ and $\lambda_c h$. In the curves of figure (7) moment of inertia of all the members are taken to be equal .If I_b is not equal to I_c the graph will consist of a series of curves for different combinations of members moment of inertia.

To demonstrate the use of the design curves an example of practical use is given in the appendix.

DISCUSSION:

The strength prediction is basically based on the assumption of replacing the wall by an equivalent compression strut where the bottom beam acts as a tie member (strut and tie model). Unlike the concentrated reaction of a true inclined strut hinged to the frame, the wall will induce some change in the mode of deformation of the frame. Before the appearance of the first crack, this change in the mode of deformation has a little effect on the increase of the over all composite stiffness or strength. However, after the appearance of more cracks, the initial behavior of the infill will be modified by creating two or more struts in place of the original one. Most of the load carried by the wall will be due to the wedging, friction and arching action. A remarkable change in the deformation will result from the redistribution of the interface reactions. The frame will be carrying a greater percentage of the load than at the pre-cracking stage of loading.

The equations derived for cracking and ultimate loads do not include the frame contribution to the strength . The frame contribution at the cracking load can be found easily by established structural analysis methods. This contribution is small because the shear crack will occur at low vertical deflection, and may be neglected except for very stiff frames , for which the contribution of the frame will add up to the curve values, in this way the curves will over estimate the strength of the panel.

The cracking strength depends mainly on the shear bond strength of the masonry which depends in turn largely on the workmanship and suction rate of the masonry unit at time of laying and independent on the mortar and masonry unit strength .

For the ultimate strength curves; a crushed region is approximately defined to be equal to the effective width of the inclined strut, this is an approximation of the actual behavior since the crushed region can not be defined accurately especially at the stage of failure where the masonry is in a disintegrated state due to the propagation of cracks and wedging and arching actions.

In view of these facts and the complex nature of masonry composite, approximate methods are appropriate and the proposed curves may be used for preliminary design purposes for steel or reinforced concrete frames, provided that values of the parameters involved are known or estimated from experiments. Curves for panels of other height : length proportions could be easily drawn from the equations.

The same method or technique could be applied to wall/frames subjected to uniformly distributed loading with some modification concerning the length of contact with the upper beam.

Experimental tests especially for reinforced concrete frames infilled with masonry are necessary to be carried out in order to verify the validity of the design curves.

CONCLUSION:

Simple approximate methods have been presented to predict cracking and crushing strength of masonry walls bounded by frame under the action of symmetrical vertical loading based on the strut and tie model, design curves are also presented for preliminary design purposes.

Appendix: ***Design Example;***

It is required to determine the cracking and crushing strength of a wall/frame given the following information:
The wall is brickwork masonry and the frame is reinforced concrete.
Height of wall = 3 m. Length of wall = 9 m. , wall thickness = 0.24 m. , column cross section = $0.3 \times 0.3\ m^2$, E concrete = 20000 N/mm^2.

For a medium strength brick with 1:3 cement : sand mortar , crushing strength of the brick masonry perpendicular to the beds can be taken as equal to 8 N/mm^2. Modulus of elasticity of brick work can be taken approximately as $E_w = 1000\ F_m$ refs[(20,21)], then :

$E_w = 1000 \times 8 = 8000\ N/mm^2$.

$a = 9/2 = 4.5$, $\tan\theta = 4.5/3 = 1.5$, $\theta = 56.3$,

$I_c = 0.3 \times 0.3^3/12 = 0.000675\ m^4$.

$\lambda_c = (8000 \times 240 \times 0.923 / 4 \times 20000 \times 675 \times 10^6 \times 3000)^{1/4} = 0.001819 (1/mm)$

$\lambda_c h = 0.001819 \times 3000 = 5.46$,

From equation (7) or Figure (6) $P_{cr}/U\,h\,t = 5.66$,
assuming shear bond strength of the brick work to be equal to 0.2 N/mm^2 which is a conservative value , then the load to cause cracking of the panel

$P_{cr} = 5.66 \times 0.2 \times 3000 \times 240 = 815\ kN.$

From equation (10) or Figure (7), $P_u / r\,F_m\,t\,h = 0.38$, *taking the reduction factor* $r = 0.6$ *, then the load to cause crushing of the panel*

$P_u = 0.38 \times 0.6 \times 8 \times 240 \times 3000 = 1313.2\ kN.$

The strength of the frame must then be checked to ensure that premature failure of its members would not occur prior reaching these loads. Enough reinforcement must be provided or the members re designed.

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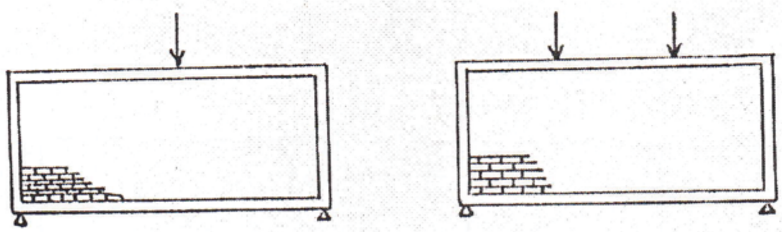

Fig. 1 Wall / frame under vertical loading

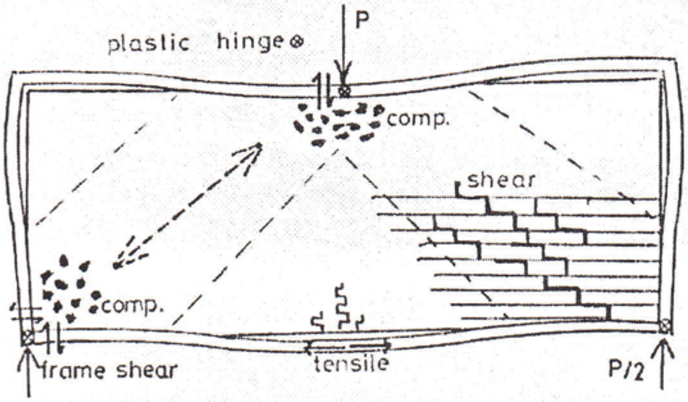

Fig. 2 Modes of failure

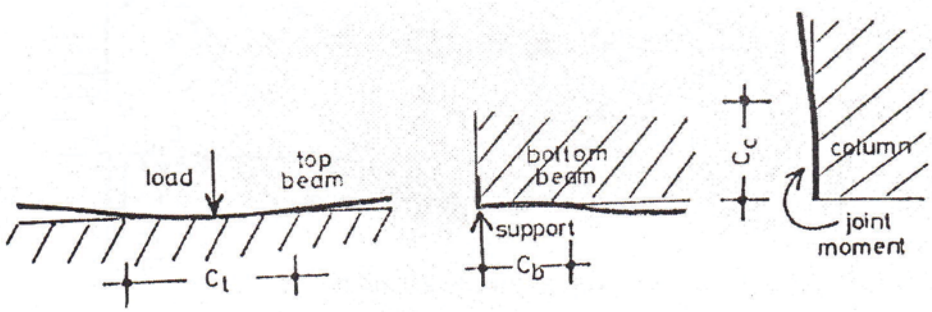

Fig. 3 Lengths of contact

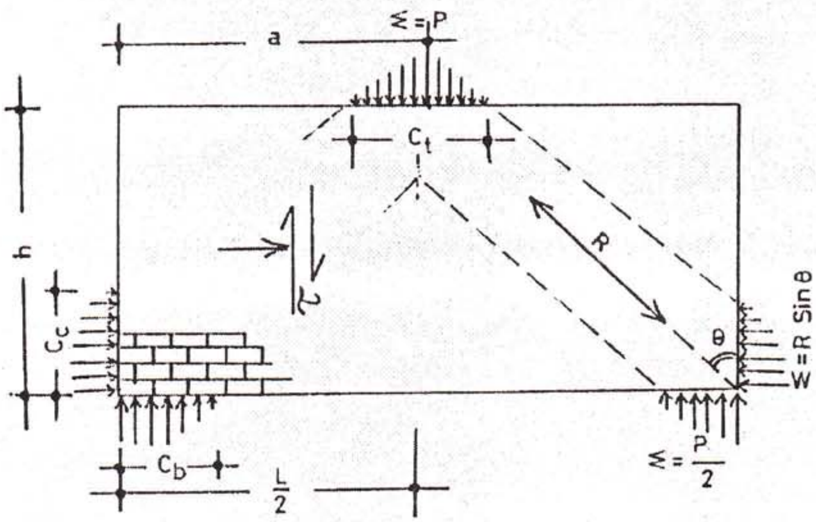

Fig. 4 Forces transmitted to the wall

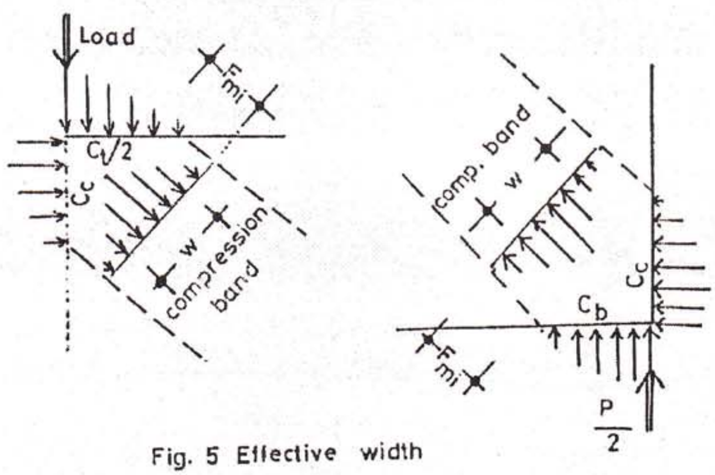

Fig. 5 Effective width

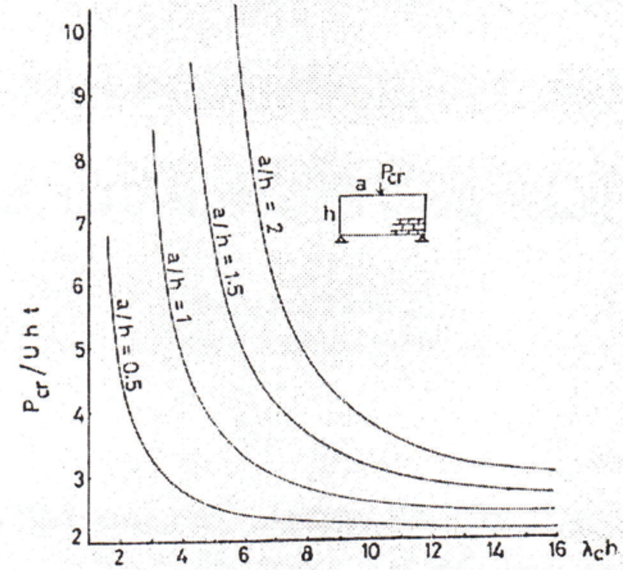

Fig. 6 Cracking load vs. $\lambda_c h$

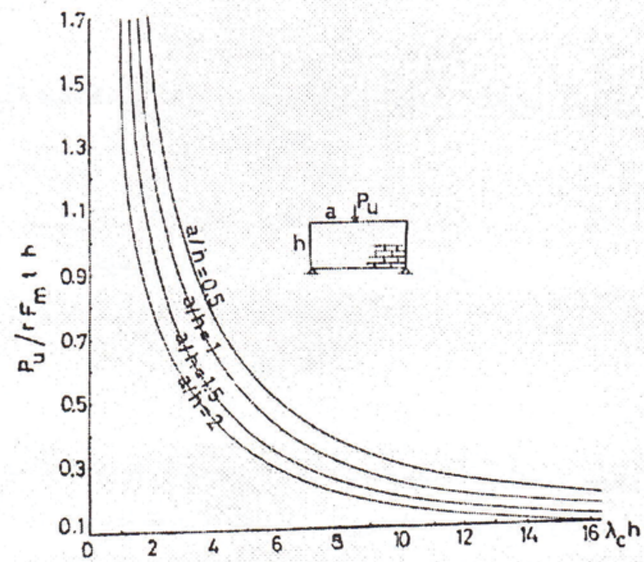

Fig. 7 Ultimate load vs. $\lambda_c h$

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مقاومة الهياكل الحاوية على الجدران تحت تأثير الأحمال العمودية

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الملخص

يقدم البحث طريقة مبسطة لحساب الأحمال العمودية التي تسبب التشقق والأحمال القصوى التي تتحملها الـهياكل المملـوءة بالجدران، تعتمد الطريقة على بعض الفرضيات أخذا بنظر الاعتبار صلابة الهيكل الإنشائي وخواص مواد الجدران. وقد تم تحضير منحنيات لإغراض التصميم الإنشائي.

بەرگری ئەو بینا هەیکەلیانەی کە دیواریان تیادایە لەژێر هێزی ستونیدا

د.محمد رؤوف عبدالقادر
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کورتە

لەم توێژینەوەیەدا ڕێگەیەکی ئاسان پێشکەش کراوە بۆ دۆزینەوەی ئەو هێزە ئەستونیانەی دەبێتە هۆی درزبون وە تێـک شکاندنی ئەو بینا هەیکەلیانەی دیواریان تیدایە . لەم ڕێگە دۆزینەوەیەدا بتەوی هەیکەلەکە و جۆری مـادەی دیوارەکـە ڕەچـاو کراوە و وە هیلـی هاوکیشـی بـۆ مەبەستی تصمیم پێشکەش کراوە.