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تمثيل الحوائط المقاومة للقوى الأفقية بأعمدة في المباني المتعددة الطوابق

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ملخص

يعمل هذا البحث على تمثيل الحوائط المعقدة والتي تحتوي على فتحات بنموذج إنشائي مبسط مكون من أعمدة وجسور رابطة لها باستحداث معامل ضرب المساحة بحيث يصبح التصرف الإنشائي للحوائط الأصلية وللنموذج الإنشائي المبسط متماثل.

إن جساءة الحوائط الأصلية والنموذج المبسط المكون من الأعمدة والجسور قد تم حسابها بتعرضها لنفس القوى الأفقية وحساب الحركة الأفقية الناتجة عنها. وعلى هذا الأساس تم استحداث معامل (معامل ضرب المساحة) ويتم ضربه في مساحة الأعمدة بحيث يتم تعويض الإزاحة في النموذج المبسط لتكون مساوية للإزاحة في الحوائط الأصلية. لقد تم استحداث طريقة العناصر المحدودة في الحوائط الأصلية بينما في النموذج المبسط تم استخدام الجسور والأعمدة. إن استبدال تمثيل الحوائط الأصلية ذات الفتحات بأعمدة وجسور في النماذج الإنشائية للمباني المتعددة الطوابق تسهل عملية التحليل الإنشائي لهذه المباني بشكل ملحوظ وخصوصاً عندما تكون سعة الكمبيوتر محدودة.

1. استاذ مساعد – كلية الهندسة – جامعة البلقاء التطبيقية – السلط- الأردن.

Abstract

This paper deals with simplifying the modeling of complex walls with openings as separate columns and beams accompanied with correction factors so that the structural behavior of the complex walls with openings and the simplified models is almost the same.

The wall rigidities of the original wall and the simplified modeling case are calculated by subjecting these walls to the same horizontal force and then calculating the horizontal displacement. A multiplication factor is introduced for the area of the columns of the simplified model to account for the difference in the horizontal displacement in order to have almost the same displacement in the prototype and the model. Finite element model was carried out on the original wall while beams and columns are adopted in the simplified structural models. This modeling is very important in designing multistorey buildings especially when the capacity of the computer is limited.

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Introduction

The distribution of horizontal forces due either to earthquake or wind which the structure is subjected to are distributed according to the rigidities of shear walls UBC [1]. Rigidity is the reciprocal of deflection.

According to Qaqish [2] a comparison of walls rigidities with opening by three methods were presented. The first is based on calculating the wall rigidity as the sum of the registries of individual piers between openings. The part of the wall above and below the opening is neglected. The second method based on evaluating wall rigidities by calculating deflections from standardized values of forces, thickness, and modulus of elasticity. The third method is by using finite element method. It was concluded that the second method for evaluating wall rigidity with openings give acceptable results with good accuracy compared to finite element method. Qaqish [2] present results for wall with only two openings.

Park [3] discussed openings in one way and two way slabs and the diaphram behavior of these slabs.

ACI [4)] also shows the detailings of steel around the openings in slabs and shear walls.

NCR [5] discuss the practical lessons which can be developed from the loma prieta earthquake. Detailings in shear walls, columns, and beams were developed to have better performance during earthquake.

Kenneth [6] discussed the braced and unbraced frames. Braced frames due to shear walls was discussed. If a designer is uncertain as to the effectiveness of bracing elements, two quantitative criteria, only one of which must be satisfied:

Method 1. Columns in a given story may be considered braced or nonsway elements if the column end moments produced by a second-order structural analysis are not more that 5 percent larger than the moments predicted by a first-order analysis. A first-order analysis is based on the initial geometry of the structure and assumes behavior is elastic. A second-order analysis, which is more complicated, includes the influence of joint displacements and changes in geometry on the forces in structures. Today more and more computer programs have the capability to carry out both a first- and second-order analysis. If lateral displacements are small, both types of analysis produce about the same results.

Method 2. A story may be considered braced if:

$$Stability\ Inde, Q = \frac{\Sigma P_u \Delta_0}{V_u l_c} \leq 0.05$$

where

ΣP_u and V_u = the total vertical load and story shear, respectively, in the story being investigated

l_c = the length of column, measured from center to center of joints

Δ_0 = the relative lateral deflection betwee the top and bottom floors of the story due to V_u computed using a first-order elastic analysis. In this analysis, ACI Code [11] specifies that the influence of flexural cracking, creep, and other factors on member stiffness be accounted for by using reduced values of moment of inertia based on the gross area of the cross section, i.e., 0.70 I_g for columns and 0.35 I_g for beams.

Fintel [7] showed the effect of infill as a braced elements for multistorey structures. The interaction between the structural elements and the infill is clearly presented. Infill had considerable effect on the overall behavior of the structure.

Reynolds [8] presented the behavior of braced walls and the unbraced walls. The walls are either slender or stocky (short) for both cases. The Joints and intersections between members are also considered.

Lindeburg [9] presents structural members resist horizontal forces composed of the same material and almost have the same thickness, the deflection is calculated with arbitrary values of applied forces, modulus of elasticity and wall thickness. The lateral loads will be distributed to vertical members according to their relative rigidity.

$$R = 1/x$$

Naeim [10] presents the deflection of walls due to both shear V and moment Vh (h is the height of wall) of walls with different boundary conditions.

For fixed pier at both ends

$$X_{Fixed} = \frac{Fh^3}{12EI} + \frac{1.2Fh}{AG}$$

For wall fixed at bottom but free to rotate at top

$$X_{Cantilever} = \frac{Fh^3}{3EI} + \frac{1.2Fh}{AG}$$

Where:

F = Applied horizontal Forces

E = Young's Modulus

I = Moment of Inertia

A = Area of Wall

G = Shear Modulus of Elasticity

h = height of wall

According to ACI [11], for normal weight Concert

$E = 3 \times 10^6$ psi (2.1×10^7 Kpa)

G = 0.4 E

According to wakabayashi [12] the horizontal forces distributed to each wall will be in proportional to their relative rigidity.

$$Rigidity_{Fixed} = \frac{1}{0.1(h/d)^3 + 0.3(h/d)}$$

$$Rigidity_{Cantilever} = \frac{1}{0.4(h/d)^3 + 0.3(h/d)}$$

Where:

d is the depth of wall

h is the height of wall

The total seismic force is distributed to the walls in proportion to their relative rigidities

$$V_A = \left(\frac{RA}{R_A + R_B}\right) F$$

$$V_B = \left(\frac{RB}{R_A + R_B}\right) F$$

Where:

RA rigidity of wall A

RB rigidity of wall B

In this paper a wall with many openings are considered and method 1 in (Qaqish [2]) will be adopted with certain correction factor (area multiplication factor) so that the rigidity of the wall will be closed enough to the finite element model.

Idealization and Description of Structural Model

Fig. (1) shows a long shear walls subjects to horizontal force equal 100 KN.

A finite element mesh was constructed for this wall as shown in Fig.. (2)

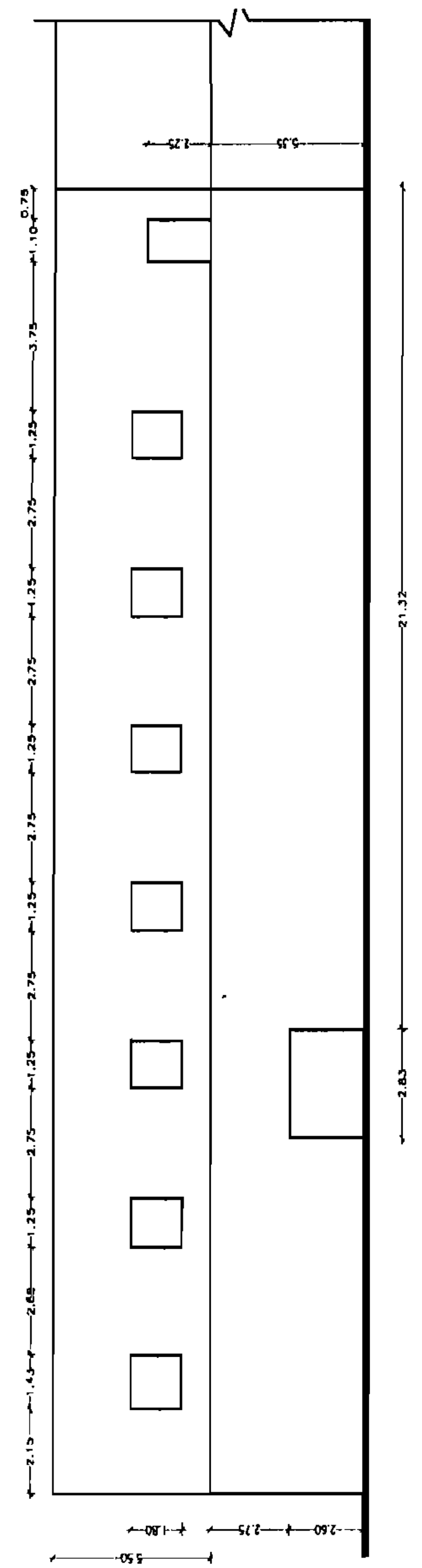

Fig. (1) Long Shear Wall 1 With openings.

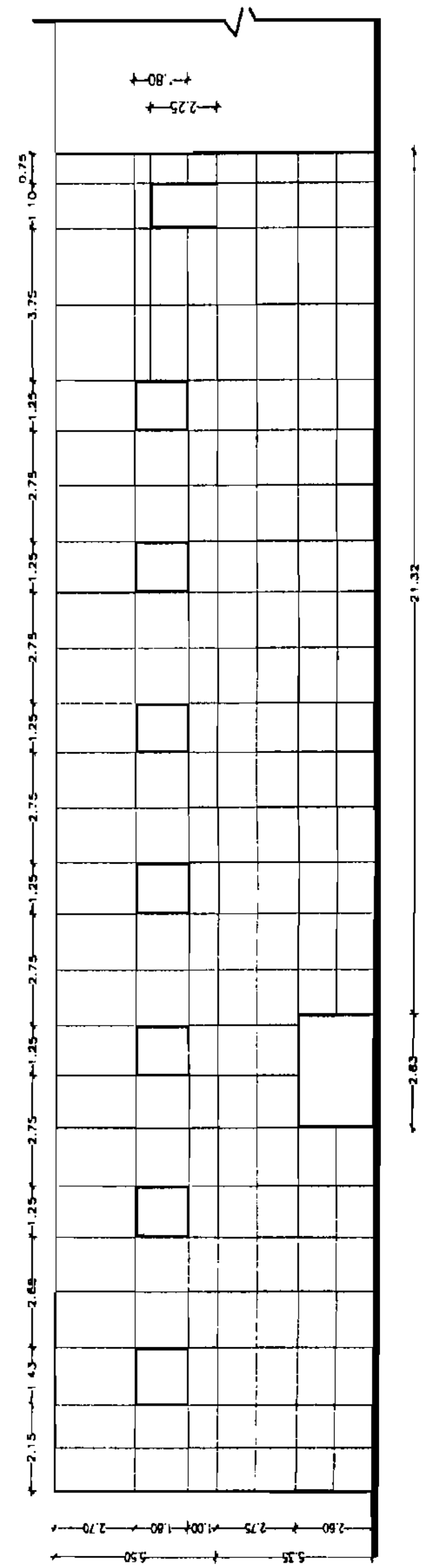

Fig. (2) Finite element mesh for shear wall 1.

The horizontal displacement is calculated and then a simplified model was suggested as shown in Fig. (3) and Table 1 , where the wall is modeled by columns and beams and this model is subjected to the same horizontal force at the top of wall. Then a multiplication area factor was introduced to be multiplied by the area of the columns to give the same displacement as the original wall.

The horizontal displacement for the finite element model was 0.01 mm at the top and then for simplified model the cross sectional area, moment of inertia around tow axes of the part of the wall between openings were calculated and the part of wall between openings were neglected. In the vertical directions horizontal beams connecting these columns between openings are introduced.

The same horizontal load 100 KN was pointed at the top of the tow models.The horizontal displacements were almost the same for the tow models and approach 0.01 mm. The multiplication area factor for the columns in the simplified model is 1.0 .

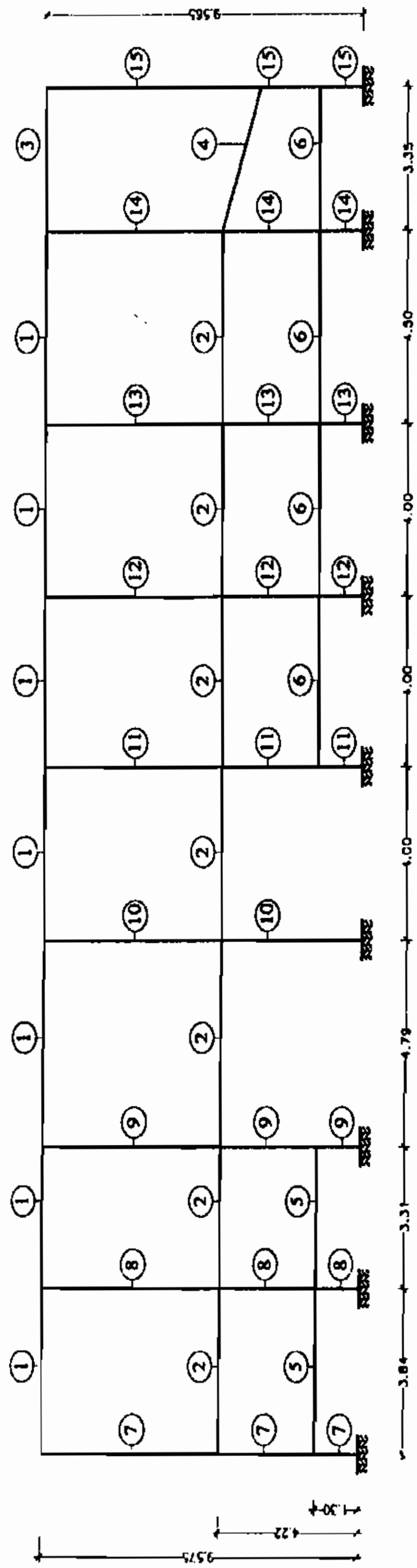

Fig. (3) Simplified model for shear wall 1.

The other long shear wall is shown in Fig. (4). Finite element model and simplified model are shown in Fig. (5) and Fig. (6) and Table 2.

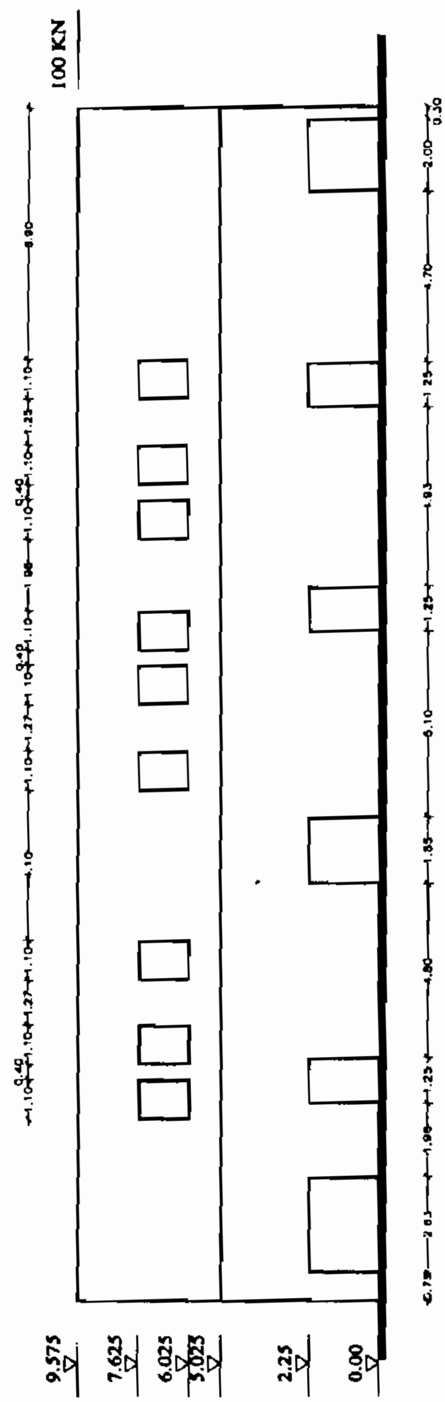

Fig. (4) Long shear wall 2 with openings.

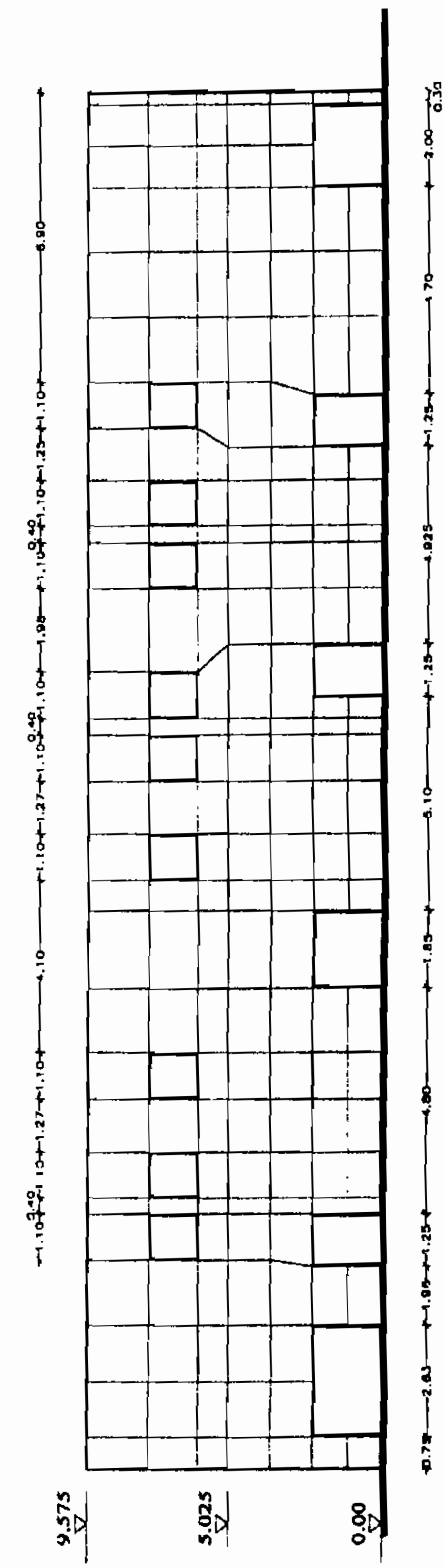

Fig. (5) finite element mesh for shear wall 2.

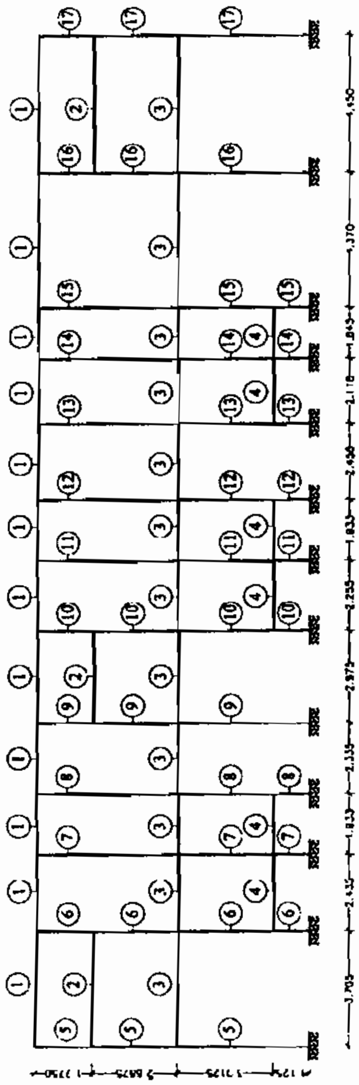

Fig. (6) Simplified model for shear wall 2.

The same procedure is followed for this long shear wall as the previous one.

A multiplication area factor of 1.33 to the area of columns in the simplified model is adopted so that the same horizontal displacement is occured at the tip of both models.

The displacement of the finite element method is 0.016 mm while the displacement of the model is 0.012 mm, so the correction factor is 1.33.

Conclusions and Recommendations

It can be concluded that modeling of such walls in simplified manner is very practical as the final model of the multistory structures will be easy to handle and the results will also be easy to analyze, especially in three dimensional model of multistorey buildings where shear walls are presented.

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Table 1: Member Properties for shear Wall 1

Member No 1	Area m^2	Moment of Inertia Iz m^4	Moment of Inertia Iy m^4
1.	0.95	0.286	0.02
2.	1.61	1.397	0.034
3.	1.05	.385	.022
4.	1.22	.54	0.029
5.	1.3	0.73	0.027
6.	1.3	0.73	0.027
7.	1.11	.414	.022
8.	1.34	0.8	0.028
9.	0.725	0.127	0.015
10.	1.24	.63	.026
11.	1.38	.867	.028
12.	1.38	.867	.028
13.	1.38	.867	.028
14.	1.87	2.19	0.039
15.	0.375	0.018	0.0078

Table 2: Member Properties for Shear Wall 2

Member No 1	Area m^2	Moment of Inertia Iz m^4	Moment of Inertia Iy m^4
1.	0.975	0.31	0.02
2.	0.8	0.17	0.017
3.	1.8875	2.24	0.039
4.	1.125	0.47	0.023
5.	0.1	0.0033	0.002
6.	0.98	0.314	0.02
7.	0.2	0.0027	0.0042
8.	0.63	0.084	0.013
9.	0.6025	0.073	0.0126
10.	0.523	0.048	0.011
11.	0.63	0.084	0.013
12.	0.2	0.0077	0.0042
13.	0.8175	0.182	0.017
14.	0.2	0.0027	0.0042
15.	0.345	0.014	0.007
16.	2.35	4.33	0.05
17.	0.1	0.0033	0.002