Lateral Load Distribution in Masonry Buildings

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Abstract - On the basis of the results of two single story masonry building model tests the suppositon of the mathematical model for the calculation of the story H-δ diagram has been verified - the supposition of the distribution of the story shear to individual, separately treated walls, according to their individual stiffnesses. Good correlation between the calculated and measured displacements and forces has been obtained, in the proportional range and up to the limit, failure state as well.

1. INTRODUCTION

When calculating the resistance of masonry buildings to seismic loading we generally have to know:

- the distribution of horizontal inertial forces over the height of the building;

- the distribution of horizontal story shear to individual walls in the critical story, in the proportional range and up to the ultimate, failure state as well. For this aim we must know both the flexural and the shear resistance of walls to horizontal loading as well as their deformability characteristics;

- the distribution of bending moments over the height of individual walls in the critical story;

- the failure mechanism of the building as a whole.

In this paper, the problems of distribution of lateral load, i.e. the horizontal story shear to individual walls in the story will be discussed.

When distributing the lateral load onto the walls, the supposition of the rigid floor diaphragm action, which causes uniform deformations of all the walls in the story considered, and the supposition of distribution of the stroy shear to individual walls according to their stiffnesses, are usually used.

The first supposition is usually satisfied by means of monolithic floor structures. On the other side, an insight into the possible distribution of the story shear to individual walls, according to their deformability characteristics, can only be obtained experimentally:

- on the basis of tests, carried out on wall elements under combined vertical and horizontal load conditions, the idealized H-δ diagrams can be calculated. By means of H-δ diagrams, which should take into account both extreme

cases of failure of wall elements, the construction of the story H-δ diagram becomes possible.

- on the basis of the subsequent failure of walls of one-story masonry building models, subjected to the programmed cyclic horizontal displacements, the supposition of treating the walls of L and T shape as being disconnected at vertical joints in both directions, can be verified.

2. IDEALIZATION OF THE H-δ DIAGRAMS OF WALLS

2.1 The suppositions

When calculating the idealized H-δ diagrams of walls, the following suppositions are taken into account:

- the wall is considered to be a beam element. In the case of slender walls (the walls with the height to width ratio $h/d \geq 1.5$) this supposition is acceptable, in the case of wider walls the contribution of the flexural part of deformation is rather small;

- the height of the moment inflection point is supposed to be at midheight of the wall. The analytical studies have shown [1] that in the case of shear failure of wall elements, which is typical for the buildings up to 5 stories high, the height of the moment inflection point is very close to the midheight of the wall;

- the influence of the vertical joints between the orthogonal walls is neglected, i.e., the walls of L or T shape are treated as being disconnected at vertical joints in both directions. This suppositon can be accepted because the shear part of deformations is prevalent.

2.2 The strength and deformability characteristics of walls

By means of vertical load carrying capacity tests on walls the compressive strength "β" and the deformation modulus of the wall "D" are obtained.

By means of combined vertical and horizontal load tests on walls the H-δ diagrams (Fig.1) are obtained, which define the strength and deformability caracteristics of walls, subjected to lateral load, with:

K_o - initial, elastic stiffness of the wall,

δ_o - deformation at the idealized elastic limit,

H_{max} - the resistance of the wall to horizontal load,

$\delta_{H_{max}}$ - deformation when the resistance of the wall H_{max} is reached,

δ_{max} - deformation at the failure of the wall.

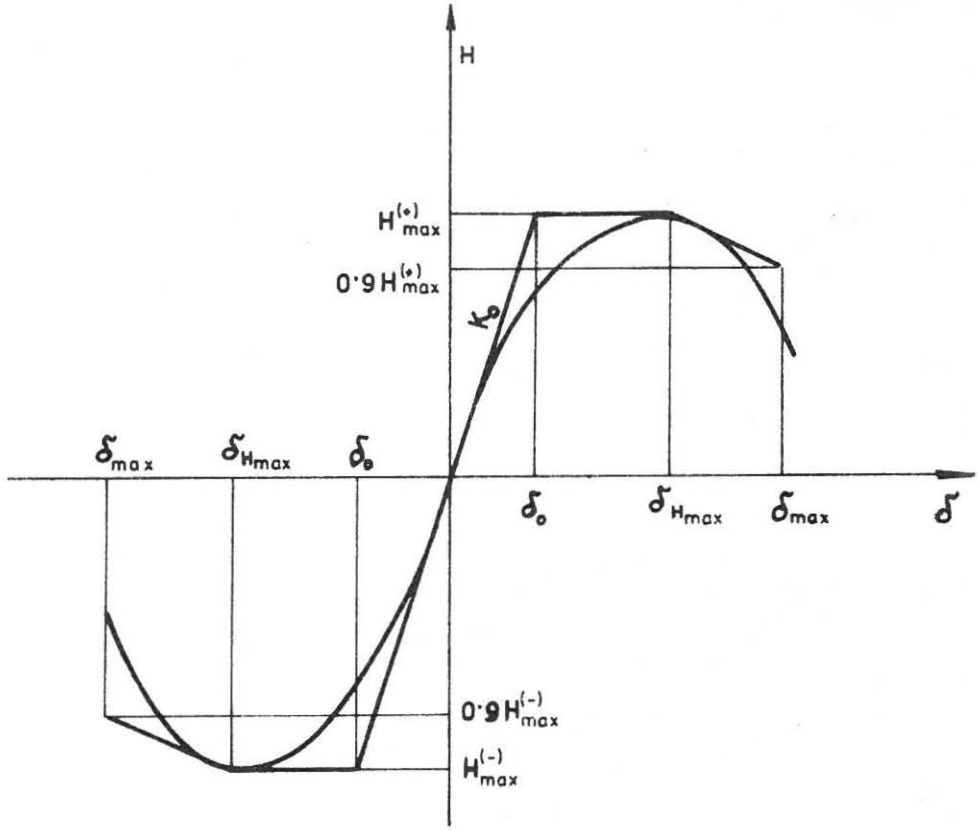

Fig.1 - Idealization of an experimentally obtained H-δ diagram

On the basis of these experimentally obtained data, taking into account the boundary conditions (both ends of wall remain parrallel during the test), the values of the shear modulus "G" and the tensile strength of the wall "σ_n" can be evaluated, using equations (1) and (2):

$$G = K_o \frac{1.2\,h}{F} \frac{1}{1 - \frac{K_o\,1.2\,h}{F} \frac{1}{D} \left(\frac{h}{d}\right)^2} \quad , \tag{1}$$

$$\sigma_n = \sigma_o \left[\sqrt{\left(1{,}5 \frac{\tau_o}{\sigma_o}\right)^2 + 0{,}25} - 0{,}50 \right] \text{(}^1\text{)} \quad . \tag{2}$$

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(1) Equation (2) is valid for walls with the height ot width ratio $h/d \geqslant 1.5$.

where:

F - cross-sectional area of the wall,

σ_o - the average normal stress in the wall due to vertical loading,

τ_o - the average shear stress in the horizontal cross-sectional area of the wall at failure.

Knowing the compressive strength "β" and the tensile strength "σ_n" of the wall it becomes possible to calculate the resistance of wall to horizontal loading, expressed in the form of the so called interaction diagrams [1], [2]. An example of interaction diagram is presented in Fig. 2, where both the flexural and the shear resistance of wall to horizontal loading (ordinate) as a function of vertical loading (abscissa) are plotted in a nondimensional form.

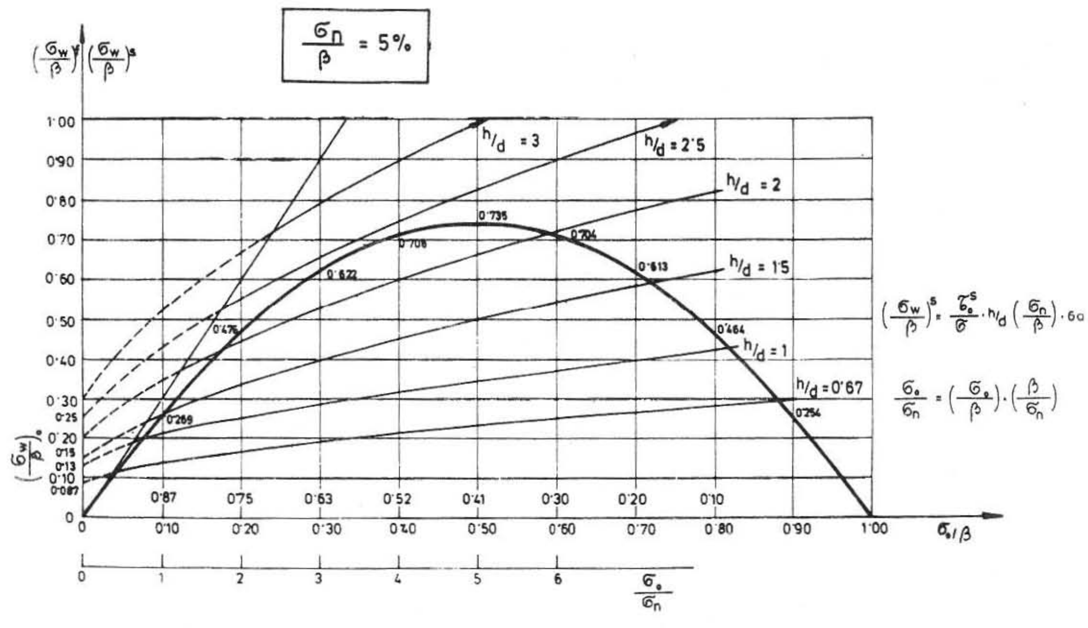

Fig.2 - Interaction diagram

The flexural resistance of wall is defined by means of the flexural interaction diagram. A nondimensional coordinate system is used for the convenience: the abscissa (σ_o/β) represents the vertical load and the ordinate (σ_w/β) the horizontal load. The flexural resistance of wall is expressed by:

$$(3) \qquad \left(\frac{\sigma_W}{\beta}\right) = \frac{M_u \, 6}{F \, d \, \beta} \quad .$$

Supposing the moment inflection point to be at midheight of the wall, the shear force, causing flexural failure, is given by:

(4) $$H_u^f = \frac{(\sigma_w/\beta)\ \beta\ t\ d^3}{3\ h} ,$$

where

t - thickness of the wall.

In the case of the shear failure the ultimate shear

(3) $$H_u^s = \tau_o^s\ F ,$$

causes the bending moment at the lower boundary of the wall:

(4) $$M^s = \tau_o^s\ F\ h\ a.$$

In equation (3), the average shear stress at failure is defined by:

(5) $$\tau_o^s = \frac{\sigma_n}{b} \sqrt{\frac{\sigma_o}{\sigma_n} + 1} ,$$

where the coefficient "b" represents the ratio between the maximum and the average shear stress in the cross-section of the wall and is depending on both geometrical characteristics of wall (height to width ratio h/d)and load conditions (load eccentricity to width of the wall ratio e/d):

(6)
$$b = 1.5 \quad \text{for the} \quad h/d \geqslant 1.5,$$
$$b = 1.54 - 0.48\ (\frac{\tau_o}{\sigma_o}) \ \text{for the}\ h/d = 1.$$

The value of the bending moment depends both upon the value of the shear "H^s" and its arm "h a", i.e. upon the height of the moment inflection point (coefficient "a" defining its position).

The shear resistance is expressed in a nondimensional coordinate system (σ_o/σ_n) and $(\tau_o/\sigma_n)^s$. The ordinate, the shear resistance can also be defined with a bending moment caused by the ultimate, failure shear at the lower boundary of the wall:

(7) $$(\frac{\sigma_w}{\beta})^s = \frac{M^s}{F\ d}\ \frac{6}{\beta} = (\frac{\tau_o}{\sigma_n})^s\ (\frac{\sigma_n}{\beta})\ (\frac{h}{d})\ 6\ a ,$$

which represents the shear resistance of wall in the coordinate system of the flexural interaction diagram. If the abscissa of the shear interaction diagram (σ_o/σ_n) is transformed to:

(8) $$(\frac{\sigma_o}{\beta}) = (\frac{\sigma_o}{\sigma_n})\ (\frac{\sigma_n}{\beta}) ,$$

the shear and the moment resistance can be plotted together in the same diagram.

The tests carried out on 5 different groups of walls have shown [1] that the ratios between the shear and deformation moduli G/D as well as the ratios

between the tensile and compressive strength σ_n/β of the walls are practically contant:

$$G/D = 1/10, \quad \sigma_n/\beta = 0.06 - 0.07 .$$

31 H-δ diagrams were evaluated statistically in order to obtain the ductility ratios δ_{max}/δ_o and $\delta_{H_{max}}/\delta_o$. The tests have shown that there is no significant difference between the 5 groups of walls, even when the quality of mortar widely varied (2 - 24 N/mm^2):

$$\delta_{H_{max}}/\delta_o = 2.4 \quad \text{(standard deviation 0.54)},$$

$$\delta_{max}/\delta_o = 3.2 \quad \text{(standard deviation 0.71)}.$$

For the strength and deformability characteristics of models of masonry buildings, the following values were obtained:

$$\sigma_n = 0.15 \text{ N/mm}^2 ,$$

$$G = 650 \text{ N/mm}^2 ,$$

$$\sigma_n/\beta = 0.065 ,$$

$$G/D = 1/10,$$

$$\delta_{max}/\delta_o = 4.6, \text{ and}$$

$$\delta_{H_{max}}/\sigma_o = 3.5$$

3. TESTS OF SINGLE STORY MASONRY BUILDING MODELS

3.1 Description of 1:2 scale models

The proposed numerical method for construction of the story H-δ diagram has been experimentally verified on two single story masonry building models, subjected to constant vertical and dynamic horizontal load.

The models represented the ground floor of a 5 story masonry building. The models were cast of normal format bricks according to the laws of model similarity. 1:2 modeling scale was used as the most suitable for the testing equipment at disposition in the Institute.

Vertical load of the upper 4 stories was substituted by means of vertical prestressing, and the horizontal load was programmed in the shape of sinusoidal displacements of the floor slab. During the tests, both relative displacements between the foundation and floor slab and the reactive total story shear were measured.

Two different models have been tested:

- model I (Fig .3), composed of two T-shape walls. The cross sectional area of the walls in the direction of horizontal forces represented 32% of the total cross sectional area of all the walls in the story. Vertical load was uniformly distributed onto the walls, the vertical stresses in the walls amounted to $\sigma_o = 0.38$ N/mm^2.

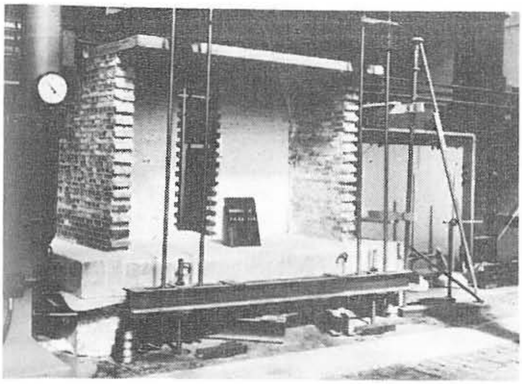

Fig.3 - Model I. during the testing

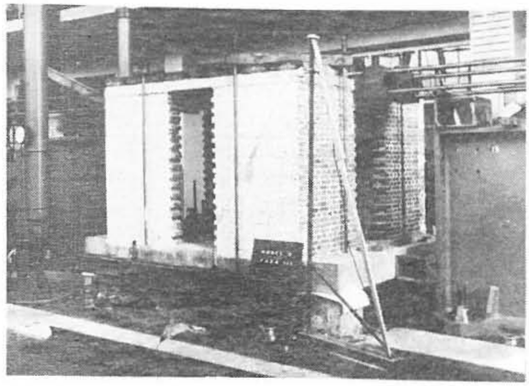

Fig.4 - Model II. during the testing

- model II (Fig.4), composed of four L - shape walls. The cross sectional area of the walls in the direction of horizontal forces represented 45% of the total cross sectional area of the walls in the story. Vertical load was uniformly distributed onto the walls, the vertical stresses in the walls amounted to σ_o = 0.40 N/mm^2.

3.2 Calculation of the story H-δ diagrams

For both models, tested up to the failure, the story H-δ diagrams have been calculated as the summation of the H-δ diagrams of individual walls of each model, taking into account the experimentally obtained values of the strength and deformability characteristics, as given in chapter 2.2.

The theoretical values of the story H-δ diagrams are presented in Tables 1 and 2 for models I and II., respectively.

Table 1 - Calculation of the story H-δ diagram for model I.

Parameter	Unit	Walls in the direction of horizontal force 2 walls	Walls perpendicular of the direction of horizontal force 2 walls
Cross sectional area - F	m^2	0.1700	0.3625
Height to width ratio - h/d	-	1.81	9.80
Vertical stress - σ_o	N/mm^2	0.38	0.38
Shear resistance - $H_{max} = H_u^s$	kN	30.6	-
Flexural resistance -$H_{max} = H_u^f$	kN	-	12.7
Initial stiffness - K_o	kN/mm	59.0	16.0
Idealized elastic limit - δ_o	mm	0.40	0.79
Deformation at H_{max} - $\delta_{H_{max}}$	mm	1.40	-
Ultimate deformation - δ_{max}		1.84	-

Table 2 - Calculation of the story H-δ diagram for model II.

Parameter	Unit	Walls in the direction of horizontal force 4 walls	Walls perpendicular to the direction of horizontal force 4 walls
Cross-sectional area - F	m^2	0.3100	0.3725
Height to width ratio - h/d	-	1.9	9.8
Vertical stress - σ_o	N/mm^2	0.40	0.40
Shear resistance - $H_{max} = H_u^s$	kN	59.0	-
Flexural resistance - $H_{max} = H_u^f$	kN	-	14.0
Initial stiffness - K_o	kN/mm	103.0	16.0
Idealized elastic limit - δ_o	mm	0.75	0.88
Deformation at H_{max} - $\delta_{H_{max}}$	mm	2.00	-
Ultimate deformation - δ_{max}	mm	2.68	-

3.3 Comparison of the results

As it can be seen in Figs.5 and 6, shear failure of the walls in the direction of horizontal loading occurred in the case of both model tests.

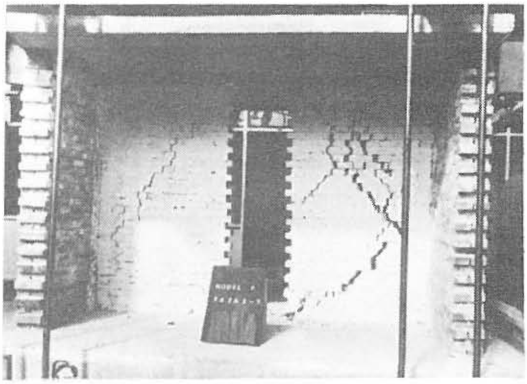

Fig.5 - Failure of model I.

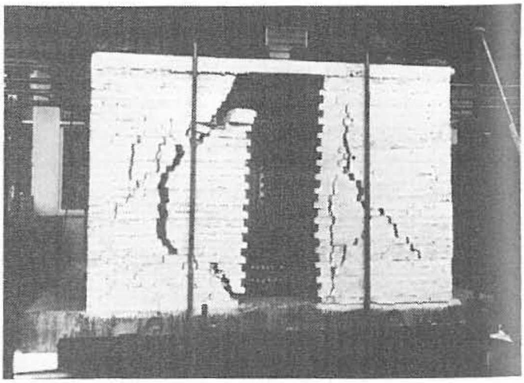

Fig.6 - Failure of model II.

The observed phenomenon was confirmed with calculation also, as it can be seen in Tables 1 and 2.

The test results are presented in the shape of the story H-δ diagrams for each of the two models tested in Figs.7 and 8. In the same Figures, the experimentally obtained H-δ diagrams are compared to the calculated ones, obtained as the summation of the H-δ diagrams of individual walls of each model (see Tables 1 and 2).

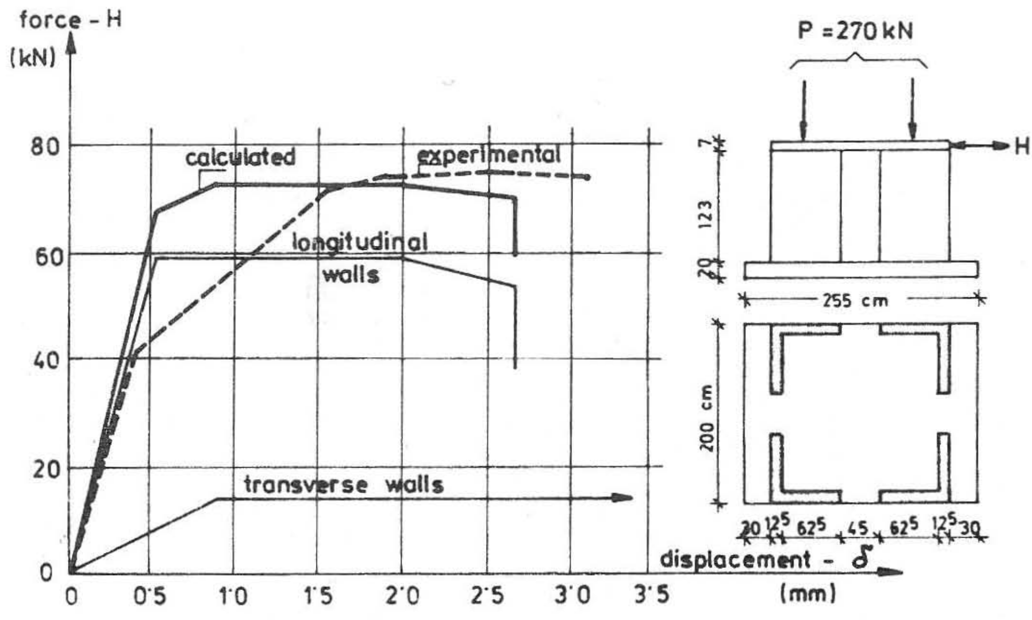

Fig.7 - Comparison of the experimentally obtained and calculated story H-δ diagrams - model I.

As it can be seen, a satisfactory correlation between the experimental and the calculated results has been obtained, especially in the case of model I. The correlation is very good in both cases, comparing the experimentally obtained and calculated values of the story resistance, but is not the best in the case of model II.,comparing the deformability characterstics.

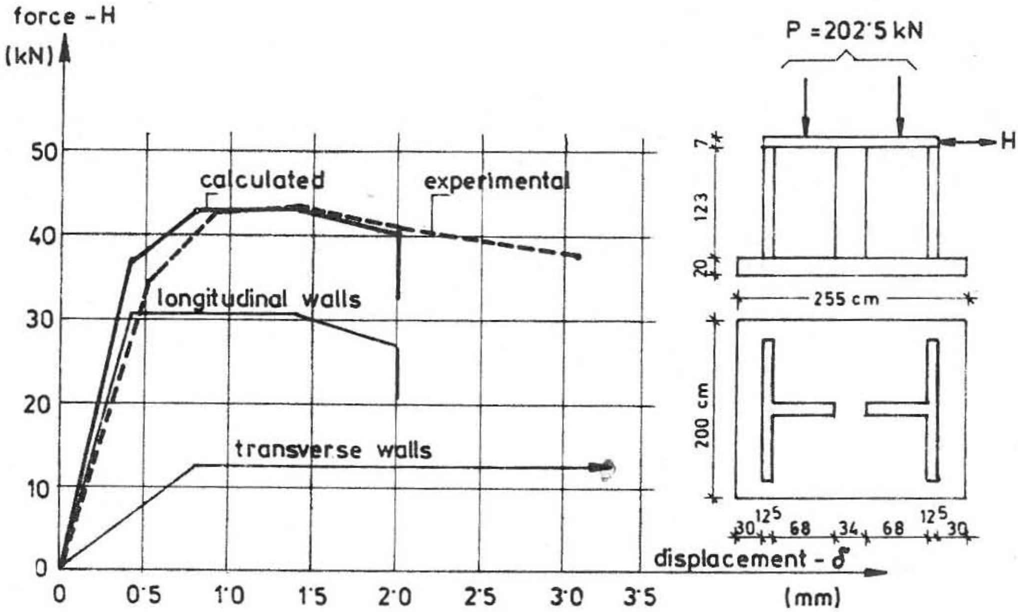

Fig.8 - Comparison of the experimentally obtained and calculated story H-δ diagrams - model II.

4. CONCLUSIONS

With the described model tests, the supposition of the distribution of the story shear to individual walls has been verified: the supposition of treating the walls of L or T shape as being disconnected at vertical joints in both directions and the supposition of distribution of the story shear to individual, disconnected walls according to their stiffnesses.

Taking into account this suppositions, a rather simple method for verification of the seismic resistance of masonry buildings can be elaborated, based on the superposition of the H-δ diagrams of individual walls in the critical story. Of course the method - which is an ultimate state method - can only be used in the case when the shear failure of individual walls defines the failure of the building as a whole.

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