Proposed Method of Distribution of Horizontal Loads on Stiffening Walls

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Abstract Numerical methods are commonly used to determine internal forces in stiffening walls. Extreme internal forces can be easily generated not only in slab-and-wall structure, but also in the bar system. It is always time-consuming to build such a structure, thus it is not used in single- or multi-family buildings with a simple wall arrangement. Then, a simple analytical model can be used to calculate internal forces in walls. Eurocode 6 (prEN 1996-1-1:2017) does not contain specific guidelines to calculate internal forces in walls and to use numerical methods or other reliable methods. This paper presents the procedure for determining internal forces in a building with a simple wall arrangement. The proposed method is based on the division of a wall with openings into components. The results were compared with values of internal forces determined by the linear elastic FE shell model. Rotation centre (RC) of the wall plan was demonstrated to be a significant factor which, besides bending and shear stiffness, had an impact on load distribution.

1. Introduction

According to the definition given in point 3.9.12 of Eurocode [1], a stiffening wall is a wall to resist lateral forces in its plane. The main aim of the stiffening wall is to ensure the geometrical stability of a building, reduce horizontal displacements of structural components, maintain good appearance, and provide the comfort in use. The adequate design of stiffening wall arrangement should eliminate horizontal displacements. Then, supporting walls, piers between openings or columns of frame structures can be dimensioned while neglecting the effect of eccentricity caused by horizontal loads. All supporting walls function as stiffening walls, and other walls can be stiffening provided that the building structure can transmit external horizontal loads [2].

The mentioned standard term of stiffening walls shows that horizontal shear forces caused mainly by wind are the key loads. These loads can be accompanied by vertical loads caused by dead load or imposed loads. Values of vertical loads depend on whether the stiffening wall is also the supporting wall and on the building location. If the stiffening wall is an infill of the reinforced concrete or steel frame, and is connected to this frame with compensatory connectors, additional horizontal forces can occur due to the restrained wall deformations. In the past the design spatial stiffness did not have to be verified in masonry structures, in which supporting walls were thick enough as they also could function as stiffening walls. Nowadays, numerical methods are often used for the design purposes. Time-consumption of FEM can be justified in case of complex buildings. However, FEM seems to be undesirable for single- or multi-family houses with a simple wall arrangement. Then, the analytical method is required to determine easily internal forces. The analytical method also seems necessary to
control the results obtained from numerical calculations. The main aim of this paper is to introduce the methodology of load distribution on stiffening walls in a building with a simple wall arrangement following recommendations presented in Eurocode 6 and other standards which contain more detailed information on this aspect. At first, Eurocode 6 recommendations were presented for stiffening walls. Then, the more detailed specification of the design method was proposed. This procedure was compared with the calculated FE results for the building model with a simple arrangement of walls.

2. Requirements of Eurocode 6

The design principles for stiffening walls are described in chapter 7.5.5 in Eurocode 6 [1]. The wall stiffness was assumed to be determined for the linear elastic model of the material. With relation to the standard provisions for calculating shear walls using the method of boundary conditions, a rectangular diaphragm is used as the design model for the stiffening wall or the wall strip in case of a wall with an opening. A distribution of shear stresses in this diaphragm, which occurs in the compressed part of the wall can be regarded as constant. The presence of openings should be taken into account while determining wall stiffness and values of internal forces. The standard does not specify in details this important issue. However, it can be assumed that the traditional method typical for analysing this type of structure can be also applied in this case. Values of internal forces in the stiffening walls should be determined taking into account interactions with intersecting walls. According to the standard, a length of each intersecting wall, which is regarded as an interacting element (Fig. 1), depend on thickness of the stiffening wall \( t \), the height \( h_{\text{tot}} \) and the distance between other stiffening walls \( l_s \) and the opening height \( h \). These values are determined from the following relationship:

\[
b_{\text{eff}1} = \min \left\{ \frac{h_{\text{tot}}}{5}, \frac{l_s}{2}, \frac{h}{2}, 8 \cdot t \right\}
\]

Figure 1. Thickness of an interacting part of the intersecting wall, which can be taken for the stiffening wall: 1 – part of a wall interacting with the stiffening wall, having a length \( b_{\text{eff}} \), 2 – intersecting wall, 3 – stiffening wall

If the walls perpendicular to the stiffening wall have window or door openings whose dimensions are smaller than \( h/4 \) or \( l/4 \) (\( h, l \) – height and length of the intersecting wall), then the intersecting wall can be considered as the wall without openings. Otherwise, they should be regarded as ends of the intersecting wall when the openings are greater than \( h/4 \) or \( l/4 \). Neglecting the presence of smaller openings is beneficial as stiffness of the stiffening set is higher. However, the calculations can become complicated due to the presence of interacting sections of the wall.

In buildings with floors that are regarded as rigid diaphragms, that is, which have solid reinforced concrete slabs based on walls using ring beams, horizontal loads can be distributed proportionally to the bending stiffness of individual walls. All walls parallel to external load are assumed to be stiffening walls, and displacements of their edges for one floor are the same – Fig. 2a.
3. Proposed method for calculating stiffening walls

3.1. Distribution of loads on stiffening walls

The adequate arrangement of supporting walls which can take the imposed horizontal forces, should provide the required spatial stiffness of the building. The construction system should be designed in such a way to obtain the symmetrical arrangement of stiffening walls. Then, the load distribution on individual walls in a specific direction is similar, and at the same time spatial stiffness of the building is the highest. However, when the building plan is irregular in many cases, the 1-way floors are present or the arrangement of supporting walls is not symmetrical due to the functional properties, horizontal loads can result in torsion of a building and an increased displacement of walls. The standard [1] clearly states that if coordinates of the point, at which the applied external loads – load centre (LC) differ from coordinates of the rotation centre (RC) (specified in EC-6 as stiffness centre) of the plan arrangement of stiffening loads, then the torsional effect should be taken into account. No detailed information has been specified regarding the method of determining the rotation centre of a building or a way to include the torsional effect while providing the freedom of the calculation method. To simplify the calculations, Eurocode assumes that loads can be only taken by walls parallel to external loading, and the load distribution proportional to wall stiffness can only take place in case of rigid floors. Based on this assumption, the integrated system can be divided into separate stiffening sets or even wall strips which interact with parts of intersecting walls.

The place of rotation centre (RC) of the wall plan is determined by dividing the system into single and separate wall strips in the direction of the building length and width [2, 3] Then, stiffness $K_{xi}$ and $K_{yi}$ are calculated for each strip, and distance $a_{xi}$, $a_{yi}$ is determined between their rotation centres and the point, at which imposed loads (LC) are applied. Coordinates of the rotation centre (RC) of the walls are determined from the conditions of force equilibrium and calculated from the following equations:

Figure 2. Stiffening walls in the masonry building with different types of floors: a) solid slabs of a 2-way behaviour considered as rigid diaphragms, b) floors of a 1-way behaviour (ribbed floor, precast floor other than solid slabs, timber-framed floor)
where: \(a_{xi}, a_{yi}\) – distance from the point of applied external load and rotation centres of individual sets or stiffening walls, \(K_{xi}, K_{yi}\) – stiffness of sets or stiffening walls.

The position of the building rotation centre is determined assuming that walls in the length and width direction are not connected. Thus, rotation centres of individual wall strips are in the point of gravity of wall strips. Generally, the preliminary correctness of the assumed stiffening system of the building is assessed at this stage. Fig. 3 illustrates a plan of the building in the regular shape and different arrangements of stiffening walls at constant point of the applied load (LC) and rotation centres (RC) of the walls. In the systems with symmetrical arrangements of walls Fig. 3a there is no eccentricity of loading with reference to the rotation centre of a building, which overlaps the point of applied load. For the unsymmetrical arrangement of walls Fig. 3b, the horizontal force acting at eccentricity causes a significant torsion of the building. However, in the building with irregularly arranged walls Fig. 3c the generated torsional moment is not compensated by shear forces in the walls.

Figure 3. The system of stiffening walls in the building: a) the recommended systems, b) the undesirable systems, c) the unstable systems

Internal forces in individual stiffening sets can be determined when the rotation centre of the building is known. While choosing the stiffening sets, any wall fragments or strips which do not determine the spatial stiffness of the building, can be neglected. Thus, the following elements can be excluded: slender piers between windows, non-structural infill walls that do not interact with the infilling structure, walls or their fragments connected with stiffening walls by connectors, or single masonry columns.

Horizontal forces \(H_x\) and \(H_y\) applied to the floor level at the mid-length of ring beams cause torsion of the building by the moment \(M_{sx} = H_xR\) and \(M_{sy} = H_yR\). Values of shear forces in individual stiffening sets at any floor are the sum of shear forces caused by external forces \(H_x\) and \(H_y\) and the torsional moment \(M_{sx}\) and \(M_{sy}\) are calculated from the following relationships:

- shear forces caused by loads \(H_x\) and \(H_y\):
  \[
  H_{x,i} = H_x \frac{K_{y,j}}{\sum_j K_{y,j}}, \quad H_{y,i} = H_y \frac{K_{x,i}}{\sum_i K_{x,i}},
  \]

- shear forces caused by torsional moments \(M_{sx}\) and \(M_{sy}\) acting on the building
  \[
  H_{sx,i} = \pm M_{sx} \frac{a_{yi}K_{y,j}}{\sum_i \sum_j a_{xi}^2K_{x,i}^2 + \sum_i \sum_j a_{yi}^2K_{y,j}^2}, \quad H_{sy,i} = \pm M_{sy} \frac{a_{xi}K_{x,i}}{\sum_i \sum_j a_{xi}^2K_{x,i}^2 + \sum_i \sum_j a_{yi}^2K_{y,j}^2},
  \]
Bending moments are equal to:

- bending moment under loads \( H_x \) and \( H_y \):
  \[
  H_{x,j} = \pm M_{sx}(\bar{a}_{x,j}K_{x,j}), \quad H_{y,j} = \pm M_{sy}(\bar{a}_{y,j}K_{y,j}),
  \]

- bending moments caused by torsional moments \( M_{sx} \) and \( M_{sy} \) acting on the building:
  \[
  M_{sx,j} = \pm M_{sx}(\bar{a}_{x,j}), \quad M_{sy,j} = \pm M_{sy}(\bar{a}_{y,j}).
  \]

where: \( \bar{a}_{x,j}, \bar{a}_{y,j} \) – distance of centroids of wall strips against the rotation centre RC, \( h_m \) – height of the wall.

A single wall was used to determine stiffness of the wall with openings. It was divided into weaker zones due to the presence of openings and zones of greater stiffness (lintels and spandrel panels) – Fig. 4. Stiffness of the set could be determined knowing displacement of the unit load at the top edge of the wall caused by the concentrated force and the bending moment. The combined displacements include displacement of spandrel panels, piers between openings, and lintels. The total displacement of the wall is defined by the following relationship:

\[
\Delta_w = \Delta_w^A + \Delta_w^B + \Delta_w^P
\]

where: \( \Delta_w^A, \Delta_w^B, \Delta_w^P \) – displacement of spandrel panel and lintel, \( \Delta_w^P \) – displacement of part of the wall with openings of height \( h_0 \) and length \( l \).

Displacements of wall components are determined from the relationship between geometry and boundary conditions for the wall. If the ratio of the wall height to length is \( h/l > 2 \), then the effect of shear stresses can be neglected while determining the wall stiffness. Otherwise, the wall stiffness should be determined with the included shear strains. Stiffness values of the walls fixed in a different way and at different size ratio, whose ends are loaded with concentrated force are shown in Table 1.

**Figure 4.** Method for determining the total stiffness of the stiffening set: a) the wall with openings divided into components, b) wall deformation under horizontal loads

After determining the total displacement of the wall, its stiffness is calculated from the following relationship:

\[
K_w = \frac{1}{\Delta_w^P}
\]

where: \( \Delta_w^P \) – total displacement of the top edge of the wall under the unit load \( P=1 \).
Table 1. Stiffness of walls subjected to shearing and bending [4, 5, 6]

| Wall scheme                  | $h/l \leq 2$ | $h/l \geq 2$ |
|------------------------------|-------------|-------------|
|                              | Force $P$  | Moment $M$  | Force $P$  | Moment $M$  |
| Cantilever wall W type       | $K_P = \frac{1}{h_m^3} + \frac{1.2h_m}{3EI} + \frac{1}{GA}$ | $K_M = \frac{2EI}{h_m^3}$ | $K_P = \frac{3EI}{h_m^3}$ | $K_M = \frac{2EI}{h_m^2}$ |
| Wall fixed at both ends OU type | $K_P = \frac{1}{h_m^3} + \frac{1.2h_m}{12EI} + \frac{1}{GA}$ | -- | $K_P = \frac{12EI}{h_m^3}$ | -- |

4. Numerical example

The calculations were conducted for a one-storey masonry building made of autoclaved aerated concrete units with the elasticity modulus $E=2041$ N/mm$^2$, shear modulus $G=475$ N/mm$^2$, and they were based on the author’s own tests [7,8,9]. The building plan had a square shape with the outer size $B = L = 4.0$ m and the wall thickness $t_1=t_2=t_A=t_B=0.18$ m. The reinforced concrete floor of thickness $h_s = 0.16$ m, supported on the ring beam with a height $h_w = 0.22$ m was made for the walls with a total height $h_m = 2.4$ m. A door opening of $L_0=1.0$ m in width and $h_0 = 1.92$ m in height was made in the wall along the axis A. The piers between windows had the same length $l_{1C} = l_{1D} = 1.32$ m. The walls were placed on the bottom ring beam of 0.22 m, made of reinforced concrete. The construction was loaded along the floor axis with the force $H_x=200$ kN. Geometry of the design building is presented in Fig. 5.

![Figure 5. Geometry of the design building: a) layout plan, b) fragment of the wall with an opening along the axis A](image)
At first, widths of interacting walls were determined, and then moments of inertia (along with effective lengths) and areas. The next determined value was the wall stiffness $K$. Table 2 contains the key geometrical parameters of the building walls.

### Table 2. Geometrical parameters and stiffness of the building walls

| wall or its component | Moment of inertia $I$, m$^4$ | Area $A$, m$^2$ | Wall model | Distance between the gravity centre of the wall and the LC point $a$, m | Wall stiffness $K$, MN/m |
|----------------------|-----------------------------|----------------|------------|--------------------------------|---------------------|
| 1                    | $I_1 = 1.59$ m$^4$          | 4.18 m$^2$     | OU         | $a_{a1} = -1.91$ m            | $K_{1,1} = 114.0$ MN/m |
| 2                    | $I_2 = 1.59$ m$^4$          | 4.18 m$^2$     | OU         | $a_{a2} = 1.91$ m            | $K_{1,2} = 114.0$ MN/m |
| A                    | $I_1 = 0.09$ m$^4$          | 0.09 m$^2$     | W          | $a_{aA} = 1.91$ m            | $K_{1,1} = 32.4$ MN/m |
| D                    | $I_2 = 0.09$ m$^4$          | 0.09 m$^2$     | W          | $a_{aD} = -1.91$ m           | $K_{1,2} = 592.8$ MN/m |
| A'                   | $I_A = 1.59$ m$^4$          | 4.18 m$^2$     | OU         | $a_{aA} = -1.91$ m           | $K_{1,1} = 140.0$ MN/m |
| B                    | $I_B = 1.59$ m$^4$          | 4.18 m$^2$     | OU         | $a_{aB} = -1.91$ m           | $K_{1,2} = 140.0$ MN/m |

The equation (1) was used to determine coordinates of the rotation centre of the building $x_R=0.0$ m, $y_R= -0.61$ m. Values of internal forces in individual walls were obtained from the equations (2)–(5). The obtained results are presented in Table 3.

### Table 3. Results for internal forces in the building walls obtained from the analytical model

| wall | $H_{x1}$, kN | $H_{x2}$, kN | $H_{y1}$, kN | $H_{y2}$, kN | $H_{x1} = (H_{x1} + H_{x2})$, kN | $H_{y1} = (H_{y1} + H_{y2})$, kN | $M_{x1}$, kNm | $M_{x2}$, kNm | $M_{y1}$, kNm | $M_{y2}$, kNm | $M_{x1} = (M_{x1} + M_{x2})$, kNm | $M_{y1} = (M_{y1} + M_{y2})$, kNm |
|------|--------------|--------------|--------------|--------------|---------------------------------|---------------------------------|----------------|----------------|----------------|----------------|---------------------------------|---------------------------------|
| 1    | --           | --           | -19.2        | --           | -19.2                           | 0                               | 0              | --             | 48.1           | --             | 48.1                             | 48.1                             |
| 2    | --           | --           | 19.2         | --           | 19.2                            | 0                               | 0              | --             | -48.1          | --             | -48.1                            | -48.1                            |
| A    | -76.8        | -13.0        | -80.8        | --           | -170.3                          | 0                               | -32.6          | --             | 202.9          | --             | 202.9                            | --                               |
| B    | -132.2       | 13.0         | --           | -119.2       | --                              | -331.7                         | 32.6           | --             | -299.1         | --             | -299.1                           | --                               |

Apart from the analytical approach, the numerical FE model was developed for the analysed building. Four-node finite shell elements were used with six degrees of freedom for each node. The model was placed on hinge supports at each node in the bottom edge of the wall. The bottom ring beam was not taken into account. The finite elements representing the walls took the floor and bond beams having parameters of the AAC masonry and C20/25 concrete. Numerical FEM models are shown in Fig. 6, Tab. 4.

![Figure 6](image-url)

**Figure 6.** Numerical FE model: a) model drawing, b) deformed model; 1 – wall made of AAC masonry units, 2 – ring beam, 3 – floor slab
Table 4. Results for internal forces in the building walls obtained for the FE model

| wall | \( H_x^{\text{MES}} = (H_{x,i} + H_{x,0}), \) kN | \( H_y^{\text{MES}} = (H_{y,i} + H_{y,0}), \) kN | \( M_x^{\text{MES}} = (M_{x,i} + M_{x,0}), \) kNm | \( M_y^{\text{MES}} = (M_{y,i} + M_{y,0}), \) kNm | \( H_x', \) kN | \( H_y', \) kN | \( M_x', \) kNm | \( M_y', \) kNm |
|------|-----------------|-----------------|-----------------|-----------------|-------|-------|-------|-------|
| 1    | -15.6           | --              | 36.9            | --              | 1.22  | 1.30  | --    | --    |
| 2    | 15.6            | --              | -36.9           | --              | 1.22  | 1.30  | --    | --    |
| A    | -84.4           | --              | 184.3           | --              | 0.96  | 1.10  | --    | --    |
| B    | -115.6          | --              | 323.7           | --              | 1.03  | 0.92  | --    | --    |

By knowing shear forces in the individual walls, the rotation centre of the FEM model could be determined. Assuming that the torsion centre is symmetrical to the vertical axis Y and unsymmetrical to the horizontal axis X, the moment equation for the RC point can be used:

\[
H_x^{\text{MES}} y_R^{\text{MES}} = H_{x,A}(\alpha_{YA} + y_R^{\text{MES}}) + H_{x,B}(\alpha_{YA} - y_R^{\text{MES}}) + H_{y,1} \alpha_{X1} + H_{y,2} \alpha_{X2}.
\]

(8)

Coordinate of the rotation centre of the FE model was \( y_R^{\text{MES}} = -0.35 \) m.

5. Discussion
The coordinate of the rotation centre RC of the building was determined with the analytical method and was -0.61 m which corresponded to ca.15% of the wall length and was greater than the conventional limit of 5% for the side length, at which the torsional effect can be neglected. The coordinate of the rotation centre in the FE model was 0.35 m and was lower by ca. 42% than the coordinate determined by the analytical method.

The values of shear forces acting on individual walls were determined by the analytical method in accordance with Eurocode 6 [1] and then compared with the numerical FE model. Shear forces were overestimated by ca. 22% in unloaded walls 1 and 2 parallel to the axis Y. However, forces in the loaded walls A, B parallel to the axis X, which were calculated using the FE model, did not considerably differ from the forces determined by the analytical method. Close test results were also obtained for bending moments in the loaded walls 1 and 2. The bending moments in total differed by more than 30%. The smallest difference in total bending moments was found for loaded walls. In the wall A weakened by an opening, the bending moments were overestimated by more than 10% using the analytical method. This method resulted in an over 8% underestimation of the bending moment along the axis of the wall B. Although the greatest difference between the analytical and FE models was obtained for the rotation centre of the building, the values of internal forces in the walls in the direction of the applied loads did not significantly differ. The biggest difference was found in the unloaded walls, in which the force value only depended on the position of the rotation centre. A noticeable difference was also demonstrated for the bending moments in the walls A and B in the direction of the applied load. In this case not only the effect of a difference in position of the rotation centre, but also the effect of wall stiffness was important.

6. Conclusions
This paper proposes the analytical model to determine internal forces in the stiffening walls in accordance with the Eurocode 6 draft [1]. The model included the standard recommendations on determining the geometry of walls and effective length of walls perpendicular to the stiffening walls. Stiffness of the stiffening walls without openings was suggested to be determined using the traditional model of the cantilever or fixed bar taking into account shear deformations [4, 5, 6]. The method of dividing the wall into components, as in the standards [10, 11] was proposed to calculate stiffness of the stiffening wall with openings. Stiffness of the wall with openings could be determined using this method on the basis of total displacement of the wall. The internal forces in the wall could be
determined only if the rotation centre of the building is localised, and its coordinates are obtained from the equilibrium equations. Also, relevant relationships were proposed to determine values of shear forces and bending moments in each wall.

This method was verified on a specific example. A model of the building with a simple arrangement of walls was used in the calculations. A door opening was performed in one wall. The model was loaded only with one force parallel to the wall with an opening. The mechanical parameters of the wall made of autoclaved aerated concrete units were taken on the basis of the author’s own tests [7, 8, 9]. Apart from the analytical approach, also the numerical FE model was developed for the linear-elastic stage. The calculations demonstrated that the highest consistency of the results was obtained for the coordinate of the rotation centre of the building. The FEM calculations indicated that the rotation centre RC was closer to the load point than in case of the analytical model. A change in the position of the rotation centre significantly affected the values of shear forces in the walls that were not along the axis of the load. Forces greater by ca. 22% were obtained from the FE model compared to the analytical method. A difference in forces did not exceed 4% in the walls in the direction of the applied load. However, some differences were identified in the bending moments. The FEM-calculated bending moments in the wall with an opening were smaller by 10% compared to the values determined by the analytical method. And the moment was overestimated by ca. 8% in case of the wall without an opening. To sum it up, the proposed method can be applied to the buildings with a relatively simple arrangement of walls to:

a) estimate safely internal forces in the walls which stand in the direction of the applied load,

b) underestimate internal forces in the walls perpendicular to the direction of the applied load,

c) estimate safely bending moments in the walls which are weakened by the presence of openings.

This method obviously requires further analyses to:

a) develop the methodology for various ways of connecting walls,

b) include varied conditions for support and load,

c) take into account changes in the wall stiffness caused by developing cracks,

d) calculate safely displacement of walls to verify ultimate limit states (ULS).

Experimental tests are currently performed on the presented model of a building to verify the proposed method. These tests are conducted in the Laboratory of Civil Engineering, Silesian University of Technology.

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