Verifying the Shear Load Capacity of Masonry Walls by the $V_{Rd} - N_{Ed}$ Interaction Diagram

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Abstract. Verification of shear load capacity is required for all shear walls that take horizontal wind loads, loads imposed by ground action or other non-mechanical (rheological or thermal) loads. Shear walls are exposed not only to shear forces, but also vertical actions caused by dead load or imposed loads as shear walls also usually function as bearing walls. This load combination is quite important as shear load capacity $V_{Rd}$ depends on mean design stresses $\sigma_d$ which, in turn, depend on design forces $N_{Ed}$. Interactions between shear $V_{Rd}$ and vertical load $N_{Ed}$ in shear walls are the consequence of observed combinations of actions in these types of walls. Additionally, the vertical load $N_{Ed}$ acts on the wall at certain eccentricity $e_{Ed}$, which can result in a change in the length of the compressed part of the cross-section $l_c$. This paper describes the procedure for verifying shear load capacity by means of the interaction diagram drawn as specified in Eurocode 6 (prEN 1996-1-1:2017). Necessary equations for determining load-carrying capacity of cross-section against vertical load $N_{Ed}$ were worked out. The effect of wall shape and eccentricity of vertical load on the shape of the interaction diagram was analysed.

1. Introduction

According to the definition given in point 3.9.10 of Eurocode [1], a shear wall is a wall to resist forces in its plane. The standard neither differentiates between factors that can be responsible for shearing nor specifies them. The design practice distinguishes between horizontal and vertical shear. Horizontal shear is usually caused by seismic actions, wind, sometimes by thermal or rheological effects and actions of other buildings. On the other hand, vertical shear is mainly related to non-uniform vertical displacements of ground below a building and floor resistance. Despite the common occurrence of both shear types, Eurocode 6 specifies relevant criteria and procedures for verifying load capacity of walls subjected to only vertical shear.

Requirements covered by Eurocode 6 are relatively simple. The verification of ULS can be troublesome, mainly due to the connection between shear resistance and acting loads. This paper describes the procedure for verifying shear load capacity of masonry walls using the diagram of interactions $V_{Rd} - N_{Ed}$. The fundamental aim of this work is to give an insight into the methodology of verifying shear load capacity of any wall cross-section, which can be easily implemented into traditional spreadsheets. This method was developed to verify load capacity of masonry walls in response to the trend of continuous reduction in thickness and introducing new technologies for preparing walls. As a preliminary point, the basic issues of shear given in Eurocode 6 were presented (neglecting the issues concerning the methodology of specifying shear strength parameters of the wall.
and conditions for its construction) in terms of verifying the ultimate limit states (ULS). Then, the principles of determining design internal forces in the wall were discussed. The final issue focused on specific cases of shear wall loading and resulting distribution of compressive stresses.

2. Requirements of Eurocode 6

2.1. Principles of method

The method of verifying load capacity of shear walls is described in chapter 8.3.1 in Eurocode 6 [N23]. Despite the fact that it has not been clearly specified in the standard, this method should include additional concepts of geometry and load presented in Table 1.

Table 1. Principles of method for verifying load capacity of unreinforced walls subjected to horizontal shearing

| Parameter                  | Principles acc. to prEN 1996-1-1:2017 [1] |
|----------------------------|---------------------------------------------|
| wall structure             | any                                         |
| wall cross-section         | rectangular, without the tensile area of the wall |
| eccentricity $e_{Ed}$ of load $N_{Ed}$ | any                                     |
| stress distribution        | trapezoidal or triangular                  |

These principles show that the method can be applied only to walls with a rectangular cross-section. It is not adequate for T- or I-shaped walls which are included in calculations made for stiffening walls. According to general principles specified in the standard, the tension zone is taken as zero for calculation, and the distribution of normal stresses is linear. The standard procedure admits compression of the whole cross-section when the resultant of vertical load is in the core or compression only of its part when the force is outside the cross-section core. Further, the standard method specified in Eurocode 6 is described in details.

2.2. Wall model

The stiffening wall was exposed to horizontal actions and also vertical loads as a result of constant or serviceability loading conditions. The eccentricity of vertical load resulted in bending in the plane of the wall due to the presence of openings Superpositioning of internal forces caused by wind and vertical loads could result in a monotonous increase in bending moments when the wall worked in the cantilever system – Fig. 1a. On the other hand, bending moments in the wall changed behaviour of the wall in the partly restrained system – Fig. 1b. Axial forces in both cases increased proportionally to the building height – Fig. 1c, and shear forces were derivatives of bending moments – Fig.1d.

Eurocode 6 specifies that a design model is a rectangular diaphragm, whose bottom and top edges are loaded with the design value of shear load $V_{Ed}$ with the same values but opposite directions, and the system of vertical loads $N_{Ed}$ acting under different eccentricity values applied to the bottom and top edge. External forces acting on the wall can be reduced to bending moments applied to the top $M_{Ed,1}$ and bottom $M_{Ed,2}$ edges of axial loads $N_{Ed}$ from the centre line of the wall and horizontal shear forces applied to geometrical load centre of the wall. Depending on the eccentricity value $e_{Ed}$ of load $N_{Ed}$ acting on the centre of the wall cross-section, the resulting bending moments can take different values and directions. Wall behaviour is based on the cantilever system when the bending moment under the vertical load increases the bending moment under the wind load (Fig. 2a). This situation occurs when $e_{Ed}>0$. Otherwise, when the bending moment under vertical loading reduces the bending moment under horizontal loading, partly restrained static scheme of the wall is observed, and the eccentricity value is lower than zero $e_{Ed}<0$ – Fig. 2d. Similar assumptions for the eccentricity of loading the top edge of the wall were proposed in German National Annex to DIN EN 1996-1-1/NA [2,3,4].
Figure 1. Possible models of walls subjected to horizontal shear loading: a) diagram of bending moments in the cantilever wall, b) diagram of bending moments in partially restrained wall, c) diagram of axial loads, d) diagram of lateral forces.

Figure 2. Design model for shear load specified in Eurocode 6: a) load applied to the cantilever model ($V_{Ed}>0$, $e_{Ed,1}>0$), b) the reduced system, c) diagram of bending moments, d) load applied to partly restrained model ($V_{Ed}<0$, $e_{Ed,1}>0$), e) the reduced system, f) diagram of bending moments; 1 – triangular diagram of compressive stresses, 2 – trapezoidal diagram of compressive stresses, 3 – tension zone neglected in calculations.

The standard [1] principle is that the design shear load applied to the masonry load $V_{Ed}$ in the ultimate limit state should not exceed the design value of shear load capacity $V_{Rd}$ or the limit value $V_{Rdlt}$:

$$V_{Ed} \leq V_{Rd} = f_{vd} \cdot t$$

or

$$V_{Ed} \leq V_{Rdlt}$$

where: $f_{vd}$ – design shear strength of masonry determined as specified in point 2.4.1, $t$ – thickness of the wall, $l_c$ – length of the compressed part of a wall.

The calculations do not include parts of the wall cross-section, in which tensile stresses occur. A length of the compressed part of the wall, for which the resultant of shear stresses is $V_{Ed}$ should be...
determined assuming the linear distribution of stresses and including openings and inclined chases. These general recommendations indicate that the wall can be considered as a rectangular diaphragm of constant thickness \( t \) and the height-to-length ratio \( h/l > 0.4 \), for which values of normal and shear stress can be determined using the traditional methods based on the elasticity theory for bar elements [5, 6]. It means that the flatness hypothesis for cross-sections under horizontal and vertical loading is accepted. For even slenderer walls \( h/l \leq 0.4 \), the elements of the stress state calculated on the basis of traditional rules, including the principle of flat cross-sections, can significantly differ from the real state. Thus, the determined load capacity of a wall can be dangerously overestimated.

2.3. Shear strength

The linear Coulomb-Mohr relationship is considered as the failure criterion in Eurocode. The characteristic strength of walls with all joints based on general mortar, thin joint mortar (1-3 mm thickness), and lightweight mortar depends on values of pre-shear stress and is calculated from the following equation:

\[
f_{vk} = f_{vk0} + \mu f_{d} \leq f_{vlt} = 0.065 f_{b},
\]

where: \( f_{vk0} \) – characteristic shear strength under zero compressive stress,
\( \mu \) – characteristic friction coefficient of mortar in bed joints,
\( f_{d} \) – design value of compressive stresses provided that the relevant combination of loads is based on mean vertical stresses above the compressed part of wall, where shear strength is verified;
\( f_{b} \) – normalised compressive strength of masonry units, for the direction of application of the load on the test specimens being perpendicular to the bed face.

For the walls with unfilled perpend joints, but with adjacent faces of the masonry units, shear strength can be taken from the following relationship:

\[
f_{vk} = 0.5 f_{vk0} + \mu f_{d} \leq f_{vlt} = 0.045 f_{b},
\]

Eurocode 6 allows the construction of shell bedded walls, in which masonry units are bedded on two or more strips of general purpose, each at least 30 mm. Shear strength can be taken from the following equation:

\[
f_{vk} = \frac{g}{t} f_{vk0} + \mu f_{d} \leq f_{vlt} = 0.045 f_{b},
\]

where:
\( g \) – total width of the mortar strips, \( t \) – wall thickness.

2.4. Ultimate limit states under shearing and bending

The equations (2) – (4) with limitations specify the wall resistance to horizontal shearing, which after making some generalizations and substituting to (1) can be used to calculate wall resistance from the following equation:

\[
V_{Ed} \leq V_{kd} = \min \left\{ \frac{(k f_{vk0} + \mu f_{d}) t l_c}{\gamma_M}, \frac{f_{vlt} t l_c}{\gamma_M} \right\},
\]

where: \( k \) – coefficient that considers the effect of filling perpend joints with mortar and the presence of shell bedded units, \( k = 1.0 \) for head joints filled with mortar, \( k = 0.5 \) for unfilled head joints, \( k = g/t \) for shell bedded units and head joints filled with mortar, \( k = 0.5 g/t \) for shell bedded units and unfilled head joints, \( f_{vlt} \) – ultimate shear strength of the wall determined which depends on filling of head joints.

The standard assumption indicates that the cracked cross-section at \( l_c < l \) and the uncracked cross-section at \( l_c = l \) should be analysed separately. Thus, a key issue is to determine values of the external
force couple \( V_{\text{Ed}} - N_{\text{Ed}} \) for the analysed cross-section, which can result in cracking. Assuming that stresses at any cross-section at a distance of \( \xi \) with reference to the top edge at the tension edge are equal to \( \sigma = 0 \), and at the compression edge \( \sigma \leq f_{d} \) horizontal shear load that cause cracking of the wall cross-section can be calculated from the following relationship:

\[
V_{\text{Ed,cr}} = \pm \frac{1}{6} \frac{1}{l} \left[ (N_{\text{Ed,1}} + N(\xi)) + 6N_{\text{Ed,1}}E_{\text{Ed,1}} \right].
\]

If \( V_{\text{Ed}} \leq V_{\text{Ed,cr}} \), only compressive stress occurs in the wall cross-section. No cracks reducing the length of the compressed wall area are observed. For the calculated horizontal load \( V_{\text{Ed}} \geq V_{\text{Ed,cr}} \), load strength should be verified on the basis of the triangular diagram of compressive stresses in the cracked cross-section. The equation (6) is simplified to verify only conditions for eccentricity values for the top cross-section of the wall. Cracking of the cross-section occurs only when \( |e_{\text{Ed,1}}| > l/6 \), and normal stresses have a triangular distribution of compressive stress. Otherwise, the whole top edge is compressed, and the stress distribution is trapezoidal when the vertical compressive force is in the core of the cross-section.

2.4.1. Shear action in uncracked cross-section

Assuming that the length of the compressed zone of the cross-section is \( l_{c} = l \) and developing the equation (1), the following relation is obtained:

\[
V_{\text{Rd}} = \min \left\{ \begin{array}{l}
V_{\text{Rd,1}} = \frac{+f_{Vd}l}{l} + \mu_{l} \left( N_{\text{Ed,1}} + N(\xi) \right), \\
V_{\text{Rd,\max}} = \frac{f_{Vd}l}{\gamma_{M}},
\end{array} \right.
\]

Shear load capacity of the wall \( V_{\text{Rd,1}} \) is described by the linear function that depends on the resultant of vertical load \( N_{\text{Ed,1}} + N(\xi) \) with the limitation to the value \( V_{\text{Rd,\max}} \), which is independent of external compressive load.

2.4.2. Shear action in cracked cross-section

After cracking in the cross-section, the linear distribution of compressive stress is observed. Thus, the equation (7) has two unknowns: shear load capacity \( V_{\text{Rd}} \) and the length of the compressed zone of the cross-section \( l_{c} \), which should be determined at first. After taking into account the conditions for equilibrium between forces and moments against the mid-length of any cross-section of the wall shown in Fig. 2a,d the following equation is obtained:

\[
l_{c} = \frac{l_{c}}{2} - \frac{\pm V_{\text{Rd}}l_{c} \pm N_{\text{Ed,1}}E_{\text{Ed,1}}}{N_{\text{Ed,1}} + N(\xi)}. \]

When the relationship (8) is brought into the equation (5), the following form of wall load capacity is obtained:

\[
V_{\text{Rd}}\gamma_{M} = \min \left\{ \begin{array}{l}
V_{\text{Rd,1}} = \pm \left\{ 3f_{Vd} \left( \frac{l_{c}}{2} - \frac{\pm V_{\text{Rd}}l_{c} \pm N_{\text{Ed,1}}E_{\text{Ed,1}}}{N_{\text{Ed,1}} + N(\xi)} \right) + \mu_{l} \left( N_{\text{Ed,1}} + N(\xi) \right) \right\}, \\
V_{\text{Rd,\max}} = \pm 3f_{Vd} \left( \frac{l_{c}}{2} - \frac{\pm V_{\text{Rd,\max}}l_{c} \pm N_{\text{Ed,1}}E_{\text{Ed,1}}}{N_{\text{Ed,1}} + N(\xi)} \right).
\end{array} \right.
\]

2.4.3. In-plane bending of the wall in cracked cross-section

To calculate values of normal stresses in the analysed wall cross-section, the maximum values of compressive stress at the most compressed edge should not be exceeded both in cracked and
uncracked cross-sections. Thus, the following condition has to be satisfied for the cross-section with the triangular diagram of compressive stresses:

\[
\sigma_{1,2} = \frac{2(N_{Ed,1} + N(\xi))}{3\left( \frac{l}{2} - \frac{\pm V_{Rd,1} \pm N_{Ed,1} e_{Ed,1}}{N_{Ed,1} + N(\xi)} \right)} \leq f_d. \tag{10}
\]

If stress values obtained from the equation (11) are higher than \( f \), the ultimate limit state under bending has to be considered to be exceeded. Assuming the triangular distribution of stresses in the cross-section and the limited stresses at the most compressed edge, the equation for the horizontal shear force \( V_{Rd,m,1} \) can be obtained.

2.4.4. In-plane bending of the wall in uncracked cross-section

For the trapezoidal distribution of compressive stresses, the condition of maximum stresses should be met at both edges. Stress values are determined as the eccentricity of the compressed rectangular cross-section using the following relationship:

\[
\sigma_{1,2} = \frac{N_{Ed,1} + N(\xi)}{t l} \pm \frac{6(V_{Ed,cr} \pm N_{Ed,1} e_{Ed,1})}{t l^2} \leq f_d. \tag{11}
\]

Assuming the trapezoidal in shape diagram of compressive stresses in the cross-section and the limited stress at the edge compressed to \( f \), the horizontal shear force against vertical load \( N_{Ed,1} \) is defined as \( V_{Rd,m,2} \). If the rectangular diagram of compressive stress is assumed for the cross-section, and the stress values are equal to the compressive strength of the wall \( f \), then the limit horizontal load is defined as \( V_{Rd,m,\text{max}} \). Even if normal stresses in the bottom edge of the wall are proved not to exceed the design compressive strength of the wall \( f \), which means that wind loading is lower than the values calculated from the relationship (9), the wall load capacity cannot be regarded as sufficient. Depending on the interactions between the moment and horizontal and vertical loads in the wall cross-section, the bending moment and thus, stresses can be lower than the moment at the top edge \( M_{Ed,1} \). This situation occurs when the bending moment at the top edge \( M_{Ed,1} \) is reduced by the moment caused by wind. Due to the combined effect of vertical and horizontal actions, the moment in any cross-section of the wall can be lower than the moment \( M_{Ed,1} \) or can even have the opposite direction. Thus, the verification should include not only the ultimate limit state for any cross-section of the wall subjected to bending, but also compressive stresses at the top edge of the wall.

3. Diagram of \( V_{Ed} - N_{Ed} \) interactions

The diagram of \( V_{Ed} - N_{Ed} \) interactions is much more applicable in practice as it can illustrate ultimate limit curves with reference to shear and bending and these interactions can be compared with calculated values of the force couple \( (V_{Ed}; N_{Ed}) \). This diagram can be drawn for any cross-section of the wall. However, it usually refers to the top (1-1) and bottom (2-2) cross-sections. This diagram requires values of maximum calculated forces \( V_{Ed} \) which are presented against the calculated vertical loads \( N_{Ed} \). In case of the top edge of the wall, the in-plane bending strength does not depend on the horizontal load. Thus, the moment \( M_{Ed} \) is used instead of the horizontal force \( V_{Ed} \). Table 2 presents the equations required to form the positive branch of the interaction diagram and the graphical illustration of the interaction diagram is presented in Figs. 3 and 4.
Table 2. Equations of the positive branch of the interaction diagram ($V_{Ed}>0$, $e_{Ed,1}>0$)

| Condition | Equation | Applicable to the range |
|-----------|----------|-------------------------|
| **TOP EDGE OF WALL** | | |
| **Shearing** | | |
| Uncracked cross-section | $V_{Rd1} = \frac{kf_{ck,0}l + \mu_1 N_{Ed,1}}{y_M}$ | $|e_{Ed,1}| < l/6$ |
| | $V_{Rd1,\text{max}} = \frac{f_{ck}l}{y_M}$ | |
| Cracked cross-section | $V_{Rd2} = \frac{3kf_{ck,0}\left(\frac{l}{2} - e_{Ed,1}\right)}{y_M} + \frac{\mu_1 N_{Ed,1}}{y_M}$ | $|e_{Ed,1}| \geq l/6$ |
| | $V_{Rd2,\text{max}} = \frac{3f_{ck}l}{y_M}\left(\frac{l}{2} - e_{Ed,1}\right)$ | |
| **Bending** | | |
| Uncracked cross-section | $M_{Rd,m,1} = \pm \frac{1}{6}\left(f_{d}t^{2} - N_{Ed,1}\right) + N_{Ed,1}e_{Ed,1}$ | $|e_{Ed,1}| < l/6$ |
| Cracked cross-section | $M_{Rd,m,1} = \pm N_{Ed,1}\left(\frac{l}{2} - \frac{2N_{Ed,1}}{3f_{d}}\right) + N_{Ed,1}e_{Ed,1}$ | $|e_{Ed,1}| \geq l/6$ |
| **BOTTOM EDGE OF WALL** | | |
| **Shearing** | | |
| Uncracked cross-section | $V_{Rd1} = \frac{kf_{ck,0}l + \mu_1 N_{Ed,2}}{y_M}$ | $V_{Ed} \leq V_{Ed,cr}$ |
| | $V_{Rd1,\text{max}} = \frac{f_{ck}l}{y_M}$ | |
| Cracked cross-section | $V_{Rd2} = \left[0.5 - N_{Ed,1}e_{Ed,1} + \frac{\mu_1 N_{Ed,2}}{N_{Ed,1}l}\right] + \frac{N_{Ed,1}e_{Ed,1}}{N_{Ed,1}l}$ | $V_{Ed} > V_{Ed,cr}$ |
| | $V_{Rd2,\text{max}} = \left[\frac{y_M}{3f_{ck}l} + \frac{h}{N_{Ed,2}l}\right]$ | |
| **Bending** | | |
| Uncracked cross-section | $V_{Rd,m,2} = \frac{1}{h}\left[f_{d}t^{2} - \left(N_{Ed,1} + N_{Ed,2}\right) - N_{Ed,1}e_{Ed,1}\right]$ | $V_{Ed} \leq V_{Ed,cr}$ |
| Cracked cross-section | $V_{Rd,m,1} = \frac{1}{h}\left[N_{Ed,2}\left(\frac{l}{2} - \frac{2N_{Ed,2}}{3f_{d}}\right) - N_{Ed,1}e_{Ed,1}\right]$ | $V_{Ed} > V_{Ed,cr}$ |
| Cracked cross-section with rectangular diagram of compressive stresses | $V_{Rd,m,\text{max}} = \frac{1}{h}\left[N_{Ed,2}\left(\frac{2}{2} - \frac{N_{Ed,2}}{f_{d}}\right) - N_{Ed,1}e_{Ed,1}\right]$ | $V_{Ed} > V_{Ed,cr}$ |
As a result of the in-plane bending of the wall, the ultimate limit state envelope is composed of the parabola \(V_{Rd,m,1}\) that defines bending in the cross-section for the triangular diagram of compressive stresses and the straight line \(V_{Rd,m,2}\) describing the trapezoidal distribution of compressive stresses in the cross-section. The common point (2) represents a situation, in which the compressed zone covers the whole cross-section, has a triangular shape, and the maximum stress at the compressed edge is equal to \(f_d\). The obtained cigar-shaped closed curve \((0-1-2-D)\) which is symmetrical to the horizontal axis \(N_{Ed}\) is the effect of combining the parabolic diagram in the cracking phase and the linear diagram in the pre-cracking phase. The obtained envelope describing the cross-section bending at the linear distribution of stresses is always smaller than the curve \(V_{Rd,m,max}\) which specifies shear forces at the rectangular distribution of stresses in the compressed zone.

**Figure 3. Interaction diagrams** \(V_{Rd} - N_{Ed}\) at \(V_{Rd}>0, e_{Ed,1}>0\) and \(V_{Rd}<0, e_{Ed,1}<0\)

Both the envelope with the linear distribution of compressive stresses due to bending and the curve with the rectangular diagram of stresses depend on the eccentricity \(e_{Ed,1}\) of load \(N_{Ed}\) applied to the top edge of the wall. Only at the eccentricity \(e_{Ed,1} = 0\), these envelopes are adjacent at one point, at which the vertical load is equal to \(N_{Rd,2,Ed} = N_{Rd,max} = f_d l\). Both the eccentricity \(e_{Ed,1}\) value and the position on the top edge of the wall affect a shape of the interaction diagram. Fig. 4 presents the situation when \(e_{Ed,1} \geq 0\) and \(V_{Ed} > 0, e_{Ed,1} > 0\), and when \(e_{Ed,1} \leq 0\) and \(V_{Ed} < 0, e_{Ed,1} < 0\), and the bending moment in the bottom edge is always higher than in the top edge. On the other hand, Fig. 5 illustrates the opposite situation when \(e_{Ed,1} \leq 0\), \(V_{Ed} > 0\), and \(e_{Ed,1} \geq 0\), \(V_{Ed} < 0\) and the bending moment in the bottom edge of the wall \(M_{Ed,2}\) is lower than \(M_{Ed,1}\). Considering the latter case, compressive stresses determining compressive strength of the wall are observed at a significantly greater horizontal load. Thus, this situation is desirable.
considering load capacity of the wall as the bending moment $M_{Ed,1}$ is increasing due to horizontal loading. At high values of vertical loading $N_{Rd}$, also horizontal loading $V_{Rd}$ responsible for compressive stress $f_d$, is adequately lower. When force $N_{Rd,2,D}$ is acting upon the top edge of the wall, compressive stresses equal to compressive strength of the wall are observed at one edge. The wall could not take any horizontal load $V_{Rd}$ under such loading only when the bending moment in the bottom edge is increased by the moment of horizontal loading. However, at the moment $M_{Ed,2}$ lower than the moment in the top edge, the wall can take horizontal loads.

![Interaction diagrams](image)

**Figure 4.** Interaction diagrams $V_{Rd} - N_{Ed}$ at $V_{Rd}>0$, $e_{Ed,1}<0$ and $V_{Rd}<0$, $e_{Ed,1}>0$

For that reason, at the maximum vertical loading $N_{Rd,2,D}$ the diagram is limited and the maximum load capacity is determined at the point (D') as shown in Fig. 5. The envelope curves specifying failure due to shearing also have to be plotted on the interaction diagrams. At first, the straight line $V_{Ed,cr}$, should be illustrated in the diagram. It defines intervals, in which the cross-section is cracked. The area above this line represents the cracked cross-section, while the point below the line defines the uncracked cross-section. In the cracked cross-section with the triangular diagram of compressive stresses, the curve (0-A-B) is composed of a straight line $V_{Rd2}$ and a curve $V_{Rd2,max}$, which is relevant within in interval to the intersection point (B) with the straight line $V_{Ed,cr}$. The condition defining the maximum shear force $V_{Rd1,max}$, represented by the straight line is critical for this range. If the bending moment in the bottom edge is increased by horizontal loads, then the straight line $V_{Rd1,max}$ intersects the branch limiting normal stresses in more compressed edge at point C. Additionally, the straight line $V_{Rd1}$ representing shear load capacity of the uncracked cross-section should be plotted below the
straight line $V_{Ed,cr}$ and should intersect the straight line $V_{Ed,ld, max}$ w at point E. Load capacity of the cross-section under higher loads is determined by in-plane bending. As it has been already mentioned, the maximum force which the wall can take is equal to $N_{Ed,2,D}$. The area of safe (Fig.4) limit states is inside the curve O-A-B-C-D as compressive stresses at more compressed edge are not exceeded formal requirements for ultimate limit state under shear are specified. The hatched area in Fig. 4 illustrate safe limit states for the bending moment $M_{Ed,2} > M_{Ed,1}$. In the second case when the bending moment in the bottom edge is lower than bending moment in the top edge, the straight line $V_{Ed,ld, max}$ can intersect the straight line which limits compressive stresses $V_{Ed,m,2}$, and at the same time cannot be greater than vertical force $N_{Ed,2,D}$. Hence, the safe area of ultimate limit states is limited by the curve O-A-B-C-D’'-D', which illustrates the hatched area. More detailed information about the structure of the interaction diagram is described in the monograph [7].

A shape of the interaction diagram depends heavily on, apart from wall resistance to compression and shearing, the wall slenderness and the value of design eccentricity applied to the top edge of the wall. Fig. 5 presents the design simulation of shapes of interaction diagrams and ultimate limit state envelopes with reference to bending and shearing against the eccentricity of vertical load $e_{Ed,1}$ and wall slenderness $h/l$ assuming that $M_{Ed,2} \geq M_{Ed,1}$.

![Figure 5. Compared shapes of interaction diagrams: a) constant slenderness $h/l$, variable eccentricity of vertical loading $e_{Ed,1}$, b) constant eccentricity of loading $e_{Ed,1} = 0$ and variable wall slenderness $h/l$](image)

This comparison indicates that the eccentricity (Fig. 5b) had the most significant impact on load capacity of the wall. The effect of changes in wall slenderness (Fig. 5a) is not so strong. Shear load capacity at low compressive stresses decreases as the eccentricity is increasing. A similar situation can be observed at high values of compressive stress. Changes in slenderness result in lower shear load capacity at relatively low compressive stresses. However, this effect is much greater at high compressive stresses. In both cases load capacity decreased by reduced shear strength of the wall generally does not limit shear load capacity of the wall. The previous considerations indicate that at $M_{Ed,2} < M_{Ed,1}$ a shape of the interaction diagram only differs in the area under high compressive loads.

4. Conclusions

The method was proposed to determine load capacity of the wall cross-section subjected to horizontal shear using the interaction diagram $V_{Ed} - N_{Ed}$ in accordance with the draft of Eurocode 6 [1]. This method took into account the conditions of ultimate limit states under shear and in-plane bending. It was shown that both the direction of shear load $V_{Ed}$ and the eccentricity of vertical loads $N_{Ed}$ acting on
the top edge of the wall are important factors which are neglected in the standard. The conducted analysis shows that relatively simple regulations specified in the standard can be relatively troublesome for the proper verification of ultimate limit states.

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