Strengthening of existing masonry structures: Design models

16

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16.1 Introduction

Building on the concepts presented in Chapter 15, this chapter aims to present simplified design procedures and models for unreinforced masonry structures (URM) strengthened with fiber composites comprising textiles in inorganic matrices. This strengthening system is given the term textile-reinforced mortar (TRM). The following topics, also presented in Fyfe Europe SA and Triantafillou (2012), are covered herein:

- General safety principles and safety verifications
- Strengthening of masonry walls for out-of-plane or in-plane loads
- Strengthening of curved masonry elements
- Confinement of masonry columns

16.2 General safety principles

Masonry elements and structures strengthened with TRM should have design strength (R_d) at all sections that is at least equal to the required strength (E_d) calculated for the factored loads and forces in combinations as stipulated by the relevant building code:

$$E_d \leq R_d. \tag{16.1}$$

The design values are obtained from the characteristic values through appropriate partial factors for each limit state. For instance, in Eurocode format (EN 1996-1-1, 2005), for the generic property X of a TRM material, the design value (X_d) is expressed as:

$$X_d = \frac{X_k}{\gamma_m}, \tag{16.2}$$

where X_k is the characteristic value of the property being considered, and γ_m is the partial factor of the material. Values for X_k should be provided by the material supplier, and, until more experimental data become available, $\gamma_m = 1.5$ ($=\gamma_t$).

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According to the aforementioned format, the design strength (R_d) is expressed as follows:

$$R_d = \frac{1}{\gamma_{Rd}} R\left(X_{d,i}; a_{d,i}\right). \tag{16.3}$$

In Equation (16.3), $R()$ is a suitable function for the specific mechanical model being considered, and γ_{Rd} is a partial factor covering model uncertainties (1.0 for bending or combined bending and axial load, 1.2 for shear, and 1.1 for confinement). The arguments of the function $R()$ are typically the design values ($X_{d,i}$) of the materials used for strengthening or the existing materials, and the nominal values ($\alpha_{d,i}$), of the geometrical parameters involved in the model.

16.3 Safety verifications

Failure modes of masonry structures strengthened with TRM may involve excessive cracking due to tensile stresses, shear slip of masonry, crushing of masonry, TRM rupture, and TRM debonding.

Masonry exhibits a brittle behavior when subjected to tensile loading; the corresponding tensile strength is quite low and, for design purposes, can be neglected. The shear strength of masonry depends on the level of applied axial load, as it relies upon cohesion and friction of the material. The design stress–strain relationship of masonry in compression may be idealized as parabolic-rectangular (Figure 16.1), linear-rectangular, or even just rectangular for the sake of cross section analysis. Unless experimental data are available, the masonry ultimate strain (ε_{mu}) may be assumed to be 0.35% and $\varepsilon_{m1} = 0.2\%$.

The characteristic values for masonry strength are f_{mk} for vertical compression, f_{mk}^h for horizontal compression, and f_{vk} for shear. In the absence of specific information or experimental data, f_{mk}^h may be taken as 50% of f_{mk}.

TRM materials stressed parallel to a principal fiber direction may be approximately idealized through the adoption of a trilinear stress–strain curve. In this curve, the slope

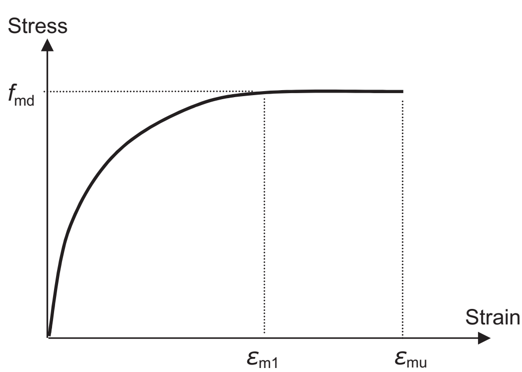

Figure 16.1 Idealized stress–strain curve for masonry in uniaxial compression.

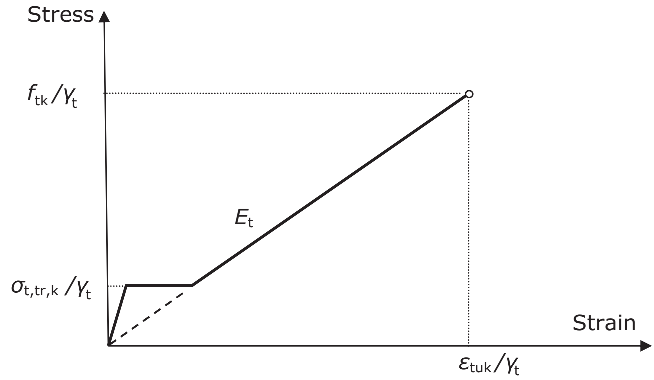

Figure 16.2 Idealized stress–strain curve for TRM system in uniaxial tension.

of the second branch corresponding to multiple cracking may be taken as zero (horizontal branch). The elastic modulus of the cracked material is $E_t = f_{tk}/\varepsilon_{tuk}$ (Figure 16.2), where f_{tk} = characteristic strength and ε_{tuk} = characteristic strain at failure. $\sigma_{t,tr,k}$, in Figure 16.2, is the characteristic stress at the transition zone.

The maximum design stress allowed to the TRM is expressed as follows:

$$f_{td} = \min\left(\frac{f_{tk}}{\gamma_t}, f_{tbd}\right) \tag{16.4}$$

where f_{tbd} is the TRM (design) stress once debonding takes place. Reliable models for the evaluation of the debonding stress of TRM are not available yet. Unless reliable experimental data are provided for a specific TRM system (with possible anchorage), it is suggested to assume that debonding initiates with the TRM at a strain $\varepsilon_{tb} = 0.003$ (which usually is in the transition zone, stage IIa in Figure 15.3). The deboning stress (f_{tbd}) is equal to $0.003E$, where E is the tensile modulus of the TRM that, depending on whether it corresponds to stage IIa or IIb, is defined by the following equation:

$$E = \max\left(\frac{\sigma_{t,tr,d}}{\varepsilon_t}, E_t\right) \tag{16.5}$$

For ultimate limit state analysis, two possible approaches may be followed, depending on the type of structural analysis performed. If nonlinear models are used, the member's carrying capacity shall be larger than the factored applied load. Care shall be taken to ensure that the proposed solution is not affected by the particular discretization adopted in the analysis. If linear elastic models or simplified methods adopting an admissible distribution of stresses (that satisfy equilibrium but not necessarily strain compatibility) are used, the resulting stresses on each structural member shall be verified. In particular, for bi-dimensional members (slabs, shells), the unit stress shall be considered (i.e., the per unit length of the member shall be evaluated). Assuming that a plane section, before loading, remains a plane after loading, the safety is verified when factored shear forces and bending moments (calculated as a function of the applied axial force), due to the applied loads, are smaller than the corresponding design factored shear and flexural capacities. Note that forces and moments may be calculated through the integration of stresses.

16.4 Strengthening of masonry walls for out-of-plane loads

As discussed in Chapter 15, out-of-plane collapse can develop as *overturning*, *vertical flexural failure*, or *horizontal flexural failure*. Design equations for each case are presented next.

16.4.1 Overturning

Consider the masonry wall shown in Figure 16.3, subjected to the following forces (design values): P_d = wall self-weight; N_d = axial force acting on top of the wall; Q_d = horizontal force due to seismic effects; and T_d = force exerted on the top part of the wall by the TRM system. The wall has a height (h) and a thickness (t).

Assuming that floors and walls perpendicular to the wall provide negligible restraint, T_d can be calculated via moment equilibrium as follows:

$$T_d = \frac{1}{2d}(Q_d h - N_d t - P_d t), \tag{16.6}$$

where d = distance from center of TRM to the bottom of the wall.

To design against overturning, it shall be verified that

$$T_d \leq 2T_{Rd} = 2A_t f_{td}, \tag{16.7}$$

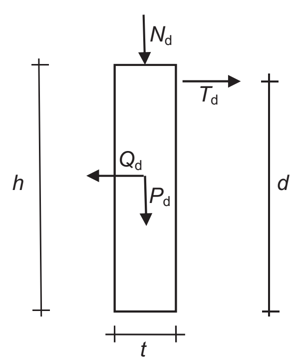

Figure 16.3 Overturning: forces acting on wall.

where A_t is the TRM cross section area and f_{td} is the maximum design stress. Note that in the case of a fully wrapped structure, TRM will fail by tensile rupture. Hence, f_{td} is the design tensile strength, possibly reduced, for example, due to stress concentrations. But in case of open TRM systems involving anchorage on the orthogonal walls, failure may be governed by TRM debonding, hence $f_{td} = f_{tbd}$.

16.4.2 Vertical flexural failure

Consider a unit width masonry wall (Figure 16.4), subjected to the following external forces (design values): $P_{u,d}$ = weight of the wall upper part; $P_{l,d}$ = weight of the wall lower part; N_d = axial force acting on the wall; $Q_{u,d}$ = seismic force related to the upper part of the wall; $Q_{l,d}$ = seismic force related to the lower part of the wall; and Q_d = force related to additional horizontal loading.

The masonry wall at cross section *B*, where the TRM is applied to prevent formation of the hinge, is subjected to axial force (N_{Ed}) and bending moment (M_{Ed}) equal to:

$$N_{Ed} = N_d + P_{u,d}, \tag{16.8a}$$

$$M_{Ed} = H_{C,d} h_u - Q_{u,d} \frac{h_u}{2}, \tag{16.8b}$$

where $H_{C,d}$ is the horizontal reaction at C, equal to:

$$H_{C,d} = \frac{\left(2Q_d + Q_{l,d}\right)h_l + Q_{u,d}(2h - h_u) - \left(N_d + P_{u,d} + P_{l,d}\right)t}{2h}. \tag{16.9}$$

The masonry wall flexural capacity is verified when the following condition is met:

$$M_{Ed} \le M_{Rd}. \tag{16.10}$$

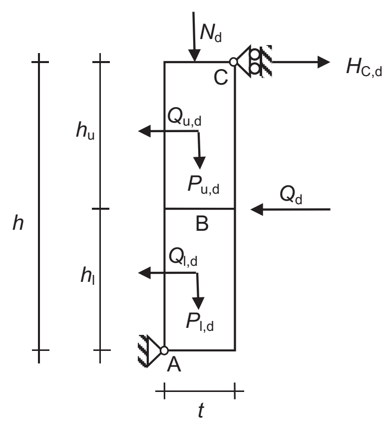

Figure 16.4 Vertical flexural failure: forces acting on wall.

The flexural capacity M_{Rd} of the strengthened masonry wall may be calculated on the basis of cross section analysis (Figure 16.5). The calculation may be done as a function of the mechanical properties of the masonry and TRM, the thickness (t), the width (l) and the value of the applied axial force (N_{Ed}). Failure is defined when either the masonry fails in compression (ε_{mu} is reached) or the TRM fails in tension [$\varepsilon_{t,lim} = \min(\varepsilon_{tu}, \varepsilon_{tb})$ is reached, where $\varepsilon_{tu} = \varepsilon_{tuk}/\gamma_t$], whichever comes first.

For the case of the parabolic-rectangular stress–strain relationship shown in Figure 16.1 (with $\varepsilon_{m1} = 0.2\%$ and $\varepsilon_{mu} = 0.35\%$), the resulting expressions for M_{Rd} are obtained as follows (assuming that the TRM is cracked, i.e., in stage IIa or IIb):

$$\text{Force equilibrium: } k_1 f_{md} l x - A_t \sigma_{td} = N_{Ed} \tag{16.11}$$

with

$$\sigma_{td} = E\varepsilon_t.$$

$$\text{Strain compatibility: } \varepsilon_t = \varepsilon_m \frac{t-x}{x} \leq \varepsilon_{t,\,\lim} = \min(\varepsilon_{tu}, \varepsilon_{tb}). \tag{16.12}$$

1. Compression failure of masonry ($\varepsilon_m = \varepsilon_{mu}$, $\varepsilon_t < \varepsilon_{t,lim}$):

$$\frac{M_{Rd}}{lt^2 f_{md}} = \frac{1}{\gamma_{Rd}} \left[\frac{1}{2} \frac{\left(1 - \frac{x}{t}\right)}{\frac{x}{t}} \omega_t + \frac{1}{2} k_1 \frac{x}{t} \left(1 - 2k_2 \frac{x}{t}\right) \right], \tag{16.13}$$

where x/t, the normalized neutral axis depth, is

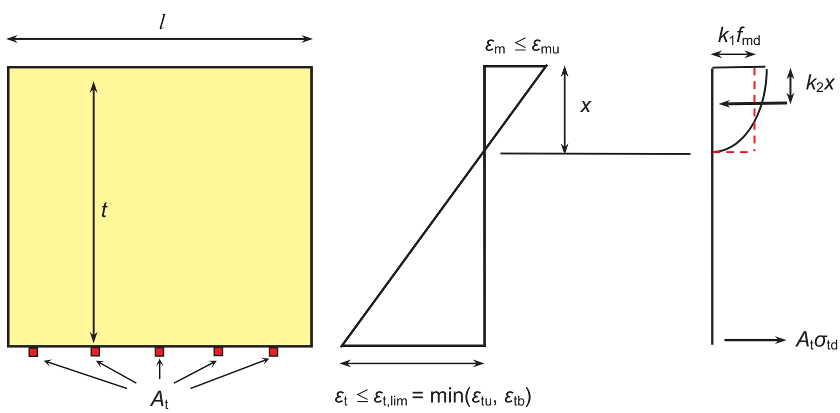

Figure 16.5 Cross-section analysis at the ultimate limit state.

$$\frac{x}{t} = \frac{1}{2k_1}\left[-\omega_t + \frac{N_{Ed}}{ltf_{md}} + \sqrt{\left(\omega_t - \frac{N_{Ed}}{ltf_{md}}\right)^2 + 4k_1\omega_t}\right] \tag{16.14}$$

and ω_t is defined as

$$\omega_t = \frac{A_t}{lt}\frac{\varepsilon_{mu}E}{f_{md}}. \tag{16.15}$$

Finally, $k_1 = 0.8$, $k_2 = 0.4$, and γ_{Rd} = partial factor for flexure (=1.0).

2. Failure of TRM by rupture or debonding ($\varepsilon_m < \varepsilon_{mu}$, $\varepsilon_t = \varepsilon_{t,lim}$):

$$\frac{M_{Rd}}{lt^2 f_{md}} = \frac{1}{\gamma_{Rd}}\left[\frac{1}{2}\frac{\varepsilon_{t,\lim}}{\varepsilon_{mu}}\omega_t + \frac{1}{2}k_1\frac{x}{t}\left(1 - 2k_2\frac{x}{t}\right)\right], \tag{16.16}$$

where

$$\frac{x}{t} = \frac{1}{k_1}\left(\omega_t\frac{\varepsilon_{t,\lim}}{\varepsilon_{mu}} + \frac{N_{Ed}}{ltf_{md}}\right) \tag{16.17}$$

and

$$k_1 = \begin{cases} 1000\varepsilon_m\left(0.5 - \dfrac{1000}{12}\varepsilon_m\right) & \text{if } \varepsilon_m \le 0.002 \\ 1 - \dfrac{2}{3000\varepsilon_m} & \text{if } 0.002 \le \varepsilon_m \le 0.0035 \end{cases} \tag{16.18}$$

$$k_2 = \begin{cases} \dfrac{8 - 1000\varepsilon_m}{4(6 - 1000\varepsilon_m)} & \text{if } \varepsilon_m \le 0.002 \\ \dfrac{1000\varepsilon_m(3000\varepsilon_m - 4) + 2}{2000\varepsilon_m(3000\varepsilon_m - 2)} & \text{if } 0.002 \le \varepsilon_m \le 0.0035 \end{cases} \tag{16.19}$$

$$\varepsilon_m = \varepsilon_{t,\lim}\frac{\frac{x}{t}}{1 - \frac{x}{t}} \le 0.0035. \tag{16.20}$$

16.4.3 Horizontal flexural failure

In the case of horizontal flexural failure, the applied bending moment M_{Ed} is due to earthquake loads, wind pressure and other possible horizontal loads that are due to the presence of other structural members. The masonry wall flexural capacity is verified when the condition (16.10) is met. The flexural capacity (M_{Rd}) of the strengthened masonry wall corresponding to a strip of unit width may be calculated on the basis of cross section analysis (Figure 16.5) as a function of the mechanical properties of masonry and TRM, the thickness (t) and the width (l) of the wall. Note that unless a more detailed analysis is available, the horizontal force due to the presence of transverse walls (axial force in the horizontal direction) may be assumed to be zero. Failure is defined when either the masonry fails in compression (ε_{mu} is reached) or the TRM fails in tension ($\varepsilon_{t,lim}$ is reached), whichever comes first.

For the case of the parabolic-rectangular stress–strain relationship shown in Figure 16.1 (with $\varepsilon_{m1} = 0.2\%$ and $\varepsilon_{mu} = 0.35\%$), the resulting expressions for M_{Rd} are as given in Section 16.4.2. $N_{Ed} = 0$, f_{md} is replaced by the design value of the masonry strength in the horizontal direction (f_{md}^{h}) and length of the wall (l) replaced by the height of the wall (h).

16.5 Strengthening of masonry walls for in-plane loads

The following verifications shall be carried out for masonry *walls subjected to in-plane loads: combined bending and axial load* or *shear* force. Furthermore, *lintels and tie areas* should be given special attention.

16.5.1 Combined bending and axial load

The flexural capacity (M_{Rd}) of a strengthened masonry wall may be calculated on the basis of cross section analysis (Figure 16.6). The calculation may be performed as a function of the mechanical properties of the masonry and TRM, the wall dimensions (length l, thickness t) and the value of the applied axial force (N_{Ed}). Again, failure is defined when either the masonry fails in compression (ε_{mu} is reached) or the TRM fails in tension ($\varepsilon_{t,lim}$ is reached), whichever comes first.

For the case of two-sided full coverage by TRM (Figure 16.6), and considering the parabolic-rectangular stress–strain relationship shown in Figure 16.1 (with $\varepsilon_{m1} = 0.2\%$ and $\varepsilon_{mu} = 0.35\%$), the resulting expressions for M_{Rd} are as follows (assuming that the TRM is cracked, i.e., in stage IIa or IIb):

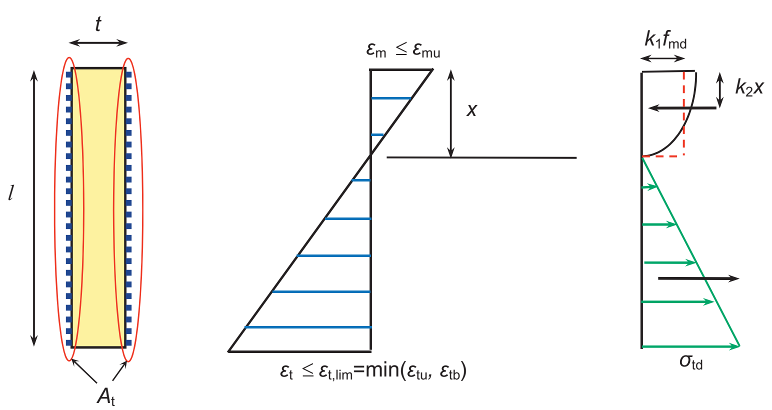

Figure 16.6 Cross-section analysis at the ultimate limit state.

$$\text{Force equilibrium:}\ k_1 f_{\text{md}} t x - \frac{1}{2} A_{\text{t}} \sigma_{\text{td}} \frac{l-x}{l} = N_{\text{Ed}} \tag{16.21}$$

with

$$\sigma_{\text{td}} = E\varepsilon_{\text{t}}.$$

$$\text{Strain compatibility:}\ \varepsilon_{\text{t}} = \varepsilon_{\text{m}} \frac{l-x}{x} \leq \varepsilon_{\text{t, lim}} = \min\left(\varepsilon_{\text{tu}}, \varepsilon_{\text{tb}}\right) \tag{16.22}$$

1. Compression failure of masonry ($\varepsilon_{\text{m}} = \varepsilon_{\text{mu}}$, $\varepsilon_{\text{t}} < \varepsilon_{\text{t,lim}}$):

$$\frac{M_{\text{Rd}}}{t l^2 f_{\text{md}}} = \frac{1}{\gamma_{\text{Rd}}} \left[\frac{1}{12} \frac{\left(1-\frac{x}{l}\right)^2 \left(1+2\frac{x}{l}\right)}{\frac{x}{l}} \omega_{\text{t}} + \frac{1}{2} k_1 \frac{x}{l} \left(1 - 2k_2 \frac{x}{l}\right) \right] \tag{16.23}$$

where x/l, the normalized neutral axis depth, is

$$\frac{x}{l} = \frac{1}{2\left(k_1 - \frac{\omega_{\text{t}}}{2}\right)} \left[-\omega_{\text{t}} + \frac{N_{\text{Ed}}}{l t f_{\text{md}}} + \sqrt{\left(\omega_{\text{t}} - \frac{N_{\text{Ed}}}{l t f_{\text{md}}}\right)^2 + 2\left(k_1 - \frac{\omega_{\text{t}}}{2}\right)\omega_{\text{t}}} \right] \tag{16.24}$$

ω_{t} is defined as before,

$$\omega_{\text{t}} = \frac{A_{\text{t}}}{l t} \frac{\varepsilon_{\text{mu}} E}{f_{\text{md}}} \tag{16.15}$$

and $k_1 = 0.8$, $k_2 = 0.4$.

2. Failure of TRM by rupture or debonding ($\varepsilon_{\text{m}} < \varepsilon_{\text{mu}}$, $\varepsilon_{\text{t}} = \varepsilon_{\text{t,lim}}$):

$$\frac{M_{\text{Rd}}}{t l^2 f_{\text{md}}} = \frac{1}{\gamma_{\text{Rd}}} \left[\frac{1}{2} \frac{\varepsilon_{\text{t, lim}}}{\varepsilon_{\text{mu}}} \omega_{\text{t}} \frac{\left(1-\frac{x}{l}\right)\left(1+2\frac{x}{l}\right)}{6} + \frac{1}{2} k_1 \frac{x}{l} \left(1 - 2k_2 \frac{x}{l}\right) \right], \tag{16.25}$$

where

$$\frac{x}{l} = \frac{\frac{1}{2}\omega_{\text{t}} \frac{\varepsilon_{\text{t, lim}}}{\varepsilon_{\text{mu}}} + \frac{N_{\text{Ed}}}{l t f_{\text{md}}}}{k_1 + \frac{1}{2}\omega_{\text{t}} \frac{\varepsilon_{\text{t, lim}}}{\varepsilon_{\text{mu}}}} \tag{16.26}$$

k_1 and k_2 are as given by Equations (16.18) and (16.19) and

$$\varepsilon_{\text{m}} = \varepsilon_{\text{t, lim}} \frac{\frac{x}{l}}{1 - \frac{x}{l}} \leq 0.0035. \tag{16.27}$$

16.5.2 Shear force

The shear resistance (V_{Rd}) of masonry walls strengthened with TRM (e.g., Figure 15.7b) is computed as the sum of the masonry contribution ($V_{Rd,m}$) and TRM contribution ($V_{Rd,t}$) up to the maximum value ($V_{Rd,max}$) corresponding to compression failure of the struts in the truss:

$$V_{Rd} = \min\left(\frac{V_{Rd,m} + V_{Rd,t}}{\gamma_{Rd}}, \frac{V_{Rd,max}}{\gamma_{Rd}}\right) \tag{16.28}$$

The above contributions may be evaluated as follows:

$$V_{Rd,m} = f_{vd} t l \tag{16.29}$$

$$V_{Rd,t} = 0.9 l (n t_f) f_{td} \tag{16.30}$$

$$\frac{V_{Rd,max}}{tl} = 2 \quad (\text{MPa}) \tag{16.31}$$

where γ_{Rd} = partial factor for shear (=1.2), t = thickness of masonry wall, l = length of wall, f_{vd} = design shear strength of masonry, f_{td} = design strength of TRM, defined in Equation (16.4), and n = number of sides strengthened with a jacket of thickness t_f each (n = 2 for two-sided jacketing, n = 1 for one-sided jacketing).

16.5.3 Lintels and tie areas

As an approximation, a TRM-strengthened lintel may be idealized as a beam fixed at both ends, under the action of factored dead loads. The flexural capacity M_{Rd} of this beam, calculated on the basis of cross section analysis as described above, shall exceed the applied moment M_{Ed}:

$$M_{Ed} = \gamma_G \frac{1}{24} g t L^3 \tag{16.32}$$

where g = masonry weight/m^3, t = thickness of masonry wall, L = clear span of the opening, and γ_G = partial factor for self-weight at the ultimate limit state.

Moreover, by considering the TRM-strengthened lintel as an axially loaded element, the TRM shall be designed to resist the following axial force (Figure 16.7):

$$N_{Ed} = \frac{q_d L^2}{8h'} \tag{16.33}$$

where q_d is the design vertical load acting on the lintel (the sum of factored dead and live loads) and h' is the internal lever arm. This is not to be taken larger than the span (L) of the opening, nor the height (h) of the tie area.

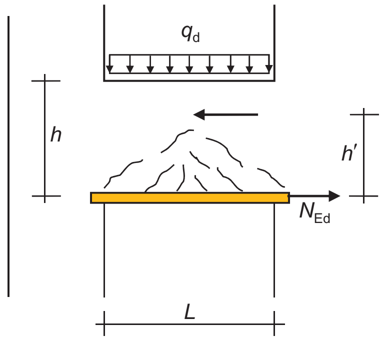

Figure 16.7 Tensile force in lintel.

Tie areas strengthened with TRM shall be verified for the bending moment, shear force and axial force (if any) acting at the connection with vertical masonry walls. The flexural and shear capacity shall be calculated on the basis of the procedures described in previous sections (Sections 16.5.1 and 16.5.2), with proper modifications.

16.6 Strengthening of curved masonry elements: Arches, barrel vaults, domes

When computing internal forces in an arch strengthened with TRM, the formation of hinges at supports shall always be considered (unless such formation is prevented). Moreover, the formation of hinges located on the opposite side of the TRM installation shall be taken into account. Arches strengthened with TRM can be designed by checking their overall stability and by applying the procedures described above for masonry subjected to combined out-of-plane bending and axial force. Special attention should be given to debonding, which may be controlled through the application of anchors.

Barrel vaults have a single curvature; hence, the design of their strengthening is similar to that of arches.

Domes are double curvature vaults and exhibit both membrane-type and flexural-type stresses. In a dome subjected to vertical loads, membrane-type tension stresses develop along the dome parallels and may cause cracking along the meridians. This is especially true near the connection with the supporting structure. In this case, the required cross section of TRM in a circular configuration around the lower part of the dome's perimeter shall be calculated for the full magnitude of tensile stresses. TRM strengthening for flexural-type stresses can be designed by applying the procedures described above for masonry that has been subjected to combined bending and axial force to unit dome elements.

16.7 Confinement of masonry columns

The axial capacity (N_{Rd}) of a strengthened column shall exceed the design axial force due to applied loads (N_{Ed}):

$$N_{Rd} \geq N_{Ed} \tag{16.34}$$

$$N_{Rd} = \frac{1}{\gamma_{Rd}} A_m f_{mcd} \geq \frac{1}{\gamma_{Rd}} A_m f_{md} \tag{16.35}$$

where A_m = cross sectional area of confined column, f_{md} = design compressive strength of unconfined masonry, f_{mcd} = design compressive strength of confined masonry and γ_{Rd}, the partial factor, is 1.1.

The design compressive strength (f_{mcd}) for confined masonry may be related to f_{md} through the use of an appropriate confinement model that has the following general form:

$$\frac{f_{mcd}}{f_{md}} = 1 + k\left(\frac{\sigma_{lud}}{f_{md}}\right)^{m} \tag{16.36}$$

where σ_{lud} is the design effective confining pressure and k, m are nondimensional empirical constants obtained from calibration with test results for the masonry material under consideration. Because of the lack of adequate test data, these parameters are not calibrated yet. Nevertheless, an estimate for the values could equate m to 1 and k to 1.5. However, those values should be used with circumspection until verification by experimental data becomes available.

To restrict axial deformation and prevent damage at the serviceability limit state, the increased axial capacity due to TRM shall be limited to 50% of f_{md}.

In columns with *circular* cross section, the confining stress σ_l is given in terms of the TRM jacket tensile stress σ_t as follows (Figure 16.8):

$$\sigma_l = \frac{2t_t}{D}\sigma_t \tag{16.37}$$

where D = cross section diameter and t_t thickness of TRM jacket. σ_{lud} is obtained from Equation (16.37), with σ_t equals the design value of the stress in the jacket at failure

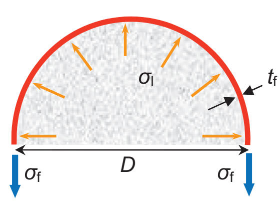

Figure 16.8 Confining stress.

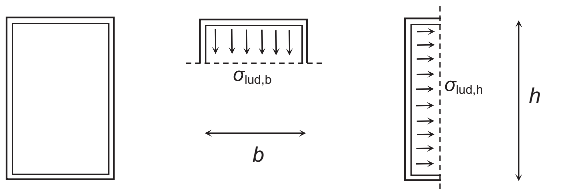

Figure 16.9 Approximate confining stresses in a rectangular cross-section.

$f_{\text{ted}} = \eta_e f_{\text{tk}}/\gamma_t$. η_e is a strength reduction factor to account for the multi-axial state of stress in the jacket, stress concentrations etc. Unless experimental support is available, η_e is 0.8.

In columns with a *rectangular* cross section, the confining stress is not uniform, especially near the corners. As an average for σ_{lud}, one may write (Krevaikas and Triantafillou, 2005; Bournas et al., 2007; Figure 16.9):

$$\sigma_{\text{lud}} = \frac{\sigma_{\text{lud,h}} + \sigma_{\text{lud,b}}}{2} = \frac{1}{2}\left(k_e \frac{2t_t}{h} f_{\text{ted}} + k_e \frac{2t_t}{b} f_{\text{ted}}\right) = \frac{1}{2} k_e \frac{(b+h)}{bh} t_t f_{\text{ted}}, \tag{16.38}$$

where k_e is the effectiveness coefficient, which, for continuous jackets with fibers in the direction perpendicular to the column axis (in addition to those parallel to the column), is defined as the ratio of the effectively confined area (A_e in Figure 16.10) to the total cross sectional area A_g:

$$k_e = 1 - \frac{b'^2 + h'^2}{3A_g}. \tag{16.39}$$

In columns with *section enlargement* (Figure 15.13), by application of a thick mortar layer on the masonry prior to the application of the TRM, the total axial load capacity of the composite section is calculated as follows:

$$N_{\text{Rd}} = \frac{1}{\gamma_{\text{Rd}}}(A_m f_{\text{mcd}} + A_l f_{\text{ld}}), \tag{16.40}$$

where A_m = cross sectional area of confined masonry, A_l = cross sectional area of thick mortar layer, f_{mcd} = design compressive strength of confined masonry, and f_{ld} = design compressive strength of thick mortar layer.

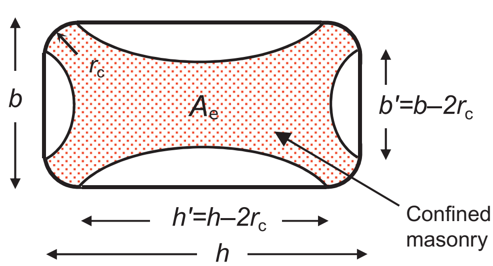

Figure 16.10 Effectively confined area in columns with a rectangular cross-section.

16.8 Summary

Building on the concepts presented in the previous chapter, this chapter aimed to present simplified design procedures and models for URMs strengthened with TRM. The following topics were covered: general safety principles and safety verifications; strengthening of masonry walls for out-of-plane loading (overturning, vertical flexural failure and horizontal flexural failure) or in-plane loading (combined bending and axial loading, shear forces, lintels and tie areas); strengthening of curved masonry elements and confinement of masonry columns with circular or rectangular cross sections. Design equations were given for most of the cases listed above, within the framework of Eurocode format.

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