Chapter 10

Concluding Remarks

This thesis describes the development and use of surface beams to represent masonry buildings in three-dimensional numerical analyses of tunnelling. It is common practice to use linear elastic surface beams to represent buildings in 2D finite element analyses, or include full building models in complex 3D analyses. This thesis investigates the extension of the surface beam approach for use in 3D, and the development of beams which capture the behaviour of a masonry building. To achieve this, the research includes the implementation of 3D capable beams in the OXFEM finite element program; development of a method for determining equivalent elastic properties to more accurately model building response; and development of a non-linear equivalent masonry beam model to capture masonry specific responses in hogging and sagging. Three-dimensional numerical models of tunnelling using the equivalent surface beams to represent masonry buildings are compared to numerical models with full masonry buildings and with case study data from the Jubilee Line Extension project.

It is common practice to use elastic beams on the ground surface to represent a building in 2D finite element analyses. When representing masonry buildings using elastic surface beams, elastic properties based on the masonry material, are assigned to the beams. Tests show that this simple assignment of properties results in beams that do not respond to imposed base displacements in a manner similar to 2D facades. The test results show that the overall geometry of the facade (L/H ratio), and the amount and position of openings affects the facade response. Generally, the more openings and the lower the L/H ratio, the lower the stiffness of the facade compared to a traditional elastic beam. To model the facade using a surface beam, effective properties need to be used that account for these factors. The equivalent elastic beam procedure developed in this thesis is a geometric procedure to determine effective cross sectional area (A^*) and second moment of area (I^*) for assignment to equivalent surface beams by considering the amount and location of openings and the overall facade geometry. The procedure is shown to lead to predicted responses that are a significant improvement on a traditional deep beam and to lead to results that closely match the finite element response of facades to imposed displacements.

Further investigation of the equivalent elastic procedure for facades with more openings would be worthwhile as there is some deviation of the equivalent beam response from the observed finite element facade results in this region. Differences between the new theory and finite element results are also evident around the critical length to height ratio. Further investigation of the choice of this value may also lead to improvements in the procedure.

It is clear from the literature that different mechanisms operate in masonry facades in sagging and hogging leading to different facade responses in each of those modes. The suite of finite element analyses reported in this thesis confirm these differing responses, the key feature of which is the loss of flexural rigidity in masonry facades in hogging. This feature has previously not been captured in equivalent surface beams representing buildings in numerical analyses. The equivalent masonry beam model developed in this research is a non-linear elastic model including loss of stiffness in hogging. The parameters of critical curvature (K_{crit}) and residual bending stiffness factor (f_b) are introduced to control this loss of stiffness.

The implementation of 3D Timoshenko beam elements into the existing in-house finite element program OXFEM represents and enhancement of the finite element code. The equivalent elastic and masonry constitutive beam models implemented are also new additions. Tests of equivalent elastic Timoshenko beams confirmed their correct implementation and the response of the beams to imposed displacements conformed to the implemented theory. When subjected to Gaussian displacements the beam response mostly matched that of elastic facades under the same conditions, however they displayed differing end effects. This should be the subject of further investigation. Such investigation could consider the use of higher order beams such as three-node beams with linear shear variation. Such beams could be combined with the use of 20 node tetrahedral soil elements (for combined soil-structure interaction analyses) for improved compatibility and accuracy. With advancing computer technology the additional run times inherent in such an approach should soon be only a minor consideration. Tests showed that the moment-curvature response of the equivalent masonry beams conformed to the implemented theory. Alternative implementations for the masonry beams could be considered as part of future research which might include a sudden reduction of stiffness instead of a gradual reduction or models which consider the variation of axial or shear stiffness as appropriate.

The use of the parallel computers at the Oxford Supercomputing Centre was a key feature of the time spent on this research. At the commencement of the project, the use of the parallel systems enabled runs to be completed significantly faster than on standard PC or Unix systems and the analysis of multiple runs to be undertaken simultaneously. For the type of 3D analyses under investigation, however, it is recommended that the use of a local high specification PC or cluster of PCs (combined with the ability to queue and batch jobs) be investigated. Hardware advances in recent years have rendered high end PCs capable of running these analyses in a reasonable length of time and their use would remove dependence on shared and at times unreliable university resources.

Three-dimensional finite element models were used to investigate the use of the equivalent elastic and masonry beams for modelling masonry buildings. The example analyses were chosen to be comparable with previous work carried out by members of the Civil Engineering Research Group at Oxford University. Greenfield analyses predicted settlements that were shallower and wider than a traditional Gaussian profile. This was not considered a problem for their use as controls for the testing of the equivalent beams. For both symmetric and oblique analyses the use of the equivalent elastic beams resulted in settlement profiles with excellent agreement to the response of full masonry facades in sagging regions. The beams showed less local settlement variation due to the lack of windows and their consistent stiffness, but the magnitude and differential settlement of the beams was generally very close to that of the masonry facades. The equivalent elastic beams did not replicate the response of the masonry facades well in hogging zones. The beams suffered minimal curvature and deflection ratios compared to the facades. The affect of assigning different stiffness values was shown by the parametric stiffness study. Reducing the stiffness of the equivalent elastic beams to 1% of their original value was found to lead to a response very close to that of the masonry facade for hogging regions.

The equivalent masonry beams responded in the same manner as elastic beams in sagging, but were more flexible in hogging. They did not, however, respond in hogging in the same manner as the masonry facades. From the results of the oblique analysis, significantly less hogging was evident in the equivalent masonry beams with the range of K_{crit} and f_b values used than the masonry facades. This indicates that further work is required to investigate the interaction of the critical curvature and residual bending stiffness parameters with lower values of critical curvature thought to lead to more realistic responses in hogging. Limitations of availability of computing systems prevented further analyses from being carried out and these are recommended for the future.

It was satisfying to be able to utilise one of the published case studies of the Jubilee Line Extension project in London. The case study material is extremely thorough and further research using this material is encouraged. The case study analysis using equivalent masonry beams to represent three masonry buildings simultaneously gave good agreement with the observed settlements and damage. In particular, the flexible response of hogging sections of the building were replicated by the equivalent masonry beams. The response in sagging regions was essentially rigid as was observed during construction. Modelling in 3D of the staged construction underneath multiple buildings simultaneously was an efficient way to analyse this problem. The case study buildings chosen suffered from being subject to smaller magnitudes of displacement than might otherwise be desirable to fully test the masonry surface beams. The problem of selecting a site with no compensation grouting made this situation unavoidable. Further case study verification of the surface beam approach for modelling masonry buildings above tunnels in more severe situations would be useful.

The developments of this thesis could be improved upon or used as a basis for future research. The most direct application of this research is that the equivalent elastic and masonry beam approaches can be used to determine properties for elastic and masonry beams for the modelling of masonry facades in two and three dimensions. A drawback to the simplification of a full masonry building as a surface beam is that no information regarding plane stresses and strains or crack profiles is available as a result of the simplified analysis. This is equally true for 2D analyses of buildings using surface beams, but these have been used extensively in previous analyses, in particular for parametric studies. Further investigations could make use of the 3D equivalent beams developed in this thesis for 3D parametric analyses of the interaction between buildings and tunnels. Such parametric studies would be useful for investigating the affect of variables such as the angle of an oblique building relative to the tunnel (and its eccentricity) and could assess: whether simply projecting the building onto the settlement profile (as is current practice) is appropriate; and the interaction between the building twist and damage. This could potentially lead to further extension of the relative stiffness approach to include oblique 3D considerations.

The use of the 3D surface beams as presented in this thesis would also be useful if a full model of a building is required where the building is part of a congested urban physical situation. Equivalent masonry surface beams could be used to represent buildings not of immediate interest but adjacent to the main building and thus important for correct modelling of the overall soil-structure interaction situation. This development would make full 3D models of buildings more accurate, by the inclusion of surrounding buildings in a simplified manner.

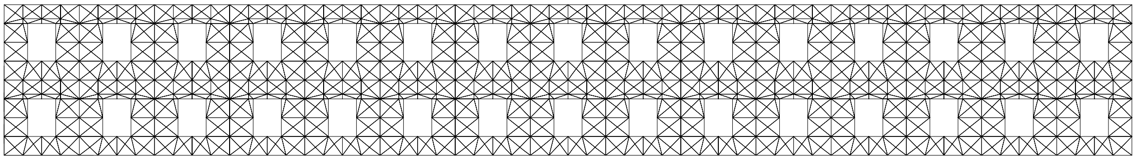
Soil structure interaction problems involve overlap between structural and geotechnical engineering. In the traditional structural engineer's view, such problems often involve simplification of the soil and a detailed model of the structure. The approach adopted in this thesis turns the tables on this view with the use of a complex soil and tunnel model and the development of a simplified building.

However such problems are considered, it is clear that the need for assessment of the influence of urban tunnel construction on existing structures will continue to be important. The simplified beam approach developed here has room for improvement and further development, but it represents a new contribution to the ongoing development of numerical modelling of building response to tunnelling.

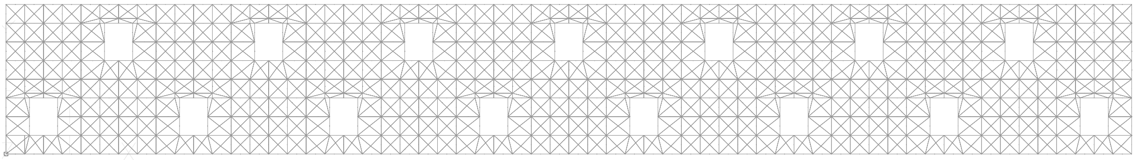
Appendix A

Facade meshes with windows

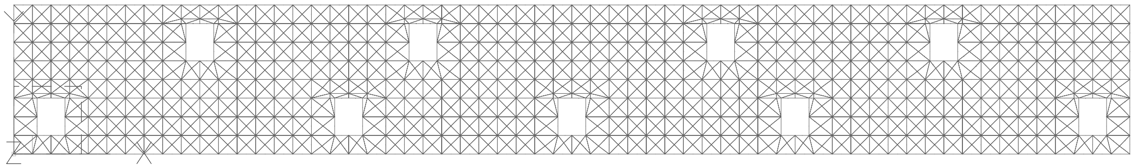
The facade meshes shown in this Appendix are those, with windows, described in tables 4.2 and 5.1. They are given here without the E or M descriptor which is simply used to designate the material used in each of the finite element analyses in Chapters 4 and 5.



(a) Mesh 31

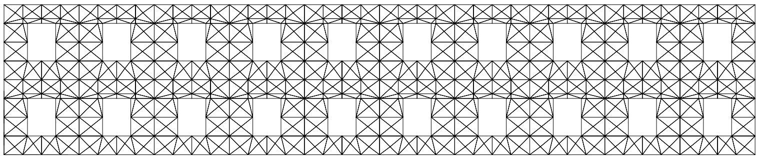


(b) Mesh 51

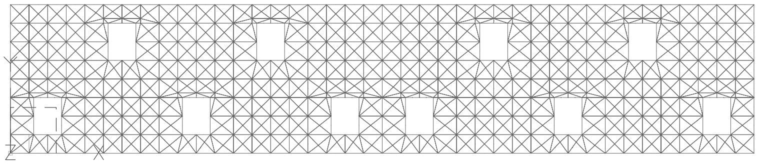


(c) Mesh 71

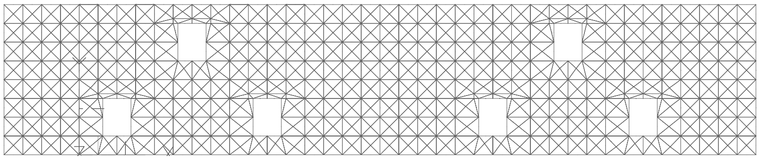
Figure A.1: **Meshes with L=60m, H=8m**



(a) Mesh 32

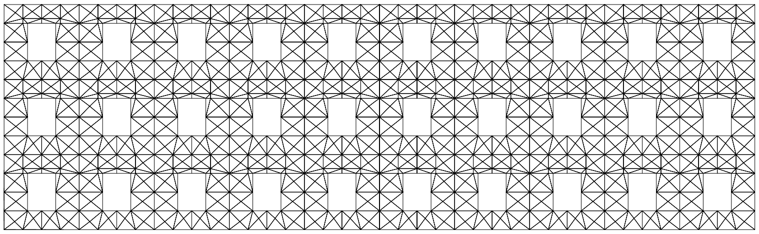


(b) Mesh 52

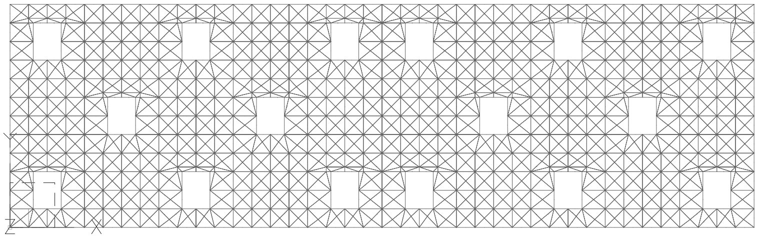


(c) Mesh 72

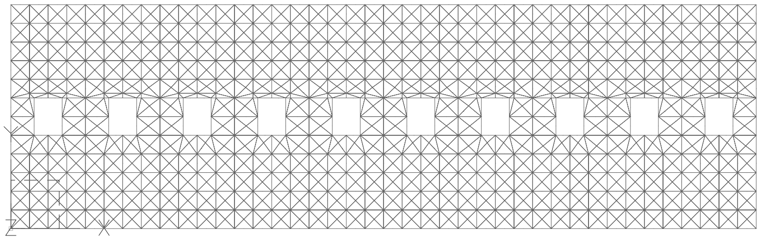
Figure A.2: **Meshes with L=40m, H=8m**



(a) Mesh 34

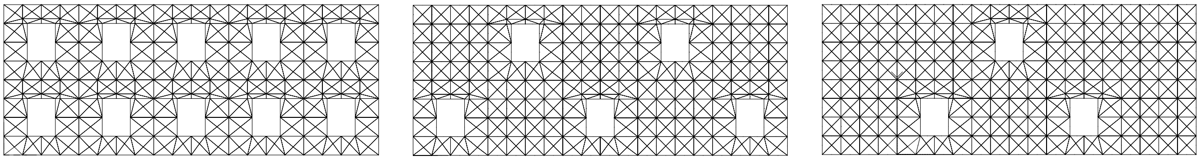


(b) Mesh 54



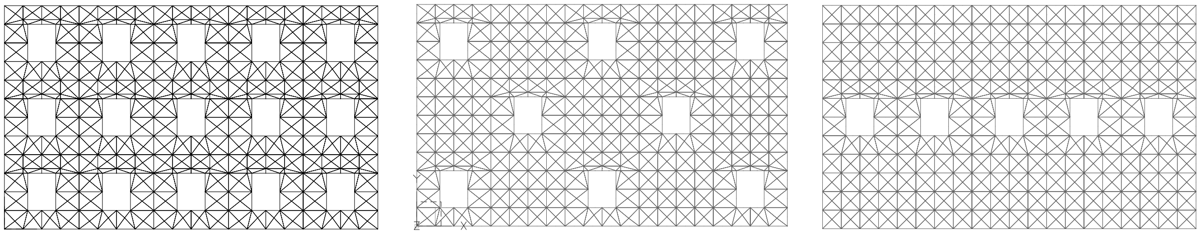
(c) Mesh 74

Figure A.3: **Meshes with L=40m, H=12m**



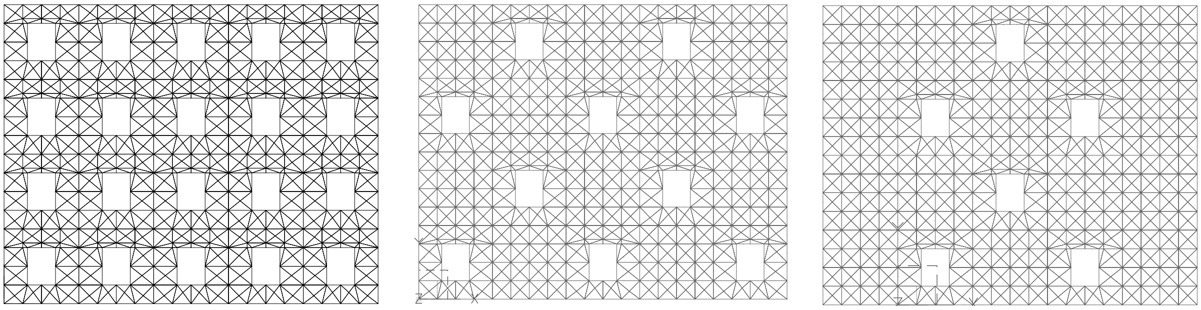
(a) Mesh 33 (b) Mesh 53 (c) Mesh 73

Figure A.4: **Meshes with L=20m, H=8m**



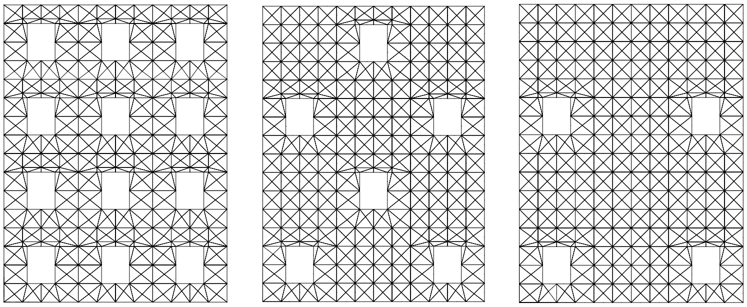
(a) Mesh 35 (b) Mesh 55 (c) Mesh 75

Figure A.5: **Meshes with L=20m, H=12m**



(a) Mesh 36 (b) Mesh 56 (c) Mesh 76

Figure A.6: **Meshes with L=20m, H=16m**



(a) Mesh 37 (b) Mesh 57 (c) Mesh 77

Figure A.7: **Meshes with L=12m, H=16m**

Appendix B

Formulation of beam element stiffness matrix components

B.1 Axial stiffness contribution

For the two-node straight beam of length, L, with nodal axial degrees of freedom u_1 and u_2 shown in figure 6.3 the choice of a linear axial displacement field leads to the following relationship for axial displacement, u, between the nodes

$$u = \boldsymbol{N_a^e d_a^e} \tag{B.1}$$

where

$$\boldsymbol{d_a^e}^T = [u_1,\ u_2] \tag{B.2}$$

$$\boldsymbol{N_a^e} = [1 - \frac{x}{L},\ \frac{x}{L}] \tag{B.3}$$

are the nodal displacements and axial element shape functions respectively.

Using the strain-displacement relationship $\boldsymbol{B} = d\boldsymbol{N}/dx$ gives

$$\boldsymbol{B_a^e} = \left[\frac{1}{L},\ \frac{1}{L}\right] \tag{B.4}$$

where $\boldsymbol{B_a^e}$ is the axial strain displacement matrix for an element. If A represents the cross sectional area and E is Young's modulus we obtain the axial stiffness matrix $\boldsymbol{K_a^e}$ for an element in the usual manner as

$$\boldsymbol{K_a^e} = \int_0^L \boldsymbol{B_a^{eT}} \left[EA\right] \boldsymbol{B_a^e} dx = \begin{bmatrix} \frac{EA}{L} & \frac{-EA}{L} \\ \frac{-EA}{L} & \frac{EA}{L} \end{bmatrix} \tag{B.5}$$

B.2 Torsional stiffness contribution

For the two-node straight beam of length, L, with nodal torsion degrees of freedom θ_{x1} and θ_{x2} shown in figure 6.3 the choice of a linear torsion displacement field leads to the following relationship for torsional rotation, θ_x, between the nodes

$$\theta_x = \boldsymbol{N_t^e d_t^e} \tag{B.6}$$

where

$$\boldsymbol{d_t^{eT}} = [\theta_{x1},\, \theta_{x2}] \tag{B.7}$$

$$\boldsymbol{N_t^e} = [1 - \frac{x}{L},\, \frac{x}{L}] \tag{B.8}$$

are the nodal displacements and torsional rotation element shape functions respectively. Using the strain-displacement relationship $\boldsymbol{B} = d\boldsymbol{N}/dx$ gives

$$\boldsymbol{B_t^e} = \left[-\frac{1}{L},\, \frac{1}{L}\right] \tag{B.9}$$

where $\boldsymbol{B_t^e}$ is the torsional strain displacement matrix for an element. If J represents the polar moment of inertia and G is modulus of rigidity we obtain the axial stiffness matrix $\boldsymbol{K_t^e}$ for an element in the usual manner as

$$\boldsymbol{K_t^e} = \int_0^L \boldsymbol{B_t^{eT}} \left[GJ\right] \boldsymbol{B_t^e} dx = \begin{bmatrix} \frac{GJ}{L} & \frac{-GJ}{L} \\ \frac{-GJ}{L} & \frac{GJ}{L} \end{bmatrix} \tag{B.10}$$

B.3 Derivation of bending shape functions

The bending shape functions for one independent plane of the two-node beam element shown in figure 6.3 are derived. The second plane in bending is independent and thus makes use of the same shape functions derived below but in the orthogonal direction. Shape functions for the axial degrees of freedom above are Lagrangian polynomials and are required here to derive the shape functions for the bending degrees of freedom as described in section 6.4.1. From equation 5.17, these are the order one Lagrange polynomials

$$L_1(x) = \frac{x - x_2}{x_1 - x_2} \tag{B.11}$$

$$L_2(x) = \frac{x - x_1}{x_2 - x_1} \tag{B.12}$$

For the two-noded beam with nodes at $x = 0, L$

$$L_1(x) = 1 - \frac{x}{L} \tag{B.13}$$

$$L_2(x) = \frac{x}{L} \tag{B.14}$$

Using the convention that $\bar{x} = x/L$

$$L_1(\bar{x}) = 1 - \bar{x} \tag{B.15}$$

$$L_2(\bar{x}) = \bar{x} \tag{B.16}$$

and the derivatives are

$$L_1'(\bar{x}) = -1 \tag{B.17}$$

$$L_2'(\bar{x}) = 1 \tag{B.18}$$

The bending shape functions are found using equations 5.15 and 5.16 as

$$N_1(\bar{x}) = H_{01}^1 \tag{B.19}$$

$$N_2(\bar{x}) = H_{11}^1 \tag{B.20}$$

$$N_3(\bar{x}) = H_{02}^1 \tag{B.21}$$

$$N_4(\bar{x}) = H_{12}^1 \tag{B.22}$$

It thus follows that

$$N_1(\bar{x}) = [1 - 2\,(\bar{x} - 0)\,(-1)]\,[1 - \bar{x}]^2 = 1 - 3\bar{x}^2 + 2\bar{x}^3 \tag{B.23}$$

$$N_2(\bar{x}) = L\,(\bar{x} - 0)\,[1 - \bar{x}]^2 = L(\bar{x} - 2\bar{x}^2 + \bar{x}^3) \tag{B.24}$$

$$N_3(\bar{x}) = [1 - 2\,(\bar{x} - 1)\,(1)]\,[\bar{x}]^2 = 3\bar{x}^2 - 2\bar{x}^3 \tag{B.25}$$

$$N_4(\bar{x}) = L\,(\bar{x} - 1)\,[\bar{x}]^2 = L(-\bar{x}^2 + \bar{x}^3) \tag{B.26}$$

and the second derivatives with respect to $\bar{x}$ required for the strain-displacement matrix are

$$N_1''(\bar{x}) = -6 + 12\bar{x} \tag{B.27}$$

$$N_2''(\bar{x}) = L(-4 + 6\bar{x}) \tag{B.28}$$

$$N_3''(\bar{x}) = 6 - 12\bar{x} \tag{B.29}$$

$$N_4''(\bar{x}) = L(-2 + 6\bar{x}) \tag{B.30}$$

Appendix C

Example equivalent elastic beam properties calculation

C.1 Introduction

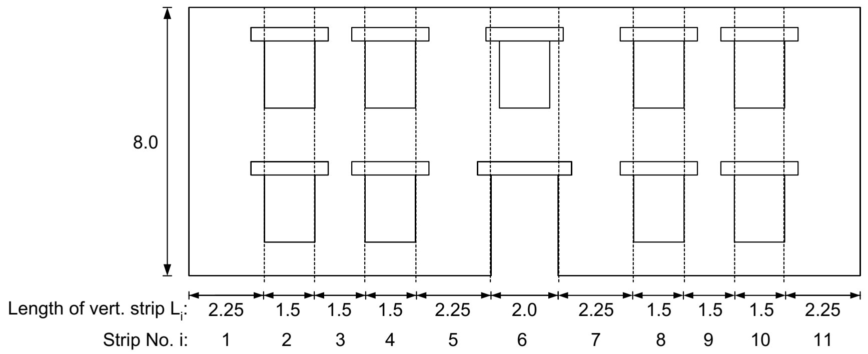
A fully worked example of the use of the procedures described in Chapter 4 for determining the effective cross sectional area, A^*, and effective second moment of area, I^*, for the beam approximation of a masonry facade is given in this section. The building facade analysed is the front facade of the building from the symmetric and oblique example problems described in Chapter 8 and shown here as figure C.1

C.2 Worked example

Part A - Shear

Take 11 vertical strips as shown and assume a wall thickness of 1m and thus a nominal cross section area, A, of 8m^2. Approximate the door as a window, thus making the assumption that all openings are windows of dimensions 1.5m wide and 2m high. The cross sectional area of each window is thus 2m^2. Facade L/H=2.5 is less than the critical L/H=3.0, thus

Part A Shear - Vertical strips i



Part B Bending - Horizontal strips j

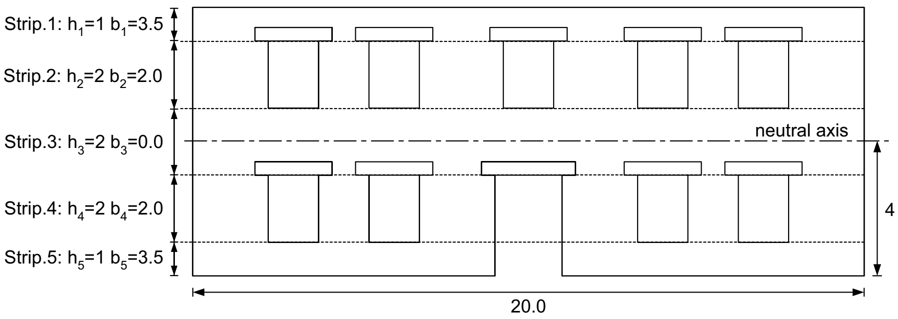


Figure C.1: **Facade dimensions (metres) for calculation of A^* and I^***

use the linear depression method (see section 4.3.2). For each strip, i, calculate L_i/A_i, where A_i is the cross sectional area net of windows, then sum and calculate A^* using equation 4.23. A strip wise calculation is shown in table C.1.

Table C.1: **Calculation of effective cross sectional area A^***

Strip.i	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
L_i	2.25	1.50	1.50	1.50	2.25	2.00	2.25	1.50	1.50	1.50	2.25
A	8	8	8	8	8	8	8	8	8	8	8
No.windows	0	2	0	2	0	2	0	2	0	2	0
$A_{windows}$	0	4	0	4	0	4	0	4	0	4	0
A_i	8	4	8	4	8	4	8	4	8	4	8
L_i/A_i	0.281	0.375	0.188	0.375	0.281	0.500	0.281	0.375	0.188	0.375	0.281
$\sum(\frac{L_i}{A_i}) =$	3.500										
$A^*=$	**4.762**										

Part B - Bending

Take five horizontal strips as shown and assume a wall thickness of 1m. Again, approximate the door as a window, thus making the assumption that all openings are windows of dimensions 1.5m wide and 2m high. The face area of each window is thus 3m^2. Facade L/H=2.5 is less than the critical L/H=3.0, thus use the linear depression method (see section 4.3.2). For each strip, j, calculate the effective height of the strip $h'_j = A_{fj}/L$, where A_{fj} is the face area net of windows. Calculate I_j for each strip using equation 4.20, then sum and calculate I^* using equation 4.24. A strip wise calculation is shown in table C.2.

Table C.2: **Calculation of effective second moment of area I^***

Strip.j	**1**	**2**	**3**	**4**	**5**
h_j	1	2	2	2	1
A_f	20	40	40	40	20
No.windows	0	5	0	5	0
$A_{windows}$	0	15	0	15	0
$A_{fj}(m^2)$	20	25	40	25	20
h'_j	1.00	1.25	2.00	1.25	1.00
b_j	3.5	2.0	0.0	2.0	3.5
I_j	12.333	5.163	0.667	5.163	12.333
$\sum(I_j) =$	35.659				
I^*=	**29.716**				

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