# Masonry walls with flanges

EDGe, "Martin Marietta Prism Testing," Technical Report Y/EN 4619, Martin Marietta Energy Systems, Inc., Oak Ridge, TN, 1989.

Hathcock, B.D., and Skillman, E., "Tests of Hollow Building Tiles," Technologic Papers of the Bureau of Standards No. 120, 1918.

Johnson, F.B., and Matthys, J.H., "Structural Strength of Hollow Clay Tile Assemblages," Journal of the Structural Division, ASCE, 1973, pp. 259-275.

Matthys, J.H., "Structural Performance of Hollow Clay Tile Wall Assemblages," PhD Dissertation, The University of Texas at Austin, 1972.

Parsons, D.E., "Factors Affecting the Strength of Masonry of Hollow units," Bureau of Standards Journal of Research, 6, Research Paper RP310, 857-867, 1931.

Plummer, H.C., "Brick and Tile Engineering," Brick Institute of America, Reston, VA, 1962.

Plummer, H.C., and Wanner, E.F., "Principles of Tile Engineering," Structural Clay Products Institute, Reston, VA, 1947.

Stang, A.H., Parsons, D.E., and Foster, H.D., "Compressive and Transverse Strength of Hollow Tile Walls," U.S. Bureau of Standards, Technologic Papers No. 311, Washington, D.C., 1926.

SCPI, "Compressive, Transverse, and Racking Strength Tests of Four-Inch Structural Clay Facing Tile Walls," Structural Clay Products Institute, Research Report No. 11, Reston, VA, 1967.

SCPI, "Compressive, Transverse and Shear Strength Tests of Six and Eight-Inch Single-Wythe Walls Built with Solid and Heavy Duty Hollow Clay Masonry Units," Structural Clay Products Institute, Research Report No. 16, Reston, VA, 1969.

Whittemore, H.L, and Hathcock, B.D., "Some Compressive Tests of Hollow-Tile Walls," U.S. Bureau of Standards, Technologic Papers No. 238, Washington, D.C., 1923.

Whittemore, H.L, Stang, A.H., and Parsons, D.E., "Structural Properties of Six Masonry Wall Constructions," U.S. National Bureau of Standards, Building Materials and Structures Report No. 5, Washington, D.C., 1938.

THE SIXTH NORTH AMERICAN MASONRY CONFERENCE

June 6-9, 1993

Philadelphia Pennsylvania

# MASONRY WALLS WITH FLANGES

H.J. Vekemans[1], O.A. Arce[2]

## ABSTRACT

This paper describes the state of development of a current research project at the University of Technology of Eindhoven in The Netherlands. The influence of flanges on the stiffness of unreinforced brick masonry walls is examined. The DIANA Finite Element code is used for the numerical analysis of the walls according to two different basic models. Bricks and interface elements are considered first in a two dimensional analysis of the walls. Second, a three dimensional model is considered. Preliminary results are presented as plots of displacements and stresses at certain sections of interest for loads in the elastic range. A description of the next phases of the project is included.

## INTRODUCTION

A substantial amount of work on the in-plane capacity of masonry walls has been published during the years through congresses, seminars and publications all over the world. Much of the work deals with plain walls and their many versions, with little consideration of the effect of the presence of flanges, or of the interaction of incoming elements.

Flanged walls do occur very frequently in real construction. Perhaps the most common case is the intersection of internal partitions and external walls. A work has been presented

[1]Ph.D. student, Faculty of Architecture Building and Planning, Department of Structural Design, University of Technology of Eindhoven, The Netherlands.

[2]Ph.D.student, Faculty of Architecture Building and Planning, Department of Structural Design, University of Technology of Eindhoven, The Netherlands. Senior Lecturer and Researcher, Department of Construction Engineering, Institute of Technology of Costa Rica.

by Rahman and Suter (1985) on such a respect. The authors consider the effect of temperature gradients between the external flange and the internal partition on the distribution of stresses on the entire building structure. Reported results showed how flange and web behave integrally in carrying stresses over the structure. Of particular interest is the T-wall, not only because of its rate of occurrence but also because of the different characteristics they exhibit depending upon the direction of the in-plane horizontal forces. The disposition of vertical reinforcement plays a significant role in the capacity and postpeak behaviour of such walls (Hart, e.a., 1987). The importance of the analysis of this type of walls has been already acknowledged in the US-TCCMAR program (Priestley and Limin, 1988; 1990).
A comparison of the provisions of six european codes for masonry design and construction (Vekemans, 1992) showed the need for further research on the structural behaviour of flanged plain masonry walls under in-plane lateral loads . It was also found that the scarce code provisions for flanged walls, in the codes analyzed (Anonymous, 1978; 1980, 2,3; 1984; 1988; 1989; 1990), have mostly an empirical basis, which offers limited advantages to the consideration of flanged walls in the calculations (Vekemans, opus cit.).
Flanges affect the lateral capacity of walls and their stiffness. This fact may significantly affect the distribution of forces among the elements of the building and it may add unaccounted torsional effects. A current project at the University of Technology of Eindhoven aims to studying the influence of flanges on the stiffness of unreinforced brick walls by numerical analysis using the Diana Finite Element code (Anonymous, 1991). Some preliminary results are presented in this paper.

## TWO DIMENSIONAL ANALYSIS

### Study Cases

For gaining a better understanding of the problem a T-shaped wall with a length of 540 mm was examined. The height of the walls was in one case 982 mm and in the other case 1 974 mm. Both the length and the height of the wall were the result of the dimensions of the bricks in the Netherlands, which are about 210 mm x 52 mm x 100 mm (l x h x w). By using joints with a thickness of 10 mm, and one length of 2½ bricks and two heights of 16 and 32 layers, this resulted in the chosen dimensions of the walls (Fig. 1). The dimensions of wall w6 are in agreement with previous numerical research in the Netherlands (Coenraads, 1991). Table T-1 presents an overview of the cases considered for the initial 2-D analysis.

Table T-1 : Cases under study in the 2-D analysis.

| code | w6 | w8 |
|---|---|---|
| overall dimensions | 540 x 982 mm | 540 x 1 874 mm |
| flange dimensions | (100) 200 300 400 500 | (100) 200 300 400 500 |
| vertical loads | 25 N/mm | 25 N/mm |
| horizontal loads | 1 210 and 2 383 N | 590 and 1 201 N |
| non linear analysis characteristics | step by step,deformation controlled | step by step,deformation controlled |



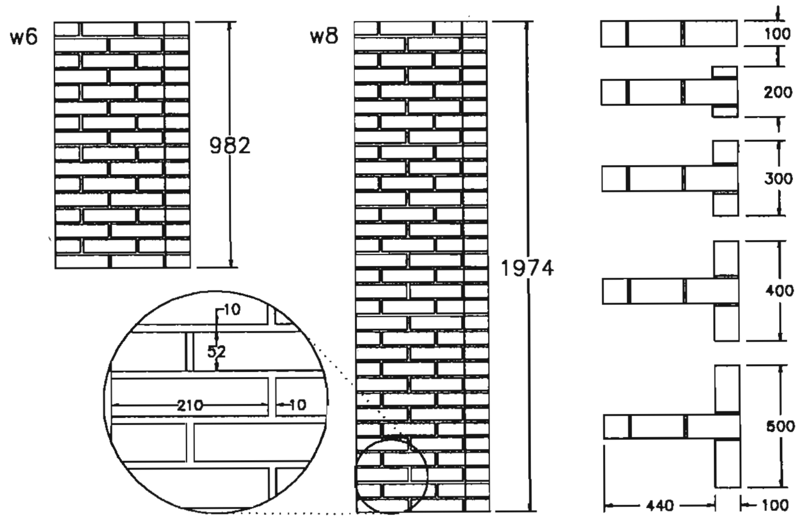

Fig.1: Dimensions of bricks, joints and walls w6 and w8.

### Model Description

Some research has taken place in the Netherlands, investigating the possibilities of modelling masonry with the Diana Finite Element code (Rots, 1991). The results showed that several models can be used, depending on the required degree of accuracy or simplicity and on the time needed for the calculation (Eijkman, 1991). The research showed also that the use of continuous elements in combination with discontinuous elements for the bricks and only discontinuous elements for the joints, leads to acceptable results in predicting ultimate capacity (Coenraads, opus cit.). This model was chosen to start with.

### Geometry and elements

The two walls (w6 and w8) considered in the 2-D analysis were modelled by plane stress and interface elements. The two types of elements were:

- CQ16M — A quadrilateral, quadratic, plane stress element, with 8 nodes and consequently 16 degrees of freedom, and a 2x2 Gauss-integration scheme.
- CL12I — A quadratic line interface element, with 6 nodes and consequently 12 degrees of freedom, and a Lobatto-integration scheme.

The bricks were modelled by using two continuous elements and one interface element, placed vertically at the middle of the brick, allowing it to crack at that position. The joints were modelled as interface elements. Figure 2 shows an overview of the generated mesh. At the intersection of head and bed joints there is a slight imperfection in the mesh, because no element is present, but this imperfection has almost no influence on the results of the calculations (Eijkman, opus cit.; Coenraads, opus cit.).

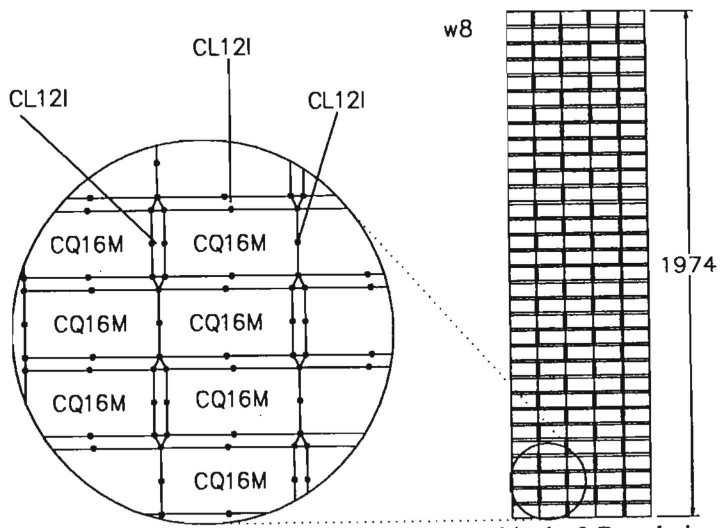


Fig.2: Geometry and elements of the model used in the 2-D analysis.

One of the limitations of the model was having only two dimensions, so flanges were modelled by only changing the thickness of the elements of the flange.

The wall considered in the 3D analysis was modelled by solid elements, namely quadratic hexahedrons with 20 nodes and 60 degrees of freedom. A 2x2x2 Gauss Integration was used. The web was modelled in a grid pattern of 100x10x100 mm in the horizontal direction and 52x10x52 mm in the vertical direction. The thickness of the web and the flange was 100 mm, the same as in the 2D analysis.

The bottom of the wall was fixed. Besides this support condition, tyings of some of the nodes needed to be used. Tying means relating nodes together by assuming proportional translation or rotation of the nodes. For the horizontal load at the top left brick, the three nodes of the brick element had to be tied, to avoid unwanted stress concentrations that would occur in the middle of the matching element. The tying was provided by modelling a proportional translation of the nodes in the surface of application of the force. In the case of the non-linear analysis, an equal horizontal translation was prescribed. This tying modelled a more realistic condition, as there is normally a reinforced concrete floor resting on top of the wall. Besides this tying assured that horizontal forces were equally spread over the top of the wall.

**Material**

Two different approaches were used in the 2-D analysis. For the linear elastic analysis bricks and joints were assumed to be isotropic and linear elastic. Material properties for the bricks were a Young's modulus $E_b$=16,700 [N/mm$^2$] and a Poisson's ratio $\nu$=0.28 [-]. The properties for the joints were a normal stiffness $E_j$=13,300 [N/mm$^2$] and a shear stiffness $G_j$= 5,980 [N/mm$^2$]. These values were taken from research carried out recently on a Dutch type of masonry, consisting of wire cut bricks (Joosten) and mortar made of cement:lime:sand, 1:½:4½ by volume (Vermeltfoort, 1991; Van der Pluijm, 1991).

In the non-linear analysis masonry was assumed to be isotropic and linear elastic until the first crack occurred. During the development of cracks it was assumed that elasticity was still valid for the rest of the material, but softening occurs at the crack region. This means that the bricks maintain a linear elastic behaviour and that all the non-linear elastic behaviour of the wall is present in the joints and the interface elements in the middle of the brick. The same values were used for the case of the linear elastic analysis as well as for the non-linear elastic analysis. On top of it a criterion of maximum normal traction (discrete cracking) was added, which means that a discrete crack arises if the normal traction exceeds the tensile strength of the joint ($f_{c,j}$), or the interface element in the middle of the brick ($f_{c,b}$). A linear tension softening diagram was added, described by the amount of fracture energy ($G_f$). This criterion was combined with a shear criterion after cracking, by which is meant that the shear modulus does not drop to zero after cracking. A reduced value of the shear modulus of $\beta$ times $G$ was assumed, in both interfaces a value of $\beta$=0.1 was used. The properties for the 2D brick interface model were the same as the properties for the 3D model, namely $E_b$=16,700N/mm$^2$, $G_b$=6,523N/mm$^2$, $\nu_b$=0.28, $f_{c,b}$=2.36N/mm$^2$ and $G_{f,b}$=0.12J/mm$^2$. The properties for the joint interface were taken as $E_j$=13,300N/mm$^2$, $G_j$=5,980N/mm$^2$, $\nu_j$=0.112, $f_{c,j}$=0.44N/mm$^2$ and $G_{f,j}$=0.012J/mm$^2$.

**Loading**

Walls were loaded with a constant uniformly spread normal force on the top, in combination with either a horizontal force or a controlled deformation of the top of the wall. The normal force was 25 N/mm and it was located only over the continuous elements, resulting in a total force of 12,980 N ( 519.2 mm times 25 N/mm).

The horizontal force on the wall varied, depending on the case which was under examination. In the linear elastic analysis of the flanged walls two different situations were scrutinized.

First no tensile stresses were allowed in the wall without a flange and, second, it was assumed that the lowest bed-joint was cracked (tensile stresses do arise ) over a length of one third of the length of the wall. The resulting horizontal forces were 1,210 N and 2,383 N respectively for wall w6, and for wall w8 forces of 590 N and 1,201 N. These forces were concentrated at the middle of the top left brick of the wall, at a height of 956 mm or 1948 mm from the bottom of each wall. In the non-linear analysis walls were stressed by controlled horizontal deformation of the top.

**2-D RESULTS**

Under the assumption of full elasticity of the components, walls were first examined to observe the effect of different flange widths on the stiffness.

Figure 4 shows the effect of changing the flange width on wall w6 accordingly to the conditions in table T-1. In the figure it is possible to observe that the predictions of the 2D Model ( 2DFEL1 and 2DFEL2 for loads 1 and 2 respectively ) lay well under those of a simple elastic calculation ( E.Th.L1 and L2 ) though the difference becomes smaller

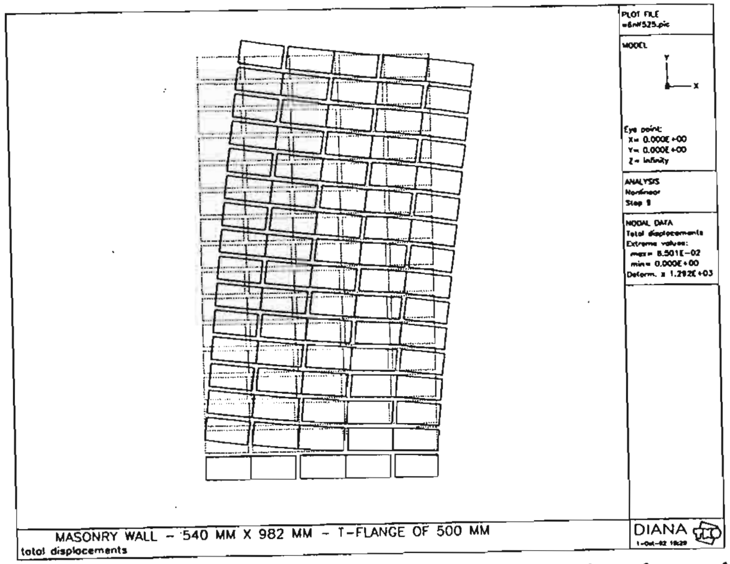


Fig.3: Total displacements of non-elastic analysis of wall w6, just after cracking.

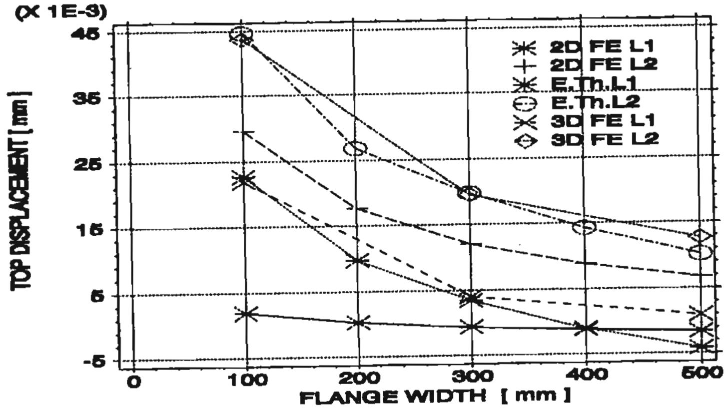


Fig.4: Flange width influence on top deformation of wall w6, elastic analysis.

as the flange width increases. This is probably due to the fact that interface elements in the 2D Model do not adequately represent real deformations in the joints, while average masonry properties used in the elastic calculation account better for the overall behaviour, at least in the load range under consideration. The 3D Model seems to match better expected deformations. At and below the 300 mm width flange it gives good agreement with the elastic calculation results.



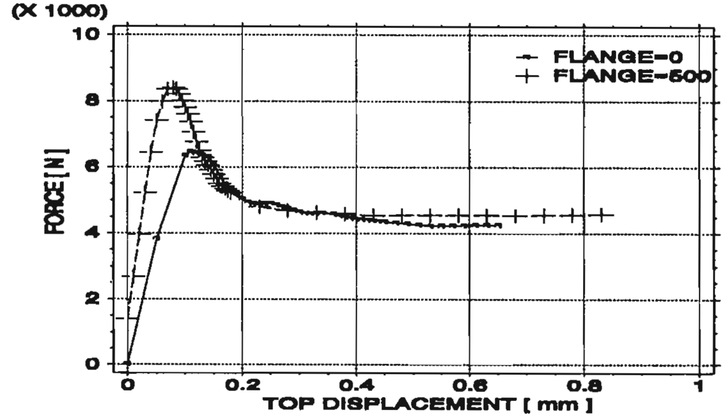


Fig.5: Force versus deformation of wall w6, non-elastic analysis.

A non elastic analysis was undertaken to observe the influence on the behaviour of the wall. The result of the last is shown in figures 3 and 5.
Figures 6 and 7 show both the result of the elastic and the non-elastic analysis of wall 8.

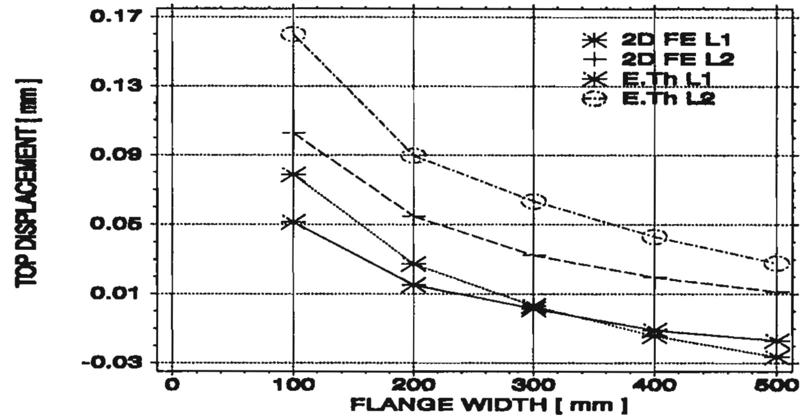


Fig.6: Flange width influence on top deformation of wall w8, elastic analysis.

Compared to the shorter wall the flange of wall w8 increased its participation to the overall stiffness. It has to be noted that the 2D Model does not take into account any possible influence of shear lag effects in the flanges, nor the deterioration that may occur due to cracking in the plane of the flange, problems that will be accounted for as the research proceeds. In any case, the contribution of the flange remains almost unchanged because it influences mainly bending deformations, being these ones the largest component of all in the cases under consideration.

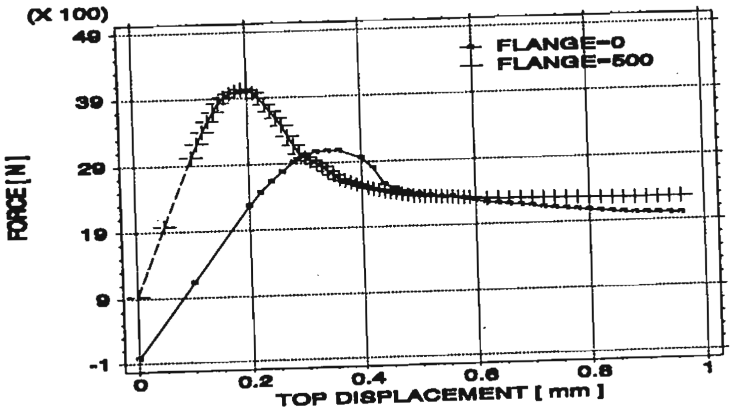


Fig.7: Force versus deformation of wall w8, non-elastic analysis.

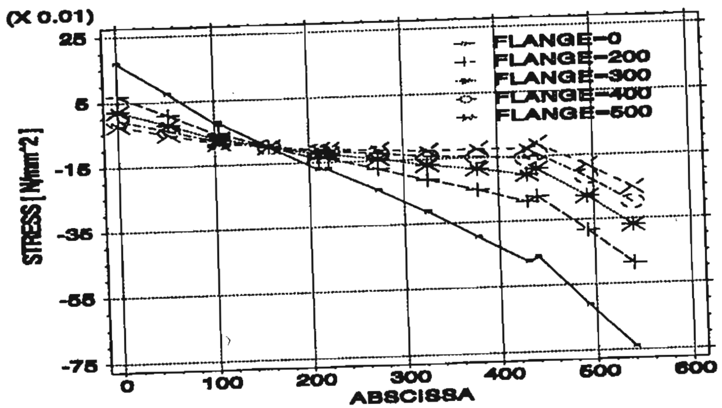


Fig.8: Stress distribution at the first bottom bedjoint of wall w6, horiz. load is 2 383 N.

**Stress distribution**

The way the 2D Model accounts for the effect of flange width on the capacity of the wall is also illustrated in the results depicted in figures 8 and 9, for the two walls w6 and w8. Sections were taken to examine the stress distribution on top of the first layer of bricks, at the first bedjoint level.
For the sake of comparison, stresses were also calculated at other levels accordingly to elastic beam theory. The different stress distributions are shown in figure 10 for wall w6.



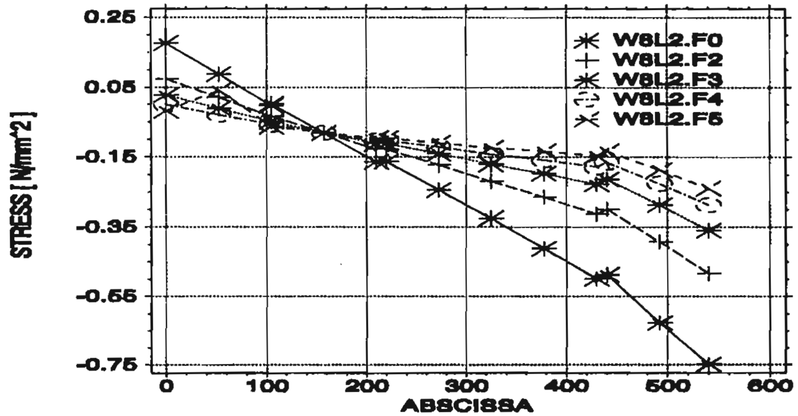


Fig.9: Stress distribution at the first bottom bedjoint of wall w8, horiz. load is 1 201 N.

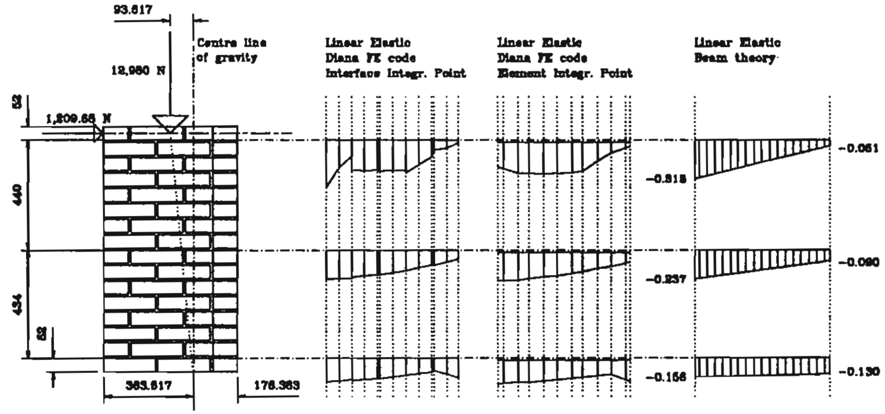


Fig.10: Different stress distributions for wall w6 (flange width is 500 mm) as a result from the Diana analysis and accordingly to elastic beam theory.

As said before, the 2D Model produces stresses and peak capacity predictions in good agreement with experimental data.

**3-D MODEL**

The 3-D model supports the need for further examining the influence of the flange on the stiffness of the walls. Among its advantages over the 2-D analysis, the late one fails to account for shear lag effects and other mechanisms arising from the non homogeneous distribution of stresses in the flange plane, or within the thickness of the flange (warping, shear concentrations in the connection between flange and web, etc).
A 3-D model is currently under consideration and development. The approached mesh is

illustrated in figures 11 and 12. The model will be calibrated with experimental results in agreement to other research going on at the University of Technology of Eindhoven (Vermeltfoort; e.a., 1993).

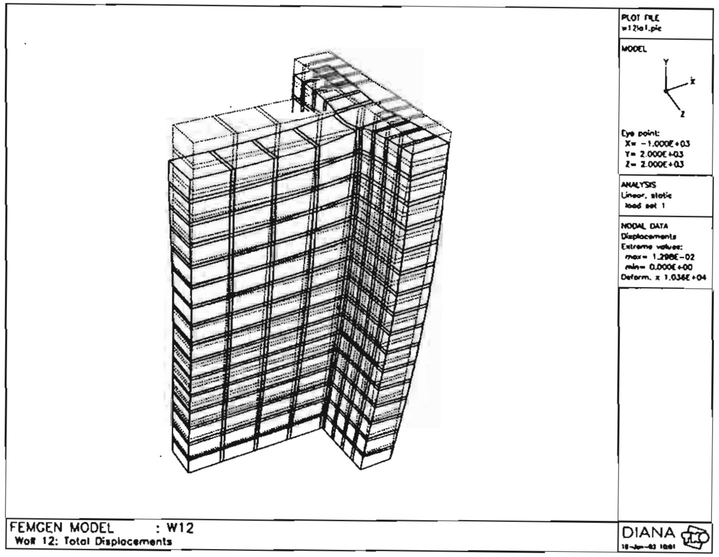


Fig.11: Isometric presentation of the deformed 3-D model, comparable with wall w6 loaded with a horizontal load of 1 210 N.

## CONCLUSIONS AND PERSPECTIVES

The importance of the analysis of masonry walls with flanges was already acknowledged in several research reports. This research provides some more ground to those acknowledgements. The results of the 3-D analysis show that this may be a good tool in the analysis of the stiffness of the wall. When the project is finished, the results from the numerical analysis will be employed for the production of design tools for the calculation of the stiffness of flanged unreinforced brick walls.

It has to be bore in mind that the results presented in this paper are dependant upon the models and material parameters that were chosen. Especially the material model in combination with the values for the material properties influenced the results of the calculations. These values, taken from tests on masonry specimens, are sensitive to the test methods and the test setup, as has already been acknowledged in previous research (Page; e.a., 1985). This is why in the general proposal for this research project it is strongly advised to give some more profound attention to the testing of material properties. On the other hand tests are also proposed for checking the final results of the finite element analyses, by using full scale tests on masonry walls.



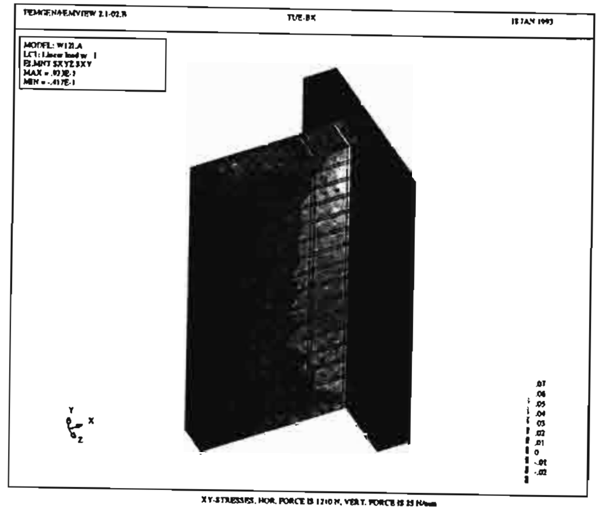

Fig.12: Isometric presentation of the xy-stresses in the 3-D model.

## NOTATION

| | |
|---|---|
| $f_{c,b}$, $f_{c,j}$ | is the tensile strength of the brick or the joint |
| $\beta$ | is the reduction factor for the shear modulus after cracking |
| $G_{f,b}$, $G_{f,j}$ | is the fracture energy for the brick or the joint |
| $E_b$ | is the Young's modulus and normal stiffness for the brick |
| $E_j$ | is the normal stiffness for the joint |
| $G_b$, $G_j$ | is the shear stiffness for the brick or the joint |
| $\nu_b$, $\nu_j$ | is the Poisson's ratio for the brick or the joint |


## ACKNOWLEDGEMENTS
This work was carried out at the University of Technology of Eindhoven (TUE) in The Netherlands as part of a project dealing with the structural design of masonry. The work is sponsored by the Technology Foundation (STW) and steered by working-group 1 of steering committee C76 of the Centre for Civil Engineering, Research and Codes (CUR).


## REFERENCES


1. Anonymous; 1978: "Code of practice for: Use of masonry. Part 1 Structural use of unreinforced masonry". BS 5628 Part 1, British Standard Institution (BSI).
2. Anonymous; 1980: "Ontwerp en berekening van metselwerk". NBN B 24-301, Belgisch Instituut voor Normalisatie.
3. Anonymous; 1980: "Mauerwerk. (Backstein, Kalksandstein, Zementstein, Gasbetonstein) Verständigung, Planung, Berechnung und Bemessung, Material, Ausführung". SIA 177, Schweizerischen Normen-Vereinigung (SNV).
4. Anonymous; 1984: "Mauerwerk. Mauerwerk nach Eignungsprüfung Berechnung und Ausführung". DIN 1053 Teil 2, Deutsches Institut für Normung e.V. (DIN).

5. Anonymous; 1988: Eurocode No 6, "Common unified rules for masonry structures". Report EUR 9888 EN, Commission of the European Communities.
6. Anonymous; 1989: "Bemessung von Mauerwerkswänden". SIA V 177/2, Schweizerischen Normen-Vereinigung (SNV).
7. Anonymous 1990: "TGB 1990 - Steenconstructies. Basiseisen en bepalingsmethoden". Ontwerp NEN 6790, Nederlands Normalisatie Instituut.
8. Anonymous; 1991: "DIANA Finite Element Analysis - User's Manual". TNO Building and Construction Research, Rijswijk, July.
9. Coenraads, A.C.; 1991: "Numerieke Detailberekeningen van Penanten in Metselwerk - Onderzoek naar Modeleringsaspecten binnen DIANA" (in Dutch). Graduate Thesis, TNO Building and Construction Research Report BI-91-073 / TU Delft Report 25.2.91-2.07.
10. Eijkman, S.M.; 1991: "Numeriek Onderzoek van Scheurvorming in Metselwerk. Deel I: Fysisch niet-lineaire berekeningen aan metselwerk, Deel II: Voeg-steen modellering op micro-niveau" (in Dutch). Graduate Thesis, TNO Building and Construction Research Report BI-91-206 / TU Delft Report 25.2.91-2.19.
11. Hart, G.C., R.E. Englekirk, R.L. Mayes, T. Kelly; 1987: "1988 Uniform Building Code, Limit State Strength Design Criteria Shear Walls". Proceedings of the Fourth North American Masonry Conference, August 16-19, University of California, Los Angeles, California, USA, pp 4.1-4.18.
12. Page, A.W., P.W. Kleeman, M. Dhanasekar; 1985: "An In-plane Finite Element Model for Brick Masonry". New Analysis Techniques for Structural Masonry. CI-proceedings of a Session held in conjunction with Structures Congress '85, ASCE, New York, USA, pp 1-18.
13. Pluijm, R. van der; 1991: "Material Properties of Masonry and its Components under Tension and Shear". Proceedings of the 6th Canadian Masonry Symposium, Saskatoon, Saskatchewan, Canada, 15-17 June, pp 675-686.
14. Priestley, M.J.N., He Limin; 1988: "Seismic behaviour of Flanged Masonry Shear Walls: Preliminary studies". Structural Systems Research Project Report N.SSRP88-01, University of California, San Diego, 119pp.
15. ___________; 1990: "Seismic Response of T-Section Masonry Shear Walls". Proceedings Fifth North American Masonry Conference, June 3-6, pp359-372.
16. Rahman, A.H., G.T. Suter; 1985: "Thermal Stresses in Intersecting Masonry Walls". Proceedings of the Third North American Masonry Conference, June 3-5, University of Texas, Arlington, Texas, USA, pp 57.1-57.16.
17. Rots, J.G.; 1991: "Computer Simulation of Masonry Fracture: Continuum and Discontinuum models". Proceedings of the International Symposium on Computer Methods in Structural Masonry, Swansea, UK, 3-5 April, pp 93-103.
18. Vekemans, H.J.; 1992: "Structural Design Methods in European Masonry Codes". Proceedings of the 6th Canadian Masonry Symposium. Saskatoon, Saskatchewan, June 15-17, pp 573-584.
19. Vermeltfoort, A.Th.; 1991: "Mechanical Properties under Compression of Masonry and its Components". Proceedings of the 6th Canadian Masonry Symposium, Saskatoon, Saskatchewan, Canada, June 15-17, pp 657-662.
20. Vermeltfoort, A.Th., Th.M.J. Raijmakers, H.J.M. Janssen; 1993: "Shear Tests on Masonry Walls of 1x1 m$^2$". Proceedings of the 6th North American Masonry Conference, Philadelphia, Pennsylvania, June 6-9.





THE SIXTH NORTH AMERICAN MASONRY CONFERENCE

June 6-9, 1993

Philadelphia Pennsylvania


# DIFFERENTIAL MOVEMENT IN CAVITY WALLS AND VENEER WALLS DUE TO MATERIAL AND ENVIRONMENTAL EFFECTS


Ajay Goyal[1], Michael A. Hatzinikolas[2] and Joseph Warwaruk[3]



## ABSTRACT

Two common cavity walls used in buildings are assemblies consisting of an exterior weathering veneer, a 25 mm air space, 25 to 75 mm or more insulation, and an interior backup wall of concrete masonry or steel stud. The two wythes are connected by ties of which a variety of shapes and function exist. Due to climatic effects and material properties the brick veneer may expand and the backup wythe may shrink or remain dimensionally stable. The brick veneer is also subjected to a variation in temperature while the backup wythe is under relatively constant temperature. These temperature differences may lead to differential movement between the two wythes, which in turn may cause stresses within the assembly, if movements are restricted. Provided that stresses remain relatively low it is desirable to partially restrict the differential vertical movement.

Two types of backup systems are evaluated, namely: concrete block and steel stud, with both connected to the veneer by ties which provide resistance to vertical movement. A analytical study has been carried out and a approximate method to evaluate the forces induced to the assembly is proposed. The restrain to differential movement provided by the ties causes stresses on both wythes. These stresses are in general in the form of compression in the brick wythe and tension in the backup system. By designing these ties for stiffness, the differential movement can be of benefit to the structural performance of the assembly and it can also improve serviceability by reducing cracking of the veneer.



1. Research Associate, Prairie Masonry Research Institute, Edmonton, Alberta, Canada.
2. Executive Director, Prairie Masonry Research Institute, Edmonton, Alberta, Canada.
3. Professor, Civil Engg., University of Alberta, Edmonton, Alberta, Canada.