Non-Linear Structural Analysis of Shear Connected Cavity Walls subject to Wind Load

by

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ABSTRACT

This study presents a comprehensive structural analysis of shear connected cavity walls, vertically spanned, subject to lateral load. The cavity wall investigated in this study is a masonry assembly comprising two wythes separated by a continuous cavity and tied together, via non-conventional metal connectors. Since the introduction of the new Block Shear™ Connector the role and the structural behaviour of traditional cavity walls with flexible ties changed significantly. Also, the new, Canadian Standards Association Standard CSA CAN3-S304.1-M94 Masonry Design for Buildings - Limit States Design *introduces strength and serviceability requirements that must be met in design. For both reasons, the author recognized a great need for a rational approach and more realistic prediction of structural performance of the cavity wall. Currently, the masonry industry is looking into a method to take advantage of the unused structural potential of the outer wythe by reducing the material and construction costs.*

The realistic determination of the response of either a plain or reinforced shear connected cavity wall demands knowledge of the inelastic behaviour of all constituent parts and the ability to incorporate these into a rational analysis of the real structure. Since a precise analysis is highly complex, this requires a reasonable compromise between reality and the use of simplifying assumptions: firstly, in the formulation of material and geometric properties, secondly in simulating the structure with a mathematical model and finally, in the use of the principles of mechanics.

In this study, the proposed method is conceptually founded on the premise that the method of analysis should be independent of the procedure for estimating material properties in order to be valid for current as well as for possible future knowledge of these properties.

The proposed Method of Imposed Rotations which falls into the category of Separation Methods is a special type of non-linear analysis. It is based on the Principle of Superposition, with material non-linear stress-strain relationships, and consequently non-linear constitutive relationships accounted for.

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LIST OF SYMBOLS

A_{st} area of reinforcement

ds unit length

EI stiffness

EI_{act} actual stiffness

EI_{eff} effective stiffness

E_m modulus of elasticity for masonry in compression

E_{mt} modulus of elasticity for masonry in tension

E_s modulus of elasticity for steel

f'_{mt} flexural tensile masonry strength

f_t tensile masonry stress

f'_m compressive masonry strength

f_m compressive masonry stress

f_{mft} modulus of rupture

f_y steel yielding stress

f_s steel stress

H hollow unit

Kn kilonewton

L cavity width

L_1 V-tie protrusion length

L_2 steel plate protrusion length

M	moment
M_u	ultimate moment
M_{cr}	cracking moment
M_p	moment of plasticity
M_{el}	moment of elasticity
M^f	fictituous moment
M_{tot}	total moment
M_y	yielding moment
mm	millimeter
Mpa	megapascal
N	axial force
P^f	equivalent fictituous concentrated force
q	wind load
R	reaction force
R^f	fictituous resultant force
S	solid unit
S_f	fully solid unit
T	transverse force
T^f	fictituous transverse force
X_i	redundant unit moment
z	plastic section modulus of V-tie
ε_{mu}	ultimate masonry strain

ε_{st}	steel strain
ε_{su}	ultimate steel strain
ε	strain
ε_{cen}	strain at the centroid
ε_{in}	strain at inner face
ε_{out}	strain at outer face
ε_{u}	ultimate strain
ε'_{m}	strain due to compressive masonry strength
γ	shear strain
δ	unit displacement
θ_{i}	mutual total rotation
θ_{t}	slope due to shear strain
θ	slope
Θ, Ψ	plastic rotation of cross-section
ϕ	curvature
ϕ_{el}	elastic portion of curvature
ϕ_{pl}	plastic portion of curvature
ϕ_{u}	ultimate curvature
Ω	area of curvature due to plastic deformation

1. INTRODUCTION

1.1 Cavity Wall

The cavity wall investigated in this study is a masonry assembly comprising two wythes separated by a continuous cavity and tied together, via non-conventional metal connectors (See Figure 1.1-1). The term wythe can be defined as *a masonry wall of one masonry unit in thickness*[1]. It belongs to the category of non-load bearing walls, since no vertical loads are involved in analysis. The tie, which can transfer shear and is also refered to as the Block Shear™ Connector (See Figure 1.1-2), provides a certain degree of composite structural action between the two wythes. How much depends on the properties of the wall's components. Generally, it enhances the integrity of the whole assembly by mobilizing the structural potential of the exterior wythe. It is clearly apparent that by providing the stiffer connector, which introduces composite action, a more structurally rigid cavity wall is obtained. However the effective stiffness of the connector is somewhat limited by the different deformation properties of brick and concrete block due to changes in humidity and temperature effects. It should be borne in mind that the cavity wall is an indeterminate structure and environmental factors can generate significant undesirable stresses.

This study presents a comprehensive structural analysis of shear connected cavity walls, vertically spanned, subject to wind load. The new shear-connector[2] changes significantly the role and the structural behaviour of traditional cavity walls with flexible ties. Also, the new Standard, CSA CAN3-S304.1 - M94 *Masonry Design for Buildings - Limit*

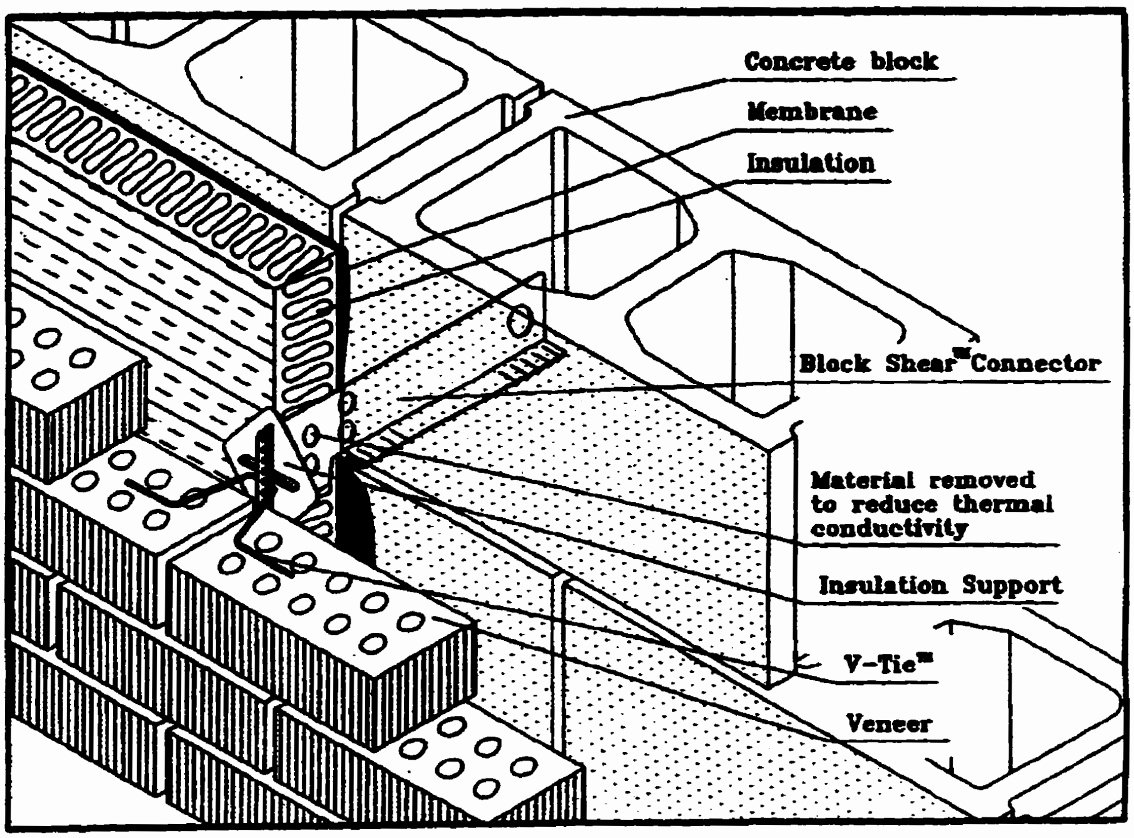

Figure 1.1-1 Cavity Wall (*courtesy "Tallcrete"*)

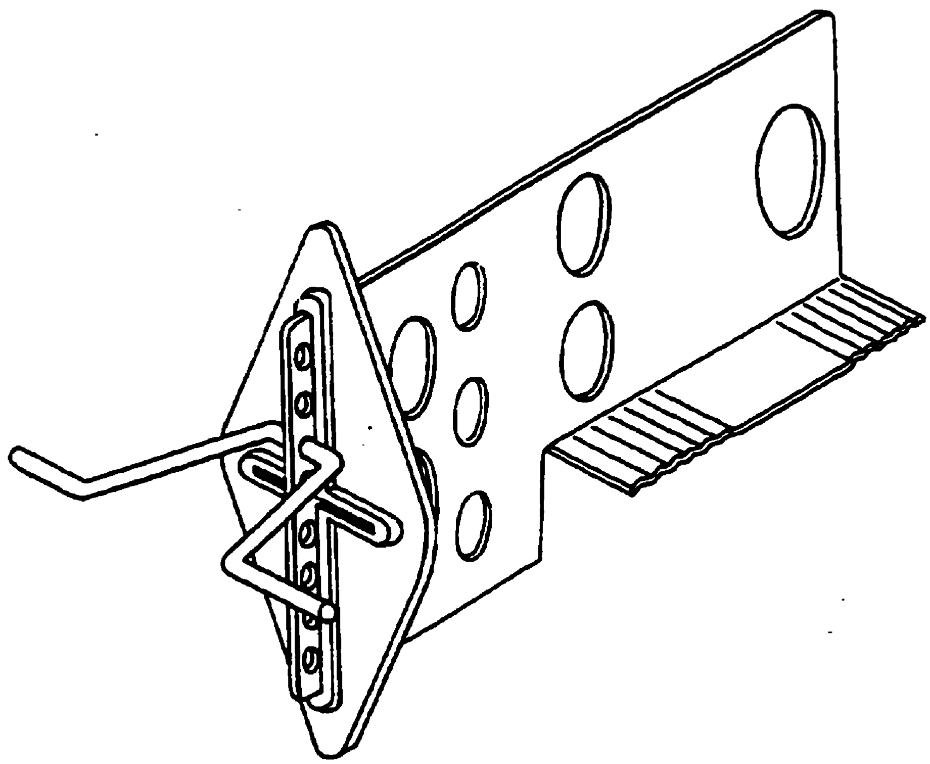

Figure 1.1-2 Shear™ Connector (*courtesy "Fero"*)

States Design[3], introduces requirements, such as strength and serviceability, that must be met in design. For both reasons, there is a great need for a rational approach and more realistic prediction of structural performance of the cavity wall.

Apparently, any method that ignores the elasto-plastic nature of the cavity wall's component materials cannot provide satisfactory prediction of strength. The realistic determination of the response of either a plain or reinforced cavity wall demands knowledge of the inelastic behaviour of all constituent parts and the ability to incorporate these into a rational analysis of the real structure. Since a precise analysis is highly complex, this requires a reasonable compromise between reality and the use of simplifying assumptions.

Initially the structure possesses a certain "amount" of stiffness, which depends on material and geometric properties. Due to external influence, wind load for example, the structure is forced to deflect laterally. Since only reversible elastic deformations take place, the relationship between the maximum deflection and applied load is linear. At one point, due to a higher load, one section starts to behave elasto-plastically. The plastic components of internal deformations are irreversible and the original stiffness of the structure is affected. The overall stiffness of the cavity wall decreases and consequently the load-deflection relationship continues in a non-linear fashion. With a further load increase new phenomena may occur. These could take the form of cracks at the mortar joints and their propagation, and/or plastification of highly stressed sections of V-ties. Both processes contribute to a considerable loss of overall stiffness of the cavity wall and its ability to resist further loads.

This stage is characterized by large deflection. The theory of small deformations is still valid, and without significant axial forces, all equilibrium equations can be expressed in terms of the original geometry of the structure. Therefore, there is no need for a second order analysis. The structure collapses when it reaches ultimate carrying capacity. At which load and in what mode a cavity wall will fail depends on geometry, and material and sectional properties of all components. If a construction factor is excluded, the material failure will likely occur at an ultimate load.

Shear connectors have the ability not only to transfer a lateral load from the veneer to the backup wall, but also to generate shear forces, which in turn produce beneficial positive moments in both wythes. The induced axial compression forces in the veneer wall and the induced axial tension forces in the backup wall are relatively small and for simplicity can be neglected in calculations. Dividing the bending moment by the axial force yields a large eccentricity close to pure flexure. Moments created by shear forces are important since they enhance the capacity of both wythes.

1.2 Proposed Method

In this study, the proposed method is conceptually founded on the premise that the method of analysis should be independent of the procedure for estimating material properties in order to be valid for current as well as for possible future knowledge of these properties.

A computer program has been developed that performs the first order non-linear structural analysis of shear-connected cavity walls taking account of non-linear material properties of the wythes. A rational approach has been employed and the analysis is based on the combination of compatibility method, stiffness method and method of imposed rotations[4] (modified compatibility method).

What the program can do:

I. It generates five points on the moment curvature diagram of any plain, partially or fully grouted, with or without reinforcement, brick or block section;

II. It establishes rotation-force relationship of Block Shear™ Connector;

III. It generates a non-linear load-deflection diagram of a masonry simply supported wall, subject to lateral load, with "tension stiffening" factor accounted for;

IV. It generates a load-deflection diagram of the cavity wall due to a lateral load up to the crack limit;

V. Theoretically, it has the potential to generate a non-linear load-deflection diagram of the cavity wall up to the failure, subject to lateral load.

1.3 Objectives

The main objective of this study was to develop a computer program which has a twofold purpose:

I. To aid the engineer in the design process of the cavity wall, respecting the limit states design requirements;

II. Usage in evaluating the test results of the cavity wall.

Other objectives are:

- To predict more realistic structural behaviour of the cavity wall;
- Better understanding of non-linear deformation phenomena;
- To determine distribution of moments and consequently load-deflection relationship at any load stage, not only at ultimate;
- To define modes of failures.

2. LITERATURE REVIEW

2.1 Introduction

Masonry cavity walls are frequently used to provide superior moisture resistance and energy efficiency for building envelope design. The wall system consists of an air cavity sandwiched between an outer veneer wythe and an inner structural back-up wythe. Traditionally, the back-up wythe has been designed to resist the full lateral imposed load, while the veneer wythe has been regarded as just an architectural facing without any structural importance. By introducing the non-conventional connectors that have the ability to transfer a shear and enable a composite action between two wythes, a contribution of veneer wythe in increasing the stiffness of the system has been achieved. Currently, the masonry industry is looking into a method to take advantage of the unused structural potential of the outer wythe by reducing the material and construction costs. In recent years many tests have been done with encouraging results and conclusions[5] [6] [7] [8] [9]. It triggered a need for developing a new approach in the design of cavity walls. Also, new definitions and revisions have been introduced and incorporated in Standard CSA S304-1 M94 *Masonry Design for Buildings*, that reflect the new structural concept of the cavity wall system.

2.2 Cavity Walls

For a long time the advantages of the arrangement of the cavity walls have been recognized[10]. The cavity between the wythes is a convenient place for installing the continuous air and vapor membrane and insulating material. Also, the cavity acts as a superior rain screen, allows free air flow for ventilation purposes and allows penetrated rainwater and/or condensed vapor to runoff freely through the weep holes. The most critical parts of the cavity wall assembly are the ties, which enable the wythes to act together. What degree of composite action is provided, structurally speaking, depends mainly upon tie stiffness, its spacing and tie interaction with the masonry at the location of its embedment(1) (11)(12). Due to the existence of a large variety of connectors on the market Canadian Standard Association made an effort to develop a code exclusive to the categorization, design and specification of masonry connectors. According to CAN-A370-M84(13), the masonry connectors were divided into two groups, the standard and non-standard connectors. Most of traditional ties, such as the corrugated strip, Z-shape wire or rectangular wire tie, fall into the category of standard connectors. They are also known under the common name "weak ties", which means they cannot transfer a measurable amount of shear force, and consequently zero composite action can be achieved between two wythes. The drawbacks of the weak ties led researchers(14) to develop a new type of tie with improved features. One attempt was to introduce a stiffer tie that would increase the rigidity of the assembly, thus reducing the lateral deflection and crack width, in other words enhancing the serviceability limit criteria.

The ideal type of connector for a cavity wall will, therefore, be one that is stiff enough to transfer the load to the backup wythe and flexible enough to accommodate the vertical movements of the two wythes. The main objective of the researchers was to develop a shear connector which will partially restrain the vertical movements between the two wythes without inducing large stresses due to material properties *and temperature effects*[2].

The shear connector used throughout this study was developed by Dr. M. Hatzinikolas and his team at The Praire Masonry Research Institute (PMRI) in Edmonton. The superiority of this type of tie is well recognized and it is being widely used in the masonry construction industry.

The evolution of the tie prompted the need for a revision of the old CSA A370-84. The new edition was published in 1994[15]. Some notable changes of interest include:

- harmonization with CSA A371, the masonry construction standard, and S304.1, the new masonry LSD design standard
- a re-thinking of terms, so that, a 'standard connector' in the 1984 edition became a "conventional connector", and a "non-standard connector" became a "non-conventional connector"
- enhancement of existing performance requirements, and the introduction of new performance requirements

The Block Shear™ Connector, which belongs to the non-conventional connector category according to the revised classification, is an "engineered connector" with superior performance, constructibility (easy of installation and placing of installation), ability to provide little or no impact on the air and vapor barrier system, ability to prevent disengagement and ability to reduce the heat loss due to thermal bridging.

Many tests had been conducted by different authors[16] [17] [18] in determining two parameters: the compressive strength and the flexural tensile strength of concrete block and brick masonry. Since the masonry is a multi-component assembly, having an insight into the relative importance of various geometric and physical properties of the block, brick, grout and mortar is indispensable. The conclusions drawn from the experimental work will be supported by analytical moment-curvature relationship established for block and brick section in this study.

In order for the performance of the cavity wall to be accurately quantified and verified two different issues should be explored: first, a need for obtaining more accurate information about material properties and secondly, a need for rational structural analysis.

2.3 Structural Analyzing Methods

Traditional methods for designing the cavity walls were based on the assumption that non-composite action is provided between two wythes, the applied moment is distributed proportional to the stiffness of the wythes and working stress method is employed[10] [27]. The engineers had a difficulty in the design process, since no clear guidelines and standards had been established.

The extensive research and testing of shear connected cavity walls have been conducted by PMRI[2] [9] [12] in the late 80's. Theoretical analysis was based on the twodimensional model by assuming constant stiffness of the wall assembly along its length and analyzing only a portion having a one metre width. Any 2-D structural program for elastic frame analysis could perform the analysis, and it gives good results only for low level loads up to the elastic limit.

As a result of experimental and analytical studies done by PMRI, design curves and tables[19] were obtained by varying the different parameters involved. They also helped to establish suitable guidelines for the design of the cavity wall. For a first time an effective moment of inertia for the cracked section was used in the analysis, taken and modified from S304-M84.

Meanwhile, a few simplified analysis techniques were developed to generate design aid tables to facilitate the design process[20].

Later, there was an attempt by the PMRI research team to produce the idealized load-deflection curve that consists of four characteristic points with changing of material and section properties depending on the stress level. The analysis procedure is based on the review of the load-deflection curve and observation during the testing of cavity walls subject to a lateral load.

Software[21] package *Shear Truss* for analysis of cavity walls, developed by Canadian Masonry Research Institute (CMRI) was introduced on the market recently.

3. CAVITY WALL AS A STRUCTURE

3.1 Geometry

Masonry is a modular product, that is, units are manufactured in standard overall sizes. Basic standard sizes come in modules of 100 mm. This modular approach has to be maintained in all three directions of a masonry building. The dimensions should be planned to multiples of 100 or 200 mm. There are two different categories in modular dimensioning of the masonry units: the nominal vs. actual dimensions. Actual dimensions of a unit are 10 mm smaller in all three dimensions than the nominal dimensions to fit a standard mortar joint of 10 mm.

Theoretically, this program allows any value for floor-to-roof height, except one condition, the modular block height of 200 mm. has to fit in overall height. The typical selected heights would be from 2400 mm up to 9000 mm.

The veneer wythe is assumed to be constructed of a standard clay brick unit. The unit has actual dimensions of 90 mm wide by 57 mm high by 190 mm long (See Figure 3.1-1). Provisions have been built in the program for analyzing a reinforced veneer wall. The same dimensions for units apply, which have larger voids to accommodate the reinforcement and adequate grouting.

The backup wythe is considered to be constructed of a standard hollow concrete block

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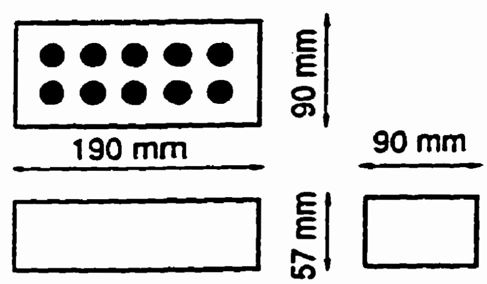

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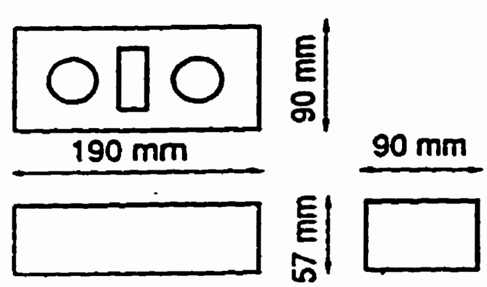


Figure 3.1-1 Standard Brick Unit

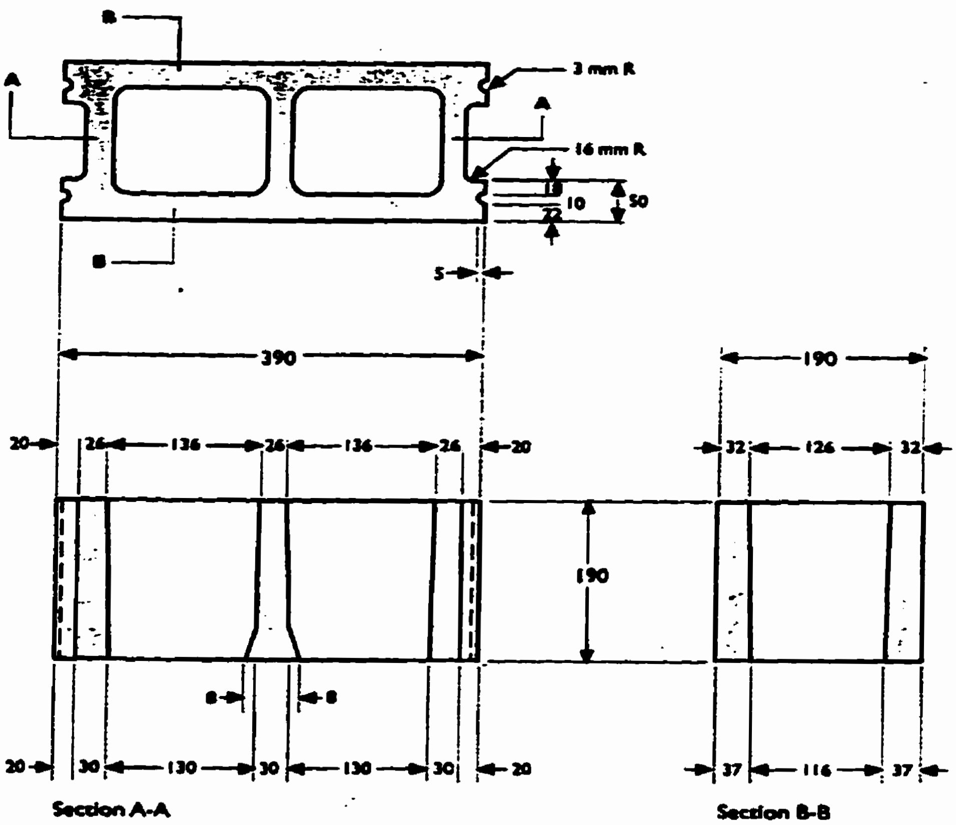


Figure 3.1-2 Standard Concrete Block

unit with widths of 140, 190, 240 or 290 mm. A typical block is shown on Figure 3.1-2. The other two dimensions are constant: 190mm high by 390 mm long. The dimensions of the face shell, web width and core length are listed on Table 5.5-2 for different unit size.

The mortar joint thickness is assumed to be 10 mm in both wythes. This makes the nominal distance between two vertical mortar bed joints to be 200 mm in backup wythe and 67 mm in the veneer wythe.

Cavity width varies from 25mm up to 100 mm.

Shear connector comes in different sizes, depending on design requirements. The program prompts for input on protrusion lengths of V-Tie and steel plate. Spacing of connectors is dictated by the maximum recommended spacings. Vertical spacing is 200 and 400 mm respectively top and bottom, followed by equal spacings of 600 mm or 800 along the height of a veneer wall. Horizontal spacing can vary from 600 mm to 1000 mm.

3.2 Constituent Parts and Properties of the Cavity Wall

Masonry strength. One of the most important material properties required in the limit states design of masonry is the specified *compressive strength of block or brick masonry* f'_m. Two sources for obtaining the values f'_m are suggested[1]. The first source is the Tables in S304.1, based on the specified unit strength, the type of mortar and the number of grouted cores. The second source would involve the testing of stack bonded masonry prisms made from the material components used in the actual structure that is to be analyzed. Table values for compressive strength of brick and block masonry can be obtained from Table 3 and 5, S304.1-94. Due to lack of consistent data there is no tabulated information about the compressive strength for hollow clay brick masonry. This should be determined by prism testing. *Flexural tensile strength* f_t is another very important engineering property required in flexural analysis of the cavity wall. The values for flexural tensile strength of block masonry, tabulated on Table 5.5-1, represent the summary of the extensive research done by different authors(17) (18) (22). They are in general agreement with the values shown in Table 6, S304.1-94. It is worth mentioning that the current Code, does not reflect the higher f_t strength of the grouted masonry versus solid masonry. Also, it does not address the fact that f_t is not only a function of the strength characteristics of the component materials, but also a function of their geometric characteristics. The values for flexural tensile strength of brick masonry are derived by analogy on the behaviour of block masonry and they are listed on Table 5.5-1.

Brick masonry unit. It is assumed that a standard brick unit satisfying the requirement of CSA Standard A82.1 has been used. It can be solid or hollow. The flexibility of the program allows a structural analysis of cavity walls where veneer and backup wythes can be made of non standard units, assuming parameters such as, compressive and flexural tensile strengths obtained through testing.

Concrete masonry unit. CSA Standard A165 covers all aspects of material properties of concrete masonry units. The most important physical properties of a concrete block are considered to be:

i. *solid content* -If net cross-sectional area is less than 75% of the gross cross-sectional area, the concrete block falls into the category of hollow units. Otherwise, it is classified as a solid unit. A hollow unit is designated by the letter H, and a solid unit is designated by the letter S. To distinguish the unit with cores from the really solid unit, A165 has included the designation S_f for fully solid unit.

ii. *compressive strength* - The most typical values range from 15MPa to 35MPa.

iii. *density* - Not addressed in this study.

iv. and *moisture content* - Not addressed in this study.

Mortar. The latest A179 Standard recognizes two types of mortar: Type N and Type S. The mortar type has to be specified if the compressive strength of masonry is selected from the Table. Although the mortar accounts only 2-3% of total masonry volume, it plays a significant roll in the tensile strength of the masonry.

Grout. The assumption is that grout conforms to all the requirements from CSA Standard A179. It serves two purposes in masonry: first, to bond the reinforcing steel and enable it to act with the rest of the assembly and secondly, to increase the effective area of the masonry section, thus enlarging the load-carrying capacity.

Reinforcement. Only standard reinforcing bars are assumed to be used. The program allows either or both wythes to be reinforced. The required parameters are yield strength and the total sectional area of the bars per one metre wall width. The assumption is that the bars are placed in the centre of the wall.

Block Shear™ Connector assembly consists of a Shear Connector Plate, a V-Tie and an optional insulation support of rigid plastic (See Figure 1.1-2). It has been recognized by CSA Standard A370-94, and belonging in the category of non-conventional connectors. The shear plate is produced from 16 gauge sheet metal. The different heights of 60, 70 and 75 mm exist on the market. The yield strength of the plate is 230 MPa. The V-Tie is manufactured from 4.76 mm diameter wire.

3.3 Boundary Conditions

The veneer wythe is supported at the bottom by means of steel shelf angle, or rests on the floor or foundation structural elements. At the top, the veneer is free to move, and a backup wall is supported by a steel channel that allows the wythe to move vertically but restrains the horizontal movement. Both wythes are connected by shear-connectors spaced at certain intervals.

In the analysis, both supports are modeled as hinged at the bottom end. At the top, the backup support is simulated by a roller that allows vertical movement. Embedded ends of the V-Tie and the steel plate in the mortar joints are represented as a fixed support in the structural analysis.

3.4 External effects

The positive uniform wind pressure is considered to be the only one that acts upon the windward face of the structure. Self weight of the masonry units has been neglected. Neither wythe is subject to vertical loads. The exterior wythe acts as a veneer wall, and the interior wythe acts as a backup wall. However, the program itself possesses enough flexibility to be upgraded to accommodate the effect of vertical loads.

4. RATIONAL APPROACH IN UNDERSTANDING THE STRUCTURAL BEHAVIOUR OF CAVITY WALL DUE TO WIND LOAD

4.1 Cause of Non Linearity in the Load-Deflection Relationship

A primary goal of any structural analysis is to predict the ultimate resistance, which a wall can transmit given a certain loading pattern, in this study a uniformly distributed wind load. Although load carrying capacity is of utmost importance, excessive deflections can also lead to structural problems. For that reason serviceability requirements that include crack and deflection control must be satisfied. The analysis can be completed if the load-deflection relationships can be determined for all stages that a structure (wall) passes through, from a zero-load stage to the ultimate-load stage.

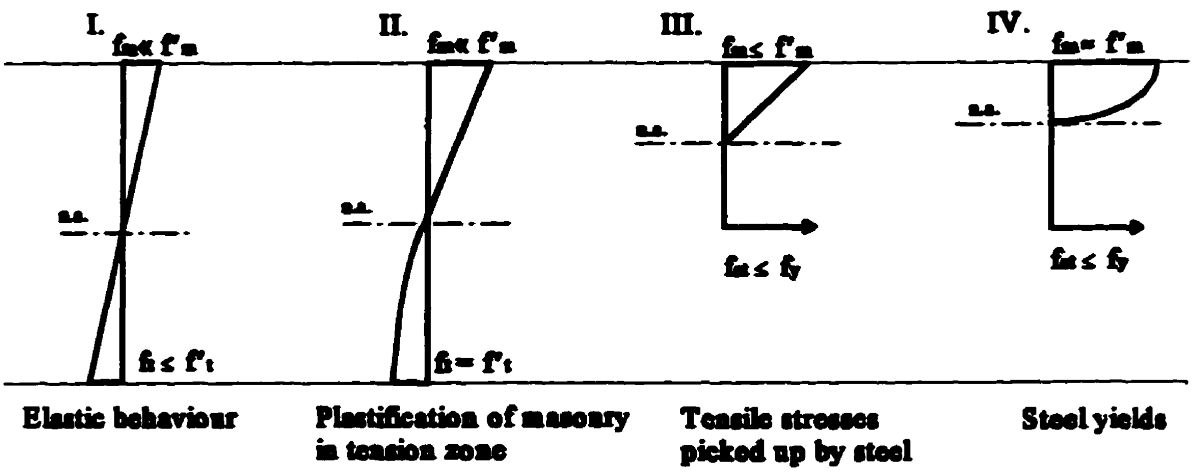


Figure 4.1-1 Characteristic Stress States due to Bending

A cross-section of masonry wall, subject to pure flexure, passes through different stress-deformation states, which directly depend on the magnitude of the moment (See Fig. 4.1-1). At low load level, defined as Stage I, a small moment causes low stresses in the masonry. The distribution of stresses along the cross-section is linear and there is no cracking, as long as the stress remains below pure tensile strength. At this stage all internal deformations are elastic. Once the pure tensile strength at the tension face has been reached the distribution of tensile stress is no longer linear. Under a further load increase, the tensile stress curve deviates more from the straight line, and the neutral axis shifts slightly toward the compression zone. Stage II is characterized by developing elasto-plastic deformations along the tension zone. The mechanism of plastification is a complex phenomenon[22] which starts by the generation of microcracking in the mortar joint or in the interface between the mortar bed joint and the block or brick and subsequently in the grout mass. When the most stressed fibres in the tension zone exceed the pure tensile strength of the mortar, plastic deformations commence. Further load increase causes an increase in internal forces at the cross-section, and tensile stresses gradually reach the pure tensile strength of the grout and/or mortar in the adjacent fibres along the cross-section and plastic deformations continue. Eventually, in the stage next to fracture most of the tension zone is fully plastified, tensile capacity had been exhausted and failure is imminent. Plain masonry members subject to out-of-plane bending always collapse through cracking and develop elasto-plastic strains in the loading stage next to fracture. The formation of a crack always takes place within the tension zone along a joint. Meanwhile the compression zone is strained elastically till fracture, since developed stresses are less than the elastic limit.

In the case of a reinforced cross-section, the bars are placed in the centre of the section and their position coincides with the neutral axis. At Stage I, no stresses are developed in the bars. At Stage II, because of the shifting of a neutral axis upward, small stresses develop, but they are too small to affect overall behaviour and can be neglected in calculations. A transition phase from Stage II to Stage III is characterized by an abrupt change of the section stiffness due to a higher moment. Since the fully plastified tension zone can no longer take part in resisting the tensile stresses, the bars become effective. A crack has a tendency for further propagation toward the compression zone and the neutral axis shifts far upward.

Depending on the amount of reinforcement, different possible situations may develop:

(a) When a very small amount of steel is present, the bars are not able to pick up all tensile stresses produced at the end of Stage II ($A_{st} < \min A_{st}$), and the section fails without any warning. In this case, Stage III is never reached;

(b,1) The minimum amount of reinforcement has been provided (the value for the moment capacity of plain section will serve as a condition for determination of the minimum amount of reinforcement, $\min A_{st} f_y\, jd \geq 1.2\, M_{cr}$), and it can resist the tensile stresses. Since the bars pick up most of the tensile stresses the section regains equilibrium. From the condition for determination of the minimum amount of reinforcement it can be shown that the stresses in the bars reach the yielding stress. Practically, it means that the capacity of the section is almost exhausted, because from now on the bars are being strained constantly with a small load increase. At Stage IV, deformation of the tensile bars continues at a much faster rate than the deformation of the compressed fibres. This causes very large rotation of the section, large deflection of the wall with a clear sign of the crack widening. It also causes a further reduction of the active compression zone. Therefore, ultimately the section fails with crushing of mortar and parts of brick or block at the compressed face shell. This type of failure in the literature is known as a "tension failure". Experimental work show that after the bars reach the yielding stress, an increase in the steel strain occurs very rapidly. From the aforementioned reasons, such as very large rotation, an excessive deflection and the width of a crack, the ultimate steel strain at failure, would be significant only from a theoretical point of view. For practical calculations, the critical steel strain from 5-10‰ can be used as the criterion for "tension failure", as it is done in some European Codes(23) (24).

b,2) When the area of steel A_{st} is considerably greater than minimum ($A_{st} \gg$ min.A_{st}),the section is capable of attracting more load. At some load level when the most strained fibres exceed their elastic strain limit the section enters Stage IV. With a further load increase both the bars and the most strained fibres in the compressed zone deform at the similar rate. There is further reduction in stiffness, shifting of the neutral axis and propagation of cracking. The fibres in compression start to strain plastically, the stress curve is more curved, while the stress in the bars is still less than yielding stress. As the most strained fibres approach the ultimate compressive strength and the top fibre reaches the ultimate strain the section has exhausted its load capacity. The section experienced a failure, the so called "compression failure", with crushing of mortar and grout. This mode of failure is undesirable because there are no clear signs and prior warnings that a failure is imminent. In other words, there is no large rotation of the section, no plastic strain deformation of the bars, no clear sign of widening the crack, and it happens at rather small deflections;

b,3) The third mode of failure is the "balanced failure." This happens when the mortar and grout start crushing and the stress in the bars reaches the yielding stress simultaneously.

As a conclusion, crushing of the exterior part of the compression zone is common to all three failure modes, while the strain in the bars can either be very large, or at yield strain, or strain that is less than the elastic strain limit. Figure 4.1-2 shows qualitatively the load-deflection behaviour of a masonry wall due to a wind load.

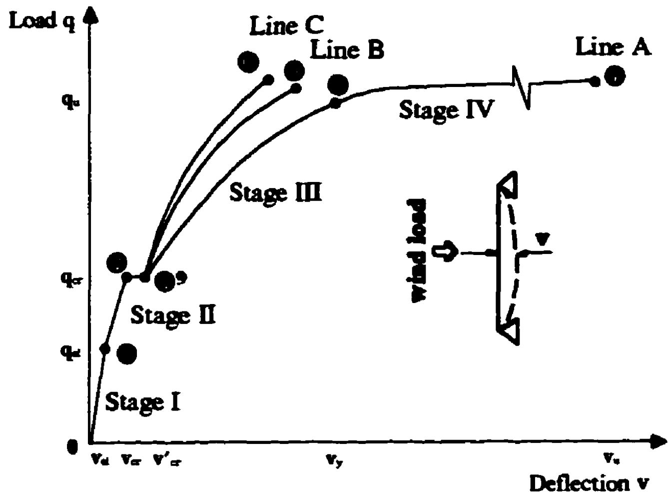


Figure 4.1-2 Load-Deflection Relationship

A wall resists deflection from external load due to its stiffness, which can be expressed as the product of the modulus of deformation of masonry (E_m), its sectional geometrical properties (I_m), and its height. How one section will respond to the imposed load and what pattern of stress-strain distribution will be developed depends directly on the moment at that section and the section stiffness. How the whole wall will respond and what the maximum deflection will be, depend on the complete performance of all sections along the wall.

Up to a certain load level (0-1), and relatively small moments, the masonry behaves elastically, all sections keep their original uncracked stiffness and the deflection is proportional to the load. From the stress-strain point of view, it means that all sections are fully functional at Stage I. As the load is increased to a value q_{el}, at least one moment exceeds a certain value defined as the limiting moment of elasticity. There is a change in the stiffness of those sections where the most tensile stressed fibres start experiencing elasto-plastic behaviour. The wall enters Stage II. Consequently, the wall becomes less stiff than the original one. The load-deflection relationship can still be idealized in a linear fashion, due very small plastic component of the total internal deformation. See load level (1-2) in Fig. 4.1-2.

Point 2. denotes the ultimate load q_{cr} for a plain wall and its correspondent deflection v_{cr}. The failure occurs at the most stressed (critical) section when the tension zone becomes fully plastified and the wall is no longer capable of resisting the tensile stresses.

At the same load level, in the case of a reinforced wall (Stage II, 2'), the bars in the critical section take over all tensile stresses. The crack propagates toward the compression zone, with a resulting reduction of the stiffness and significantly larger rotation of the section. It affects the wall total "flexure stiffness" and deflections increse, 2-2' (v_{cr} - v'_{cr}).

With a further load increase the wall becomes more stressed and more deformed. At sections where cracks have formed and the steel becomes active, different degrees of plastification of highly strained fibres in the compression zone occur due to higher stresses (Stage III). Some sections function at Stage III and some of them at Stage II. The portions of the wall where the moment is less than the moment of elasticity still function under the Stage I regime. It depends on the moment distribution. The behaviour of the portions of the wall between the cracks will be considered in the next chapter. The overall effect of all mentioned is that the wall becomes less stiff and its deflection increases at a faster rate than does the load (2'-3/5/6, shown on Fig. 4.1-2).

Point 3. denotes a moment when the bars start yielding in at least one section. The final stage has been characterized by a progressive rotation of the critical section, a clear widening of the crack and a rapid increase in the deflection. This all occurs during a very small load increment. When the capacity of the critical section is exhausted, it results in the failure (not necessarily in the statically indeterminate structures) of the whole wall at q_u (ultimate load carrying capacity), at point 4.

The nonlinear nature of a load-deflection curve comes as a consequence of the inelastic nature of the stress-strain relationship of the materials involved in masonry structures. The other causes of non-linearity, such as a geometric or long-term loading effect, are not an objective of this study, because only the wind short-static load is considered.

4.2 Response of Veneer / Back-Up Wall Subjected to Flexure

4.2.1 Reality & Simplifying Assumptions

The realistic determination of the response of plain and reinforced masonry structures demands a knowledge of the inelastic behaviour of the component materials, and the ability to incorporate these into a rational analysis of real structures. This requires a reasonable compromise between reality and simplicity: Firstly, in the formulation of material characteristics and geometric properties, secondly in simulating the structure with mathematical model and finally, in the use of the principles of mechanics. For that reason some assumptions must be made.

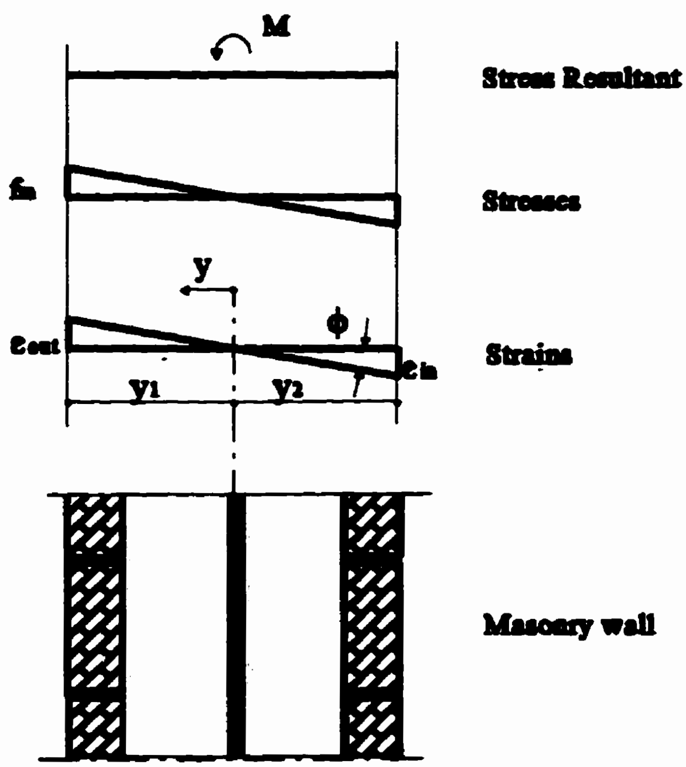


Figure 4.2.1-1 Stress and strain distribution in a masonry wall subject to bending

- **Cross-section**

The most widely used elements in cavity walls are concrete block and brick. Since the wall is vertically supported, the stresses act normal to the bed joints. Analysis is performed per metre length of the wall. The typical cross-section can be idealized as either I-shape or rectangular shape depending on the degree of grouting, with one metre width.

- **Distribution of strains (across the cross-section)**

It is assumed that the member is subjected to strains in only the axial direction. These strains are uniform over the width of the section, but vary linearly over the depth of the section (i.e.plane section remains plane). Shear strain caused by transverse forces is neglected ($\gamma = 0$).

The masonry strain distribution can be defined by just two variables[25]: strain at an outer face(ε_t) and strain at the inner face(ε_b). The two variables that will be chosen to define the linear strain distribution are the strain at the centroid $\varepsilon_{cen.}$ and the curvature ϕ (See Fig. 4.2.1-1).

$$\phi = \frac{d(\theta - \theta_t)}{ds} = \frac{d\theta}{ds} = \frac{d^2y}{dx^2} = \frac{M(x)}{E_m I(x)} \qquad \textit{assumption: } \gamma = \frac{d\theta_t}{ds} = 0$$

The curvature is equal to the change of the slope per unit length along the member and is also equal to the strain gradient over the depth of the member.

$$\varepsilon = \varepsilon_{cen.} + \phi y$$

$$\varepsilon_{st.} = \varepsilon_{cen.} + \phi y$$

$$\varepsilon_t = \varepsilon_{cen.} + \phi y_1$$

$$\varepsilon_b = \varepsilon_{cen.} + \phi y_2$$

Compatibility Conditions

(Tensile strains are positive, compressive strains are negative and a positive curvature is associated with the inner face having an algebraically smaller strain than an outer face)

- **Distribution of stresses (across the cross-section)**

A curve that depicts the distribution of the stresses across the masonry section is approximated by using standard stress-strain curves (See Fig.4.2.1-2).

The inelastic nature of this relationship can generally be represented by the expression:

$$\mathbf{f}_{mas.} = \mathbf{f}\,(\varepsilon_{mas.})$$

$$\mathbf{f}_{steel} = \mathbf{f}\,(\varepsilon_{steel})$$

Stress-Strain Relationship

It should be mentioned that each masonry assemblage with well defined sectional and material properties and loading history has a unique stress-strain curve. The most basic measure of the stress-strain behaviour of masonry is the uniaxial compression and tension curves obtained from prism and/or cylinder tests. A typical uniaxial stress-strain curve is shown qualitatively on Figure 4.2.1-2. At low levels of stress up to a certain point, the masonry exhibits elastic

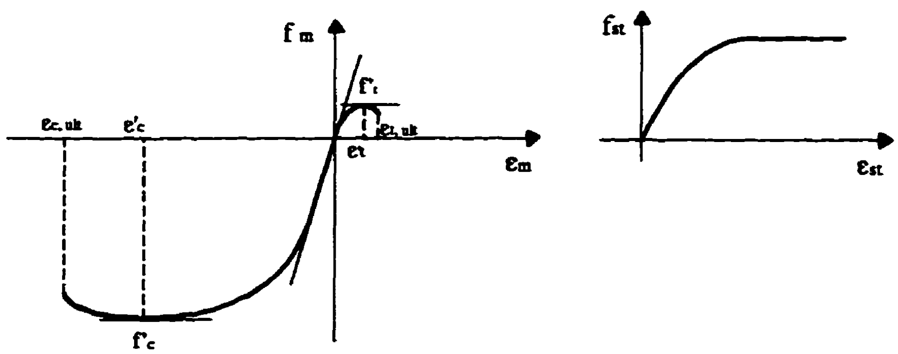


Figure 4.2.1-2 Typical Stress-Strain Curve for Masonry and Steel

behaviour and the stress-strain curve follows Hooke's law, that is, stress is proportional to strain. At a certain load level that can be defined as a level when the masonry starts to behave elasto-plastically, the strain increases at a faster rate than the stress. This stage is characterized with a nonlinear ascending branch of the stress-strain curve, up to the maximum stress and the correspondent maximum load which the cross-section can sustain. After this peak has been reached, the masonry can no longer sustain that load. It crushes, or in the pure flexure case, because of a high strain gradient, the descending branch of the curve is formed and ultimately the masonry crushes, but at a much larger strain. The crushing of masonry is represented by an unstable portion of the compressive stress-strain curve.

The diagram on Fig. 4.2.1-2 shows that during tension masonry behaves in a similar fashion, but at a much smaller scale. The maximum stress is about 8-12 times less than the maximum stress in compression. Similar proportions[22] exist between corresponding strains (ε'_t) and (ε'_c): whereas (ε'_c) takes on values of about 0.2%, (ε'_t) does not usually exceed 0.01%.

Two sets of values have important significance: the compressive strength of the masonry and the ultimate compressive strain, and the tensile strength and the ultimate tensile strain.

- **Internal forces** (Equilibrium conditions)

At any section the stresses when integrated over the section must add up to the required sectional forces M and N. The axial load N is positive if tensile and negative if compressive. The moment M is positive if it causes tensile stresses on the inner face.

$$N = \int_A f\,dA + \int_{A_{st}} f_{st}\,dA_{st}$$
$$M = \int_A f\,y\,dA + \int_{A_{st}} f_{st}\,y\,dA_{st}$$

Equilibrium Conditions

4.2.2 Moment-Curvature Response of the Masonry Cross Section

If a member is subjected to flexure, which is a case with a veneer and backup wall due to wind load, the moment-curvature response can be easily determined by using equilibrium and compatibility conditions with the known material stress-strain relationship. The analytical process is shown schematically as follows.

Compatibility Conditions ► Strain Distribution (across the section)

⇦

Stress-Strain Relationship ► Distribution of Stresses (across the section)

⇦

Equilibrium Conditions ► Axial Force & Moment

The assumption that axial load (force) is zero, that is, pure flexure, enables a creation of a unique moment-curvature relationship for any section with different geometrical and material

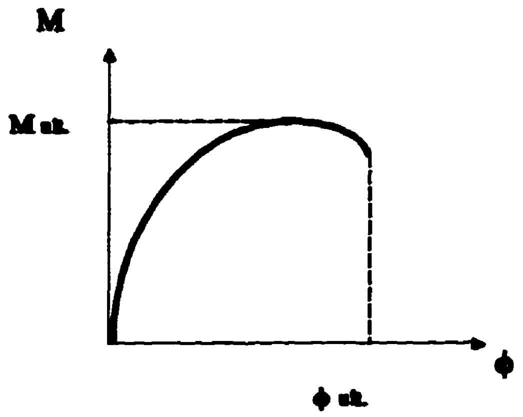


Figure 4.2.2-1 Moment-Curvature Curve

properties (See Fig. 4.2.2-1). A convenient procedure is to choose an arbitrary value of outer face compressive strain and then find, by trial and error, the corresponding inner face tensile strain which gives zero axial force. For each set of strain values, obtained from this procedure, there is one, and only one value for curvature and its associated moment. If these calculations are repeated for different values of outter section strain, up to the ultimate value of compressive strain which the section can sustain, the complete moment-curvature curve can be defined. The maximum value of the moment (the peak of the curve) is the ultimate resistance moment of the cross-section, while the maximum curvature shows at what curvature the cross-section can no longer sustain further deformation (ultimate curvature of a critical cross-section at failure).

Although the pure tensile strength is of high practical interest, this characteristic quantity is rather seldom used, because it is difficult to test masonry in pure axial tension[22]. The tensile strength is usually determined using indirect ways. One way to establish the tensile strength is through the testing a specimen subject to bending stresses. Then, conventional tensile strength or modulus of rupture can be calculated using Navier's expression (the hypothesis of the elastic body and linear stress distribution across the cross-section up to failure), when the maximum moment is divided by the section modulus. This approach, at the same time, represents a convenient way for expressing flexural cracking strength of the section and load carrying capacity of the plain section, but only for elastic method of structural analysis. More rational analysis requires a more refined approach. In reality, actual stress distribution at the stage next to failure is other than linear, and the maximum tensile stress is lower than the modulus of rupture. In analysing either plain or reinforced masonry walls, such behaviour is particularly important.

The stress-strain compatibility method is based on the hypothesis of the elasto-plastic nature of the body. To make this method applicable for the whole range of loading, besides the compression portion, the tension portion of the stress-strain curve must be also known.

4.2.3 Moment-Curvature Relationship for a Finite Length of Member

In Section 4.2.2 only the relation between moment and curvature at a cross-section was presented and defined. To understand development of curvature along the wall axis and its relation with a moment further explanation is needed.

A finite (discrete) length of a reinforced wall, is shown on Fig. 4.2.3-1, subject to constant bending moments. It is assumed that the moment is large enough to cause cracking. From the nature of masonry, the cracks always form along the joints, therefore a regular pattern of cracks eventually will form at the joint spacing. At the cracks the masonry has exceeded its tensile strength. No tensile stresses exist in the masonry, and therefore all tensile stresses must be carried by the reinforcement. As for masonry between the joints, it is less stressed and there is no appearance of the primary cracks which are visible at the joints. The

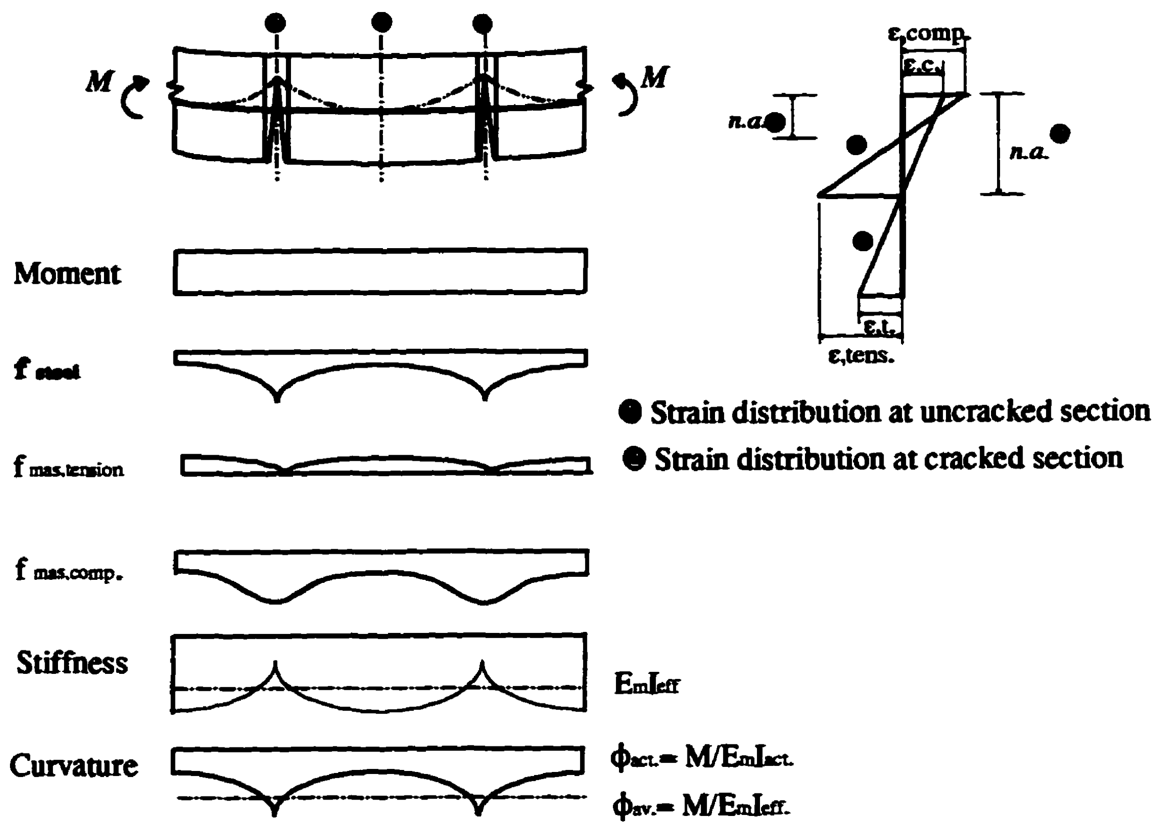


Figure 4.2.3-1 Stress and Curvature Distribution along a Segment of the Masonry Wall

bond between the bars and grout will enable some tensile stresses to be transferred from the bars into the adjacent zone. It will cause formation of internal secondary cracks. As a result, a very complex stress-strain state exists between two external cracks. So far, no convenient analytical method can describe and calculate the moment-curvature relation at a section between two cracks.

Figure 4.2.3-1 shows qualitatively the stress distribution in the masonry and the bars, and the change of curvature and stiffness along one typical segment of wall:

i. The average stress in the compressive portion of the masonry changes along the the height of the wall. The largest average stress occurs at the cracked section, where the neutral line is closest to the compression face. Between the cracks, due to involvement of the tension portion of the masonry, the average compressive stress is much smaller, and the neutral line is close to the centroidal axis;

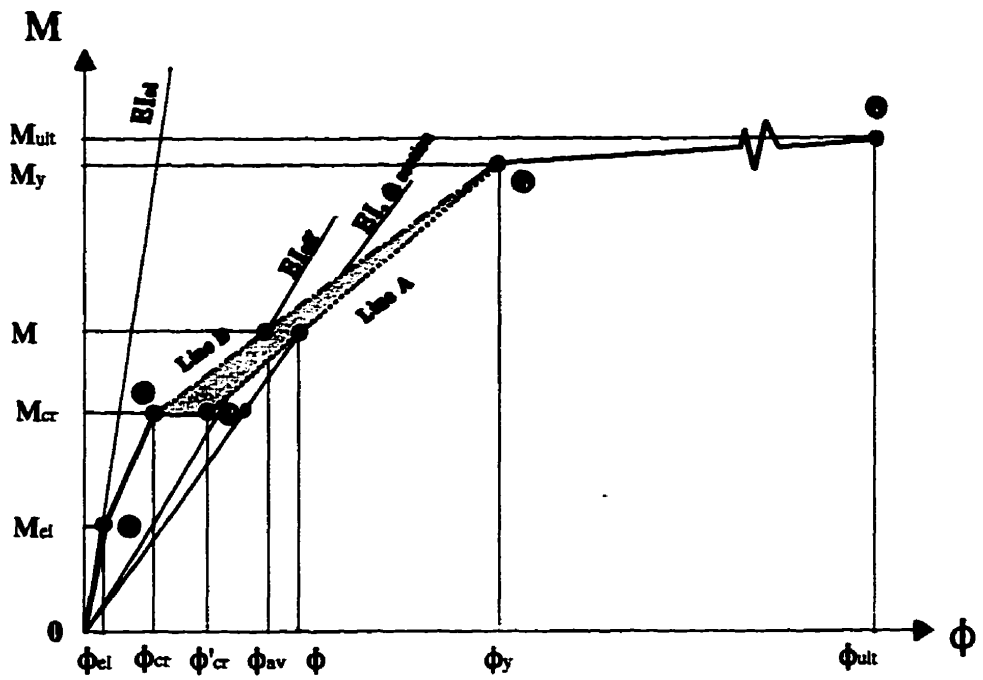
ii. The masonry cannot carry any tensile stresses at the crack, while between the cracks, below the neutral axis it is in the tension with a very complex stress distribution;

iii. The bars are most stressed at the location of the cracks and the least stressed mid way between two cracks;

iv. Curvature also changes its value, with the highest value at the cracked *Section #2* and the smallest value at the least strained *Section#1;*

v. The actual stiffness has a inverse proportional relation with curvature. For practical reason, an average stiffness is used in calculation.

Line A (O-❶-❷-❷'-❸-❹) on Figure 4.2.3-2 represents an idealized moment-curvature response of a reinforced masonry member at a cross-section (*Section#2*, Fig.4.2.3-1) subject to bending with tension failure expected. This type of diagram is used in this study in calculating effective stiffness of a finite height of the wall. The effective stiffness is expressed as a ratio between the applied moment and the average curvature (Line B).



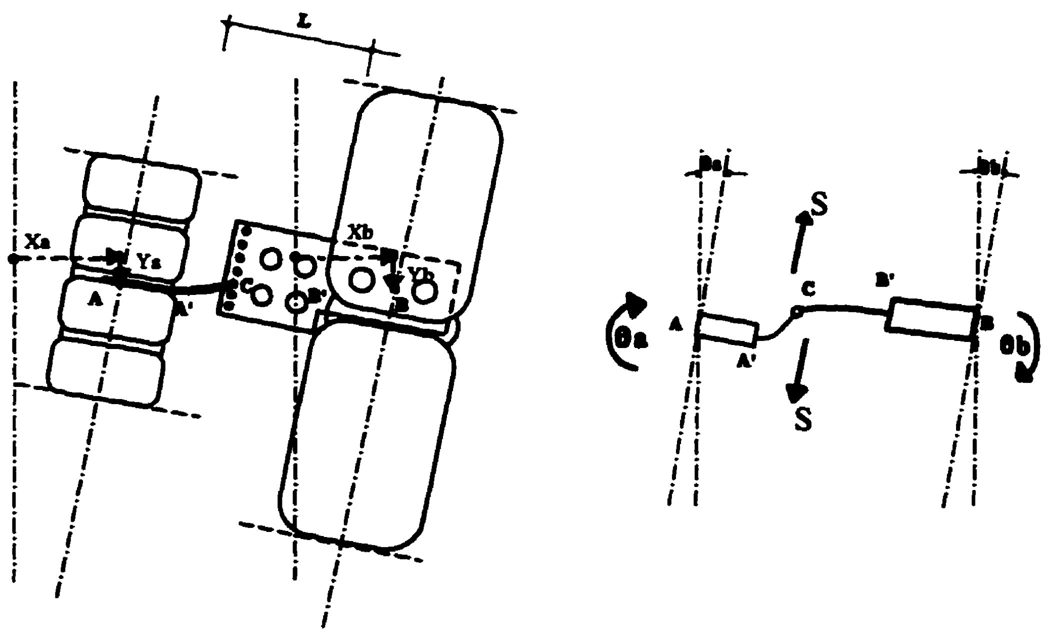
Uncracked segment:	0-1	Elastic portion
	1-2	Plastification in tensile masonry zone
Cracked segment:	2'-3	EI @ section
	2-3	EI_{eff} @ finite length of a member
Steel yields:	3-4	Section at crack controls

Figure 4.2.3-2 Moment-Curvature and "Tension Stiffening" Effect

As long as the applied moment is less than the cracking moment the average stiffness will be equal to the initial stiffness of *Section#2*. Up to the limit of elasticity this statement is valid, while for the section ●-●, up to the moment of cracking it represents a good approximation. At the cracking moment level, the section experiences significant loss of stiffness and the curvature changes its value (●-●'). At higher moments up to the moment of yielding, curvature increases linearly at one rate (●'-●), and at the final stage up to failure for small moment increment, curvature also increases linearly, but at much faster rate. The effective stiffness will follow the moment-curvature relation expressed by line ❷-❸ (part of line B), instead of line ❷-❷'-❸ (part of line A). The calculated value for stiffness and curvature, taking into account the contribution of masonry in tension between two cracks (hatched area), will be the effective stiffness and the average curvature associated with an applied moment. Once the steel starts yielding (region ❸-❹), the section at the crack is assumed to control the calculation of the stiffness. This is a stage close to failure, and it justifies this kind of approximation.

4.3 Origin and Effect of Generated Shear Force in Connectors

Shear connectors have the ability not only to transfer a lateral load from veneer to a backup wall, but also to generate shear force which in turns produces beneficial effects in both wythes. The structural response of a shear connectors to an imposed load and/or deformation depends mainly on material and geometric properties of its two components, V-tie and steel plate. These two parts, linked via adjustable hinges and well embedded in the mortar joints of veneer and backup wall respectively, renders the shear connector close to "state of the art" in a large family of connectors. It is sufficiently stiff to enhance stability of a cavity wall system and flexible enough not to produce undesirable effects in the mortar joints due to bending. When a cavity wall is subjected to uniformly distributed wind load, a large number of experiments[9][11][12] showed that both wythes follow similar deflected shapes. In terms of displacements, the joints in both wythes at the same level undergo almost the same horizontal displacements ($X_a \approx X_b$), almost the same rotations ($\theta_a \approx \theta_b$), and, without significant axial forces, negligible vertical displacements ($Y_a \approx 0$ and $Y_b \approx 0$). The effect of the small difference in horizontal displacements results from axial force in the shear connector, the effect of vertical displacements can be neglected and the effect of rotation will result in a generating shear force in the connector. Knowing material and geometrical properties of V-tie, steel plate, veneer and backup wall, and having a well-defined cavity width, the relation between rotation and shear force can be established. For that purpose the idealized beam model in Fig.4.3-1 is assumed. Points A and B represent the joints at the centerline of the veneer and backup wall respectively. Element AA' is the part of the V-tie embedded into the mortar bed area of the veneer wall. Element A'C is a continuation of the V-tie into the cavity, linked with



Assumptions:

$X_a \approx X_b + \Delta L$

$Y_a \approx 0$

$Y_b \approx 0$

$\theta_a \approx \theta_b$

Figure 4.3-1 Shear Connector

element CB' at hinge C. Element CB' is part of the steel plate in the cavity while element B'B is the remainder of the steel plate embedded in the mortar area of the backup wall. In reality both embedded parts of the V-tie and steel plate act as rigid bodies, and for more realistic simulation very large stiffness has been assigned to them. All connections are assumed to be fixed except the connection at hinge C which cannot transfer any moment by definition. Due to rotation of joints A and B both V-tie and steel connector are forced into bending. The generated shear force causes moment at joints A' and B'. Since the V-tie (two round wires in V shape) normally has much weaker flexural stiffness than the steel plate, the ultimate shear force will be limited by the ratio between the capacity moment of V-tie and protrusion length of V-tie [20] [29](A'C). Once the maximum shear force has been reached, the wires start yielding, associated with the formation of a plastic hinge next to point A'. Further rotation will cause no increase in shear force, except larger deformation of wires concentrated in the region of the plastic hinge. The direct stiffness method is used in this study in analysing a model beam with rigid end parts.

4.4 Compatibility Analysis - Separation Method

One structure is considered fully analysed when internal forces, internal deformations, reactions and displacements, caused by external and/or internal effects are determined.

In statically determinate structures, for any load level, the distribution of internal forces can be determined with the use of basic static conditions of equilibrium. To calculate displacements, moment-curvature relationship must be established first, for every cross-section with different material and geometric properties. The next step would be integration of curvature along the beam, assuming boundary conditions to be defined. It should be pointed out that while the distribution of forces is independent of the deformation of structure, the displacements can be determined only after distribution of forces is obtained.

For statically indeterminate structures, the problem is quite different and more complicated. The distribution of internal forces cannot be determined without considering the deformation of the structure. Deformation properties of materials from which the structure is made directly affect the distribution of forces. Generally speaking, it is possible to determine all unknowns with the help of a different numerical iterative technique.

It is well known that the Compatibility Method represents an application of the Energy Principle of Virtual Work, which is given with Equation (4.4-1):

$$\sum \overline{F}\underline{d} + \sum \overline{R}\underline{r} = \int_s (\overline{M}\underline{\theta} + \overline{N}\underline{\varepsilon} + \overline{T}\underline{\theta}t)ds \tag{4.4-1}$$

Equation (4.4-1) expresses the basic relation between possible equilibrium states and possible deformation states of a structure. From an energy point of view, work done by external forces including reaction forces on correspondent displacements is equal to work done by internal forces on internal deformations. In fact, the equation (4.4-1), which expresses Principle of Virtual Work, forms a basis for derivation of equations satisfying the requirements of equilibrium and compatibility.

The proposed Method of Imposed Rotations[4] which falls into the category of *separation methods* is a special type of non-linear analysis. It is based on the Principle of Superposition, with material non-linear stress-strain relationships, and consequently non-linear constitutive relationships are accounted for. The name *separation methods* comes from the methodology of these methods where the effects of plastic, or more accurately non-linear structural properties, are separated from linear effects. A continuous masonry wall subject to uniformly distributed load, statically indeterminate to the *n*-th degree will serve as an example to demonstrate the Separation Method, Fig.4.4-1a. To create a primary statically determinate system, *n*-redundancies are required to be released by the introduction of hinges at the inner supports. It makes the primary system, a system of $n+1$ simple beams. For compatibility requirement (relative rotations of cross-sections next to the hinges should be zero), a pair of equal and opposite external moments are applied in adjacent cross-sections of each new introduced hinge. There are as many pairs of external moments as the number of redundancies, and their magnitude is unknown.

In the analysis, the mortar joints are assumed to be the only regions where plastic deformation takes place. The segments between the mortar joints are assumed to behave linearly according to principles of elastic analysis. For methodological reasons plastic regions will be categorized into two groups: regions in the vicinity of new introduced hinges (*i*) and regions in other locations (*j*). Potential plastic regions are shown on Fig. 4.4-1b with

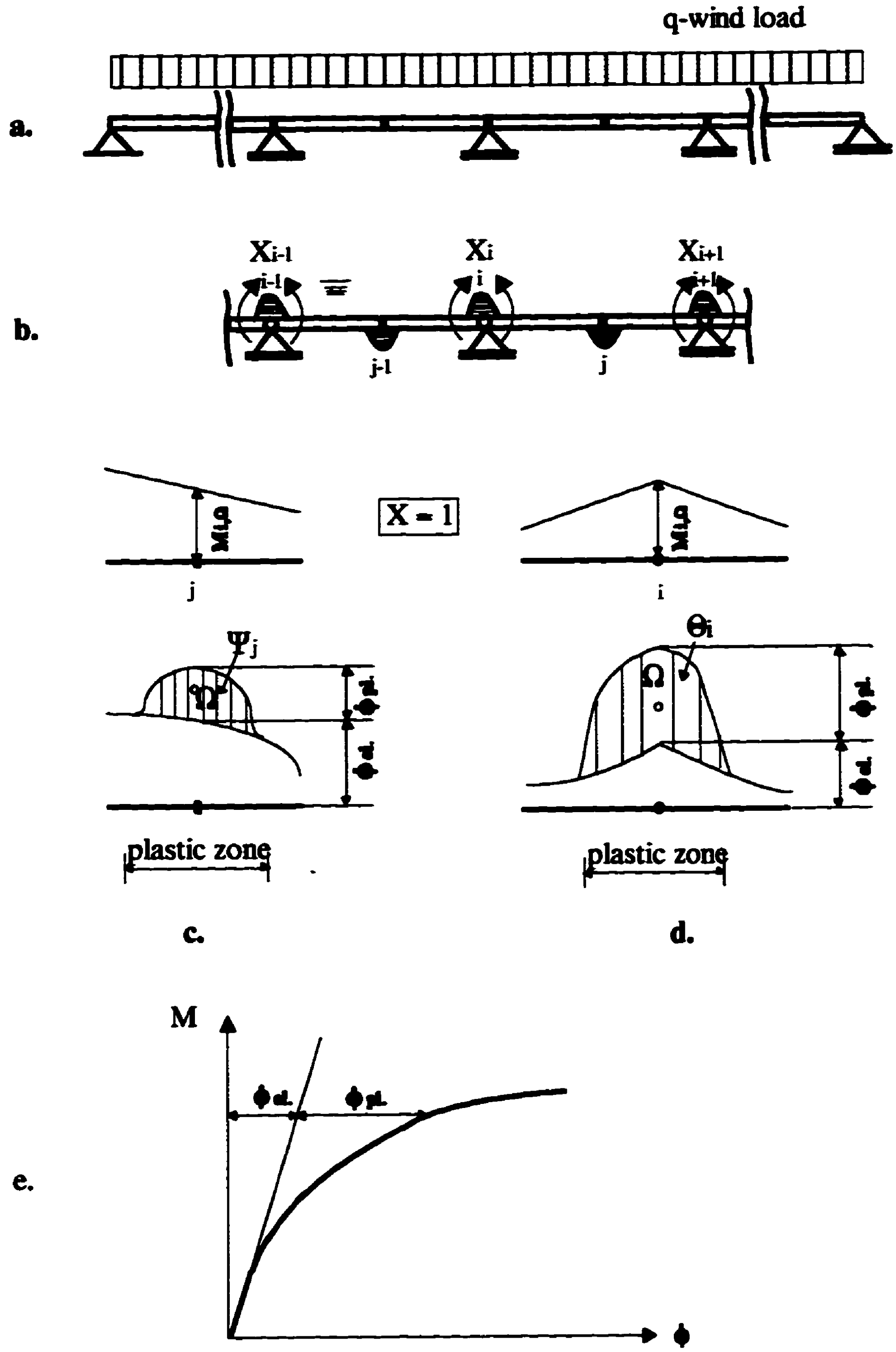


Figure 4.4-1 Plastic Zones in a Masonry Wall

help of the moment distribution diagram, which is not known beforehand, but qualitatively could be estimated. A set of *n* simultaneous compatibility equations must then be solved for *n*-number of unknown moments. The physical meaning of general compatibility equation (4.4-2) can be described as follows:

$$\theta_i = \theta_{i,o} + \theta_{ii} X_i + \sum_{k \neq i} \theta_{ik} X_k + \sum_j \Psi_j M_{i,\Omega} + \Theta_i \qquad (4.4\text{-}2)$$

where:

θ_i - mutual total rotation of end cross-sections adjacent to hinge *i;*

θ_{io} - elastic rotation at hinge *i*, caused by external load;

θ_{ii} - elastic rotation at hinge *i*, caused by unit moment force $X_i = 1$, $i = 1, 2, \ldots n$

θ_{ik} - elastic rotation at hinge *i*, caused by unit moment force $X_k = 1$, $k \neq i$

$\Psi_j\ M_{i,\Omega}$- internal work done by bending forces produced by state $X_{k=i}$ on plastic deformation in the plastic region (j) between the hinges, which affects the total rotation at hinge *i*;

Θ_i - mutual plastic rotation of end cross-sections adjacent to hinge *i*;

The compatibility condition for hinge *i* requires the rotations to be the same on both sides of support. In other words, the end sections adjacent to hinge *i* must not mutually rotate and relative total rotation θ_i must be zero.

The contribution of plastic regions in total internal work is given by the integral $\int M_i\ \phi_{pl}\, dl$ which refers to all plastic regions of the continuous beam. The estimation of this integral depends on the accuracy of the moment-curvature diagram (Fig. 4.4-1e), and the applied numerical technique. In Fig. 4.4-1 c, d, a simplified approach in calculating this integral is illustrated. The integral simply represents the area of the region of plastic curvature

For the integral between the hinges:

$$\int_i M_i \,\phi_{pl} dl = M_{i,\Omega}\,\psi_j \qquad \text{knowing that:} \qquad \psi_j = \int_i \phi_{pl} dl$$

For the integral adjacent to hinges:

$$\int_i M_i \,\phi_{pl} dl = \Theta_i \qquad \text{knowing that:} \qquad \Theta_i = \int_i \phi_{pl} dl \quad \wedge \quad M_{i,\Omega} \approx X_i = 1$$

Equation (4.4-3) may be written for every state $X_i = 1$, thus generating and solving

$$\theta_{i,o} + \theta_{ii} X_i + \sum_{k \neq i} \theta_{ik} X_k + \sum_j \Psi_j M_{i,\Omega} + \Theta_i = 0 \tag{4.4-3}$$

a system of n-linear simultaneous equations in which the elasto-plastic behaviour of a beam is accounted for. The solutions are not straightforward, because plastic rotations Θ_i and Ψ_j are functions of unknowns X_i. Through an iteration process and repeated adjustments of plastic components of rotations, solutions are possible. This is a general form of equation and it requires some rearrangements to make it more suitable for use. The modified version of the Separation Method, called the Method of Imposed Rotations (Macchi's Method) is used in this study and presented in chapter five.

4.5 Deflection

The deflection line of a loaded wythe is more realistic, if irreversible, i.e. plastic component of curvature is taken into account. Figure 4.5-1 shows that at any section where

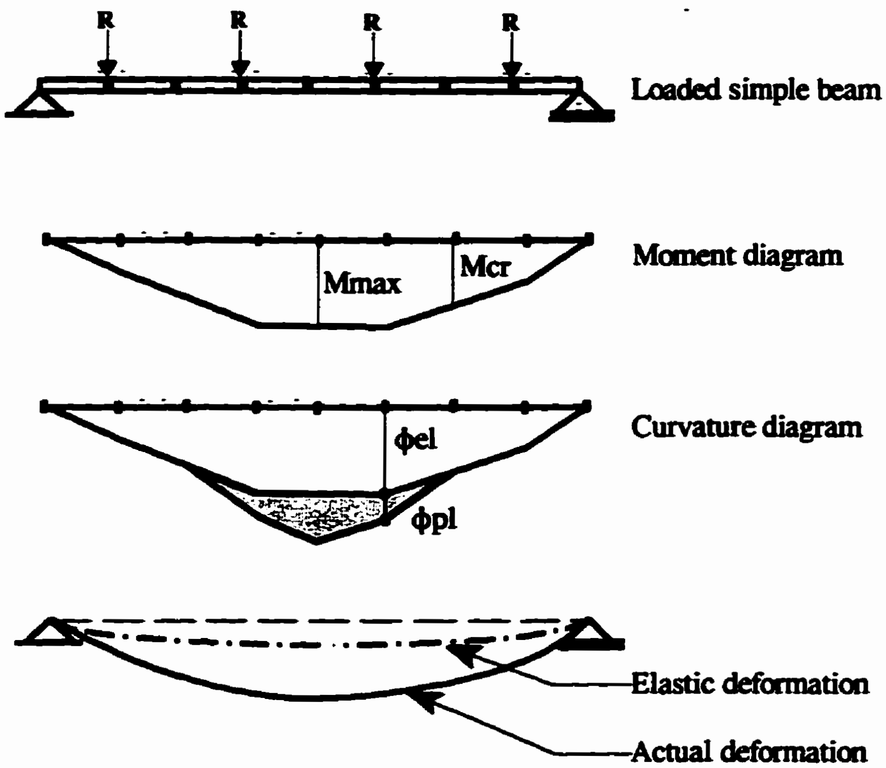


Figure 4.5-1 Deflected Beam and Curvature

the moment exceeds the elastic limit, plastic deformation takes place. It contributes significantly to the overall deformation performance of the wythe. In this study the method of the conjugate beam will be used to compute the deflections of the backup wythe. The conjugate beam is loaded with a fictitious load of intensity numerically equal to $\phi_{tot} = \phi_{el} + \phi_{pl}$ for the actual wythe. The values for curvature can be obtained from the moment-curvature relationship for a finite length of a member, elaborated in the section 4.2.3.

5. FIRST ORDER NON-LINEAR STRUCTURAL ANALYSIS /FONLSA/

5.1 Introduction

FONLSA means *First Order Non-linear Structural Analysis*. First order, because all equilibrium and kinematic relationship for the structure are expressed in terms of the undeformed geometry of the structure. Non-linear, because the assumption is the displacements and internal forces are not proportional to the applied load. The cause of

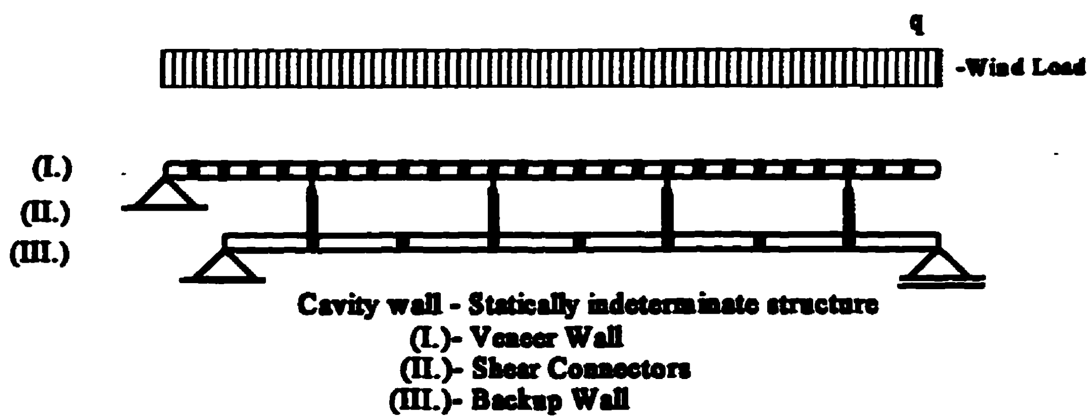


Figure 5.1-1 Cavity Wall

nonlinear structural behaviour is based on the material stress-strain relationship that will force the structural elements and the structure to have nonlinear constitutive relationships. In the analysis the mortar joints and V-Ties are assumed to act as a plastic hinges and to experience nonlinear moment-rotational deformation characteristics. Figure 5.1-1 shows a typical cavity wall used in the structural analysis. It represents a statically indeterminate structure, where forces cannot be determined without considering deformations. The program has been written in FORTRAN and can be run on UNIX system.

5.2 Mathematical Modeling / Decomposition

Figure 5.2 (A) shows the static system of the cavity wall used in this program. For methodological reasons it is decomposed into three main parts: the veneer wall (I), shear connector (II) and backup wall (III). The axial force in the wythes induced by the generated shear force in the connector is neglected. Figure 5.2-1 (B) depicts the deformation state of

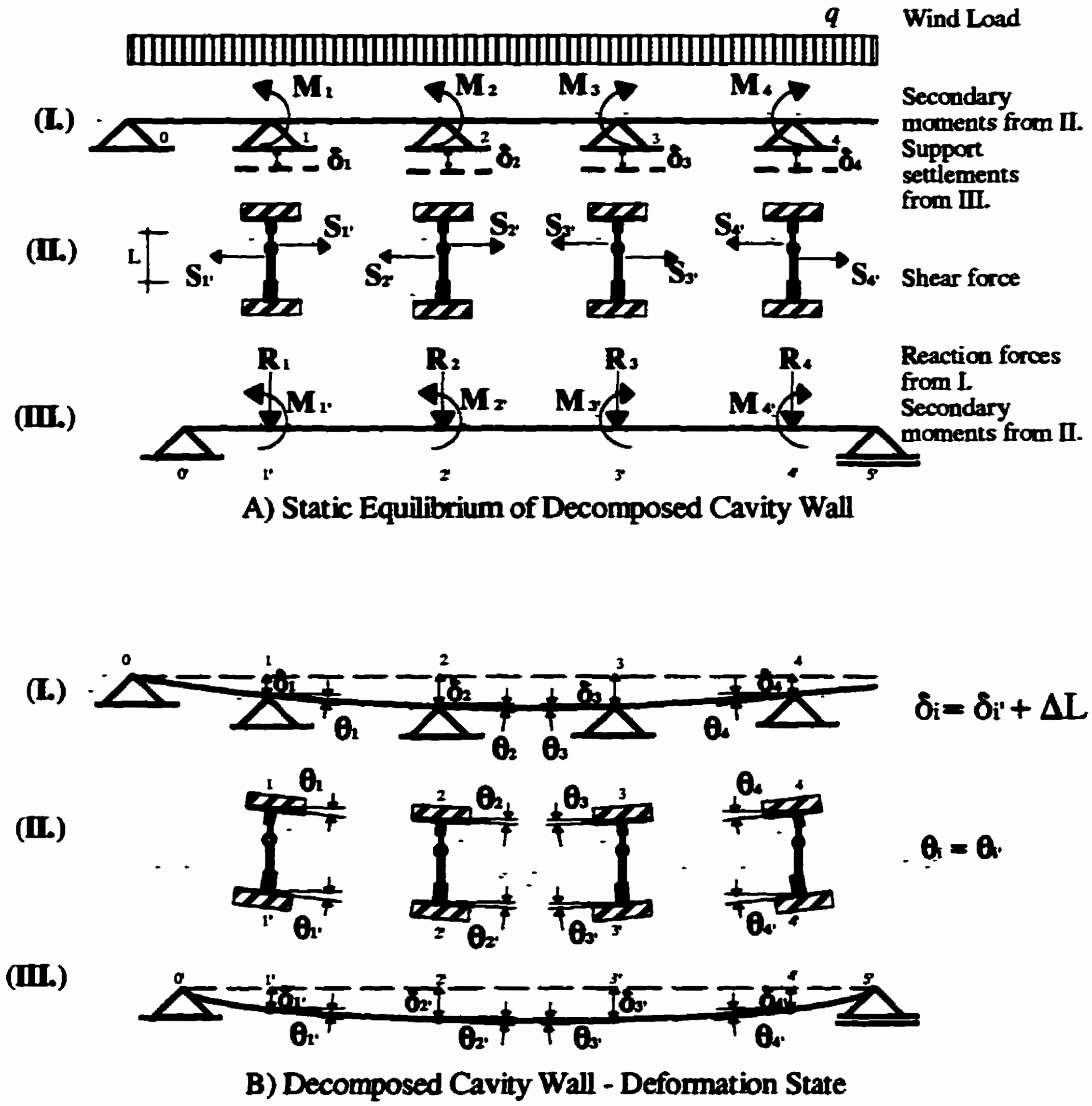
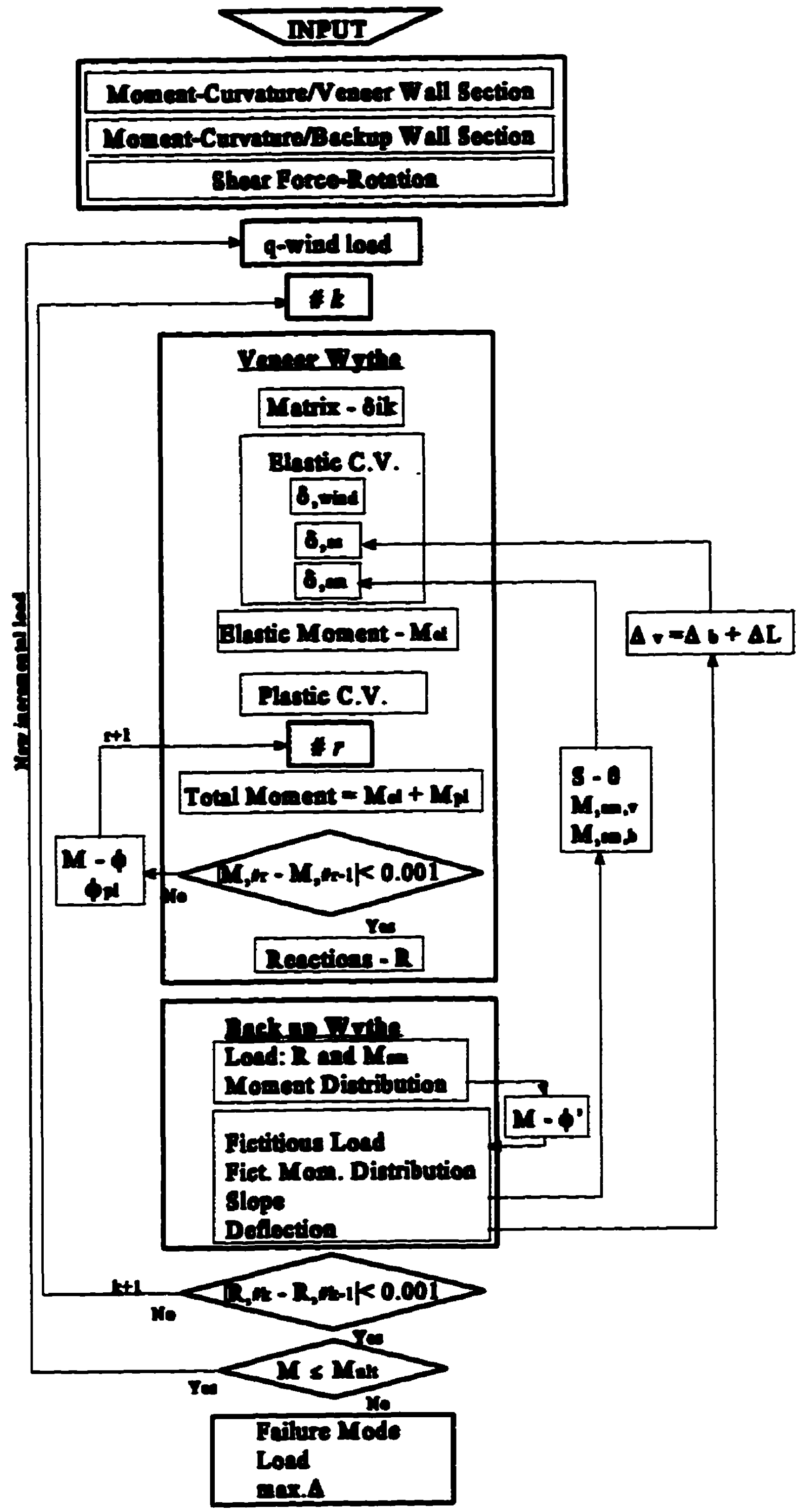


Figure 5.2-1 Decomposition of the Cavity Wall

these three main parts. The kinematic assumptions and conditions are: the deflection of the venner and backup wythe at the "supports" is defined by the expression $\delta_i = \delta_{i'} + \Delta L$ and the wythes at the "supports" experience the same rotations. Static equilibrium is maintained for each load level through the iteration process.

5.3 Flow-Chart



5.4 Section: Layer Approach

The adopted model of the masonry cross-section is shown in Fig 5.4-1, I - section with variable web width which goes from zero for ungrouted, to full flange width for a fully grouted wall.

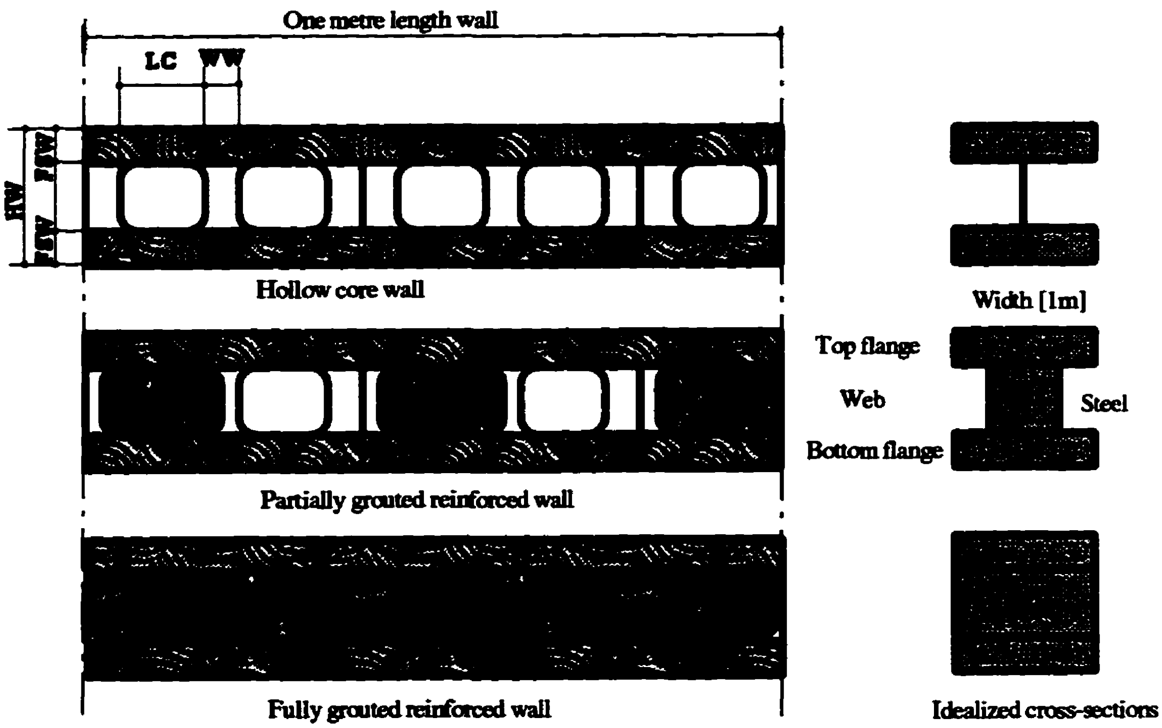


Figure 5.4-1 Equivalent and Idealized cross-section

To calculate the section moment which will produce a correspondent curvature, at the same time satisfying the constitutive law, the masonry stress integrals stated in the Sub-section 4.2.1 must be evaluated. A numerical technique has been employed[25]. The section is divided into a series of rectangular layers. The larger number of layers will give the more accurate the idealization (See Fig.5.4-2).

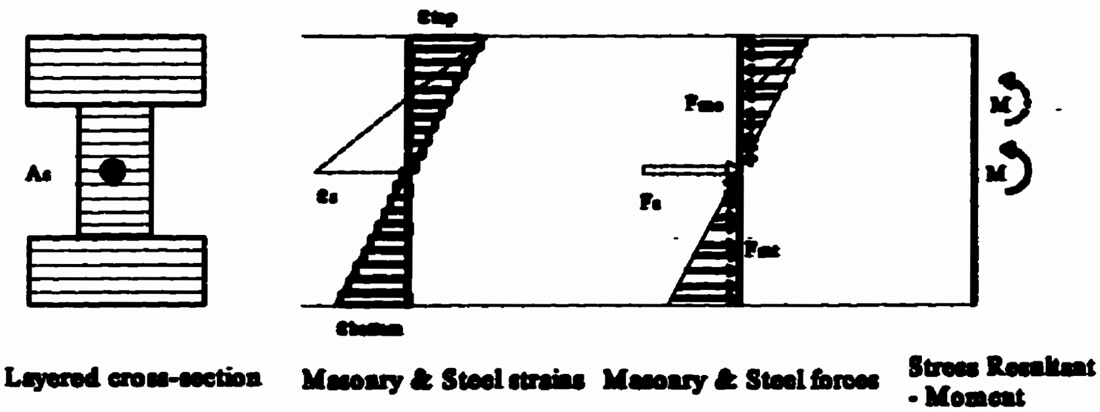


Figure 5.4-2 Calculating Sectional Moment Using Layer Approach

The next assumption is that the strain in each layer is uniform and equal to the "actual" strain at the centre of the layer. For each value of the strain, a correspondent value of the stress can easily be determined from the masonry stress-strain relationship (Fig. 5.4-3) for a

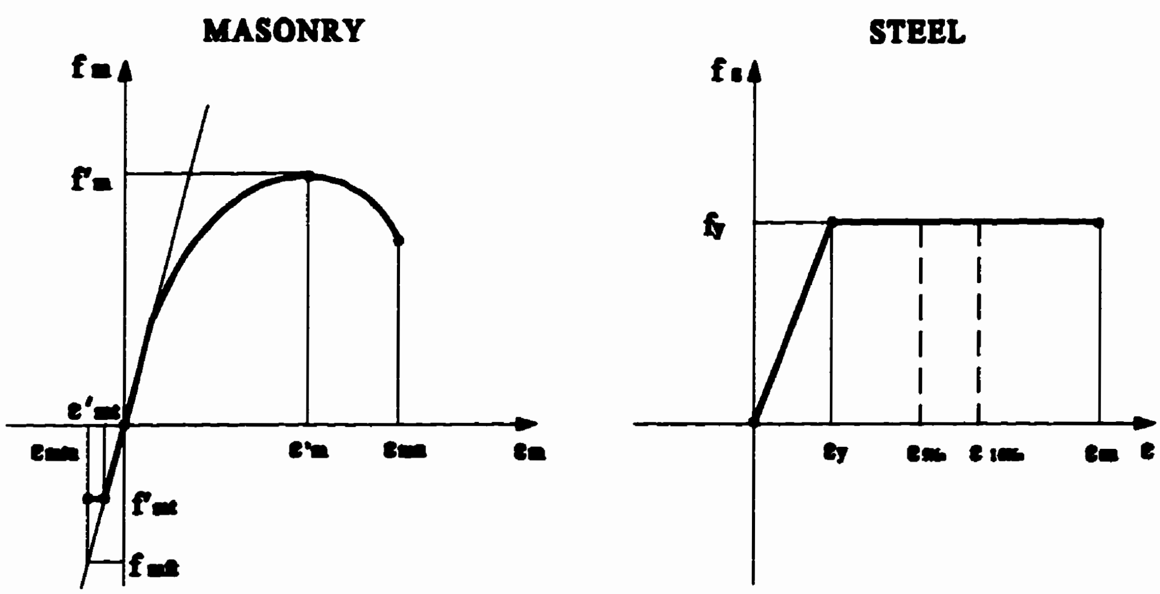


Figure 5.4-3 Stress-Strain Relationship

given section. Note that the CSA Standard S304.1-M94 allows some flexibility in the selection of a stress-strain relationship, and various physical properties of masonry are defined as follows:

> *The relationship between the masonry compressive stress and masonry strain may be assumed parabolic, trapezoidal or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests[30].*

For masonry in compression, the relationship between stress, f_m and the strain caused by this stress, ε_m is represented by the parabola shown on Fig. 5.4-3 and given by Equation[25].(5.4-1). The peak

$$f_m = f'_m\left(2\frac{\varepsilon_m}{\varepsilon'_m} - \left(\frac{\varepsilon_m}{\varepsilon'_m}\right)^2\right) \qquad (5.4\text{-}1)$$

$$\varepsilon'_m = 2\,\frac{f'_m}{E_m} \qquad (5.4\text{-}2)$$

$$\varepsilon_{mu} = 0.003 \qquad (5.4\text{-}3)$$

stress is f'_m. associated with corresponding strain ε'_m obtained from Eqn (5.4-2). The ultimate strain is adopted from the Code[3] (Eqn.5.4-3). The new Code for concrete introduces larger ultimate strain value, and it can be expected that an updated masonry code will reflect that change.

The stress-strain relationship of masonry in tension is idealized by the bi-linear curve shown on the Fig. 5.4-3. One assumption is that the masonry has the same modulus of elasticity in tension and compression[22] (Eq.5.4-4). The ratio

$$E_{mt} = E_m \qquad (5.4\text{-}4)$$

$$\frac{f'_{mt}}{f_{mft}} = 0.6 + \frac{0.4}{\sqrt[4]{h}} \approx 0.8 \qquad (5.4\text{-}5)$$

$$\varepsilon'_m = \frac{f_{mft}}{E_m} \qquad (5.4\text{-}6)$$

between ultimate tensile strength, f'_{mt}, and modulus of rupture, f_{mft}, is based on an empirical equation[23] (Eqn. 5.4-5), which is derived for concrete, but for practical purpose can be applied to the masonry as well. The ultimate tension strain is defined by Eqn. 5.4-6.

The bi-linear curve representing, the steel stress-strain relationship is shown on Fig.5.4-3. Analytically, the curve is defined by expressions

$$f_s = E_m\,\varepsilon_s, \qquad \text{if} \quad \varepsilon_s \le \varepsilon_y \tag{5.4-7}$$

$$f_s = f_y, \qquad \text{if} \quad \varepsilon_y < \varepsilon_s \le \varepsilon_{su} \tag{5.4-8}$$

5.4-7 and 5.4-8. In some European codes the ultimate strain is limited to 5‰ or 10‰.[23][24]

Then, by multiplying the stress with the area of the layer, the force in each layer can be found. The last step would be the calculation of the section moment. This can be found by a summation of all products of the layer force and the distance between the middle of the layer and the reference axis.

The procedure for calculating the forces in the reinforcing bars is as follows: the force in the bar can be found by multiplying the stress at the centre of the bar times the area of the bar, following the stress-strain relationship for a given steel grade (Fig. 5.4-3). Finally, the total moment is obtained by adding the moment caused by masonry stresses to the moment caused by bars.

5.5 Moment-curvature

In this section a subroutine is presented that calculates M-ϕ (moment-curvature) values for brick or block cross-sections (see Fig. 5.5). It encompasses the whole range of sections, from plain to fully grouted reinforced sections. The calculation pertains to one metre length of the wall. The bars are located in the middle of the section.

The required input:

Compressive unit strength, defined

Size of unit [mm]

Solid content of unit (Hollow, Solid or Solid Fully)

Type of mortar (S or N)

Number of grouted cores, per linear metre length (0-5)

Compressive strength of unit [MPa]

Compressive masonry strength [Mpa], from Table 5.5-1

Flexural tensile masonry strength [Mpa], from Table 5.5-1

Steel reinforcement [mm^2], /if plain...0 mm^2/

- Face-shell (**fsw**), Table 5.5-2
- Web width (**ww**), Table 5.5-2
- Core length, (**alc**), Table 5.5-2

Equivalent and layer's dimensions:

- Equivalent web width: ew=ng*(alc+ww)

Table 5 S304.1-94		f_m [MPa] - Compressive Strength of Concrete Block Masonry *													
	Mortar Type	S							N						
	number of grouts per m²	0	1	2	3	4	5	solid	0	1	2	3	4	5	solid
	>40	22	21	20	19	18	17	17	14	13.5	12.9	11.9	11.2	10.5	10.5
	30	17.5	16.7	15.9	15.1	14.3	13.5	13.5	12	11.4	10.8	10.2	9.6	9	9
[MPa]	20	13	12.4	11.8	11.2	10.6	10	10	10	9.5	9	8.5	8	7.5	7.5
	15	9.8	9.4	8.9	8.4	8	7.5	7.5	8	7.6	7.2	6.8	6.4	6	6
	10	6.5	6.2	5.9	5.6	5.3	5	5	6	5.7	5.4	5.1	4.8	4.5	4.5

* Linear interpolation is valid

Based on Paper(17)&(18)	f_t [MPa] - Flexural Tensile Strength of Concrete Block and Clay Brick Masonry*,**,***													
Mortar Type	S							N						
	H or S unit						S-solid	H or S unit						S-solid
number of grouts per m²	0	1	2	3	4	5		0	1	2	3	4	5	
concrete block	0.45	0.58	0.71	0.84	0.97	1.1	0.8	0.3	0.42	0.54	0.66	0.78	0.9	0.6
clay brick	0.3	0.45	0.6	0.75	0.9	1.05	0.8	0.2	0.3	0.4	0.5	0.6	0.7	0.6

* Linear interpolation is valid
** Stresses are normal to bed joints
*** For low or normal grout strength

Table 5.5-1 Table for Compressive Strength of Concrete Block Masonry and

Table Flexural Tensile Strength of Concrete Block and Clay Brick Masonry

Unit [mm]	H-hollow			S-solid			B-solid	
	FSW	WW	LC max(min)	FSW	WW	LC	FSW	WW=LC=0
brick 100	?	?	?	45			45	
block 100	26	26	156(146)	30	26	156(146)	45	
block 150	26	26	156(132)	44	30	150(126)	n/a	
block 200	32	26	136(112)	60	30	130(106)	n/a	
block 250	35	28	133(109)	75	30	130(106)	n/a	
block 300	36	32	127(113)	90	30	130(106)	n/a	

FCW - Face Shell Width
WW - Web Width
LC - Core Length

Table 5.5-2 Dimensions of Clay Bricks and Concrete Blocks

- The thickness of one face-shell layer: tfs=fsw/20.
- The thickness of one eq. web layer: hhw=(hw-2.*fsw)/20.

Reinforcing bars: Specified yield strength of 400 MPa.

Modulus of elasticity for masonry: Em = 850 f 'm

Modulus of elasticity for steel: Es = 200,000 Mpa.

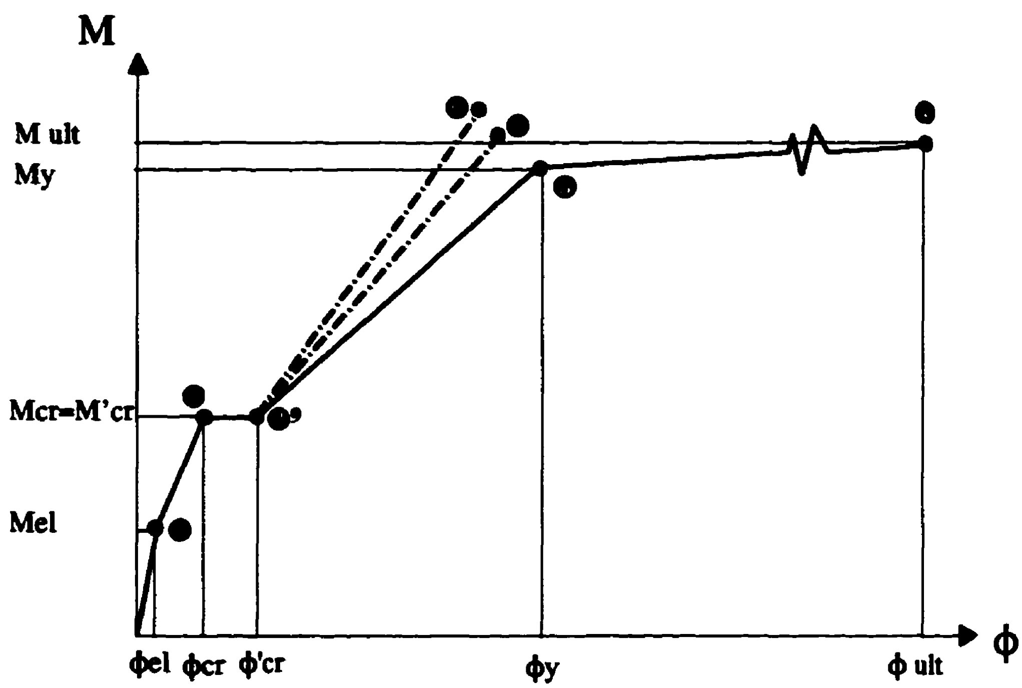


Figure 5.5-1 Idealized Moment-Curvature Diagram

POINT ● The first set of values is determined by setting the following conditions:

$\varepsilon_{,\,bottom} = f'_{mt} / E_m$

$\varepsilon_{,\,top} \ll \varepsilon_{mu}$

$\sum C = \sum F$

M_{el} & ϕ_{el}

POINT ● The second set of values is determined by setting the following conditions:

$\varepsilon,_{bottom} = f_{mft} / E_m$

$\varepsilon,_{top} \ll \varepsilon_{mu}$

$\sum C = \sum F$

M_{cr} & ϕ_{cr}

The program checks for As (amount of reinforcement). If As=0., then the program terminates.

If $A_s \neq 0.$ two new conditions are set for each case:

$f_s = E_m \, \varepsilon_s ,$ if $\varepsilon_s \leq \varepsilon_y$

$f_s = f_y ,$ if $\varepsilon_s > \varepsilon_y$

As, min The program calculates As, min. The conditions:

$M_{cr} = M'_{cr}$

$\varepsilon,_{bottom} = \varepsilon_{sy}$

If As ≤ **As, min** then <u>'Section works as a plain one'</u>.

As, bal The conditions:

$\varepsilon,_{top} = \varepsilon_{mu}$

$\varepsilon,_{bottom} = \varepsilon_y$

If **As, min ≤ As ≤ As, bal** then:

POINT ●' The third set of values is determined by setting the following conditions:

$M'_{cr} = M_{cr}$

$f_s \leq f_y$

$f_m < f'_m$

$\sum C = \sum F$

Find M'_{cr} & ϕ'_{cr}

POINT ● The fourth set of values is determined by setting the following:

$\varepsilon,_{bottom} = \varepsilon_y$

$\varepsilon,_{top} \leq \varepsilon_{mu}$*

$\sum C = \sum F$

Find M_y & ϕ_y

* If $\varepsilon,_{top} = \varepsilon_{mu}$*, then **POINT●** M_u & ϕ_u /"balance failure"/

POINT ● The fifth set of values is determined by setting the following:

$\varepsilon,_{top} = \varepsilon_{mu}$

$\varepsilon,_{bottom} > \varepsilon_y$

$\sum C = \sum F$

Find M_u & ϕ_u /"tension failure"/

POINT● The sixth set of values is determined by setting the following:

$\varepsilon_{top} = \varepsilon_{mu}$

$\varepsilon_{bottom} < \varepsilon_y$

$\sum C = \sum F$

Find M_u & ϕ_u /"compression failure"/

5.6 Shear Connector II

In this section a subroutine is presented which calculates a relationship between generated shear-force, which occurs at the location of the hinge, and imposed rotations at the member ends (at the centerline of the veneer and block wall respectively.) The Direct stiffness method is used in analysing a model beam with rigid end parts[26].

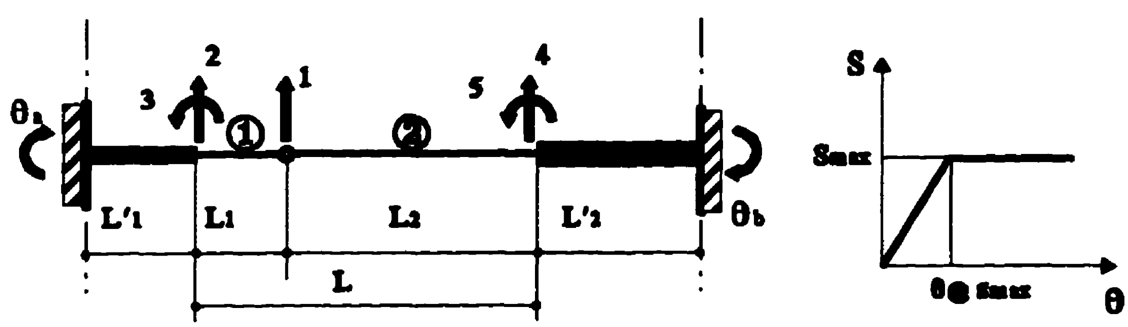


Figure 5.6 Shear-connector mathematical model and Shear force / Rotation relationship

Definition of the problem:

Due to imposed rotations $\theta_a(d_3) = \theta_b(d_5)$, find shear force ($R_1$), for incremental value of θ.

If $\theta < \theta_{s,max}$, then $S = R_1$

If $\theta \geq \theta_{s,max}$, then $S = S_{max}$

Required input:

- Cavity Width [mm] / **L** /
- V-tie Protrusion Length [mm] / $\mathbf{L_1}$ /
- Steel Plate Length [mm] / $\mathbf{L_2}$ /
- V-tie Diameter [mm] / $\mathbf{d_v}$ /
- V-tie Moment of Inertia [mm^4] / $\mathbf{I_1}$ /
- Steel Plate Moment of Inertia [mm^4] / $\mathbf{I_2}$ /
- Veneer Width* [mm] / $\mathbf{2L'_1}$ /
- Backup Width* [mm] / $\mathbf{2L'_2}$ /

The maximum shear force is limited by:

$$S_{max} = \frac{M_p}{L_1}, \qquad \textit{where} \quad M_p = f_y * z$$

$$z = \frac{d_v^3}{3}, \text{ (for two wires)}$$

Displacement vector:

Displacements:

d_1 = ? (Unknown)

$d_2 = d_3 * L'_1$

$d_3 = d_5$ (Rotation**) ** Imposed rotations

$d_4 = d_3 * L'_2$

Stiffness matrixes for ● (V-Tie) and ● (Steel plate)

$$K\ D = Q$$

$$Q = 0.$$

$$K = k_1 + k_2 \quad \text{, where} \quad k_1 = \begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix}; \quad k_2 = \begin{bmatrix} k_{11} & k_{14} & k_{15} \\ k_{41} & k_{44} & k_{45} \\ k_{51} & k_{52} & k_{55} \end{bmatrix}$$

$$D = \begin{bmatrix} D_1 \\ \\ D_0 \\ \\ \end{bmatrix} = \begin{bmatrix} \underline{d_1} \\ d_2 \\ d_3 \\ d_4 \\ d_5 \end{bmatrix}$$

$$\begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & K_{bb} \end{bmatrix} \begin{bmatrix} D_1 \\ D_0 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix}$$

$$D_1 = -K_{aa}^{-1} K_{ab} D_0 \quad \text{, where} \quad K_{aa}^{-1} = -\frac{1}{|K_{aa}|}$$

$$K\,D = R$$

$$\begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & K_{bb} \end{bmatrix} \begin{bmatrix} D_1 \\ D_0 \end{bmatrix} = \begin{bmatrix} 0 \\ R_0 \end{bmatrix}$$

$$R_0 = K_{ba} D_1 + K_{bb} D_0 = \begin{bmatrix} R_2 \\ R_3 \\ R_4 \\ R_5 \end{bmatrix} = \begin{bmatrix} T_2 \\ M_3 \\ T_4 \\ M_5 \end{bmatrix}$$

Shear force R_1 or S generated at the hinge is equal to:

$R_1 = R_2 = R_4$

Moment induced at the veneer wall due to imposed rotation $\theta = d_3 = d_5$ is equal to:

$M_v = R_3 + R_2 * L'_1$ *where*: R_3 is end moment

Moment induced at the backup wall due to imposed rotation $\theta = d_3 = d_5$ is equal to:

$M_b = R_5 + R_2 * L'_2$ *where:* R_5 is end moment

5.7 Veneer Wythe I

5.7.1 Analysis of Veneer Wythe

This subroutine performs the analysis of the veneer wythe. Statically the veneer is assumed to act as a continuous beam. The external effect consists of three parts: the positive wind load, the moments at the "support" joints induced by shear-force and "support" settlements due to flexure of the whole cavity wall. Initially, the moments at the "support" joints and support settlements are set to zero.

Vocabulary:

"Support" - joint at the connector

"Field" - wall between two "supports"

Each mortar joint is labelled, as either a "support" or a "field" joint.

Each mortar joint is designated as a plastic one

Definition of the problem:

Due to uniform wind load (q), concentrated moments ($M_{i,sm}$) and "support" settlements (Δ_i), find the total moment at each joint and reaction force at each "support".

The Method of Imposed Rotations[4] proposed by Macchi is used in this program, and represents an improved version of the Separation Method, already covered in chapter four. It is based on the principle of superposition of the distribution of moments due to external loads and the distribution of moments induced by plastic rotations.

The distribution of moments due to external loads is obtained from the system of equations:

$$\delta_{ii}\, x_i + \sum \delta_{ik}\, x_k = -\,\delta_{i,w} - \delta_{i,ss} - \delta_{i,sm} \tag{5.7-1}$$

This system of equations represents the pure application of the compatibility method, and is given in a matrix form.

Distribution of the moments due to plastic rotations is computed from:

$$\delta_{ii}\, X_i + \sum_{k \neq i} \delta_{ik}\, X_k + \sum_{j} \Psi_j\, M_{i,\Omega} + \Theta_i = 0 \tag{5.7-2}$$

This system has to be solved only for one plastic rotation, while others are taken as zero. The number of system of equations to be solved corresponds to the number of plastic regions (mortar joints) in the veneer wall

for *i=1,2.......nr*

for *j=1,2.......jpf*

Total number is equal to *nr +jpf*

Designation:

nr - number of redundancies, unknown forces

jph - number of the "supports" minus top one, equal to number of redundancies

jbvf - number of plastic joints in the bottom field

jsvf - number of plastic joints in the standard field jpf - total number of plastic joints in the fields

jphf - total number of plastic joints

Required Input, from Main:

bwh - backup wall height

sbc - spacing between two vertical connectors /400, 600, 800 mm/

Geometry:

blevel (200mm) - elevation between veneer and backup bottom support

bfvw (600mm) - bottom veneer field

bfbw - bottom backup field

tfcw(200mm) - top veneer field

sbvw (200mm/3.) - an average spacing among two bricks (57mm), including mortar joints

Total moment at each joint is calculated using the principle of superposition:

$$M_{tot} = M_{el} + \sum_j \Psi_j\, M(\Psi_j = 1) + \sum_i \Theta_i\, M(\Theta_i = 1) \qquad (5.7\text{-}3)$$

$$M_{el} = M_o + \sum_{k=1}^{nr} X_k M_k$$

$$\textit{where},\ M_o = M_w + M_{sm}$$

The rotations Θ_i and Ψ_j from Eqn. 5.7-3 are not known beforehand and an iteration procedure is used as follows:

i. Moment distribution is computed according to elastic theory (Eqn. 5.7-1, Fig. 5.7-1)

ii. Plastic rotations Θ_i and Ψ_j are found for each plastic joint from the M-ϕ diagram

iii. Distribution of moments due to imposed unit rotations is computed (Eqn. 5.7-2)

iv. Total moments are obtained from Eqn. 5.7-3

v. Rotations Θ_i and Ψ_j are corrected and subsequently the total moments

vi. Iteration is terminated as soon as the difference in moment values for two subsequent distributions do not exceed a specified value

Reactions at supports are solved via shear forces, from the moment diagram:

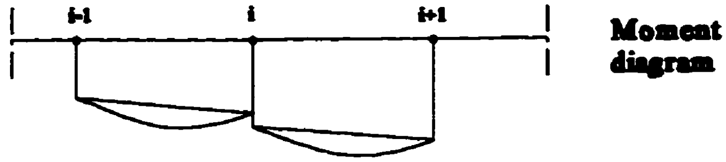
$$T = T_w + \frac{M_{i,i-1} - M_{i-1,i}}{l_{i-1,i}} \text{ ,} \textit{where}$$

$$T_{i,left} = -0.5\, q\, l_{i-1,i} + \frac{M_{i,left} - M_{i-1,right}}{l_{i-1,i}}$$

$$T_{i,right} = -0.5\, q\, l_{i,i+1} + \frac{M_{i+1,left} - M_{i,right}}{l_{i,i+1}}$$

$$R_i = T_{i,left} + T_{i,right}$$

Sign convention: Shear force is positive where the member axes tends to rotate clockwise, and



negative when it tends to rotate counterclockwise in order to coincide with the tangent at the correspondent point on the moment diagram.

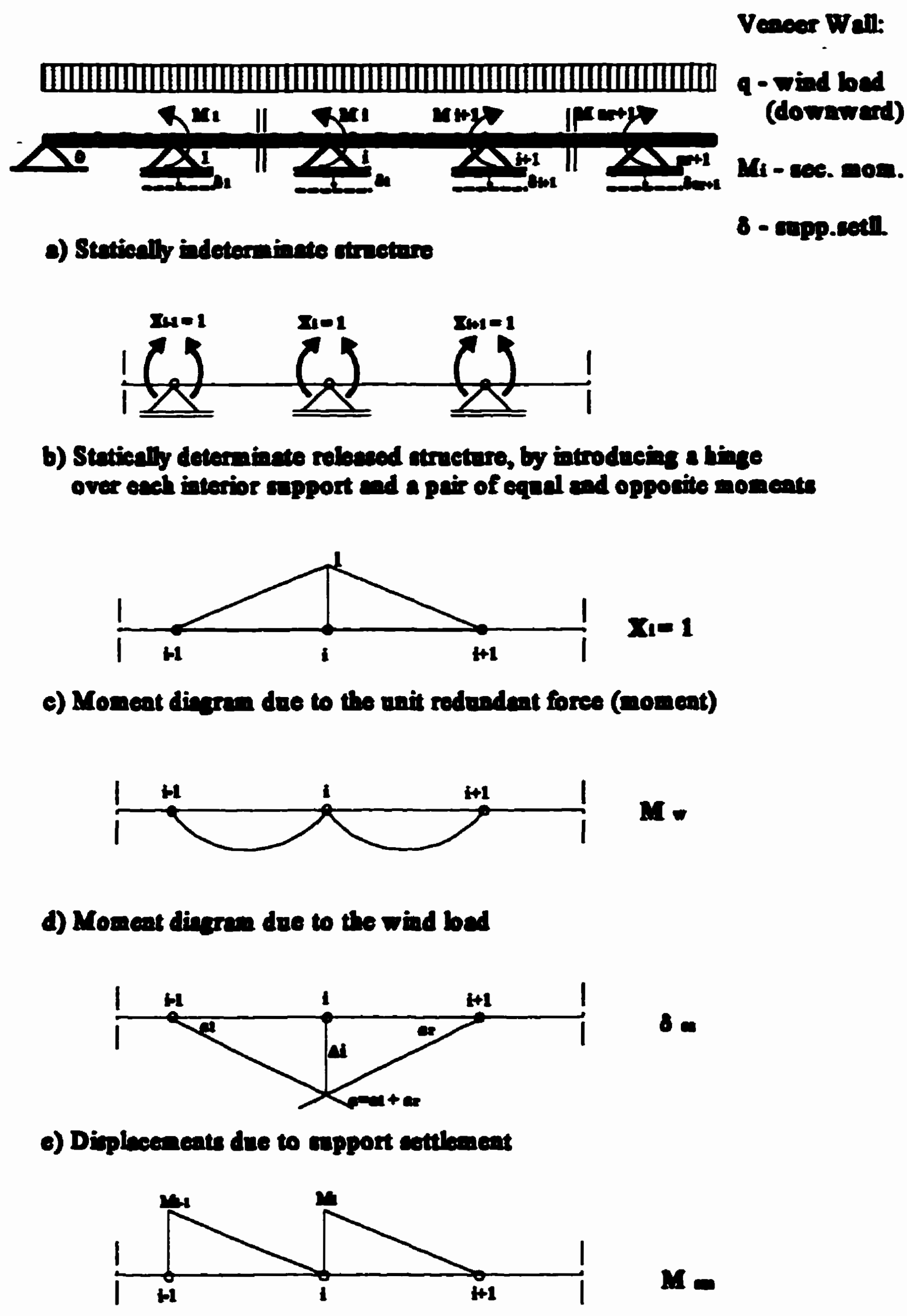
$$\delta\, X = D_{tot}$$

$$\delta_{ik} = \int_m \frac{M_i M_k}{EI}\, dl\,; \quad \delta_{iw} = \int_m \frac{M_i M_w}{EI}\, dl\,; \quad \delta_{i,ss} = \alpha = 2\,\frac{\Delta_i}{l}\,; \quad \delta_{i,sm} = \int_m \frac{M_i M_{sm}}{EI}\, dl;$$

$$\delta_{ij} = \begin{bmatrix} \delta_{11} & \delta_{12} & \cdots & \delta_{1nr} \\ \delta_{21} & \delta_{22} & \cdots & \delta_{2nr} \\ \vdots & & & \vdots \\ \delta_{nr1} & \delta_{nr2} & \cdots & \delta_{nr,nr} \end{bmatrix}; \qquad X = \begin{bmatrix} x_1 \\ x_2 \\ \vdots \\ x_{nr} \end{bmatrix}$$

$$D_{tot} = -\begin{bmatrix} \delta_1 \\ \delta_2 \\ \vdots \\ \delta_{nr} \end{bmatrix}_w - \begin{bmatrix} \delta_1 \\ \delta_2 \\ \vdots \\ \delta_{nr} \end{bmatrix}_{ss} - \begin{bmatrix} \delta_1 \\ \delta_2 \\ \vdots \\ \delta_{nr} \end{bmatrix}_{sm}$$

Notes:
nr - number of unknowns
w - wind load
ss - support settlements
sm - secondary moments



a) Statically indeterminate structure

b) Statically determinate released structure, by introducing a hinge over each interior support and a pair of equal and opposite moments

c) Moment diagram due to the unit redundant force (moment)

d) Moment diagram due to the wind load

e) Displacements due to support settlement

f) Moment diagram due to secondary moments (shear-force)

Figure 5.7-1 Analysis of a Veneer Wythe by the Compatibility Method

5.7.2 Gauss Method - Solving a System of Linear Equations

To find unknown forces in the Compatibility Method a system of linear equations has to be solved. The Gaussian elimination method is applied in this program.

Any linear system can be written as the vector equation,

$$\mathbf{A\,x = b}$$

where **A** is the coefficient matrix [N x N], **b** is the constant vector [N x 1] and **x** is the vector of unknowns [N x 1].

$$A = \begin{bmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \vdots & & & \vdots \\ a_{n1} & a_{n2} & \cdots & a_{nn} \end{bmatrix}; \quad b = \begin{bmatrix} b_1 \\ b_2 \\ \vdots \\ b_n \end{bmatrix}; \quad x = \begin{bmatrix} x_1 \\ x_2 \\ \vdots \\ x_n \end{bmatrix}$$

<u>Gaussian elimination algorithm:</u>

a. Form the N x (N + 1) augmented matrix AUG by adjoining B to A:

$$AUG = [\,A \,\vdots\, B\,]$$

b. For I ranging from 1 to N, do the following:

   i. If AUG(I,I) = 0, interchange the Ith row of AUG with any row below it for which the coefficient of X(I) is nonzero. (If there is no such row, matrix A is said to be singular, and the system does not have a unique solution.)

   ii. For J ranging from I +1 to N, do the following:

Add -AUG(J,I) / AUG(I,I) times the Ith row of AUG to the Jth row of AUG to eliminate X(I) from the Jth equation.

c. Set X(N) equal to AUG(N, N + 1) / AUG(N,N).

d. For J ranging from N -1 to 1 in steps of -1, do the following:

Substitute the values of X(J + 1),......,X(N) in the Jth equation and solve for X(J).

5.8 Backup Wall III

- Backup wall is modeled as a simple beam;
- Loads:

  Concentrated forces (axial force in the connector);

  Concentrated moments (due to shear force in the connector);
- Axial forces, equal to generated shear forces in the connectors are neglected;
- Initially, concentrated moments have zero value;
- Initially, the model has a constant stiffness. When $M > M_{cr}$, the program calculates the effective stiffness, following the M-ϕ relationship with "tension stiffening" included.

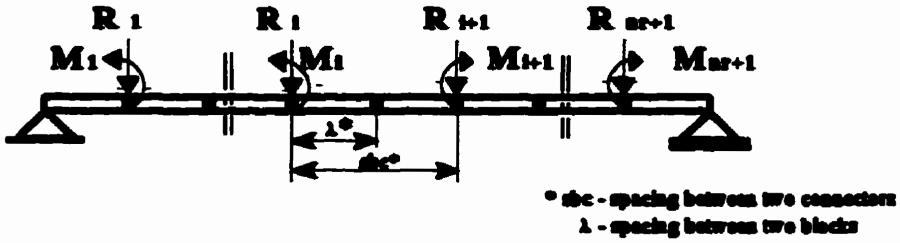


Figure 5.8-1 Back-Up Wythe

Find: I.) Moment Distribution

II.) Slopes & Deflections

I.) Moment distribution

Assuming standard spacing between two blocks (200mm), for each joint (total number of joints: ii=1,.......,nr+1), the moment is found using the recursive formulae:

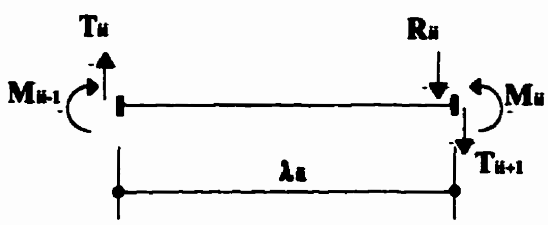
$T_{ii+1} = T_{ii} - P_{ii}$

$M_{ii} = M_{ii-1} + T_{ii} * l_{ii}$

$M_{ii,left} = M_{ii-1}$

$M_{ii,right} = M_{ii-1} \pm M_{i,\,sm}$

$M_{ii,@\text{"support"}} = 0.5\ (M_{ii,left} \pm M_{ii,right})$



b) Slopes & Deflections

The Conjugate - Beam Method is used for solving slopes and deflections of the backup wall.

Slope: $\theta - \theta_t = T^f$

Deflection: $\delta = M^f$

where the fictitious load: $p^f = \phi = M / EI_{eff.}$

Effective stiffness ($EI_{eff.}$) is based on the moment-curvature diagram for a finite length of the wall, following Line 0-2-3-4 (Fig. 5.8-2).

If:

$$M < M_{cr} \rightarrow EI_{eff} = \frac{M_{cr}}{\phi_{cr}}$$

$$M_{cr} \le M < M_y \rightarrow EI_{eff} = \frac{M}{\phi_{cr} + \phi_{pl,cr}}, \text{ where } \phi_{pl,cr} = (M - M_{cr})\frac{\phi_y - \phi_{cr}}{M_y - M_{cr}}$$

$$M_y \le M \le M_u \rightarrow EI_{eff} = \frac{M}{\phi_y + \phi_{pl,y}}, \text{ where } \phi_{pl,y} = (M - M_y)\frac{\phi_u - \phi_y}{M_u - M_y}$$

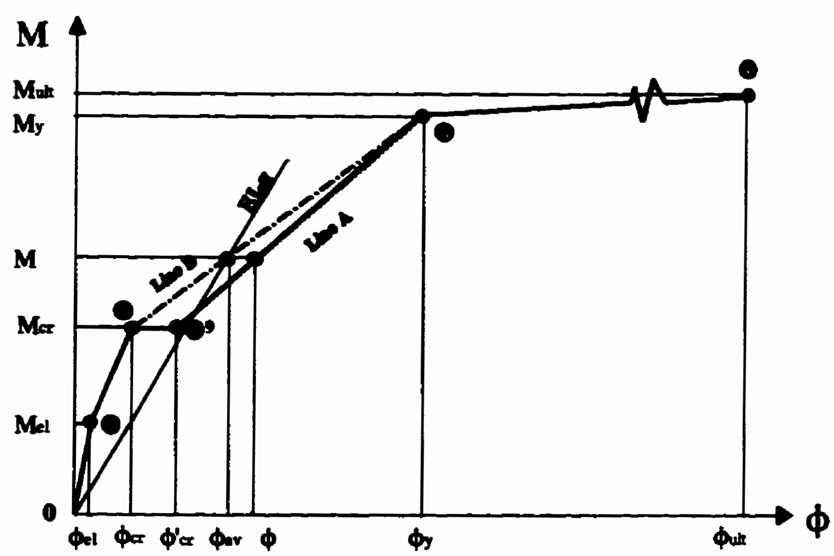
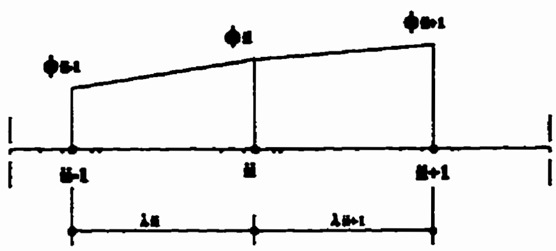


Figure 5.8-2 Calculating the Effective Stiffness

From static-kinematic analogy it can be concluded that the deflection-diagram matches moment- diagram, and the slope-diagram matches the shear-diagram of the fictitious beam loaded with fictitious load (See Fig. 5.8-3).



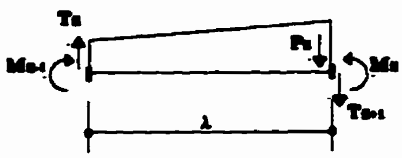
The fictitious load has a non-uniform and polygonal shape. For practical purpose this load will be replaced in two ways:

- With equivalent concentrated force, P^f, at each joint

$$P^f_0 = \frac{\lambda}{6}(2\phi_0 + \phi_1)$$

$$P^f_{ii} = \frac{\lambda}{6}(\phi_{ii-1} + 4\phi_{ii} + \phi_{ii+1})$$

$$P^f_{nr+1} = \frac{\lambda}{6}(\phi_{nr} + 2\phi_{nr+1})$$



then, *fictitious moment ≈ deflection*

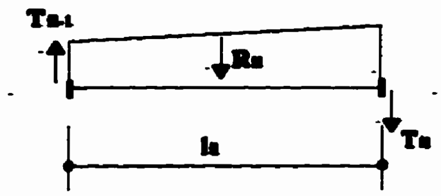
$M^f \approx \delta$

- With resultant force, R^f, which replaces load between two joints

$$R^f_1 = \frac{\phi_1}{2}\lambda$$

$$R^f_{ii} = \frac{\phi_{ii-1} + \phi_{ii}}{2}\lambda$$

$$R^f_{nr+1} = \frac{\phi_{nr+1}}{2}\lambda$$



then, *fictitious shear force* ≈ *slope*

$$T^f = \theta$$

At the "support" joints, axial strain due to axial force in the connectors is taken into account:

$$\Delta L_i = \frac{R_i}{E_s}\left(\frac{L_1}{A_1} + \frac{L_2}{A_2}\right), \quad \textit{for } i=1....nr+1$$

$$\delta_{i,veneer} = \delta_{i,backup} + \Delta L_i$$

6. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This study presents a comprehensive structural analysis of shear connected cavity walls, vertically spanned, subject to a wind load. Since the introduction of the new Block Shear™ Connector the role and the structural behaviour of traditional cavity walls with flexible ties changed significantly. The author recognized a great need for a rational approach and more realistic prediction of structural performance of the cavity wall.

The analysis is based on the combination of the Method of Imposed Rotations and the Stiffness Method. For methodological reasons the structure is decomposed into three main parts: the veneer wall, shear-connector and backup wall. The main assumption is that the mortar joints of both wythes and V-tie are assumed to be the only regions where a plastic portion of the deformation takes place.

The proposed Method of Imposed Rotations which falls into the category of Separation Methods is a special type of non-linear analysis. It is based on the Principle of Superposition, with material non-linear stress-strain relationships.

The main objectives of the study were: more realistic prediction of the structural behaviour of the cavity wall, better understanding of non-linear deformation phenomena, to determine distribution of moments and consequently load-deflection relationship at any load stage, not only at ultimate, and to define modes of failures.

The program computes five points on the moment-curvature diagrams of any plain, partially or fully grouted, with or without reinforcement, brick or block whether standard or non-standard section. It calculates the maximum of the induced shear force in a function of geometric and material properties. It also generates a non-linear load-deflection diagram of a masonry wall, subject to flexure, with the "tension stiffening" factor accounted for. Furthermore, it generates a non-linear load-deflection diagram of the cavity wall due to a lateral load up to the moment of cracking, and has been largely developed to predict behaviour through to failure.

The program has been written in FORTRAN and can be run on the UNIX system.

6.2 Conclusions

The advantage of the proposed method is that it is conceptually founded on the premise that the method of analysis should be independent of the procedure for estimating material properties in order to be valid for current as well as for possible future knowledge of these properties.

The stress-strain compatibility method is based on the hypothesis of the elasto-plastic nature of the body. To make this method applicable for the whole range of loading, besides the compression portion, the tension portion of the stress-strain curve must be also known.

Due to the recognition of the plastic deformation component in a tension zone of the masonry, two "elastic moments" are introduced: the moment of the limit of elasticity and the moment of cracking. Up to the moment of elasticity only linear distribution of stresses and strains are present in the tension zone. The moment of cracking is a load level when the tension zone is fully plastified and reaches its full capacity. In a reinforced section, an abrupt change of the section stiffness occurs and the bars become effective.

A rational procedure is presented in a computer form that calculates M-ϕ (moment-curvature) values for brick or block cross-sections. It encompasses the whole range of sections, from plain to fully-grouted reinforced sections. The procedure facilitates designing the optimal combination of the masonry assembly for a specific application. It is shown that:

- the larger block unit sizes of reinforced masonry section significantly increase the flexural capacity of the masonry wall;
- using the units with higher compressive strength in masonry walls subject to flexure is not economical;
- the usage of higher strength type "S" mortar is justified, especially for crack control, since the masonry with type "S" has a 25% greater cracking moment limit compared with the masonry with type "N";
- there is no justification for specifying more reinforcement, unless the moment capacity

governs the design. On the contrary, it is shown that the increase of the area of the reinforcement decreases the ductility of the section;

- the number of grouted cores does not increase the ultimate flexural capacity of the reinforced masonry wall section; however, the grout significantly increase the capacity of a non-reinforced section.

The program recognizes and analytically formulates the tension stiffening factor for a finite length of a masonry wall.

It is shown that the Block Shear™ Connector has the ability not only to transfer a lateral load from veneer to a backup wall, but also to generate shear forces that in turn produces positive moments in both wythes. Also, it is shown that the endmost shear connectors attract considerably higher axial forces than the others.

The analysis shows that the critical limit state for unreinforced shear connected masonry walls subject to wind load is the tensile failure of the mortar bed of the backup wythe. It occurs at the central portion of the wythe, where the sections are subject to the maximum moments. The cavity walls with reinforced backup wythe are a very effective combination.

6.3 Recommendations

I. The structural analysis of the cavity wall presented should include the effect of the axial load and environmental loads. However, the program itself possesses enough flexibility to be upgraded to accommodate the effect of vertical loads.

II. The computer program should be refined and made more user-friendly.

III. In order for the performance of the cavity wall to be accurately quantified and verified there is a need for obtaining more accurate information about material properties.

IV. It is worth mentioning that the current Code does not reflect the higher tensile strength f_t of the grouted masonry versus solid masonry. Also, it does not address the fact that f_t is not only a function of the strength characteristics of the component materials, but also a function of their geometric characteristics. More testing is required to resolve this matter.

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APPENDIX: A

Program Listing /FONLSA/

```
C   MASTER THESIS "NON-LINEAR STRUCTURAL ANALYSIS OF
C   SHEAR-CONNECTED  CAVITY WALLS DUE TO WIND LOAD"
*********************************************************************
C                    April 1997, University of Manitoba
C *********************************************************************
C
      implicit real*8(a-h,o-z)
      dimension del(20,20),delmo(20),delms(20),tcvel(20),xel(20),
     *        tdelss(20),
     *        caf(20),cmh(20,2),cmf(100),caff(20,50),
     *        vmom(20),bmom(20),
     *        amelh(20,2),amelf(100),
     *        defll(20),
     *        cvplh(20,20),cvplf(20,100),xplh(20,20),xplf(20,100),
     *        plrh(20),plrf(100),totmh(20,2),totmf(100),
     *        amplf(100),amplh(20),axplh(20,20),axplf(20,20),
     *        totmhh(20,50),ddel(20,20),xxel(20)
      character*4 vtm,btm
      parameter(diter=100000.,number=20)
      common /mmfistr/fy,est
c -----------------------------------------------------------------
C I N P U T:
c -----------------------------------------------------------------
C Input needed for Main
C q        - wind load
C bwh      - backup wall height [mm], from floor to floor
C sbc      - vertical spacing between two connectors/400;600;800/[mm]
C -----------------------------------------------------------------
C Input needed for moment-curvature relation of the veneer wall /MOMCURV/
C
C vs       - size(width) of unit[mm]
C ivprs    - solid precentage of unit (hollow/50% & 75%/, solid/100%/)
C vtm      - type of mortar ( S or N )
C ngcv     - number of grouted cores(0-5)
C vcms     - compressive masonry strength[MPa]
C vftms    - flexural tensile masonry strength[MPa]
C vstr     - reinforcment[mm2]/if plain...0./
c -----------------------------------------------------------------
C Input needed for moment curvature relation of the backup wall/MOMCURV/
C
C bs       - size(width) of unit[mm]
C ibprs    - solid precentage of unit (hollow/50% & 75%/, solid/100%/)
C btm      - type of mortar ( S or N )
```

```
C ngcb    - nmber of grouted cores(0-5)
C bcms    - compressive masonry strength[MPa]
C bftms   - flexural tensile masonry strength[MPa]
C bstr    - reinforcment[mm2]/if plain...0./
c -----------------------------------------------------------------
C Input needed for Shear force-Rotation relation/SHEARFR/
C
C cw      - cavity width [mm]
C tiepl   - V-tie protrusion length [mm]
C areav   - sectional area of V-tie [mm2]
C areas   - sectional area of Steel plate [mm2]
C tiesma  - V-tie,momentof inertia [mm4]
C spsma   - steel plate, moment of inertia [mm4]
c -----------------------------------------------------------------
      read*,q
      read*,bwh
      read*,sbc
c -----------------------------------------------------------------
      read*,vs
      read*,ivprs
      read*,vtm
      read*,ngcv
      read*,vcms
      read*,vftms
      read*,vstr
c -----------------------------------------------------------------
      read*,bs
      read*,ibprs
      read*,btm
      read*,ngcb
      read*,bcms
      read*,bftms
      read*,bstr
c -----------------------------------------------------------------
      read*,cw
      read*,tiepl
      read*,areav
      read*,areas
      read*,tiesma
      read*,spsma
c -----------------------------------------------------------------
      print*,'Wall Height =',bwh
      print*,'Connector Space =',sbc
```

```
C -------------------------------------------------------------
C Description:
C nr     - number of redundants
C bfvw    - bottom veneer field [mm]
c tfcw    - top field    [mm]
C vwh     - veneer wall height [mm]
C blevel  - elevation between veneer and backup bottom support
C bfbw    - bottom backup field
C sbvw    - spacing among two bricks,including mortar joints
C jph     - number of plastic joints at the supports(the last one not
c         included), jph=nr
C jbvf    - number of plasic joints in the bottom field
C jsvf    - number of plastic joints in the standard field
C jpf     - number of plastic joints in the fields
c jphf    - totalnumber of plastic joints
C est     - modulus of elasticity, steel [N/mm2]
C fy      - yielding stress [N/mm2]
C -------------------------------------------------------------
   blevel=200.
   bfvw=600.
   bfbw=bfvw-blevel
   tfcw=200.
   vwh=bwh+blevel
   nr=(vwh-bfvw-tfcw)/sbc
   sbvw=200./3.
   jph=nr
   jbvf=bfvw/sbvw-1
   jsvf=sbc/sbvw-1
   jpf=jbvf+nr*jsvf
   jphf=jph+jpf
   est=200000.
   fy=400.
c -------------------------------------------------------------
   print*,'          '
   print*,'Moment-Curvature for Veneer Wall'
   CALL MOMCURV(vs,ivprs,vtm,ngcv,vcms,vftms,vstr,
  *     vmel,vfiel,vmcr,vficr,vmcrp,vficrp,vmy,vfiy,vmu,vfiu,eiv)
   print*,'          '
   print*,'Moment-Curvature for Backup Wall'
   CALL MOMCURV(bs,ibprs,btm,ngcb,bcms,bftms,bstr,
  *     bmel,bfiel,bmcr,bficr,bmcrp,bficrp,bmy,bfiy,bmu,bfiu,eib)
c  go to 1500
   print*,'          '
```

```
      CALL SHEARFR(cw,tiepl,areav,areas,tiesma,spsma,vs,bs,
     *        xarea,sfmax,rotmax)
c -----------------------------------------------------------------
c     go to 1500
      CALL MATRIX (nr,bfvw,sbc,
     *        del)
c     go to 1500
C loop ,if failure is not declared new value for q!!!!
      idefl=0
  700 print*,'_______________________'
      print*,'WIND LOAD q=',q
      print*,'_______________________'
c ----------------------------------------------------
C constant vector-wind load-delmo(i)
c ----------------------------------------------------
      CALL CVWIND (nr,q,tfcw,bfvw,sbc,
     *        amoh,delmo)
c ----------------------------------------------------
C loop from backup wall  NEW DEFLL(ii)
C constant vector-due to support setllments-tdelss(i)
c support settlments are positive downward
c ----------------------------------------------------
      icirc=1
  650 print*,' '
c     if((q.eq.qq).and.(icirc.eq.ici))then
c     print*,q,icirc
      go to 1500
c     end if
      print*,q,icirc
      print*,'ITERATION No.= ',icirc
      print*,'----------------------'
      CALL CVSUPSET (nr,bfvw,defll,sbc,eiv,
     *        tdelss)
c -------------------------------------------------------
C const. vector-due to secondary moments-delms(i) i=1...nr
C input secondary moments vmom(i)  i=1,nr+1
c vmom(i),assumed must be all positive
c constant vector is positive(tcvel(i)=+delms(i))
c -------------------------------------------------------
      CALL CVSECMOM (nr,sbc,vmom,
     *        it,delms)
c -------------------------------------------------------
c     print*,'Total constant vector elastic-tcvel(i)'
```

```
c -----------------------------------------------------
      CALL CVTOTAL (nr,delmo,tdelss,delms,
     *        tcvel)
      CALL GAUSS (nr,del,tcvel,
     *        xel)
c -----------------------------------------------------
c redundancies, moments-positive, bottom side in tension
c -----------------------------------------------------
      do 100 i=1,nr
      print*,'Joint No:',i,' Moment-Xel=',xel(i)
 100  continue
c -----------------------------------------------------
c elastic portion of total moment amelh(i,j)-hinges-i=1...jph+1
c -----------------------------------------------------
      CALL TOTALMEL (jph,amoh,q,tfcw,it,jbvf,jsvf,bfvw,sbvw,vmom,
     *        sbc,xel,
     *        amelh,amelf)
      do 371 ii=1,jph
       if((totmhh(ii,iter).lt.0).and.(ii.le.jph))then
        go to 445
       end if
 371  continue
      go to 1000
 445  print*,'  '
      print*, 'REACTIONS'
      CALL REACT (amelh,jph,bfvw,tfcw,sbc,q,vwh,
     *        summcaf,sumq,caf)
C -----------------------------------------------------
      iflag=1
      do 150 ii=1,jph+1
       caff(ii,icirc)=abs(caf(ii))
 150  continue
      do 155 ii=1,jph+1
       if(abs(caff(ii,icirc)-caff(ii,icirc-1)).gt.100)then
        iflag=1
        go to 160
       else
       iflag=0
       end if
 155  continue
 160  CALL BACKUP_M (caf,bwh,tfcw,jph,sbc,bfbw,sumq,bmom,
     *        sbbw,jbbf,jsbf,jtbf,ntotal,cmh,cmf)
      print*,'  '
```

```
    print*,'FI /curvature/ from M-FI '
    ifail=0
    CALL SLO_DEFL (ntotal,bwh,sbbw,jbbf,jsbf,jtbf,
   *          cmh,cmf,est,caf,icirc,
   *          bmcr,bficr,bmy,bfiy,bmu,bfiu,
   *          sfmax,rotmax,cw,tiepl,vs,bs,xarea,
   *          vmom,bmom,defll,ifail,idefl)
C if idefl=0 then elastic analysis
    idefl=1
    idefl=0
C ----------------------------------------------------
    if(iflag.eq.1.and.icirc.lt.15)then
     icirc=icirc+1
     go to 650
    end if
    if(ifail.eq.0)then
c -------------------------------------------------------
C constant vector,plastic-hinges-cvplh(i,ii)   ii=1,jph
C total number jphf=jph+jpf,   jph=nr
c sign; if plastic rotation from `totm(ii) and xel(i)=1 on the same side
c then....
c =cv*(-1.)
c -------------------------------------------------------
    jph=nr
    do 165 ii=1,jph
     do 170 i=1,nr
      if(i.eq.ii)then
       cvplh(i,ii)=(-1.)*eiv
      else
       cvplh(i,ii)=0.0
      end if
     tcvel(i)=cvplh(i,ii)
     print*,'cvplh ',cvplh(i,ii)
 170  continue
    do 171 i=1,nr
    do 172 j=1,nr
    ddel(i,j)=del(i,j)
 172  continue
 171 continue
    call gauss(nr,ddel,tcvel,xxel)
    do 173 i=1,nr
    xplh(i,ii)=xxel(i)
    print*,'ii',ii,'  xplh  ',xxel(i)
```

```
 173  continue
 165 continue
c ------------------------------------------------------------
C constant vector,plastic-field points-cvplf(i,ii)  ii=1,jpf
C first field (0-1)
c ------------------------------------------------------------
    k=1
    jbf=bf/sbb-1
    do 175 ii=1,jbf
     do 180 i=1,nr
      if(i.eq.k)then
       cvplf(i,ii)=(-1.)/bf*ii*sbb*eiv
      else
       cvplf(i,ii)=.0
      end if
     tcvel(i)=cvplf(i,ii)
     print*,'ii ',ii,' cvplf ',cvplf(i,ii)
 180  continue
    do 181 i=1,nr
     do 182 j=1,nr
     ddel(i,j)=del(i,j)
 182  continue
 181 continue
     call gauss(nr,ddel,tcvel,xxel)
     do 183 i=1,nr
     xplf(i,ii)=xxel(i)
      print*,'k',k,' ii',ii,' i',i,'  xplf',xxel(i)
      print*,'xplf  ',xplf(i,ii)
 183  continue
 175 continue
c ------------------------------------------------------------
C    second field (1-2)........(last-1)
c ------------------------------------------------------------
    jj=jbf
    jsf=sf/sbb-1
    do 195 k=2,nr
     do 200 ii=jj+1,jj+jsf
      do 205 i=1,nr
       if(i.eq.k-1)then
        cvplf(i,ii)=-1./sf*((jj+jsf+1)-ii)*sbb*eiv
       else if(i.eq.k)then
        cvplf(i,ii)=-1./sf*(ii-jj)*sbb*eiv
       else if(i.gt.k)then
```

```
      cvplf(i,ii)=0.0
     end if
    tcvel(i)=cvplf(i,ii)
    print*,'ii ',ii,' cvplf ',cvplf(i,ii)
205  continue
   do 206 i=1,nr
    do 207 j=1,nr
    ddel(i,j)=del(i,j)
207  continue
206 continue
     call gauss(nr,ddel,tcvel,xxel)
     do 208 i=1,nr
     xplf(i,ii)=xxel(i)
      print*,'k',k,' ii',ii,' i',i,' xplf',xxel(i)
208  continue
200  continue
    jj=jj+jsf
195 continue
   k=nr+1
   do 210 ii=jj+1,jj+jsf
    do 215 i=1,nr
     if(i.eq.k-1)then
      cvplf(i,ii)=-1./sf*((jj+jsf+1)-ii)*sbb*eiv
     else
      cvplf(i,ii)=0.0
     end if
     tcvel(i)=cvplf(i,ii)
215  continue
   do 216 i=1,nr
    do 217 j=1,nr
    ddel(i,j)=del(i,j)
217  continue
216 continue
    call gauss(nr,ddel,tcvel,xxel)
    do 218 i=1,nr
    xplf(i,ii)=xxel(i)
     print*,'k',k,' ii',ii,' i',i,' xplf',xxel(i)
218  continue
210 continue
   print*,'k',k,'jpf',jj+jsf
   print*,'jphf ',jphf
c -------------------------------------------------------
C  plastic portion of moment-hinges- ii=1,jph+1;
```

```
c calculate amplh(ii); ii=1,jph+1
c plr(j) from otmh(i,j) & M-FI
c ------------------------------------------------------
440  do 375 ii=1,jph+1
      amplh(ii)=0.0
      do 380 j=1,jph
       totmhh(j)=(totmh(j,1)+totmh(j,2))/2.
       print*,'totmhh(',j,')=',totmhh(j)
       abstotmhh(j)=abs(totmhh(j,iter))
       print*,'abstotmhh(',j,iter,')=',abstotmhh(j,iter)
c ------------------------------------------------------
c this condition applies to non-reinforced veneer wall
c      if(vstr.eq.0.and.abstotmhh(j).ge.vmcr)print*,'failure'
c ------------------------------------------------------
       if(totmhh(j,iter).lt.vmcr)then
        plrh(j)=0.0
       else if(totmhh(j,iter).ge.vmcr.and.totmhh(j,iter).lt.vmy)then
        filin=(totmhh(j,iter)/vmcr)*vficr
        difcry=((vmy-totmhh(j,iter))/(vmy-vmcr))*(vfiy-vficrp)
        teta=vfiy-(filin+difcry)
        plrh(j)=teta*10.
       else if(totmhh(j,iter).ge.vmy.and.totmhh(j,iter).lt.vmu)then
        filin=(totmhh(j,iter)/vmcr)*vficr
        difyu=((vmu-totmhh(j,iter))/(vmu-vmy))*(vfiu-vfiy)
        teta=vfiu-(filin+difyu)
        plrh(j)=teta*10.
       else if(totmhh(j,iter).ge.vmu)then
        print*,'tension failure at the joint-hinge No. ',j
       end if
       amplh(ii)=amplh(ii)+xplh(ii,j)*plrh(j)
       print*,'plrh(',j,')=',plrh(j)
  380  continue
      print*,'amplh(',ii,')=',amplh(ii)
      print*,'totmhh(',ii,')=',totmhh(ii)
      do 385 j=1,jpf
c ------------------------------------------------------
c this condition applies to non-reinforced veneer wall
c      if(vstr.eq.0.and.totmf(j).ge.vmcr)print*,'failure'
c ------------------------------------------------------
       if(totmf(j).lt.vmcr)then
        plrf(j)=0.0
       else if(totmf(j).ge.vmcr.and.totmf(j).lt.vmy)then
        filin=(totmf(j)/vmcr)*vficr
```

```
      difcry=((vmy-totmf(j))/(vmy-vmcr))*(vfiy-vficrp)
      teta=vfiy-(filin+difcry)
      plrf(j)=teta*10.
     else if(totmf(j).ge.vmy.and.totmf(j).lt.vmu)then
      filin=(totmf(j)/vmcr)*vficr
     difyu=((vmu-totmf(j))/(vmu-vmy))*(vfiu-vfiy)
      teta=vfiu-(filin+difyu)
      plrf(j)=teta*10.
     else if(totmf(j).ge.vmu)then
      print*,'tension failure at the joint-fields No. ',j
     end if
     print*,'plrf(',j,')=',plrf(j)
     amplh(ii)=amplh(ii)+xplf(ii,j)*plrf(j)
 385  continue
    print*,'amplh(',ii,')=',amplh(ii)
c -----------------------------------------------------------
C TOTAL MOMENT AT HINGES/ELASTIC PORTION amelh(ii,j)
c                 PLASTIC PORTION amplh(ii)
c -----------------------------------------------------------
    do 390 j=1,2
     totmh(ii,j)=amelh(ii,j)+amplh(ii)
     print*,'totmh(',ii,',',j,')=',totmh(ii,j)
 390  continue
 375 continue
c -----------------------------------------------------------
C TOTAL MOMENT AT FIELDS/ELASTIC PORTION amelf(ii,j)
c                 PLASTIC PORTION amplf(ii)
c -----------------------------------------------------------
   k=1
   do 395 ii=1,jbf
    amplf(ii)=0.0
    do 400 j=1,jph
     axplh(k,j)=xplh(k,j)/bf*ii*sbb
     amplf(ii)=amplf(ii)+axplh(k,j)*plrh(j)
 400  continue
    do 405 j=1,jpf
     axplf(k,j)=xplf(k,j)/bf*ii*sbb
     amplf(ii)=amplf(ii)+axplf(k,ii)*plrf(j)
 405  continue
    totmf(ii)=amelf(ii)+amplf(ii)
    print*,'totmf(',ii,')=',totmf(ii)
 395 continue
c -----------------------------------------------------------
```

```
c    standard fields- k=2.....nr+1
c -----------------------------------------------------
     jj=jbf+1
     jsf=sf/sbb-1
     do 410 k=2,nr+1
      do 415 ii=jj+1,jj+jsf
       amplf(ii)=0.0
c -----------------------------------------------------
c check jph+1
c -----------------------------------------------------
       do 420 j=1,jph
        xplh(nr+1,j)=0.0
        axplh(k,j)=xplh(k-1,j)/sf*((jj+jsf+1)-ii)*sbb
     *            +xplh(k,j)/sf*(ii-jj)*sbb
        amplf(ii)=amplf(ii)+axplh(k,j)*plrh(j)
  420   continue
       do 425 j=1,jpf
        xplf(nr+1,j)=0.0
       axplf(k,j)=xplf(k-1,j)/sf*((jj+jsf+1)-ii)*sbb
     *            +xplf(k,j)/sf*(ii-jj)*sbb
        amplf(ii)=amplf(ii)+axplf(k,j)*plrf(j)
  425   continue
       totmf(ii)=amelf(ii)+amplf(ii)
       print*,'totmf(',ii,')=',totmf(ii)
  415 continue
       jj=jj+jsf
  410 continue
     do 430 ii=1,jph
      totmhh(ii,iter+1)=(totmh(ii,1)+totmh(ii,2))/2.
     if((totmhh(ii,iter+1)-totmhh(ii,iter)).gt.diter)then
      iter=iter+1
      print*,'Number of iteration  ',iter
      go to 440
     end if
  430 continue
     print*,'Total number of iteration  ',iter
 1500 end
C
C
C
C
C ******************************************************************
C  THIS  SUBROUTINE CALCULATES M-FI (MOMENT-CURVATURE) VALUES
```

```
C  FOR BRICK&BLOCK CROSS-SECTION,FROM ZERO TO ULTIMATE
C  STAGE.SUITABLE FOR C PLAIN AND FOR REINFORCED SECTIONS.
C ******************************************************************
C I N P U T:
C hw      - size(width) of unit[mm]
C ispr    - solid precentage of unit (hollow/50%&75%/, solid/100%/)
C typem   - type of mortar ( S or N )
C ng      - number of grouted cores(0-5)
C fmp     - compressive masonry strength[MPa]
C fmtf    - flexural tensile masonry strength[MPa]
C ast     - reinforcment[mm2](fy=400Mpa)/if plain...0./
C ------------------------------------------------------------
      SUBROUTINE MOMCURV(hw,ispr,typem,ng,ffmp,ffmtf,aast,
     *          amel,fiel,amcr,ficr,amcrp,ficrp,amy,fiy,amu,fiu,ei)
      implicit real*8(a-h,o-z)
      character*4,typem
      common hhw,ew,tfs,bw,ast,shw,epst,epsb,epsr,fmp,em,fmtf,fmtp,
     *     epsy,ck
     *     ,fi,fst,sumc,summ
      common /mmfistr/fy,est
C ------------------------------------------------------------
C epsb - bottom strain (positive)
C epst - top strain (negative)
C fi   - curvature (positive associated with algebrically larger
C        bottom strain)
C ------------------------------------------------------------
      fmp=ffmp
      fmtf=ffmtf
      ast=aast
      em=850.*fmp
      eps=-0.003
      epsy=fy/est
      fmtp=0.8*fmtf
C ------------------------------------------------------------
C LAYERS
C ------------------------------------------------------------
      fsw=35.
      ww=28.
      alc=109.
      bw=1000.
      ew=ng*(alc+ww)
      tfs=fsw/20.
      hhw=(hw-2.*fsw)/20.
```

```
C point 1.
      ck=1.
      shw=hw
      epsb=fmtp/em
      epst=-1.*epsb
      epsr=epsb
      CALL STR
      amel=(-1.)*summ
      fiel=fi
      print 100,amel,fi
  100 format(/10x,'Mel =',f13.2,'[Nmm]',10x,'FIel =',f13.10)
c     go to 1000
C point2.
      epsb=fmtf/em
      epst=(-1.)*epsb
      epsr=epsb
      CALL STR
  110 if(abs(sumc).gt.100)then
        epst=epst+0.0000001
        CALL STR
        go to 110
      else
        epst2=epst
        amcr=(-1.)*summ
        fi2=fi
        ficr=fi2
        ei=amcr/ficr
        print 120,amcr,fi2
  120   format(10x,'Mcr =',f13.2,'[Nmm]',10x,'FIcr =',f13.10)
      end if
C if steel=0 no more calculation
      if(ast.eq.0)stop
C calculate Ast,min (from Mcr=Mcr', Fi2<Fi2')
      ck=0.
      shw=0.5*hw
      epsb=epsy
      epst=epst2
      epsr=epsb
      CALL STR
  200 if((summ+amcr).le.0)then
       tf=-1*sumc
       asmin=tf/fy
       summ=(-1.)*summ
```

```
      print 210,asmin
      print 211,summ,fi
 210  format(/10x,'As.min=',f8.2,'[mm2]')
 211  format( 10x,'M   =',f13.2,'[Nmm]',10x,'FI   =',f13.10)
     else
      epst=epst-0.000005
      CALL STR
      go to 200
     end if
C check for min. reinforcment
     if(asmin.gt.ast)then
      print*,'       Section works as a plain one'
      stop
     else
     end if
C csalculate Ast,balance
     epst=epsu
     epsb=epsy
     epsr=epsb
     CALL STR
     tf=-1*sumc
     asbal=tf/fy
     summ=(-1.)*summ
     print 250,asbal
     print 251,summ,fi
 250 format(/10x,'As.bal=',f8.2,'[mm2]')
 251 format( 10x,'M   =',f13.2,'[Nmm]',10x,'FI   =',f13.10)
C check actualAst. and Ast.balance
     if(ast.gt.asbal)go to 500
     if(ast.eq.asbal)go to 600
C point '
     epsb=0.00001
     epst=epst2
     epsr=epsb
 300 CALL STR
     tf=fst*ast
     sumc=-1*sumc
     if(tf/sumc.gt.0.99.and.tf/sumc.lt.1.01)then
      tf=sumc
     else
      epsb=epsb+0.000001
      epsr=epsb
      go to 300
```

```
      end if
      if((summ+amcr).gt.0)then
       epst=epst-0.000001
       epsr=epsb
       go to 300
      else
       amcrp=(-1.)*summ
       ficrp=fi
       print 350,amcrp,fi
 350   format(/10x,'Mcrp=',f13.2,'[Nmm]',10x,'FIcrp=',f13.10)
      end if
C tension failure ; yielding moment
      epst=epsu
      epsb=epsy
      epsr=epsb
 440 CALL STR
      tf=fst*ast
      sumc=-1.*sumc
      if(tf/sumc.lt.0.99.or.tf/sumc.gt.1.01)then
       epst=epst+0.000001
       go to 440
      else
       print*,'          Tension failure'
       amy=(-1.)*summ
       fiy=fi
       print 450,amy,fi
 450   format(10x,'My  =',f13.2,'[Nmm]',10x,'FIy  =',f13.10)
      end if
C moment capacity at tension failure
      epst=epsu
      epsb=epsy
      epsr=epsb
 460 CALL STR
      tf=fst*ast
      sumc=-1.*sumc
      if(tf/sumc.lt.0.99.or.tf/sumc.gt.1.01)then
       epsb=epsb+0.000001
       epsr=epsb
       go to 460
      else
       amu=(-1.)*summ
       fiu=fi
       print 470,amu,fi
```

```
 470  format(10x,'Mult=',f13.2,'[Nmm]',10x,'Flult=',f13.10)
    end if
    go to 1000
C compression failure
 500 epst=epsu
    epsb=epsy
    epsr=epsb
 540 CALL STR
    tf=fst*ast
    sumc=-1.*sumc
    if(tf/sumc.lt.0.99.or.tf/sumc.gt.1.01)then
     epsb=epsb-0.0000001
     epsr=epsb
     go to 540
    else
 500  print*,'        Compression failure'
     print 550,summ,fi
 550  format(/10x,'Mc,ult=',f15.2,'     FIc,ult=',f12.10)
    end if
    go to 1000
c balance failure
 600 print*,'         Balance failure'
    stop
 1000 end
C
C
c ----------------------------------------------------------
C SUBROUTINE  STR calculates strains and stresses
c ----------------------------------------------------------
    UBROUTINE STR
    implicit real*8(a-h,o-z)
    dimension eps(60),c(60),am(60)
    common hhw,ew,tfs,bw,ast,shw,epst,epsb,epsr,fmp,em,
   *      fmtf,fmtp,epsy,ck,
   *      fi,fst,sumc,summ
    common /mmfistr/fy,est
c ---------------------------------------------------------------
    ii=60
    m=20
    n=40
    sumrr=0.
    sumc=0.
    summ=0.
```

```
      if (ck.eq.1)go to 50
      if (ast.ne.0)then
       ii=30
       m=0
       n=10
      end if
   50 do 100 i=1,ii,1
        if(i.le.m.or.i.gt.n)then
          rr=tfs
          ss=bw
        else
          rr=hhw
          ss=ew
        end if
        fi=(abs(epst)+abs(epsb))/shw
c ---------------------------------------------------------------
C strains
c ---------------------------------------------------------------
        sumrr=sumrr+rr
        yy=sumrr-0.5*rr
        eps(i)=epsr-fi*yy
        if(ck.eq.1)go to 110
        if(ast.ne.0.and.eps(i).lt.0)then
          eps(i)=-1*eps(i)
          epsp=2.*fmp/em
          fm=fmp*(2.*eps(i)/epsp-(eps(i)/epsp)**2.)
          fm=-1*fm
          go to 150
        else if(ast.ne.0.and.eps(i).gt.0)then
          fm=0.
          go to 150
        else
        end if
c ---------------------------------------------------------------
C stresses
c ---------------------------------------------------------------
  110   if (epsb.eq.(fmtp/em))then
          edif=0.
        else
          edif=(fmtf-fmtp)/em
        end if
        if((eps(i).ge.epst).and.(eps(i).le.(epsb-edif)))then
          fm=eps(i)*em
```

```
      go to 200
     else if(eps(i).gt.(epsb-edif).and.eps(i).lt.epsb)then
      fm=fmtp
      go to 200
     end if
 150  if(epsb.lt.epsy)then
      fst=est*epsb
     else
      fst=fy
     end if
 200  c(i)=fm*ss*rr
     sumc=sumc+c(i)
     am(i)=c(i)*yy
     summ=summ+am(i)
 100 continue
    end
C
c
c
c ************************************************************************
C This subroutine solves system of linear quations with constant vector
C ************************************************************************
    SUBROUTINE GAUSS(nnr,sdel,stcvel,sxel)
    implicit real*8(a-h,o-z)
    dimension sdel(20,20),stcvel(20),sxel(20)
c ----------------------------------------------------------------
    do 100 i=1,nnr
     sdel(i,nnr+1)=stcvel(i)
 100 continue
C ----------------------------------------------------------------
C locate non-zero diagonal entry
C ----------------------------------------------------------------
    do 105 i=1,nnr
     if(sdel(i,i).eq.0)then
     npivot=0
     j=i+1
 110  if((npivot.eq.0).and.(j.le.nnr))then
       if(sdel(j,i).ne.0)npivot=j
       j=j+1
       go to 110
      end if
     if(npivot.eq.0)then
      stop'matrix is singular'
```

```
      else
C -----------------------------------------------------------------
C interchange rows and pivot
C -----------------------------------------------------------------
      do 115 j=1,nnr+1
       temp=sdel(i,j)
       sdel(i,j)=sdel(npivot,j)
       sdel(npivot,j)=temp
 115   continue
      end if
     end if
C -----------------------------------------------------------------
C eliminate i-th unknown from equation i+1....nr
C -----------------------------------------------------------------
     do 120 j=i+1,nnr
      amult=-sdel(j,i)/sdel(i,i)
      do 125 k=i,nnr+1
       sdel(j,k)=sdel(j,k)+amult*sdel(i,k)
 125   continue
 120  continue
 105 continue
C -----------------------------------------------------------------
C find the solutions
C -----------------------------------------------------------------
    sxel(nnr)=sdel(nnr,nnr+1)/sdel(nnr,nnr)
    do 130 j=nnr-1,1,-1
     sxel(j)=sdel(j,nnr+1)
     do 135 k=j+1,nnr
      sxel(j)=sxel(j)-sdel(j,k)*sxel(k)
 135  continue
     sxel(j)=sxel(j)/sdel(j,j)
 130 continue
    end
C
c
c
c
C ************************************************************************
 this Subroutine calculates relationship between generated
C shear-force and rotaton at the member ends
C ************************************************************************
C al    - cavity width [mm]
C all   - V-Tie protrusion length [mm]
```

```
C al2   - steel plate length [mm]
C dv    - diameter of V-Tie [mm]
C ail   - second moment of area of V-Tie [mm4]
C ai2   - second moment of area of steel plate [mm4]
C hwv   - veneer width
C hwb   - backup width
c ---------------------------------------------------------------
      SUBROUTINE SHEARFR(al,all,areav,areas,ail,ai2,hwv,hwb,
     *               xarea,smax,rotmax)
      implicit real*8(a-h,o-z)
      dimension d(5),vk(5,5),spk(5,5),sk(5,5),skdp(5),skdpp(5),
     *        r(5)
      common /mmfistr/fy,est
c ---------------------------------------------------------------
      print 10
      print 20
      print 40,al
      print 50,all
      print 60,ail
      print 70,ai2
      print 80,hwv
      print 90,hwb
   10 format(/'SHEAR_FORCE-ROTATION (SF-TETA) RELATIONSHIP')
   20 format( 10x,'==================================================')
   40 format( 10x,'Cavity width  [mm]                    ',f6.2)
   50 format( 10x,'V-Tie protrusion length [mm]          ',f6.2)
   60 format( 10x,'V-Tie/moment of inertia/ [mm4]        ',f10.2)
   70 format( 10x,'Steel Plate/moment of inertia/  [mm4]  ',f10.2)
   80 format( 10x,'Veneer Wall width [mm]                ',f6.2)
   90 format( 10x,'Backup Wall width [mm]                ',f6.2)
c ---------------------------------------------------------------
      dv=4.76
      al2=al-all
      allp=hwv*0.5
      al2p=hwb*0.5
      xarea=all/areav+al2/areas
      s1=est*ail
      s2=est*ai2
      sml=ail/(0.5*dv)
      amp=fy*sml
      rp=amp/all
C ---------------------------------------------------------------
C stiffness matrix for V-Tie and Steel Plate
```

```
C -----------------------------------------------------------
    do 150 i=1,5
     do 20 j=1,5
      vk(i,j)=0.0
 200  continue
 150 continue
    vk(1,1)=3.*s1/al1**3.
    vk(1,2)=-1.*vk(1,1)
    vk(1,3)=-3.*s1/al1**2.
    vk(2,2)=vk(1,1)
    vk(2,3)=-1.*vk(1,3)
    vk(3,3)=3.*s1/al1
    do 250 j=1,5
     do 300 i=1,5
      vk(i,j)=vk(j,i)
c      print*,vk(i,j)
 300   continue
 250 continue
    do 350 i=1,5
     do 400 j=1,5
      spk(i,j)=0.0
 400  continue
 350 continue
    spk(1,1)=3.*s2/al2**3.
    spk(1,4)=-1.*spk(1,1)
    spk(1,5)=3.*s2/al2**2.
    spk(4,4)=spk(1,1)
    spk(4,5)=-1.*spk(1,5)
    spk(5,5)=3.*s2/al2
    do 450 j=1,5
     do 500 i=1,5
      spk(i,j)=spk(j,i)
 500   continue
 450 continue
C -----------------------------------------------------------
C total stiffness matrix
C -----------------------------------------------------------
    do 550 i=1,5
     do 600 j=1,5
      sk(i,j)=vk(i,j)+spk(i,j)
 600  continue
 550 continue
C -----------------------------------------------------------
```

```
C unknown vertical displacement(at the hinge)
C -----------------------------------------------------------------
     ak=-1.
 620 rot=ak*0.0000001
     do 630 i=1,5
       d(i)=0.0
 630 continue
     d(3)=rot
     d(2)=al1p*d(3)
     d(4)=-1.*al2p*d(3)
     d(5)=d(3)
     skd=0.0
     do 650 i=2,5
      skd=skd+sk(1,i)*d(i)
 650 continue
     d(1)=-1.*(1./sk(1,1))*skd
c -----------------------------------------------------------------
C reaction forces
c -----------------------------------------------------------------
     do 690 i=2,5
      skdp(i)=0.0
      skdpp(i)=0.0
 690 continue
     do 700 i=2,5
      skdp(i)=sk(i,1)*d(1)
 700 continue
     do 750 i=2,5
       skdpp(i)=0.0
       do 800 j=2,5
         skdpp(i)=skdpp(i)+sk(i,j)*d(j)
 800   continue
 750 continue
     do 850 i=2,5
       r(i)=skdp(i)+skdpp(i)
 850 continue
     if(r(4).lt.rp)then
      ak=ak-1.
      go to 620
     else
      amv=r(3)+r(2)*al1p
      amb=r()-r(4)*al2p
      smax=r(4)
      rotmax=abs(d(3))
```

```
c ------------------------------------------------------------
     print 890
     print 900,amp
     print 910,r(4)
     print 920,amv
     print 930,amb
     print 940,d(3)
c    print 950,(d(i),i=1,5)
 890 format(/10x,'R E S U L T S:  ')
 900 format(  10x,'Moment of Plasticity    [Nmm]          ',f10.2)
 910 format(  10x,'Maximum Shear-Force      [N]           ',f10.2)
 920 format(  10x,'Moment at Veneer Wall  [Nmm]           ',f10.2)
 930 format(  10x,'Moment at Backup Wall  [Nmm]           ',f10.2)
 940 format(  10x,'Rotation at the ends                   ',f10.6/)
c 950 format(  10x,'Displacement  ',25x,f10.6)
    end if
 1000 end
C
C
C
C ************************************************************************
C This is subroutine MATRIX
C ************************************************************************
    Subroutine MATRIX(nr,bfvw,sbc,
   *              del)
    implicit real*8(a-h,o-z)
    dimension del(20,20)
c ------------------------------------------------------------
    do 100 i=1,nr
     do 105 j=1,nr
     del(i,j)=0.0
 105  continue
 100 continue
    del(1,1)=(bfvw+sbc)/3.
    del(1,2)=sbc/6.
    do 110 i=2,nr
     j=i
     del(i,j)=(sbc+sbc)/3.
     if(i.lt.nr)then
      j=i+1
      del(i,j)=sbc/6.
     end if
 110 continue
```

```
      do 115 j=1,nr
       do 120 i=1,nr
        del(i,j)=del(j,i)
  120  continue
  115 continue
      do 125 i=1,nr
       do 130 j=1,nr
       del(i,j)=del(i,j)
  130  continue
  125 continue
      end
C
C
C
c ********************************************************************
C This is subroutine CVWIND
C ********************************************************************
      SUBROUTINE CVWIND(nr,q,tfcw,bfvw,sbc,
     *           amoh,delmo)
      implicit real*8(a-h,o-z)
      dimension delmo(20)
c ------------------------------------------------------------
      amoh=q*tfcw**2/2.
      d 100 i=1,nr
       delmo(i)=0.0
  100 continue
      delmo(1)=bfvw/3.*(q*bfvw**2/8.)+sbc/3.*(q*sbc**2/8.)
      delmo(2)=2.*sbc/3.*(q*sbc**2/8.)
      do 200 i=3,nr-1
       delmo(i)=delmo(2)
  200 continue
      delmo(nr)=delmo(2)-sbc/6.*amoh
      do 300 i=1,nr
  300 continue
      end
c
c
c ********************************************************************
c   This is subroutine CVSUPSET
C ********************************************************************
      SUBROUTINE CVSUPSET(nr,bfvw,defll,sbc,eiv,
     *            tdelss)
      implicit real*8(a-h,o-z)
```

```
      dimension defll(20),delss(20,20),tdelss(20)
c ------------------------------------------------------------------
      delsso1=defll(1)/bfvw
      delss(2,1)=defll(1)/sbc
      delss(1,1)=(-1.)*(delsso1+delss(2,1))
      do 100 i=2,nr
       delss(i-1,i)=defll(i)/sbc
       delss(i+1,i)=defll(i)/sbc
       delss(i,i)=(-1.)*(delss(i-1,i)+delss(i+1,i))
  100 continue
      delss(nr,nr+1)=defll(nr+1)/sbc
      print*,'Delta/ss/-support settlments'
      do 200 j=1,nr
       tdelss(j)=0.0
       delss(1,0)=0.0
       do 300 i=j-1,j+1
        tdelss(j)=tdelss(j)+delss(j,i)*eiv
  300  continue
  200 continue
      end
c
c
C ************************************************************************
C This is subroutine CVSECMOM
C ************************************************************************
      SUBROUTINE CVSECMOM (nr,sbc,vmom,
     *                    it,delms)
      implicit real*8(a-h,o-z)
      dimension vmom(20),delms(20)
c ------------------------------------------------------------------
      do 100 i=1,nr
       it=(nr+1)/2
       if(i.eq.1)then
        delms(i)=(-1.)*sbc/3.*vmom(i)
       else if(i.gt.1.and.i.lt.it)then
        delms(i)=(-1.)*sbc/3.*vmom(i)+(-1.)*sbc/6.*vmom(i-1)
       else if(i.eq.it)then
        delms(i)=(-1.)*sbc/6.*vmom(i-1)+(-1.)*sbc/3.*vmom(i)
     *            +(-1.)*sbc/6.*(-1.)*vmom(i+1)
       else if(i.eq.it+1)then
        delms(i)=(-1.)*sbc/6.*vmom(i-1)+(-1.)*sbc/3.*(-1.)*vmom(i)
                 +(-1.)*sbc/6.*(-1.)*vmom(i+1)
       else if(i.gt.it+1.and.i.le.nr)then
```

```
      delms(i)=(-1.)*sbc/3*(-1.)*vmom(i)+(-1.)*sbc/6.*(-1.)*vmom(i+1)
      end if
      print*,' delms(',i,')=',delms(i)
  100 continue
      end
c
c
C ***********************************************************************
C This is subroutine CVTOTAL-elastic
C ***********************************************************************
      SUBROUTINE CVTOTAL (nr,delmo,tdelss,delms,
     *                  tcvel)
      implicit real*8(a-h,o-z)
      dimension delmo(20),tdelss(20),delms(20),tcvel(20)
c ------------------------------------------------------------------
      do 100 i=1,nr
       tcvel(i)=-delmo(i)-tdelss(i)-delms(i)
       print*,' tcvel(',i,')=',tcvel(i)
  100 continue
      end
C
C
C
C ***********************************************************************
C This is subroutine TOTALMEL
C ***********************************************************************
      SUBROUTINE TOTALMEL (jph,amoh,q,tfcw,it,jbvf,jsvf,bfvw,sbvw,
     *            vmom,sbc,xel,
     *             amelh,amelf)
      implicit real*8(a-h,o-z)
      dimension vmom(20),xel(20),amelh(20,2),amelf(100),amelhh(20,50)
     *       ,amo(100),ams(100),amxel(100)
c ------------------------------------------------------------------
      iter=1
      xel(jph+1)=(-1.)*amoh
      do 100 i=1,jph+1
       if(i.ge.it+1)vmom(i)=(-1.)*vmom(i)
       do 150 j=1,2
        if((j.eq.1.and.i.le.it).or.(j.eq.2.and.i.gt.it))then
         amelh(i,j)=xel(i)
        else
         amelh(i,j)=(-1.)*vmom(i)+xel(i)
        end if
```

```
      print*,'Veneer Moment(',i,',',j,')=',amelh(i,j)
 150  continue
      amelhh(i,iter)=(amelh(i,1)+amelh(i,2))/2.
 100 continue
c     go to 1000
c -----------------------------------------------------
C elastic portion of total moment amelf(ii)-field points-ii=1,jpf
C     first term: [amo(ii);ams(ii)]
c     first field k=1
c -----------------------------------------------------
     k=1
     do 200 i=1,jbvf
      amo(ii)=(q*bfvw/2.)*ii*sbvw-(q/2.*(ii*sbvw)**2)
      ams(ii)=0.
 200 continue
c -----------------------------------------------------
c     standard field,including the last one
c -----------------------------------------------------
     jj=jbvf
     do 250 k=2,jph+1
      do 300 ii=jj+1,jj+jsvf
c -----------------------------------------------------
C amlf- moment which takes into account overhang moment amelh(jph+1,1)
c -----------------------------------------------------
       if(k.eq.jph+1)then
        amlf=1./sbc*(ii-jj)*sbvw*amelh(jph+1,1)
       else
        amlf=0.
       end if
       amo(ii)=(q*sbc/2.)*(ii-jj)*sbvw-(q/2.*((ii-jj)*sbvw)**2)
     *          +amlf
       if(k.le.it)then
        ams(ii)=(-1.)*vmom(k-1)/sbc*((jj+jsvf+1)-ii)*sbvw
       else if(k.eq.it+1)then
        ams(ii)=(-1.)*(vmom(k-1)+vmom(k))/2.
       else if(k.gt.it+1)then
        ams(ii)=(-1.)*vmom(k)/sbc*(ii-jj)*sbvw
       end if
 300  continue
     jj=jj+jsvf
 250 continue
c -----------------------------------------------------
C elastic portion including redundants-amxel(ii)-ii=1,jpvf
```

```
c ------------------------------------------------------
    xel(jph+1)=0.0
    k=1
    do  350 ii=1,jbvf
     amxel(ii)=1./bfvw*(ii)*sbvw*xel(k)
 350 continue
    jj=jbvf
    do 400 k=2,jph+1
     do 450 ii=jj+1,jj+jsvf
      amxel(ii)=1./sbc*((jj+jsvf+1)-ii)*sbvw*xel(k-1)
   *          +1./sbc*(ii-jj)*sbvw*xel(k)
c     print*,'amxel(',ii,') ',amxel(ii)
 450  continue
    jj=jj+jsvf
 400 continue
c ------------------------------------------------------
C elastic moment-field points-amelf(ii);ii=1,jpvf
c ------------------------------------------------------
    k=1
    do 500 ii=1,jbvf
     amelf(ii)=amo(ii)+ams(ii)+amxel(ii)
 500 continue
    jj=jbvf
    do 550 k=2,jph+1
     do 600 ii=jj+1,jj+jsvf
      amelf(ii)=amo(ii)+ams(ii)+amxel(ii)
 600  continue
    jj=jj+jsvf
 550 continue
    end
C
C
CC
C ****************************************************************
C This subroutine is REACT
C ****************************************************************
    SUBROUTINE REACT (amelh,jph,bfvw,tfcw,sbc,q,vwh,
   *               summcaf,sumq,caf)
    implicit real*8(a-h,o-z)
    dimension amelh(20,2),ttf(20,2),tfm(20),caf(20)
c -----------------------------------------------------------
    summcaf=0.
    amelh(0,2)=0.
```

```
      ttf(0,1)=0.
      do 100 k=1,jph+1
       if(k.eq.1)then
        pf=bfvw
       else
        pf=sbc
       end if
       tsfo=q*pf/2.
       tfm(k)=(amelh(k,1)-amelh(k-1,2))/pf
       ttf(k-1,2)=tsfo+tfm(k)
       caf(k-1)=(-1.)*ttf(k-1,1)+ttf(k-1,2)
       ttf(k,1)=(-1.)*tsfo+tfm(k)
       if(k.eq.jph+1)then
        ttf(jph+1,2)=q*tfcw
        caf(jph+1)=(-1.)*ttf(jph+1,1)+ttf(jph+1,2)
       end if
      summcaf=summcaf+caf(k-1)
      print*,'caf(',k-1,')=',caf(k-1)
  100 continue
      summcaf=summcaf+caf(jph+1)
      sumq=q*vwh
      end
C
c
c
C ***************************************************************
C This is Subroutine BACKUP_M
C ***************************************************************
C Load:Concentrated Forces CAF(i);Conc. Moments BMOM(i).
c ---------------------------------------------------------------
c bwh  - backup wythe height
c bsr  - bottom reaction
c tsr  - top reaction
c sbbw        - spacing between two blocks
c hinges: ii=1...jph+1
c field points: ii=1..njbf
c caf(i)- axial force in the connector(ii=1...jph+1)
c bmom(i)- secondary moments
C -------------------------------------------------------
      SUBROUTINE BACKUP_M (caf,bwh,tfcw,jph,sbc,bfbw,sumq,bmom,
     *              sbbw,jbbf,jsbf,jtbf,ntotal,cmh,cmf)
      implicit real*8(a-h,o-z)
      dimension caf(20),bmom(20),tfh(20),cmh(20,2),cmf(100)
```

```
c ------------------------------------------------------------
    sbbw=200.
    jbbf=bfbw/sbbw-1
    jsbf=sbc/sbbw-1
    jtbf=tfcw/sbbw-1
    njbf=jbbf+jh*jsbf+jtbf
    ntotal=jph+1+njbf
c ------------------------------------------------------------
c find reactions/load:caf+amst/
c ------------------------------------------------------------
    amcaf=0.
    sumcaf=0.
    do 100 ii=1,jph+1
     sumcaf=sumcaf+caf(ii)
c ------------------------------------------------------------
c bmom(ii) counterclokwise positive
c ------------------------------------------------------------
     amcaf=amcaf+caf(ii)*((jph+1-ii)*sbc+tfcw)+bmom(ii)
 100 continue
    bsr=amcaf/bwh
    tsr=sumcaf-bsr
    sumreact=bsr+tsr+caf(0)
C -----------------------------------------------------
c find moment distribution at each point
c tfh(ii) - transversal force at hinge
c cmh(ii,j) - moment at hinge/left & right/
C -----------------------------------------------------
    tfh(1)=bsr
    cmh(0,2)=0.
    do 200 ii=1,jph+1
     tfh(ii+1)=tfh(ii)-caf(ii)
     if(ii.eq.1)rf=bfbw
     if(ii.ne.1)rf=sbc
     do 300 j=1,2
     if(j.eq.1)then
      ampt=0.0
     else
      ampt=bmom(ii)
     end if
     cmh(ii,j)=cmh(ii-1,2)+tfh(ii)*rf-ampt
     print*,'Backup Moment(',ii,',',j,')=',cmh(ii,j)
 300  continue
 200 continue
```

```
    cmh(jph+2,1)=cmh(jph+1,2)+tfh(jph+2)*tfcw
    k=1
    jj=jbbf
    do 400 ii=1,jbbf
     cmf(ii)=cmh(k,1)/bfbw*(ii)*sbbw
 400 continue
    do 500 k=2,jph+1
     do 600 ii=jj+1,jj+jsbf
      cmf(ii)=(cmh(k,1)-cmh(k-1,2))/sbc*(ii-jj)*sbbw+cmh(k-1,2)
 600  continue
     jj=jj+jsbf
 500 continue
    end
C
C
c
C ****************************************************************
C This is Subroutine  SLO_DEFL
c ****************************************************************
c this program calculates slopes & deflections of a simple beam
C ---------------------------------------------------------------
    SUBROUTINE SLO_DEFL (ntotal,bwh,sbbw,jbbf,jsbf,jtbf,
   *          cmh,cmf,est,caf,icirc,
   *          bmcr,bficr,bm,bfiy,bmu,bfiu,
   *          sfmax,rotmax,cw,tiepl,vs,bs,xarea,
   *          vmom,bmom,defll,ifail,idefl)
    implicit real*8(a-h,o-z)
    dimension cmh(20,2),cmf(100),vmom(20),bmom(20),slo(100),
   *       cmr(100),cef(100),fib(100),rez(100),tef(100),
   *       caf(20),shearf(20),abscmr(100),
   *       ssbw(20),sdef(100),deltal(20),defll(20),slopeh(20)
c ---------------------------------------------------------------
    nh=0
    jj=jbbf
    do 100 ii=1,ntotal
     if(ii.eq.jj+1)then
      cmr(ii)=(cmh(nh+1,1)+cmh(nh+1,2))/2.
      jj=jj+jsbf+1
      nh=nh+1
     else
      cmr(ii)=cmf(ii-nh)
     end if
     abscmr(ii)=abs(cmr(ii))
```

```
      if(idefl.eq.0)then
       fib(ii)=(abscmr(ii)/bmcr)*bficr
      else if(abscmr(ii).le.bmcr)then
       fib(ii)=(abscmr(ii)/bmcr)*bficr
      else if((abscmr(ii).ge.bmcr).and.(abscmr(ii).lt.bmy))then
       xmycr=abscmr(ii)-bmcr
       ficry=xmycr*((bfiy-bficr)/(bmy-bmcr))
       fib(ii)=bficr+ficry
      else if(abscmr(ii).ge.bmy)then
       xmuy=abscmr(ii)-bmy
       fiyu=xmuy*((bfiu-bfiy)/(bmu-bmy))
       fib(ii)=bfiy+fiyu
      end if
      if(abscmr(ii).ge.bmcr.and.abscmr(ii).lt.(bmcr+0.01*bmy))then
       print*,'Crack occurs at joint',ii,' Moment=Mcracking'
      else if(abscmr(ii).gt.(bmcr+0.01*bmy).and.abscmr(ii).lt.bmy)then
       print*,'Cracked section at joint(',ii,'),Fsteel<Fy'
      else if(abscmr(ii).ge.bmy.and.abscmr(ii).lt.bmu)then
       print*,'Steel yielding at joint',ii,'Mcr<Moment<Mult.'
      else if(abscmr(ii).ge.bmu)then
       print*,'Failure at joint',ii,' Moment>Mult.'
      end if
      if(cmr(ii).lt.0)then
       fib(ii)=(-1.)*fib(ii)
      end if
      print*,'Curvature(',ii,')=',fib(ii)
  100 continue
c ----------------------------------------------------------------
c Calculate cef(ii) - concentrated equivalent force  ii=0...total+1
c ----------------------------------------------------------------
      print*,'    '
      fib(0)=0.
      fibntotal+1)=0.
      rez(0)=0.
      cef(0)=sbbw/6.*fib(1)
      do 200 ii=1,ntotal
       cef(ii)=sbbw/6.*(fib(ii-1)+4.*fib(ii)+fib(ii+1))
       rez(ii)=((fib(ii-1)+fib(ii))/2.)*sbbw
  200 continue
c ----------------------------------------------------------------
c ii=ntotal+1
c ----------------------------------------------------------------
      cef(ntotal+1)=sbbw/6.*fib(ntotal)
```

```
      rez(ntotal+1)=(fib(ntotal)/2.)*sbbw
      amcef=0.
      sumcef=0.
      do 300 ii=0,ntotal+1
       sumcef=sumcef+cef(ii)
       amcef=amcef+cef(ii)*((ntotal+1-ii)*sbbw)
  300 continue
      slbr=amcef/bwh
      sltr=sumcef-slbr
      print*,'SumCEF=',sumcef
      print*,'slbr',slbr
      print*,'sltr',sltr
      print*,'   '
      print*,'SLOPES'
C -----------------------------------------------------------------
c SLOPES
C -----------------------------------------------------------------
      slo(0)=slbr
      print*,'Slope(  0)=',slbr
      nh=0
      jj=jbbf
      do 400 ii=1,ntotal+1
       slo(ii)=slo(ii-1)-rez(ii)
       if(ii.eq.jj+1)then
        print*,'Joint No.',nh+1
        jj=jj+jsbf+1
        slopeh(nh+1)=slo(ii)
        print*,'Slope  (',nh+1,')=',slopeh(nh+1)
        shearf(nh+1)=(sfmax/rotmax)*slo(ii)
        if(shearf(nh+1).gt.sfmax)shearf(nh+1)=sfmax
        if(shearf(nh+1).lt.(-1.)*sfmax)shearf(nh+1)=(-1.)*sfmax
        vmom(nh+1)=shearf(nh+1)*(tiepl+0.5*vs)
        bmom(nh+1)=shearf(nh+1)*(cw-tiepl+0.5*bs)
        nh=nh+1
       else
        print*,'Slope/field/(',ii,')=',slo(ii)
       end if
  400 continue
      print*,'slope(',ntotal+1,')=',slo(ntotal+1)
c -----------------------------------------------------------------
c deflections/first:transversal force based on conc.eq.forces/
c tef - tr. eq. force
c -----------------------------------------------------------------
```

```
      tef(1)=slbr-cef(0)
      do 500 ii=2,ntotal+1
       tef(ii)=tef(ii-1)-cef(ii-1)
  500 continue
      ch=sltr+slo(ntotal+1)
C -----------------------------------------------------
C DEFLETION
C -----------------------------------------------------
      print*,'     '
      print*,'DEFLECTIONS'
      nh=0
      jj=jbbf
      sdef(0)=0.
      do 600 ii=1,ntotal+1
       sdef(ii)=sdef(ii-1)+tef(ii)*sbbw
       if(ii.eq.jj+1)then
        print*,'Joint No.',nh+1
        jj=jj+jsbf+1
        ssbw(nh+1)=sdef(ii)
        deltal(nh+1)=(caf(nh+1)/est)*xarea
        defll(nh+1)=ssbw(nh+1)+deltal(nh+1)
        print*,'Deflection,backup (',nh+1,')=',ssbw(nh+1)
        print*,'Deflection,veneer (',nh+1,')=',defll(nh+1)
        print*,' '
        nh=nh+1
       else
        if(sdef(ntotal+1).lt.0.000000001)sdef(ntotal+1)=0.0
        print*,'Defl./field/(',ii,')=',sdef(ii)
       end if
  600 continue
      end
```

APPENDIX: B

Example /*Fonlsa*/

Wall Height = 3000.00
Connector Space = 800.00

INPUT FOR THE VENEER WALL
Size(width) of unit = 90.00 [mm]
Solid precentage of unit = 50
Type of mortar = S
Number of grouted cores = 3
Compressive masonry strength] = 15.10 [MPa]
Flexural tensile masonry strength = 0.9 [Mpa]
Reinforcment 250.00 [mm^2]

INPUT FOR THE BACKUP WALL
Size(width) of unit = 140.00 [mm]
Solid precentage of unit = 50
Type of mortar = S
Number of grouted cores = 3
Compressive masonry strength] = 15.10 [MPa]
Flexural tensile masonry strength = 0.9 [Mpa]
Reinforcment 250.00 [mm^2]

Moment-Curvature for Veneer Wall

Mel = 965420.54[Nmm]	Flel = 0.0000012466
Mcr = 1162501.20[Nmm]	Flcr = 0.0000015416
Tension failure	
My = 3964195.04[Nmm]	Fly = 0.0000699333
Mult= 4180382.01[Nmm]	Flult= 0.0003302000

Moment-Curvature for Backup Wall

Mel = 2178348.17[Nmm]	Flel = 0.0000008014
Mcr = 2607826.46[Nmm]	Flcr = 0.0000009882
Tension failure	
My = 6333847.29[Nmm]	Fly = 0.0000407714
Mult= 6705545.41[Nmm]	Flult= 0.0003302000

Shear_Force-Rotation (S-TETA) Relationship

Cavity width [mm]	100.00
V-Tie protrusion length [mm]	15.00
V-Tie/moment of inertia/ [mm4]	85.50
Steel Plate/moment of inertia/ [mm4]	45114.00
Veneer Wall width [mm]	90.00
Backup Wall width [mm]	140.00

R E S U L T S:

Moment of Plasticity [Nmm]	14369.75
Maximum Shear-Force [N]	958.15
Moment at Veneer Wall [Nmm]	-57488.94
Moment at Backup Wall [Nmm]	-148513.09
Rotation at the ends	-0.000394

WIND LOAD q= 3.0000000000000

ITERATION No.= 1

Delta/ss/-support settlments
tdelss(1)= 0.
tdelss(2)= 0.
tdelss(3)= 0.
Delta/sm/-secondary moments
delms(1)= 0.
delms(2)= 0.
delms(3)= 0.
Total constant vector elastic-tcvel(i)
tcvel(1)= -91000000.000000
tcvel(2)= -128000000.00000
tcvel(3)= -120000000.00000

Redundants/moments-positive,

```
          bottom side in tension/
Joint No: 1 Moment-Xel=  -150463.91752577
Joint No: 2 Moment-Xel=  -155876.28865979
Joint No: 3 Moment-Xel=  -186030.92783505

Veneer Moment( 1, 1)=  -150463.91752577
Veneer Moment( 1, 2)=  -150463.91752577
Veneer Moment( 2, 1)=  -155876.28865979
Veneer Moment( 2, 2)=  -155876.28865979
Veneer Moment( 3, 1)=  -186030.92783505
Veneer Moment( 3, 2)=  -186030.92783505
Veneer Moment( 4, 1)=  -60000.000000000
Veneer Moment( 4, 2)=  -60000.000000000

F I N D  R E A C T I O N S
caf( 0)=    649.2268044123
caf( 1)=   2344.0077319588
caf( 2)=   2369.0721649485
caf( 3)=   2595.2319587629
caf( 4)=   1642.4613402062


Backup Moment( 1, 1)=   1770996.5635739
Backup Moment( 1, 2)=   1770996.5635739
Backup Moment( 2, 1)=   3437783.5051546
Backup Moment( 2, 2)=   3437783.5051546
Backup Moment( 3, 1)=   3209312.7147766
Backup Moment( 3, 2)=   3209312.7147766
Backup Moment( 4, 1)=   904656.35738832
Backup Moment( 4, 2)=   904656.35738832

FIND FI /CURVATURE/FROM M-FI
Moment   ( 1)=  885498.28178694
Curvature( 1)=  3.3553170619829D-07
Moment   ( 2)=  1770996.5635739
Curvature( 2)=  6.7106341239658D-07
Moment   ( 3)=  2187693.2989691
Curvature( 3)=  8.2895752633236D-07
Moment   ( 4)=  2604390.0343643
Curvature( 4)=  9.8685164026814D-07
Moment   ( 5)=  3021086.7697595
Cracked section at joint( 5),Fsteel<Fy
Curvature( 5)=  1.1447457542039D-06
```

```
Moment  ( 6)=  3437783.5051546
Cracked section at joint( 6),Fsteel<Fy
Curvature( 6)=  1.3026398681397D-06
Moment  ( 7)=  3380665.8075601
Cracked section at joint( 7),Fsteel<Fy
Curvature( 7)=  1.2809969141982D-06
Moment  ( 8)=  3323548.1099656
Cracked section at joint( 8),Fsteel<Fy
Curvature( 8)=  1.2593539602567D-06
Moment  ( 9)=  3266430.4123711
Cracked section at joint( 9),Fsteel<Fy
Curvature( 9)=  1.2377110063152D-06
Moment  ( 10)=  3209312.7147766
Cracked section at joint( 10),Fsteel<Fy
Curvature( 10)=  1.2160680523737D-06
Moment  ( 11)=  2633148.6254296
Crack occurs at joint 11 Moment=Mcracking
Curvature( 11)=  9.9774880328525D-07
Moment  ( 12)=  2056984.5360825
Curvature( 12)=  7.7942955419683D-07
Moment  ( 13)=  1480820.4467354
Curvature( 13)=  5.6111030510841D-07
Moment  ( 14)=  904656.35738832
Curvature( 14)=  3.4279105601999D-07


DEFLECTIONS
Defl./field/( 1)=  0.25845564953008
Joint No. 1
Deltal ( 1)=  1.3253823061868D-02
Deflection,backup ( 1)=  0.50349003081223
Deflection,veneer ( 1)=  0.51674385387410

Defl./field/( 3)=  0.72286612621360
Defl./field/( 4)=  0.90908392056167
Defl./field/( 5)=  1.0558276492990
Joint No. 2
Deltal ( 2)=  1.3395545956149D-02
Deflection,backup ( 2)=  1.1567815478682
Deflection,veneer ( 2)=  1.1701770938244

Defl./field/( 7)=  1.2068267654977
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Defl./field/(8)= 1.2056321065592
Defl./field/(9)= 1.1540632892105
Joint No. 3
Deltal (3)= 1.4674330940540D-02
Deflection,backup (3)= 1.0529860316091
Deflection,veneer (3)= 1.0676603625496

Defl./field/(11)= 0.90457722721379
Defl./field/(12)= 0.71625847068706
Defl./field/(13)= 0.49676253199246
Joint No. 4
Deltal (4)= 9.2870393268114D-03
Deflection,backup (4)= 0.25482218109352
Deflection,veneer (4)= 0.26410922042034

Defl./field/(15)= 0.

ITERATION No.= 2

Delta/ss/-support settlments
tdelss(1)= -33518431.708189
tdelss(2)= -712578174.39956
tdelss(3)= -660813333.80904
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -29660939.151402
delms(3)= -30160853.263754
Total constant vector elastic-tcvel(i)
tcvel(1)= -42151184.603758
tcvel(2)= 614239113.55096
tcvel(3)= 570974187.07279

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -356397.70898199
Joint No: 2 Moment-Xel= 931258.09690877
Joint No: 3 Moment-Xel= 1147834.7059711
Veneer Moment(1, 1)= -356397.70898199
Veneer Moment(1, 2)= -413886.64781219
Veneer Moment(2, 1)= 931258.09690877
Veneer Momet(2, 2)= 877518.51392121
Veneer Moment(3, 1)= 1090345.7671409

Veneer Moment(3, 2)= 1147834.7059711
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment(4, 2)= -60000.000000000

FIND REACTIONS
caf(0)= 306.0038184521
caf(1)= 4375.4271125378
caf(2)= 984.60313562338
caf(3)= 552.31137747382
caf(4)= 3381.6545560016

Backup Moment(1, 1)= 1915647.0119290
Backup Moment(1, 2)= 1767133.9199510
Backup Moment(2, 1)= 2098086.2537788
Backup Moment(2, 2)= 1959258.9977276
Backup Moment(3, 1)= 1502528.8230567
Backup Moment(3, 2)= 1651041.9150348
Backup Moment(4, 1)= 752462.63838480
Backup Moment(4, 2)= 900975.73036281

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 957823.50596452
Curvature(1)= 3.6293707373950D-07
Moment (2)= 1841390.4659400
Curvature(2)= 6.9773696631834D-07
Moment (3)= 1849872.0034080
Curvature(3)= 7.0095077801774D-07
Moment (4)= 1932610.0868649
Curvature(4)= 7.3230177087779D-07
Moment (5)= 2015348.1703219
Curvature(5)= 7.6365276373784D-07
Moment (6)= 2028672.6257532
Curvature(6)= 7.670164678708D-07
Moment (7)= 1845076.4540599
Curvature(7)= 6.9913365551394D-07
Moment (8)= 1730893.9103922
Curvature(8)= 6.5586777405160D-07
Moment (9)= 1616711.3667245
Curvature(9)= 6.1260189258927D-07
Moment (10)= 1576785.3690457
Curvature(10)= 5.9747319228759D-07

Moment (11)= 1426397.0958723
Curvature(11)= 5.4048828906645D-07
Moment (12)= 1201752.2767098
Curvature(12)= 4.5536620468466D-07
Moment (13)= 977107.45754728
Curvature(13)= 3.7024412030288D-07
Moment (14)= 826719.18437380
Curvature(14)= 3.1325921708174D-07

DEFLECTIONS
Defl./field/(1)= 0.17518626237847
Joint No. 1
Deltal (1)= 2.4740164453816D-02
Deflection,backup (1)= 0.33604262301510
Deflection,veneer (1)= 0.36078278746892

Defl./field/(3)= 0.47120007887153
Defl./field/(4)= 0.57813192239951
Defl./field/(5)= 0.65577169509237
Joint No. 2
Deltal (2)= 5.5672835749597D-03
Deflection,backup (2)= 0.70304070463446
Deflection,veneer (2)= 0.70860798820942

Defl./field/(7)= 0.72005909413389
Defl./field/(8)= 0.70893679001402
Defl./field/(9)= 0.67157977493208
Joint No. 3
Deltal (3)= 3.1229578180518D-03
Deflection,backup (3)= 0.60953110293884
Deflection,veneer (3)= 0.61265406075689

Defl./field/(11)= 0.52386254460689
Defl./field/(12)= 0.41676203592001
Defl./field/(13)= 0.29144687904575
Joint No. 4
Deltal (4)= 1.9121033830443D-02
Deflection,backup (4)= 0.15113437615164
Deflection,veneer (4)= 0.17025540998209

Defl./field/(15)= 0.

ITERATION No.= 3

Delta/ss/-support settlments
tdelss(1)= -125575250.29272
tdelss(2)= -418317789.59128
tdelss(3)= -326567838.97392
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -20839357.358224
delms(3)= -25408682.511805
Total constant vector elastic-tcvel(i)
tcvel(1)= 49905633.980770
tcvel(2)= 311157146.94950
tcvel(3)= 231976521.48573

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -36700.436418494
Joint No: 2 Moment-Xel= 502743.78232050
Joint No: 3 Moment-Xel= 466344.55350223
Veneer Moment(1, 1)= -36700.436418494
Veneer Moment(1, 2)= -94189.375248692
Veneer Moment(2, 1)= 502743.78232050
Veneer Moment(2, 2)= 481231.68141896
Veneer Moment(3, 1)= 410562.51394883
Veneer Moment(3, 2)= 466344.55350223
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment(4, 2)= -60000.000000000

F I N D R E A C T I O N S
caf(0)= 838.8326057265
caf(1)= 2907.3338409923
caf(2)= 1565.4970937008
caf(3)= 1758.5445939221
caf(4)= 2529.7918654155

Backup Moment(1, 1)= 1673724.6729862
Backup Moment(1, 2)= 1525211.5810081
Backup Moment(2, 1)= 2546793.8541866
Backup Moment(2, 2)= 2491220.9268576
Backup Moment(3, 1)= 2260405.5250754
Backup Moment(3, 2)= 2404509.1272550

Backup Moment(4, 1)= 766858.05033508
Backup Moment(4, 2)= 915371.14231309

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 836862.33649308
Curvature(1)= 3.1710264536027D-07
Moment (2)= 1599468.1269971
Curvature(2)= 6.0606810955988D-07
Moment (3)= 1780607.1493028
Curvature(3)= 6.7470504140196D-07
Moment (4)= 2036002.7175974
Curvature(4)= 7.7147915440469D-07
Moment (5)= 2291398.2858920
Curvature(5)= 8.6825326740743D-07
Moment (6)= 2519007.3905221
Curvature(6)= 9.5449857447758D-07
Moment (7)= 2433517.0764121
Curvature(7)= 9.2210471042752D-07
Moment (8)= 2375813.2259665
Curvature(8)= 9.0023965231003D-07
Moment (9)= 2318109.3755209
Curvature(9)= 8.7837459419254D-07
Moment (10)= 2332457.3261652
Curvature(10)= 8.8381129854209D-07
Moment (11)= 1995096.3580250
Curvature(11)= 7.5597897681658D-07
Moment (12)= 1585683.5887950
Curvature(12)= 6.0084489262402D-07
Moment (13)= 1176270.8195651
Curvature(13)= 4.4571080843146D-07
Moment (14)= 841114.59632408
Curvature(14)= 3.1871390539955D-07

DEFLECTIONS
Defl./field/(1)= 0.19942859280114
Joint No. 1
Deltal (1)= 1.6439061947160D-02
Deflection,backup (1)= 0.38636066099560
Deflection,veneer (1)= 0.40279972294276

Defl./field/(3)= 0.55051886169005
Defl./field/(4)= 0.68750127952069

Defl./field/(5)= 0.79362453117513
Joint No. 2
Deltal (2)= 8.8518570996523D-03
Deflection,backup (2)= 0.86508784417283
Deflection,veneer (2)= 0.87393970127249

Defl./field/(7)= 0.89916214199890
Defl./field/(8)= 0.89628205936832
Defl./field/(9)= 0.85739239064533
Joint No. 3
Deltal (3)= 9.9434138277228D-03
Deflection,backup (3)= 0.78318572640486
Deflection,veneer (3)= 0.79312914023258

Defl./field/(11)= 0.67451507039654
Defl./field/(12)= 0.53578726706534
Defl./field/(13)= 0.37302566802918
Joint No. 4
Deltal (4)= 1.4304310224929D-02
Deflection,backup (4)= 0.19224805544802
Deflection,veneer (4)= 0.20655236567295

Defl./field/(15)= 0.

ITERATION No.= 4

Delta/ss/-support settlments
tdelss(1)= -62143869.004933
tdelss(2)= -520282985.21597
tdelss(3)= -476748434.18389
Delta/sm/-secondary moments
delms(1)= -15330383.68853
delms(2)= -25212569.965728
delms(3)= -27936668.670917
Total constant vector elastic-tcvel(i)
tcvel(1)= -13525747.307014
tcvel(2)= 417495555.18170
tcvel(3)= 384685102.85480

Redundants/moments-positive,
bottom side in tension/

Joint No: 1 Moment-Xel= -210359.17748705
Joint No: 2 Moment-Xel= 634814.01640206
Joint No: 3 Moment-Xel= 773336.03151223
Veneer Moment(1, 1)= -210359.17748705
Veneer Moment(1, 2)= -267848.11631725
Veneer Moment(2, 1)= 634814.01640206
Veneer Moment(2, 2)= 597755.81786078
Veneer Moment(3, 1)= 715847.09268203
Veneer Moment(3, 2)= 773336.03151223
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment(4, 2)= -60000.000000000

FIND REACTIONS
caf(0)= 549.4013709222
caf(1)= 3578.9262950442
caf(2)= 1419.2864276274
caf(3)= 1138.8546935454
caf(4)= 2913.5312129280

Backup Moment(1, 1)= 1803827.3947416
Backup Moment(1, 2)= 1655314.3027636
Backup Moment(2, 1)= 2399828.0562115
Backup Moment(2, 2)= 2304094.3766466
Backup Moment(3, 1)= 1913178.9879925
Backup Moment(3, 2)= 2061692.0799705
Backup Moment(4, 1)= 759692.93648018
Backup Moment(4, 2)= 908206.02845820

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 901913.69737082
Curvature(1)= 3.4175181132113D-07
Moment (2)= 1729570.8487526
Curvature(2)= 6.5536644148162D-07
Moment (3)= 1841442.7411256
Curvature(3)= 6.9775677435475D-07
Moment (4)= 2027571.1794876
Curvature(4)= 7.6828428838853D-07
Moment (5)= 2213699.6178496
Curvature(5)= 8.3881180242231D-07
Moment (6)= 2351961.2164290
Curvature(6)= 8.9120168394694D-07
Moment (7)= 2206365.5294831

Curvature(7)= 8.3603278044832D-07
Moment (8)= 2108636.6823195
Curvature(8)= 7.9900150945884D-07
Moment (9)= 2010907.8351560
Curvature(9)= 7.6197023846937D-07
Moment (10)= 1987435.5339815
Curvature(10)= 7.5307614864055D-07
Moment (11)= 1736192.2940979
Curvature(11)= 6.5787542981046D-07
Moment (12)= 1410692.5082254
Curvature(12)= 5.3453752981972D-07
Moment (13)= 1085192.7223528
Curvature(13)= 4.1119962982899D-07
Moment (14)= 833949.48246919
Curvature(14)= 3.1599891099890D-07

DEFLECTIONS
Defl./field/(1)= 0.19113343647175
Joint No. 1
Deltal (1)= 2.0236475852553D-02
Deflection,backup (1)= 0.36878438169839
Deflection,veneer (1)= 0.38902085755094

Defl./field/(3)= 0.52202883124768
Defl./field/(4)= 0.64717542861505
Defl./field/(5)= 0.74159065444687
Joint No. 2
Deltal (2)= 8.0251318839145D-03
Deflection,backup (2)= 0.80257432573186
Deflection,veneer (2)= 0.81059945761578

Defl./field/(7)= 0.82862698822580
Defl./field/(8)= 0.82111742195174
Defl./field/(9)= 0.78164779529933
Joint No. 3
Deltal (3)= 6.4394747490081D-03
Deflection,backup (3)= 0.71151177790040
Deflection,veneer (3)= 0.71795125264941

Defl./field/(11)= 0.61182809208253
Defl./field/(12)= 0.48601697027998
Defl./field/(13)= 0.33882434728464

Joint No. 4
Deltal (4)= 1.6474104012067D-02
Deflection,backup (4)= 0.17499615788840
Deflection,veneer (4)= 0.19147026190047

Defl./field/(15)= 0.

ITERATION No.= 5

Delta/ss/-support settlments
tdelss(1)= -91543960.231138
tdelss(2)= -484723608.57779
tdelss(3)= -408942107.61947
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -23345604.014557
delms(3)= -27003185.695332
Total constant vector elastic-tcvel(i)
tcvel(1)= 15874343.919190
tcvel(2)= 380069212.59234
tcvel(3)= 315945293.31480

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -136396.72873233
Joint No: 2 Moment-Xel= 596446.12995710
Joint No: 3 Moment-Xel= 635147.75460192
Veneer Moment(1, 1)= -136396.72873233
Veneer Moment(1, 2)= -193885.66756253
Veneer Moment(2, 1)= 596446.12995710
Veneer Moment(2, 2)= 566389.05373271
Veneer Moment(3, 1)= 577658.81577173
Veneer Moment(3, 2)= 635147.75460192
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment 4, 2)= -60000.000000000

FIND REACTIONS
caf(0)= 676.6721255462
caf(1)= 3315.2426281201
caf(2)= 1426.1724556492
caf(3)= 1445.1169306611
caf(4)= 2740.7958667902

Backup Moment(1, 1)= 1747886.8949647
Backup Moment(1, 2)= 1599373.8029867
Backup Moment(2, 1)= 2442953.4904199
Backup Moment(2, 2)= 2365306.0435069
Backup Moment(3, 1)= 2067947.7664208
Backup Moment(3, 2)= 2216460.8583988
Backup Moment(4, 1)= 763009.03678377
Backup Moment(4, 2)= 911522.12876178

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 873943.44748233
Curvature(1)= 3.3115336538295D-07
Moment (2)= 1673630.3489757
Curvature(2)= 6.3416954960524D-07
Moment (3)= 1810268.7248450
Curvature(3)= 6.8594436196860D-07
Moment (4)= 2021163.6467033
Curvature(4)= 7.6585635549260D-07
Moment (5)= 2232058.5685616
Curvature(5)= 8.4576834901661D-07
Moment (6)= 2404129.7669634
Curvature(6)= 9.1096931436555D-07
Moment (7)= 2290966.4742354
Curvature(7)= 8.6808964596978D-07
Moment (8)= 2216626.9049638
Curvature(8)= 8.3992100574905D-07
Moment (9)= 2142287.3356923
Curvature(9)= 8.1175236552833D-07
Moment (10)= 2142204.3124098
Curvature(10)= 8.1172090646826D-07
Moment (11)= 1853097.9029950
Curvature(11)= 7.0217313114335D-07
Moment (12)= 1489734.9475913
Curvature(12)= 5.6448817465779D-07
Moment (13)= 1126371.9921875
Curvature(13)= 4.2680321817223D-07
Moment (14)= 837265.58277277
Curvature(14)= 3.1725544284732D-07

DEFLECTIONS

Defl./field/(1)= 0.19410914475072
Joint No. 1
Deltal (1)= 1.8745518029306D-02
Deflection,backup (1)= 0.37515973609387
Deflection,veneer (1)= 0.39390525412317
Defl./field/(3)= 0.53251848793186
Defl./field/(4)= 0.66225188408337
Defl./field/(5)= 0.76135102601518
Joint No. 2
Deltal (2)= 8.0640678463500D-03
Deflection,backup (2)= 0.82671750750749
Deflection,veneer (2)= 0.83478157535384

Defl./field/(7)= 0.85636575398348
Defl./field/(8)= 0.85119234109950
Defl./field/(9)= 0.81242208798557
Joint No. 3
Deltal (3)= 8.1711864007743D-03
Deflection,backup (3)= 0.74099415904276
Deflection,veneer (3)= 0.7491653454454

Defl./field/(11)= 0.63782750261633
Defl./field/(12)= 0.50676150215190
Defl./field/(13)= 0.35311597470115
Joint No. 4
Deltal (4)= 1.5497399164626D-02
Deflection,backup (4)= 0.18221073731578
Deflection,veneer (4)= 0.19770813648041

Defl./field/(15)= 0.

ITERATION No.= 6

Delta/ss/-support settlments
tdelss(1)= -79492304.157898
tdelss(2)= -496285621.01676
tdelss(3)= -439113866.06188
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -24151104.505974
delms(3)= -27405935.941040
Total constant vector elastic-tcvel(i)

tcvel(1)= 3822687.8459514
tcvel(2)= 392436725.52274
tcvel(3)= 346519802.00292

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -164628.79283838
Joint No: 2 Moment-Xel= 604870.93377897
Joint No: 3 Moment-Xel= 696611.97309865
Veneer Moment(1, 1)= -164628.79283838
Veneer Moment(1, 2)= -222117.73166858
Veneer Moment(2, 1)= 604870.93377897
Veneer Moment(2, 2)= 571793.23071176
Veneer Moment(3, 1)= 639123.03426845
Veneer Moment(3, 2)= 696611.97309865
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment(4, 2)= -60000.000000000

FIND REACTIONS
caf(0)= 625.6186787515
caf(1)= 3408.1171532067
caf(2)= 1450.4264226364
caf(3)= 1298.2116056431
caf(4)= 2817.6261399111

Backup Moment(1, 1)= 1769406.2178205
Backup Moment(1, 2)= 1620893.1258425
Backup Moment(2, 1)= 2433211.8389182
Backup Moment(2, 2)= 2347761.1059946
Backup Moment(3, 1)= 1999738.6809611
Backup Moment(3, 2)= 2148251.7729391
Backup Moment(4, 1)= 761660.06339118
Backup Moment(4, 2)= 910173.15536920

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 884703.10891026
Curvature(1)= 3.3523040046171D-07
Moment (2)= 1695149.6718315
Curvature(2)= 6.4232361976277D-07
Moment (3)= 1823972.8041114
Curvature(3)= 6.9113709152294D-07

Moment (4)= 2027052.4823804
Curvature(4)= 7.6808774444376D-07
Moment (5)= 2230132.1606493
Curvature(5)= 8.4503839736458D-07
Moment (6)= 2390486.4724564
Curvature(6)= 9.0579961728284D-07
Moment (7)= 2260755.4997362
Curvature(7)= 8.5664214795865D-07
Moment (8)= 2173749.8934778
Curvature(8)= 8.2367411163703D-07
Moment (9)= 2086744.2872195
Curvature(9)= 7.9070607531540D-07
Moment (10)= 2073995.2269501
Curvature(10)= 7.8587522015443D-07
Moment (11)= 1801603.8455521
Curvature(11)= 6.8266107865464D-07
Moment (12)= 1454955.9181651
Curvature(12)= 5.5130975599421D-07
Moment (13)= 1108307.9907782
Curvature(13)= 4.1995843333378D-07
Moment (14)= 835916.60938019
Curvature(14)= 3.1674429183400D-07

DEFLECTIONS
Defl./field/(1)= 0.19302466863325
Joint No. 1
Deltal (1)= 1.9270662424382D-02
Deflection,backup (1)= 0.37282770245577
Deflection,veneer (1)= 0.39209836488015

Defl./field/(3)= 0.52865965647139
Defl./field/(4)= 0.65665854561835
Defl./field/(5)= 0.75393392498756
Joint No. 2
Deltal (2)= 8.2012080880880D-03
Deflection,backup (2)= 0.81751569801554
Deflection,veneer (2)= 0.82571690610363

Defl./field/(7)= 0.84559827761382
Defl./field/(8)= 0.83930724174040
Defl./field/(9)= 0.80006924140150
Joint No. 3

Deltal (3)= 7.3405333452882D-03
Deflection,backup (3)= 0.72901541684225
Deflection,veneer (3)= 0.73635595018754

Defl./field/(11)= 0.62718247205242
Defl./field/(12)= 0.49823066532413
Defl./field/(13)= 0.34722646835608
Joint No. 4
Deltal (4)= 1.5931823860354D-02
Deflection,backup (4)= 0.17923635284694
Deflection,veneer (4)= 0.19516817670729
Defl./field/(15)= 0.
ITERATION No.= 7

Delta/ss/-support settlments
tdelss(1)= -84062715.748249
tdelss(2)= -492974124.61861
tdelss(3)= -425903751.65027
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -23813219.222101
delms(3)= -27236993.299103
Total constant vector elastic-tcvel(i)
tcvel(1)= 8393099.4363024
tcvel(2)= 388787343.84071
cvel(3)= 333140744.94937

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -154798.56755583
Joint No: 2 Moment-Xel= 604743.23221766
Joint No: 3 Moment-Xel= 669715.93056833
Veneer Moment(1, 1)= -154798.56755583
Veneer Moment(1, 2)= -212287.50638603
Veneer Moment(2, 1)= 604743.23221766
Veneer Moment(2, 2)= 572932.59896498
Veneer Moment(3, 1)= 612226.99173813
Veneer Moment(3, 2)= 669715.93056833
Veneer Moment(4, 1)= -117488.93883020
Veneer Moment(4, 2)= -60000.000000000

FIND REACTIONS
caf(0)= 642.0023875121

caf(1)= 3379.2860358477
caf(2)= 1427.8295677118
caf(3)= 1366.8759222854
caf(4)= 2784.0060867482

Backup Moment(1, 1)= 1761810.4575971
Backup Moment(1, 2)= 1613297.3656191
Backup Moment(2, 1)= 2433489.4521352
Backup Moment(2, 2)= 2351311.9828991
Backup Moment(3, 1)= 2029240.4152457
Backup Moment(3, 2)= 2177753.5072237
Backup Moment(4, 1)= 762181.20174204
Backup Moment(4, 2)= 910694.29372006

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 880905.22879855
Curvature(1)= 3.3379131331718D-07
Moment (2)= 1687553.9116081
Curvature(2)= 6.3944544547372D-07
Moment (3)= 1818345.3872481
Curvature(3)= 6.8900475900409D-07
Moment (4)= 2023393.4088771
Curvature(4)= 7.6670125369512D-07
Moment (5)= 2228441.4305062
Curvature(5)= 8.4439774838614D-07
Moment (6)= 2392400.7175171
Curvature(6)= 9.0652496020504D-07
Moment (7)= 2270794.0909858
Curvature(7)= 8.6044595618625D-07
Moment (8)= 2190276.1990724
Curvature(8)= 8.2993623503958D-07
Moment (9)= 2109758.3071591
Curvature(9)= 7.9942651389291D-07
Moment (10)= 2103496.9612347
Curvature(10)= 7.9705397390690D-07
Moment (11)= 1823860.4308533
Curvature(11)= 6.9109451121330D-07
Moment (12)= 1469967.3544829
Curvature(12)= 5.5699786735905D-07
Moment (13)= 1116074.2781125
Curvature(13)= 4.2290122350481D-07
Moment (14)= 836437.74773105

Curvature(14)= 3.1694176081121D-07

DEFLECTIONS
Defl./field/(1)= 0.19341048005632
Joint No. 1
Deltal (1)= 1.9107641405747D-02
Deflection,backup (1)= 0.37365688878768
Delection,veneer (1)= 0.39276453019343

Defl./field/(3)= 0.53003277849094
Defl./field/(4)= 0.65866089662630
Defl./field/(5)= 0.75662096461385
Joint No. 2
Deltal (2)= 8.0734377258823D-03
Deflection,backup (2)= 0.82090891788510
Deflection,veneer (2)= 0.82898235561099

Defl./field/(7)= 0.84965724752041
Defl./field/(8)= 0.84388394368911
Defl./field/(9)= 0.80491319045624
Joint No. 3
Deltal (3)= 7.7287849244248D-03
Deflection,backup (3)= 0.73377779545991
Deflection,veneer (3)= 0.74150658038433

Defl./field/(11)= 0.63145082099202
Defl./field/(12)= 0.50166764728333
Defl./field/(13)= 0.34960455888028
Joint No. 4
Deltal (4)= 1.5741724557405D-02
Deflection,backup (4)= 0.18043784032931
Deflection,veneer (4)= 0.19617956488671

Defl./field/(15)= 0.

WIND LOAD q= 3.1000000000000

Delta/io/-uniform load
delmo(1)= 94033333.333333
delmo(2)= 132266666.66667
delmo(3)= 124000000.00000

ITERATION No.= 1

Delta/ss/-support settlments
tdelss(1)= -82449822.371080
tdelss(2)= -493647257.22137
tdelss(3)= -431582529.74092
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -23951315.907745
delms(3)= -27306041.641925
Total constant vector elastic-tcvel(i)
tcvel(1)= 3746872.7257998
tcvel(2)= 385331906.46244
tcvel(3)= 334888571.38284

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -162490.96364721
Joint No: 2 Moment-Xel= 596819.91820873
Joint No: 3 Moment-Xel= 673229.60226448
Veneer Moment(1, 1)= -162490.96364721
Veneer Moment(1, 2)= -219979.90247741
Veneer Moment(2, 1)= 596819.91820873
Veneer Moment(2, 2)= 564491.42238489
Veneer Moment(3, 1)= 615740.66343428
Veneer Moment(3, 2)= 673229.60226448
Veneer Moment(4, 1)= -11948.93883020
Veneer Moment(4, 2)= -62000.000000000

FIND REACTIONS
caf(0)= 660.8182727453
caf(1)= 3461.8180486030
caf(2)= 1523.0617754541
caf(3)= 1425.0402723199
caf(4)= 2850.8981763684

Backup Moment(1, 1)= 1822994.6958239
Backup Moment(1, 2)= 1674481.6038459
Backup Moment(2, 1)= 2551016.5566112
Backup Moment(2, 2)= 2467501.2757329

Backup Moment(3, 1)= 2125586.8081350
Backup Moment(3, 2)= 2274099.9001130
Backup Moment(4, 1)= 792153.21465912
Backup Moment(4, 2)= 940666.30663714

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 911497.34791193
Curvature(1)= 3.4538323408480D-07
Moment (2)= 1748738.1498349
Curvature(2)= 6.6262928700895D-07
Moment (3)= 1893615.3420372
Curvature(3)= 7.1752593953635D-07
Moment (4)= 2112749.0802285
Curvature(4)= 8.0055977322439D-07
Moment (5)= 2331882.8184198
Curvature(5)= 8.8359360691244D-07
Moment (6)= 2509258.9161720
Curvature(6)= 9.5080469691872D-07
Moment (7)= 2382022.6588334
Curvature(7)= 9.0259252147671D-07
Moment (8)= 2296544.0419339
Curvature(8)= 8.7020308971646D-07
Moment (9)= 2211065.4250345
Curvatre(9)= 8.3781365795621D-07
Moment (10)= 2199843.3541240
Curvature(10)= 8.3356140735662D-07
Moment (11)= 1903613.2287495
Curvature(11)= 7.2131432406059D-07
Moment (12)= 1533126.5573861
Curvature(12)= 5.8093005960392D-07
Moment (13)= 1162639.8860226
Curvature(13)= 4.4054579514724D-07
Moment (14)= 866409.76064813
Curvature(14)= 3.2829871185122D-07

DEFLECTIONS
Defl./field/(1)= 0.20197339628250
Joint No. 1
Deltal (1)= 1.9574305691485D-02
Deflection,backup (1)= 0.39031904440935
Deflection,veneer (1)= 0.40989335010083

Defl./field/(3)= 0.55390851705848
Defl./field/(4)= 0.68860937091842
Defl./field/(5)= 0.79128783384939
Joint No. 2
Deltal (2)= 8.6119132667254D-03
Deflection,backup (2)= 0.85872803746174
Deflection,veneer (2)= 0.86733995072846

Defl./field/(7)= 0.88890554163366
Defl./field/(8)= 0.88287385998863
Defl./field/(9)= 0.84203405475495
Joint No. 3
Deltal (3)= 8.0576660937807D-03
Deflection,backup (3)= 0.76749412199528
Deflection,veneer (3)= 0.77555178808906

Defl./field/(11)= 0.6033169849265
Defl./field/(12)= 0.52450428323534
Defl./field/(13)= 0.36543966559387
Joint No. 4
Deltal (4)= 1.6119955357576D-02
Deflection,backup (4)= 0.18856563493878
Deflection,veneer (4)= 0.20468559029635

Defl./field/(15)= 0.

ITERATION No.= 2

Delta/ss/-support settlments
tdelss(1)= -83967121.732206
tdelss(2)= -517723023.77550
tdelss(3)= -451591460.65086
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -24383519.325732
delms(3)= -27522143.350919
Total constant vector elastic-tcvel(i)
tcvel(1)= 5264172.0869259
tcvel(2)= 409839876.43456
tcvel(3)= 355113604.00178

Redundants/moments-positive,

bottom side in tension/

Joint No: 1 Moment-Xel= -170251.23895507
Joint No: 2 Moment-Xel= 635360.62699467
Joint No: 3 Moment-Xel= 713888.17299327
Veneer Moment(1, 1)= -170251.23895507
Veneer Moment(1, 2)= -227740.17778526
Veneer Moment(2, 1)= 635360.62699467
Veneer Moment(2, 2)= 601411.36835338
Veneer Moment(3, 1)= 656399.23416307
Veneer Moment(3, 2)= 713888.17299327
Veneer Moment(4, 1)= -119488.93883020
Veneer Moment(4, 2)= -62000.000000000

FIND REACTIONS
caf(0)= 647.75206492511
caf(1)= 3532.6280709000
caf(2)= 1469.8588262872
caf(3)= 1369.5437779586
caf(4)= 2901.7213897793

Backup Moment(1, 1)= 1829287.4782778
Backup Moment(1, 2)= 1680774.3862998
Backup Moment(2, 1)= 2513246.8861353
Backup Moment(2, 2)= 2425544.6346453
Backup Moment(3, 1)= 2082130.0734511
Backup Moment(3, 2)= 2230643.1654291
Backup Moment(4, 1)= 791593.58186811
Backup Moment(4, 2)= 940106.67384612

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 914643.73913890
Curvature(1)= 3.4657546001958D-07
Moment (2)= 1755030.9322888
Curvature(2)= 6.6501373887851D-07
Moment (3)= 1888892.5112587
Curvature(3)= 7.1573637144593D-07
Moment (4)= 2097010.6362176
Curvature(4)= 7.9459618517401D-07
Moment (5)= 2305128.7611764
Curvature(5)= 8.7345599890208D-07
Moment (6)= 2469395.7603903

Curvature(6)= 9.3569980857620D-07
Moment (7)= 2339690.9943468
Curvature(7)= 8.8655226944736D-07
Moment (8)= 2253837.3540482
Curvature(8)= 8.5402073437247D-07
Moment (9)= 2167983.7137497
Curvature(9)= 8.2148919929758D-07
Moment (10)= 2156386.6194401
Curvature(10)= 8.1709484538334D-07
Moment (11)= 1870880.7695389
Curvature(11)= 7.0891138877217D-07
Moment (12)= 1511118.3736486
Curvature(12)= 5.7259075100036D-07
Moment (13)= 1151355.9777584
Curvature(13)= 4.3627011322854D-07
Moment (14)= 865850.12785712
Curvature(14)= 3.2808665661737D-07

DEFLECTIONS
Defl./field/(1)= 0.19988017750979
Joint No. 1
Deltal (1)= 1.9974689825776D-02
Deflection,backup (1)= 0.38608491782653
Deflection,veneer (1)= 0.40605960765230
Defl./field/(3)= 0.54747387956341
Defl./field/(4)= 0.68004580523471
Defl./field/(5)= 0.78083388349905
Joint No. 2
Deltal (2)= 8.3110855582612D-03
Deflection,backup (2)= 0.84679449516767
Deflection,veneer (2)= 0.85510558072593

Defl./field/(7)= 0.87606972348526
Defl./field/(8)= 0.86977208766460
Defl./field/(9)= 0.82931362246904
Joint No. 3
Deltal (3)= 7.7438698947363D-03
Deflection,backup (3)= 0.75580800809384
Deflection,veneer (3)= 0.76355187798857

Defl./field/(11)= 0.65031052725461
Defl./field/(12)= 0.51664417207224

DefL/field/(13)= 0.36007418684985
Joint No. 4
Deltal (4)= 1.6407327224486D-02
Deflection,backup (4)= 0.18586581589059
Deflection,veneer (4)= 0.20227314311507

DefL/field/(15)= 0.

WIND LOAD q= 3.2000000000000

Delta/io/-uniform load
delmo(1)= 97066666.666667
delmo(2)= 136533333.33333
delmo(3)= 128000000.00000

ITERATION No.= 1

Delta/ss/-support settlments
tdelss(1)= -87067391.602787
tdelss(2)= -509583364.94961
tdelss(3)= -442775075.87830
Delta/sm/-secondary moments
delms(1)= -15330383.688053
dems(2)= -24146631.742445
delms(3)= -27403699.559275
Total constant vector elastic-tcvel(i)
tcvel(1)= 5331108.6241729
tcvel(2)= 397196663.35872
tcvel(3)= 342178775.43758

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -164813.87334947
Joint No: 2 Moment-Xel= 616831.87140444
Joint No: 3 Moment-Xel= 687885.16711678
Veneer Moment(1, 1)= -164813.87334947
Veneer Moment(1, 2)= -222302.81217967
Veneer Moment(2, 1)= 616831.87140444
Veneer Moment(2, 2)= 583770.94120047
Veneer Moment(3, 1)= 630396.22828658
Veneer Moment(3, 2)= 687885.16711678

Veneer Moment(4, 1)= -121488.93883020
Veneer Moment(4, 2)= -64000.000000000

FIND REACTIONS
caf(0)= 686.6897889157
caf(1)= 3563.6081433959
caf(2)= 1569.3632543775
caf(3)= 1490.0007587086
caf(4)= 2931.7176324337

Backup Moment(1, 1)= 1880463.1502604
Backup Moment(1, 2)= 1731950.0582824
Backup Moment(2, 1)= 2641989.8440865
Backup Moment(2, 2)= 2556582.4410596
Backup Moment(3, 1)= 2211131.6233617
Backup Moment(3, 2)= 2359644.7153397
Backup Moment(4, 1)= 822193.29067493
Backup Moment(4, 2)= 970706.38265295

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 940231.57513021
Curvature(1)= 3.5627116518884D-07
Moment (2)= 1806206.6042714
Curvature(2)= 6.8440514921702D-07
Moment (3)= 1959460.0047334
Curvature(3)= 7.4247570225519D-07
Moment (4)= 2186969.9511845
Curvature(4)= 8.2868343645400D-07
Moment (5)= 2414479.8976355
Curvature(5)= 9.1489117065282D-07
Moment (6)= 2599286.1425731
Curvature(6)= 9.8491768109942D-07
Moment (7)= 2470219.7366351
Curvature(7)= 9.3601202844265D-07
Moment (8)= 2383857.0322107
Curvature(8)= 9.0328759953810D-07
Moment (9)= 2297494.3277862
Curvature(9)= 8.7056317063354D-07
Moment (10)= 2285388.1693507
Curvature(10)= 8.6597592288964D-07
Moment (11)= 1975281.8591735
Curvature(11)= 7.4847089606267D-07

Moment (12)= 1590919.0030073
Curvature(12)= 6.0282868807505D-07
Moment (13)= 1206556.1468411
Curvature(13)= 4.5718648008743D-07
Moment (14)= 896449.83666394
Curvature(14)= 3.3968145326046D-07

DEFLECTIONS
Defl./field/(1)= 0.2092053207584
Joint No. 1
Deltal (1)= 2.0149861773251D-02
Deflection,backup (1)= 0.40435579875187
Deflection,veneer (1)= 0.42450566052512

Defl./field/(3)= 0.57392628233248
Defl./field/(4)= 0.71361015661515
Defl./field/(5)= 0.82014669343965
Joint No. 2
Deltal (2)= 8.8737176971405D-03
Deflection,backup (2)= 0.89019545826306
Deflection,veneer (2)= 0.89906917596020

Defl./field/(7)= 0.92164039692985
Defl./field/(8)= 0.91553697963392
Defl./field/(9)= 0.87330205835646
Joint No. 3
Deltal (3)= 8.4249749472756D-03
Deflection,backup (3)= 0.79605702904592
Deflection,veneer (3)= 0.80448200399320

Defl./field/(11)= 0.68492574801369
Defl./field/(12)= 0.54404321234668
Defl./field/(13)= 0.37904752915668
Joint No. 4
Deltal (4)= 1.6576936260856D-02
Deflection,backup (4)= 0.19557680555544
Deflection,veneer (4)= 0.21215374181629

Defl./field/(15)= 0.

ITERATION No.= 2

Delta/ss/-support settlments
tdelss(1)= -86197531.336716
tdelss(2)= -536496293.70318
tdelss(3)= -469183743.48328
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -24747530.361440
delms(3)= -27704148.868773
Total constant vector elastic-tcvel(i)
tcvel(1)= 4461248.3581024
tcvel(2)= 424710490.73129
tcvel(3)= 368887892.35205

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -178420.94576810
Joint No: 2 Moment-Xel= 657932.67287413
Joint No: 3 Moment-Xel= 741578.75266650
Veneer Moment(1, 1)= -178420.94576810
Veneer Moment(1, 2)= -235909.88459830
Veneer Moment(2, 1)= 657932.67287413
Veneer Moment(2, 2)= 622618.37284892
Veneer Moment(3, 1)= 684089.81383630
Veneer Moment(3, 2)= 741578.75266650
Veneer Moment(4, 1)= -121488.93883020
Veneer Moment(4, 2)= -64000.000000000

F I N D R E A C T I O N S
caf(0)= 663.3682429468
caf(1)= 3654.6714397874
caf(2)= 1519.5361043937
caf(3)= 1404.3260843949
caf(4)= 2998.8346143709

Backup Moment(1, 1)= 1891215.9006727
Backup Moment(1, 2)= 1742702.8086947
Backup Moment(2, 1)= 2601397.4582102
Backup Moment(2, 2)= 2510168.8498118
Backup Moment(3, 1)= 2153234.6158123
Backup Moment(3, 2)= 2301747.7077904
Backup Moment(4, 1)= 821352.60627500

Backup Moment(4, 2)= 969865.69825301

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 945607.95033635
Curvature(1)= 3.5830837337228D-07
Moment (2)= 1816959.3546837
Curvature(2)= 6.8847956558390D-07
Moment (3)= 1957376.4710736
Curvature(3)= 7.4168621274607D-07
Moment (4)= 2172050.1334525
Curvature(4)= 8.2303004106888D-07
Moment (5)= 2386723.7958313
Curvature(5)= 9.0437386939169D-07
Moment (6)= 2555783.1540110
Curvature(6)= 9.6843359267465D-07
Moment (7)= 2420935.2913119
Curvature(7)= 9.1733723892757D-07
Moment (8)= 2331701.7328121
Curvature(8)= 8.8352499022035D-07
Moment (9)= 2242468.1743122
Curvature(9)= 8.4971274151312D-07
oment (10)= 2227491.1618013
Curvature(10)= 8.4403767396654D-07
Moment (11)= 1931648.9324115
Curvature(11)= 7.3193757164634D-07
Moment (12)= 1561550.1570327
Curvature(12)= 5.9170028816549D-07
Moment (13)= 1191451.3816538
Curvature(13)= 4.5146300468464D-07
Moment (14)= 895609.15226401
Curvature(14)= 3.3936290236445D-07

DEFLECTIONS
Defl./field/(1)= 0.20685590507670
Joint No. 1
Deltal (1)= 2.0664764860535D-02
Deflection,backup (1)= 0.39956705642624
Deflection,veneer (1)= 0.42023182128677

Defl./field/(3)= 0.56658545545276
Defl./field/(4)= 0.70374882476169
Defl./field/(5)= 0.80799099242787

Joint No. 2
Deltal (2)= 8.5919779142214D-03
Deflection,backup (2)= 0.87617343268532
Deflection,veneer (2)= 0.88476541059954

Defl./field/(7)= 0.90638623641598
Defl./field/(8)= 0.89979032322260
Defl./field/(9)= 0.85785341042042
Joint No. 3
Deltal (3)= 7.9405409760240D-03
Deflection,backup (3)= 0.78174040674996
Deflection,veneer (3)= 0.78968094772599

Defl./field/(11)= 0.67257539635268
Defl./field/(12)= 0.53432046429727
Defl./field/(13)= 0.37239752071525
Joint No. 4
Deltal (4)= 1.6956438679263D-02
Deflection,backup (4)= 0.19222847573810
Deflection,veneer (4)= 0.20918491441736

Defl./field/(15)= 0.

WIND LOAD q= 3.3000000000000

Delta/io/-uniform load
delmo(1)= 100100000.000000
delmo(2)= 140800000.00000
delmo(3)= 132000000.00000

ITERATION No.= 1

Delta/ss/-support settlments
tdelss(1)= -90280491.603366
tdelss(2)= -527510582.93153
tdelss(3)= -457561616.32302
Delta/sm/-secondary moments
delms(1)= -15330383.688053
delms(2)= -24438433.829932
delms(3)= -27549600.603019
Total constant vector elastic-tcvel(i)

tcvel(1)= 5510875.2914193
tcvel(2)= 411149016.76146
tcvel(3)= 353111216.92604

Redundants/moments-positive,
bottom side in tension/
Joint No: 1 Moment-Xel= -170787.50998261
Joint No: 2 Moment-Xel= 639087.84962478
Joint No: 3 Moment-Xel= 709862.75567607
Veneer Moment(1, 1)= -170787.50998261
Veneer Moment(1, 2)= -228276.44881281
Veneer Moment(2, 1)= 639087.84962478
Veneer Moment(2, 2)= 604932.66159273
Veneer Moment(3, 1)= 652373.81684587
Veneer Moment(3, 2)= 709862.75567607
Veneer Moment(4, 1)= -123488.93883020
Veneer Moment(4, 2)= -66000.000000000

F I N D R E A C T I O N S
caf(0)= 706.6458499710
caf(1)= 3678.8512230180
caf(2)= 1615.0960710194
caf(3)= 1539.0089378007
caf(4)= 3021.6896181328

Backup Moment(1, 1)= 1940700.5483841
Backup Moment(1, 2)= 1792187.4564061
Backup Moment(2, 1)= 2730507.5747598
Backup Moment(2, 2)= 2642273.3390103
Backup Moment(3, 1)= 2288516.6005485
Backup Moment(3, 2)= 2437029.6925266
Backup Moment(4, 1)= 852065.80382415
Backup Moment(4, 2)= 1000578.89580217

FIND FI /CURVATURE/ FROM M-FI
Moment (1)= 970350.27419204
Curvature(1)= 3.6768369832701D-07
Moment (2)= 1866444.0023951
Curvature(2)= 7.0723021549336D-07
Moment (3)= 2026767.4859945
Curvature(3)= 7.6797975403252D-07
Moment (4)= 2261347.5155829

Curvature(4)= 8.5686647373232D-07
Moment (5)= 2495927.5451714
Curvature(5)= 9.4575319343213D-07
Moment (6)= 2686390.4568851
Cracked section at joint(6),Fsteel<Fy
Curvature(6)= 1.8269931868931D-06
Moment (7)= 2553834.1543949
Curvature(7)= 9.6769508060747D-07
Moment (8)= 2465394.9697794
Curvature(8)= 9.3418383488384D-07
Moment (9)= 2376955.7851640
Curvature(9)= 9.0067258916022D-07
Moment (10)= 2362773.1465375
Curvature(10)= 8.9529852459725D-07
Moment (11)= 2040788.7203510
Curvature(11)= 7.7329265952697D-07
Moment (12)= 1644547.7481754
Curvature(12)= 6.2314961329603D-07
Moment (13)= 1248306.7759998
Curvature(13)= 4.7300656706510D-07
Moment (14)= 926322.34981316
Curvature(14)= 3.5100070199482D-07

DEFLECTIONS
Defl./field/(1)= 0.23569366178425
Joint No. 1
Deltal (1)= 2.0801485641889D-02
Deflection,backup (1)= 0.45686755684316
Deflection,veneer (1)= 0.47766904248505

Defl./field/(3)= 0.65161088980651
Defl./field/(4)= 0.81544745140083
Defl./field/(5)= 0.94500935404585
Joint No. 2
Deltal (2)= 9.1323067161222D-03
Deflection,backup (2)= 1.0314587737952
Deflection,veneer (2)= 1.0405910805113

Defl./field/(7)= 1.0564320534004
Defl./field/(8)= 1.0371922840443
Defl./field/(9)= 0.98058516129283

Joint No. 3
Deltal (3)= 8.7020839880928D-03
Deflection,backup (3)= 0.88776355376720
Deflection,veneer (3)= 0.89646563775530

Defl./field/(11)= 0.75990755059440
Defl./field/(12)= 0.60130742224826
Defl./field/(13)= 0.41778130937028
Joint No. 4
Deltal (4)= 1.7085668703468D-02
Deflection,backup (4)= 0.21514735260195
Deflection,veneer (4)= 0.23223302130542

Defl./field/(15)= 0.

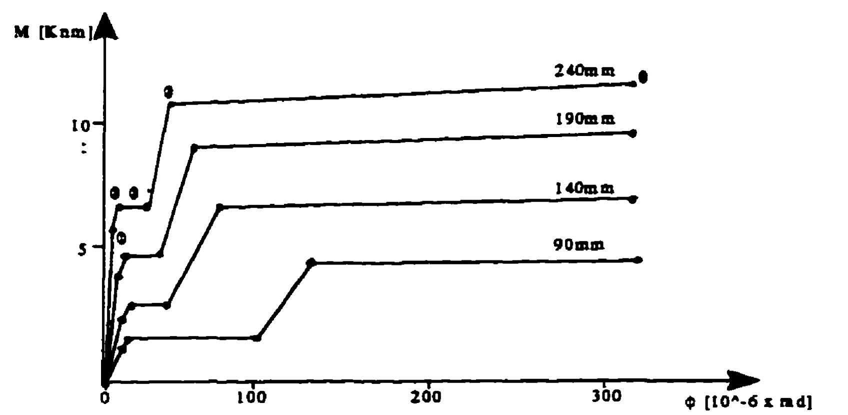
APPENDIX: C

Moment-Curvature Diagrams
Shear_Force-Rotation, Example

MOMENT-CURVATURE (M-FI) RELATIONSHIP OF MASONRY CROSS-SECTION

Block Size	[mm]	*Variable*
Solid Precentage	[%]	50
Mortar Type	[S or N]	S
Number of Grouted Cores	[0-5]	3
Compressive Strength	[MPa]	15.1
Flexural Tensile Strength	[MPa]	0.9
Area of Reinforcement	[mm^2]	250

	M [KNm] FI [rad x 10^-6]	Block Size [mm] 90	140	190	240
Points					
1	M el	0.93	2.07	3.59	5.31
1	FI el	1.2	0.8	0.6	0.5
2	M cr	1.12	2.47	4.27	6.29
2	FI cr	1.5	0.9	0.7	0.6
2'	M cr'	1.12	2.47	4.27	6.29
2'	FI cr'	19	15	14	12
3	M y	3.96	6.33	8.73	11.12
3	FI y	69	41	28	22
4	M u	4.18	6.7	9.23	11.75
4	FI u	330	330	329	329

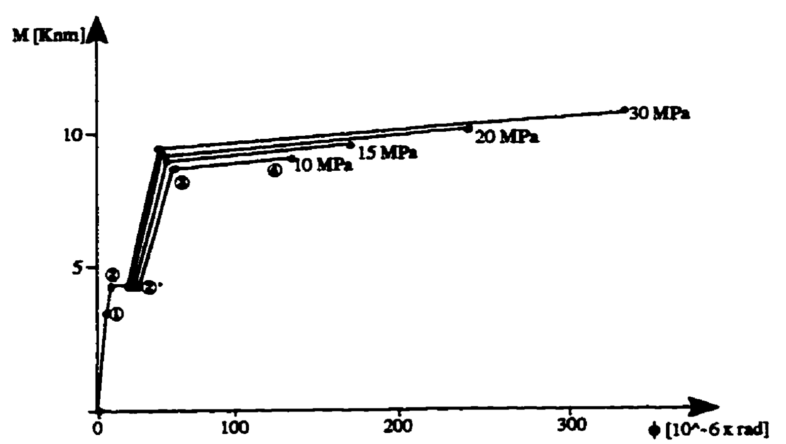


Different block size /90,140, 190 and 240 mm/

MOMENT-CURVATURE (M-FI) RELATIONSHIP OF MASONRY CROSS-SECTIO!

Block Size	[mm]	190
Solid Precentage	[%]	50
Mortar Type	[S or N]	S
Number of Grouted Cores	[0-5]	3
Compressive Strength	[MPa]	*Variable (Unit Strength Block)*
Flexural Tensile Strength	[MPa]	0.9
Area of Reinforcement	[mm^2]	250

		Compressive Strength [MPa]			
	M [KNm] FI [rad x 10^-6]	5.6	8.4	11.2	15.1
Points					
1	M el	3.64	3.64	3.64	3.64
1	FI el	1.6	1	0.8	0.6
2	M cr	4.34	4.34	4.34	4.34
2	FI cr	1.9	1.3	1	0.7
2'	M cr'	4.34	4.34	4.34	4.34
2'	FI cr'	18	16	15	14
3	M y	8.24	8.46	8.6	8.73
3	FI y	36	32	30	28
4	M u	8.61	8.94	9.1	9.2
4	FI u	121	183	244	329

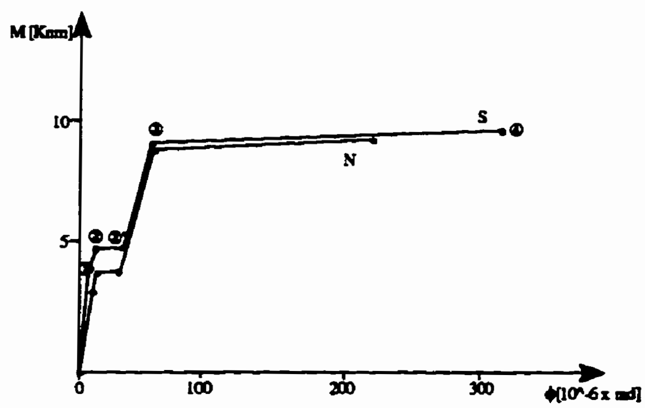


Different unit strength block / 10, 15, 20, 30 MPa /

MOMENT-CURVATURE (M-FI) RELATIONSHIP OF MASONRY CROSS-SECTION

Block Size	[mm]	190	190
Solid Precentage	[%]	50	50
Mortar Type	[S or N]	*S*	*N*
Number of Grouted Cores	[0-5]	3	3
Compressive Strength	[MPa]	*15.1*	*10.2*
Flexural Tensile Strength	[MPa]	*0.9*	*0.75*
Area of Reinforcement	[mm^2]	250	250

Points	M [KNm] / FI [rad x 10^-6]	Type of Mortar [N or S]: N	S
1	M el	3.4	3.65
1	FI el	0.7	0.6
2	M cr	3.6	4.34
2	FI cr	0.9	0.7
2'	M cr'	3.6	4.34
2'	FI cr'	13	14
3	M y	8.56	8.73
3	FI y	30	28
4	M u	9.06	9.23
4	FI u	221	329

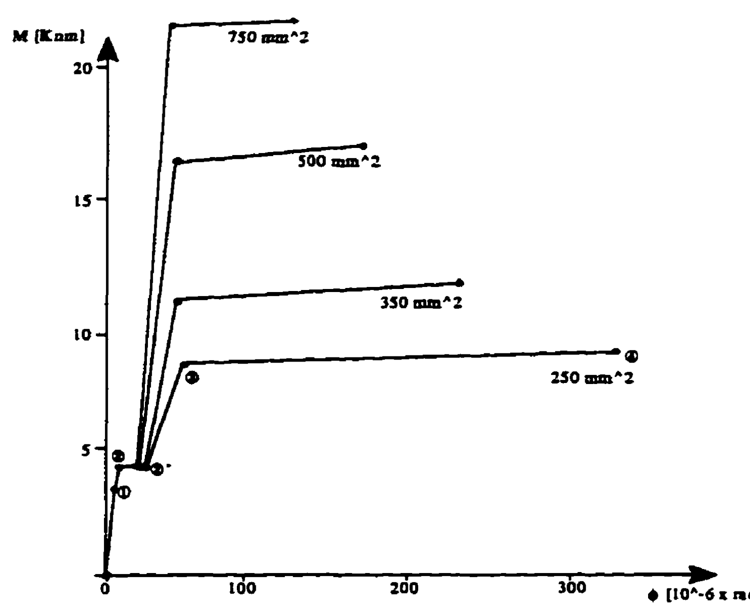


Different mortar type / S and N /

MOMENT-CURVATURE (M-FI) RELATIONSHIP OF MASONRY CROSS-SECTION

Block Size	[mm]	190
Solid Precentage	[%]	50
Mortar Type	[S or N]	S
Number of Grouted Cores	[0-5]	3
Compressive Strength	[MPa]	15.1
Flexural Tensile Strength	[MPa]	0.9
Area of Reinforcement	[mm^2]	*Variable*

		Area of Reinforcement [mm^2]			
	M [KNm] / FI [rad x 10^-6]	250	350	500	750
Points					
1	M el	3.64	3.64	3.64	3.64
1	FI el	0.6	0.6	0.6	0.6
2	M cr	4.34	4.34	4.34	4.34
2	FI cr	0.7	0.7	0.7	0.7
2'	M cr'	4.34	4.34	4.34	4.34
2'	FI cr'	14	11	8	6
3	M y	8.73	12	16.8	24.5
3	FI y	28	30	33	37
4	M u	9.2	12.7	17.7	25.5
4	FI u	329	236	164	109

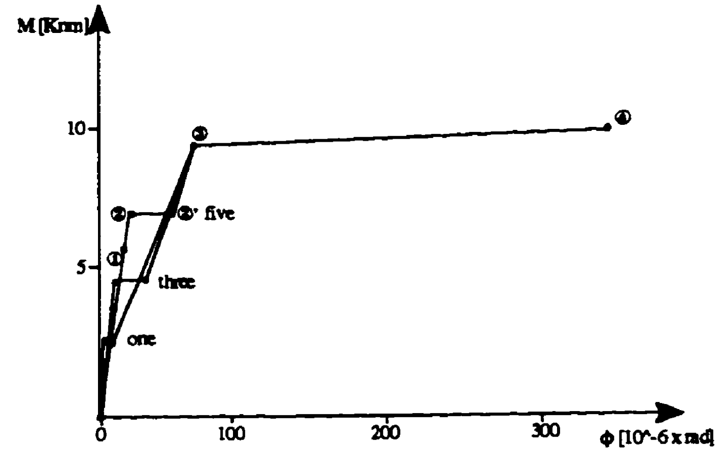


Different amount of reinforcement

MOMENT-CURVATURE (M-FI) RELATIONSHIP OF MASONRY CROSS-SECTION

Block Size	[mm]	190	190	190
Solid Precentage	[%]	50	50	50
Mortar Type	[S or N]	S	S	S
Number of Grouted Cores	[0-5]		*Variable*	
Compressive Strength	[MPa]	16.7	15.1	14.3
Flexural Tensile Strength	[MPa]	0.5	0.9	1.1
Area of Reinforcement	[mm^2]	250	250	250

Points	M [KNm] / FI [rad x 10^-6]	Number of Grouted Cores: 1	3	5
1	M el	1.8	3.6	5.8
1	FI el	0.3	0.6	0.9
2	M cr	2.1	4.3	7
2	FI cr	0.4	0.7	1.1
2'	M cr'	2.1	4.3	7
2'	FI cr'	0.7	26	23
3	M y	8.7	8.7	8.7
3	FI y	28	28	29
4	M u	9.2	9.2	9.2
4	FI u	365	329	293



Different number of grouted cores / 1, 3 and 5/

SHEAR_FORCE - ROTATION (S-TETA) RELATIONSHIP

Input

Cavity Width	[mm]	75
V-Tie Protrusion Length	[mm]	15
V-Tie, Moment of Inertia	[mm^4]	85.5
Steel Plate, Momen of Inertia	[mm^4]	45114
Veneer Wythe Width	[mm]	90
Backup Wythe Width	[mm]	190

Results

Moment of Plasticity	[Nmm]	14369.75
Maximum Shear Force	[N]	957.99
Moment at Venner Wythe	[Nmm]	57479.63
Moment at Backup Wythe	[Nmm]	148489.06
Rotation at the "supports"	[rad x 10^-6]	329.00
Displacement @ the hinge	[mm]	0.04