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COMPOSITE BEHAVIOR OF WALL-BEAM STRUCTURE

A Thesis
by

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ABSTRACT

Load bearing masonry wall supported over lintels, grade beams, floor beams etc. when subjected to superimposed vertical load act compositely with the supporting member. The wall contributes in stiffening the structure, the effect of which is reduced dimension and saving in material of bottom beam. An experimental investigation has been performed to study this composite action between wall and the supporting beam. In this experimental program a total of twelve specimens under four groups are tested. Out of these twelve specimens nine are composite wall-beam and the remaining three are reinforced concrete beams which are identical to the bottom beams of the composite wall-beam of the test series. Different height to length ratio of the wall beam has been considered in the study by varying the length of wall, while keeping the height of wall and beam constant. Different bond pattern in the brickwork of wall-beam structures and the provision of reinforcement in the brickwork are also considered in this study.

From the analysis of the test results, the effect of bond pattern of brickwork, inclusion of vertical reinforcement in the wall and H/L ratio on the failure load and deflection of composite wall-beams are discussed. The test results along with the analysis using existing formulae are also reported in this study.

Load carrying capacity of composite wall-beam structure is predicted by considering the wall-beam section uncracked and ultimate state of failure. The formulae for reinforced concrete beam and deep beam have been used with slight modification for prediction of the load.

An analytical method is also suggested for the prediction of load carrying capacity of composite wall-beam. The predicted load using this suggested method is compared with the experimental loads.

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NOTATIONS

A Gross cross-sectional area of reinforced concrete (RC) beam

a Shear Span

a_1 Depth of rectangular stress block of RC beam and composite wall-beam

A_{tc} Transformed area of RC beam into equivalent concrete

A_{tm} Transformed area of RC beam into equivalent masonry

A_s Cross-sectional area oflongitudinal reinforcement in beam

A_{sb} Cross-sectional area of balanced longitudinal reinforcement in beam

b Width of composite wall-beam and RC deep beam

C Total compressive force in beam

c Distance from neutral axis to top of beam at ultimate state (USD)

C_1 Ratio of the area of RC beam in masonry equivalent to the gross area (A_{tm}/A)

C_2 Ratio of the distance of natural axis from bottom fibre to the total height of composite wall-beam (y_b/H)

C_3 Ratio of the moment of inertia of composite wall-beam in masnry equivalent to gross moment of inertia of composite wall-beam (I_m/I_g)

C_4 Ratio of depth of bottom RC beam to total height of composite wall-beam (D/H)

D Total depth of RC beam

d Effective depth of RC beam

E_b Modulus of elasticity of RC beam

E_c Modulus of elasticity of concrete

E_m	Modulus of elasticity of brick masonry
E_w	Modulus of elasticity of wall proposed by Davis & Ahmed; S. Smith & Riddington
E_s	Modulus of elasticity of M.S bar
F	Proposed multiplying factor for predicting shear of composite wall-beam
$f_{b'}$	Compressive strength of brick
$f_{c'}$	Compressive strength of concrete cylinder
f_c	Allowable compressive strength of concrete
$f_{m''}$	Ultimate compressive strength of masonry prism
$f_{m'}$	Compressive strength of masonry prism
f_r	Modulus of rupture of concrete
f_y	Yield strength of M.S bar
f_s	Allowable stress of M.S bar
f_t	Tensile strength of brick
f_v	Maximum vertical stress in wall of composite wall-beam
H	Total height of composite wall-beam
h	Height of brick wall in composite wall-beam
I	Moment of Inertia ?
I_b	Moment of inertia of bottom (RC) beam
I_c	Moment of inertia of composite wall-beam in concrete equivalent
I_m	Moment of inertia of composite wall-beam in masonry equivalent
I_g	Gross moment of inertia of composite wall-beam
K	Axial stiffness parameter given by Davis and Ahmed
K_1	Factor for the equivalent width of rectangular stress block of composite wall-beam

L	Effective span length of beam
M	Bending moment of beam
M_u	Ultimate bending moment of beam at factor load
n	Modular ratio (E_s/E_c)
n_1	(E_c/E_m)
P	Vertical concentrated load
p	Longitudinal steel ratio of beam
R	Relative stiffness parameter given by stafford Smith and Riddington
R_f	Relative flexural stiffness parameter given by Davis and Ahmed
R_p	Relative flexural stiffness parameter suggested by the author
T	Axial force
t	Width of bottom RC beam
V_c	Ultimate shear taken by concrete in shallow RC beam
V_m	Ultimate shear taken by masonry in shallow reinforced masonry beam
V_{cb}	Ultimate shear taken by masonry in composite wall-beam
V_{dc}	Ultimate shear taken by concrete in RC deep beam
V_u	Ultimate shear force of beam at factor dead load
v	Shear stress
W	Total uniformly distributed load
w	Uniformly distributed load per unit per length
X	Length of vertical stress block assumed by Wood and Simms
y_t	Distance of neutral axis from top fibre of composite wall-beam

y_b Distance of neutral axis from bottom fibre of composite wall-beam

τ Wall and beam interface shear stress suggested by Green

τ_m Maximum shear stress at wall-beam interface suggested by Davis and Ahmed

α, β, γ Coefficients given by Davis and Ahmed for the analysis of composite wall-beam

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CHAPTER 1

INTRODUCTION

1.1 General

Masonry walls supported over other members such as lintels or beams in buildings are called the composite wall-beam structure. These composite masonry structures tend to act combinedly when subjected to vertical loading. In composite wall-beam structure, the high inplane rigidity of wall makes it act more like a tied arch or deep beam. The beams are thus required to tie the arch and hence, axial force is more predominant than the flexural action. Generally for design of lintels, grade beams, floor beams etc. the brick work lying over these members are considered as dead weights. The beams are designed to support the load of an equilateral triangular area of brick work. The base of this triangle is the span of the beam. Consideration of the composite behavior of wall-beam structure will not only lead to a rational design of beams and walls, but also ensure satisfactory performance with respect to cracking. This study is an attempt to investigate the influence of different parameters on the composite behavior of wall-beam structures.

1.2 Statement of the Problem

Some of the common examples of load bearing masonry walls supported over reinfroced concrete beams spanning between two supports (composite wall-beam) are discussed below.

1.2.1 Brick Wall over Lintel in Masonry Wall

In masonry building, reinforced concrete lintels are provided over the openings of wall such as, doors and windows to support the brickwork above it and also the floor slab. The reinforced concrete lintel and the brickwork above it act as a composite wall-beam structure as shown in Fig. 1.1.

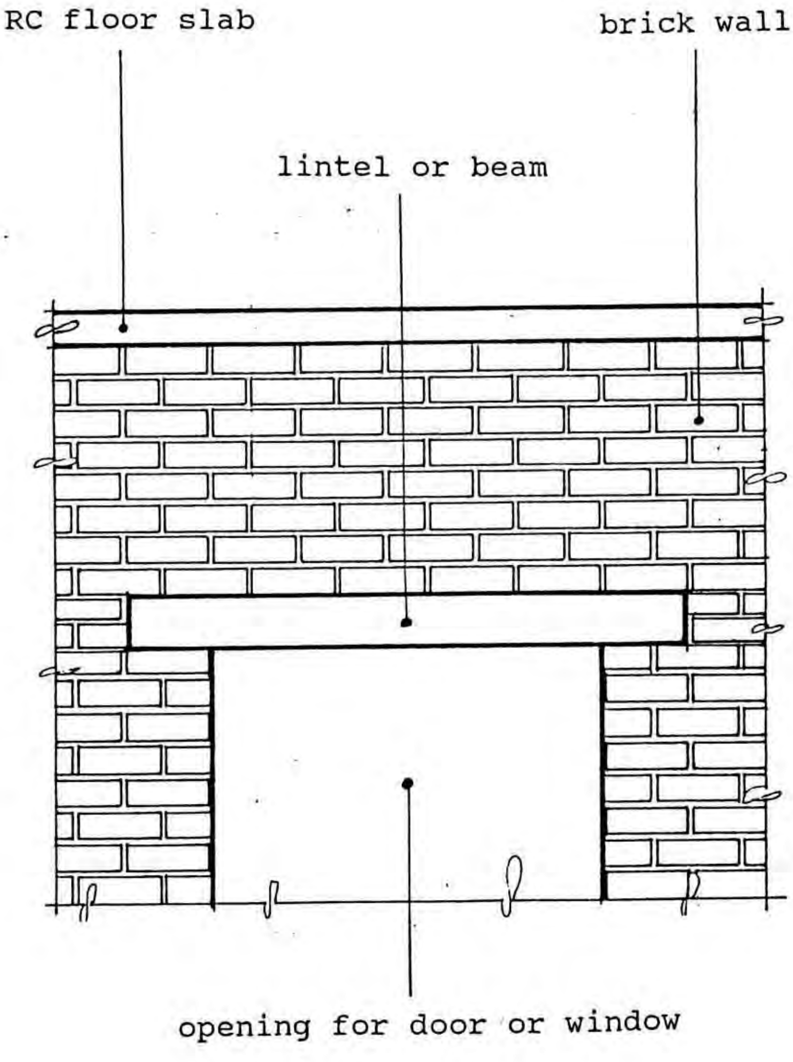


Fig.1.1 Load Bearing Brick Wall Over Reinforced Concrete Lintel

1.2.2 Load Bearing Brick Wall Over Grade Beam

In cases where piles are used for the foundation of building, the brick wall above grade beam (foundation beam) may be considered as a load bearing wall. The grade beam and the load bearing wall above it will act as a composite wall-beam structure as shown in Fig. 1.2. In such condition, the size of the grade beam can reduce if it is designed taking into account the composite action between masonry wall and RC beam, which gives the beam greatly increased lever arm over that of the RC beam acting alone.

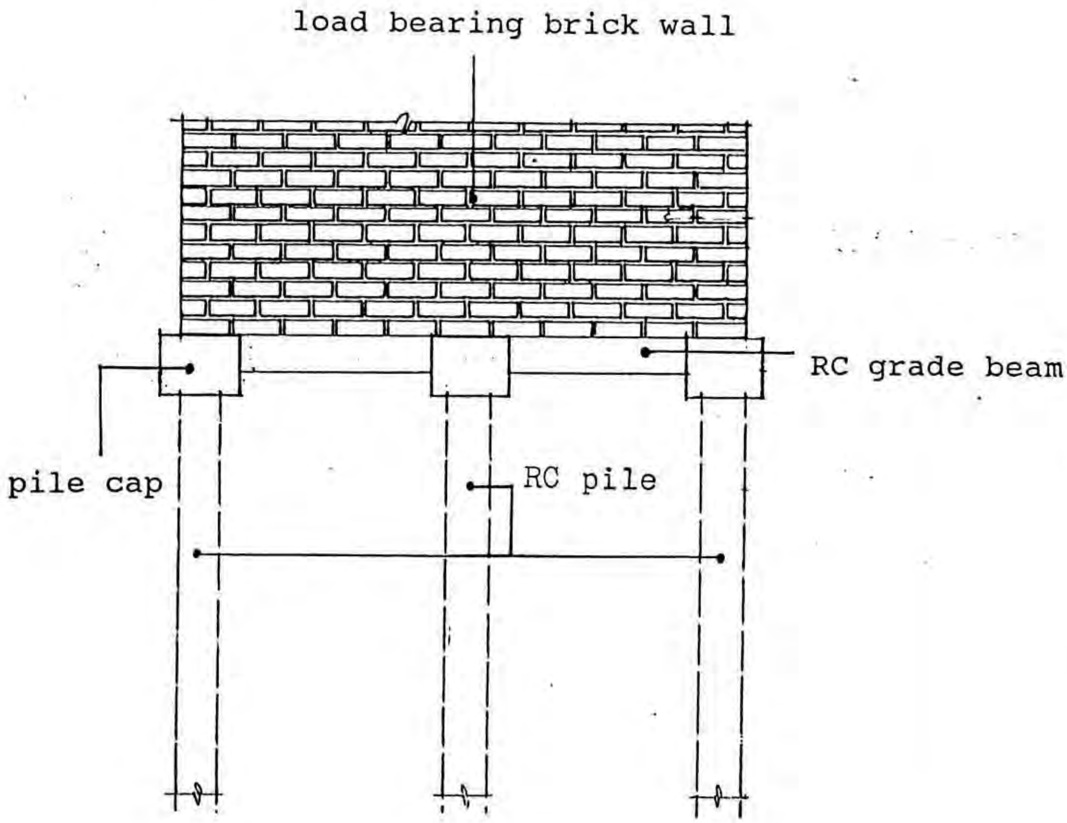


Fig.1.2 Load Bearing Brick Wall Over Reinforced Concrete Grade Beam

1.2.3 Load Bearing Brick Work Over Floor Beam in Framed Structure

In framed structure building where ground floor spaces are kept open for parking and other utility services, masonry wall above first floor beam may be considered as load bearing wall shown in Fig.1.3. The load bearing brick wall over the floor beam may be act as a composite wall-beam structure.In such case, columns may be continued upto first floor beam level and the size of beam at first floor level can reduce if composite action between the masonry wall and the floor beam is considered.

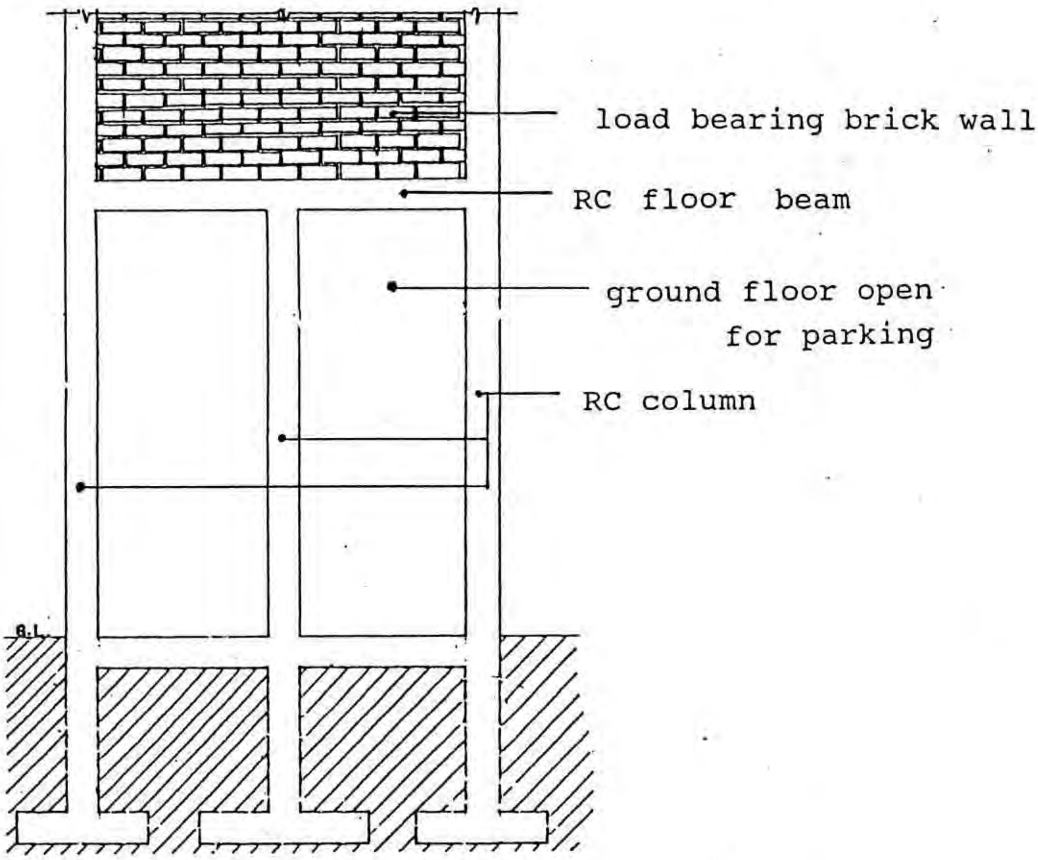


Fig.1.3 Load Bearing Brick Wall Over Reinforced Concrete Floor Beam

1.2.4 Load Bearing Brick Wall Over Continuous Lintel in Framed Structure

In reinforced concrete framed structures generally beams are placed below the slab level to carry the loads of slabs and walls. Conventionally the gap between the bottom of beam and the top of lintel is filled up with brickwork. This brickwork is considered as filler material. In such cases the continuous lintel and the brick wall above it extending upto floor level may be considered as composite wall-beam structure which may lead to replace the floor beams as shown in Fig. 1.4

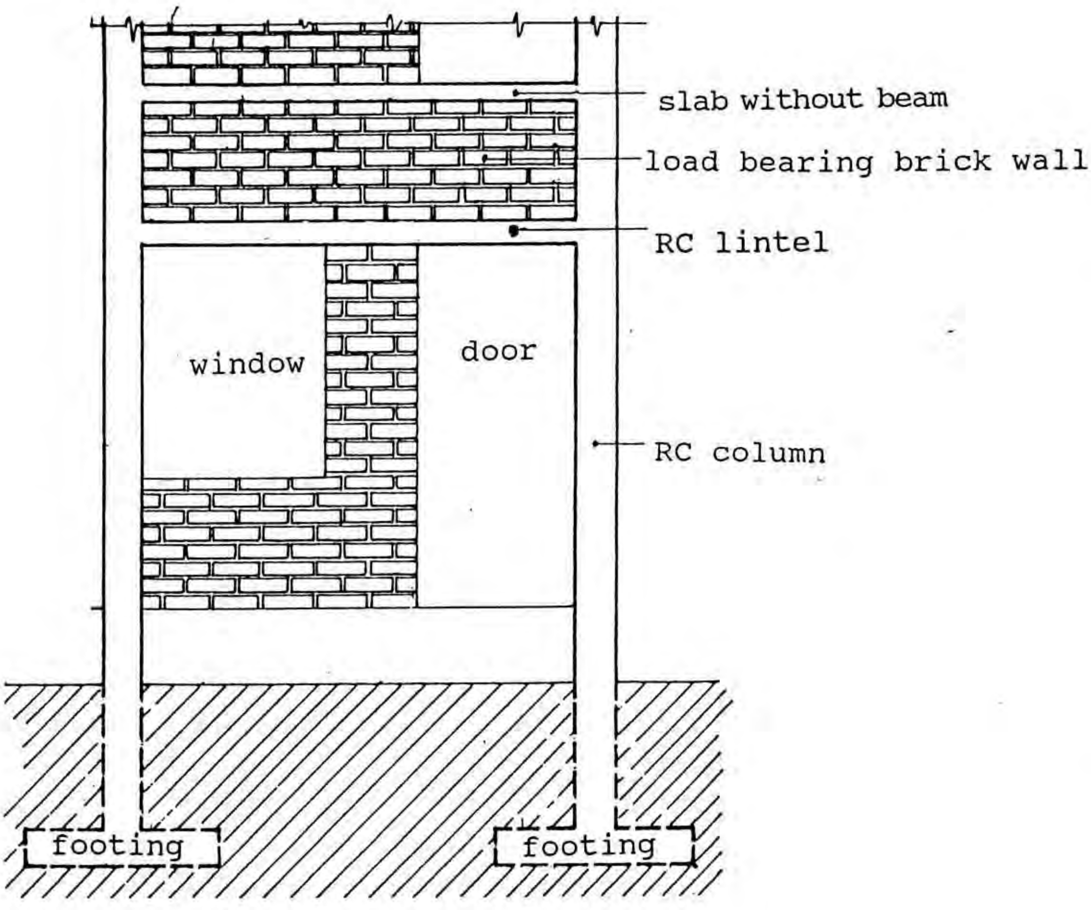


Fig.1.4 Load Bearing Brick Wall Over Reinforced Concrete Lintel In Framed Structure

1.3 Justification of this Study

In Bangladesh, the present construction of building is based on utilization of primary materials like cement, steel, bricks, stone etc. The demand for these materials is enormous and the building materials industry and trade are unable to cope with it. There is always a wide difference between demand and the supply of these materials in the market. As a result, the price of building materials is rising very high day by day with the growing demand and consequently the overall cost of building is increasing sharply. Consideration of the composite behavior of wall-beam structures lead to a rational design of the supporting beam resulting in significant saving in concrete and steel. Parametric study in this field have been made in different countries of the world leading to new design recommendations. Recently a theoretical study has been carried out in the civil Engineering department of BUET, Dhaka. It appears that there are considerable scope and need for experimental study in this field. It may be mentioned here that most of the previous researchers have restricted their work to uniformly distributed load on running bonded brick work. Very little is known about the behavior of wall-beam composite action subjected to concentrated load and stack bonded brick work. It is expected that the result of this experimental investigation would help in understanding the behavior of composite wall-beam structures. This may lead to economic design of lintel, grade beam, floor beam in framed structures.

1.4 Objective of this Study

There are immense potentialities of increasing the application of composite wall-beam structure in this country if composite behavior is duly taken into consideration. This is expected to reduce the dimension of supporting beam and its steel requirements. The present study may be considered as the beginning of a research program to be carried out in BUET in order to identify the significant parameters influencing the composite behavior of wall-beam structure. The main objectives of this study are -

i) To study the behavior of reinforced concrete beam or lintel when brick masonry wall is built over it.

ii) To study the influence of height to span (H/L) ratio of composite wall-beam on its load carrying capacity, deflection and crack pattern.

iii) To study the effect of different bond pattern in brick wall supported over reinforced concrete beam.

iv) To study the effect of shear reinforcement in composite wall-beam structure.

v) To compare the load carrying capacity, deflection, and failure pattern of the composite wall-beam with the respective supporting reinforced concrete beam.

vi) To suggest design methods on the basis of this investigation.

1.5 Scope of The Present Study

There are many factors that influence the behavior of composite wall-beams. For example height to span (H/L) ratio, bond pattern of brick wall, opening in the wall, depth of the supporting beams, amount of reinforcement in the supporting beams, presence of vertical and horizontal reinforcement in the wall, type of loading, strength of constituents materials etc. are some of the prominent parameters. The present investigation has been limited to study the effect of height to span (H/L) ratio, bond pattern (two types) and inclusion of vertical reinforcement in the wall on the behavior of wall-beam structures. It may be mentioned that different height to span ratios were achieved by varying the span of the composite wall-beams while height of all the walls were kept constant at 2 ft. 6 in. for all the test beams.

Suitability of the existing design methods for computing flexural and shear strength of reinforced concrete beams have been studied with appropriate modification to predict the ultimate load of composite wall-beams. Increase in load carrying capacity of the supporting beams due to composite actions have been studied in relation to its non-composite behavior.

1.6 Methodology of Study

Methodology of this research work is summarized as follows:

i) Available literatures were reviewed in order to know the state of the art of composite behavior of wall-beam structures.

ii) Three reinforced concrete beams of varying span having same cross sectional area and reinforcement have been made and tested in the laboratory.

iii) Three running bonded brick masonry wall have been constructed over the similar type of reinforced concrete beam (as mentioned in (ii) and tested in the laboratory to investigate the composite behavior of wall-beam structures.

iv) Three stack bonded brick wall without vertical reinforcement and three stack bonded wall with vertical reinforcement in vertical joints of wall have been constructed over the similar type of reinforced concrete beam as mentioned in (ii) in order to investigate the effect of bond pattern and vertical reinforcement in the composite wall beam structure.

v) Crack pattern, failure load, deflections at different levels of loading have been recorded during the test.

vi) Test results have been analyzed and compared with the results of similar works and recommendations have been put forward.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction

Masonry walls supported over lintels or beams spanning between two supports are called wall-beam structures. Some examples of wall-beam structures are illustrated in Figs.1.1 to 1.4. It has been recognized that structural interaction takes place between a masonry wall and a supporting beam when these are subjected to superimposed vertical load. Composite wall-beam acts similar to an arch or a deep beam due to their high inplane rigidity. It has also been recognized that superimposed vertical loadings on the wall of composite wall-beam structure are not transmitted vertically down to the beam below but are carried towards the support points by arching action[1] as shown in Fig. 2.1.

To develop the design method of the composite wall-beam structure, the important parameters such as, the maximum vertical stress in the wall, the axial force in the beam, the maximum shear stress along the interface, the maximum bending moment in the beam and its location and also the maximum deflection are to be considered. It is to be noted that a number of investigators[2,3,4] already started their work on this subject and their available papers have been reviewed here.

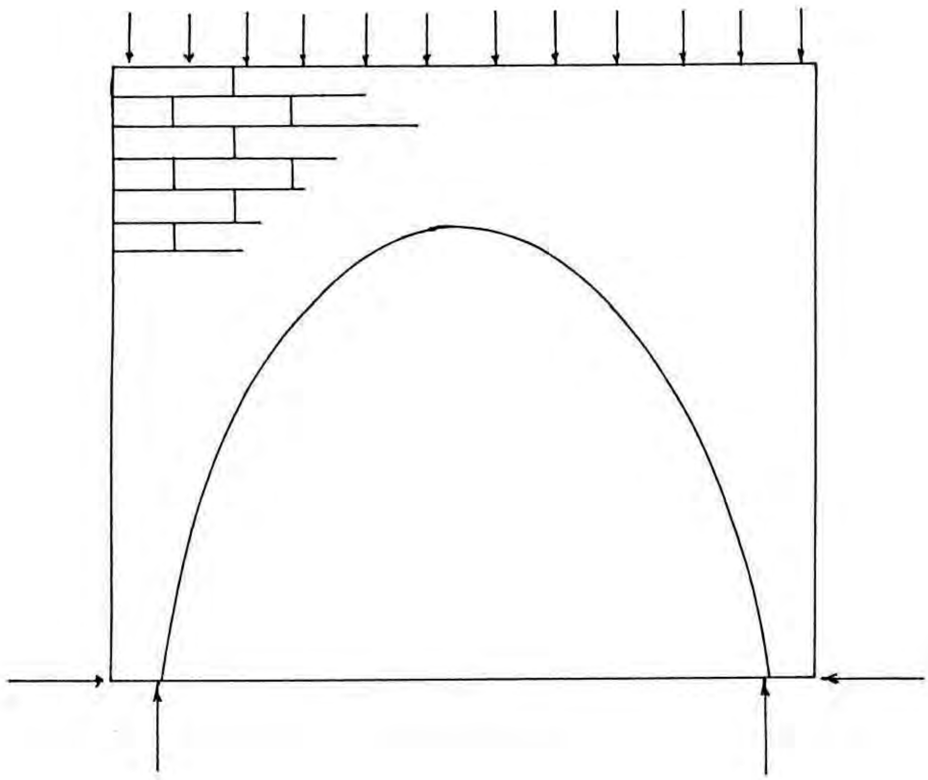


Fig.2.1 Arching Forces in Wall (Hendry)[1]

Brick masonry wall is the major part of a composite wall-beam structure, so the properties of brick masonry and its behavior in flexural, shear and diagonal tension with or without reinforcement are reviewed here. ACI code provisions for the design of reinforced concrete deep beam has been illustrated here. Concept of reinforced concrete deep beam behavior has been later introduced with appropriate modification to estimate the ultimate load of composite wall-beams.

2.2 Previous Study on Composite Wall-Beam Structure

In 1952 Wood[5] introduced an empirical design method for brick walls supported over reinforced concrete beams subjected to superimposed vertical load on top of the wall based on tests of typical house walls. He introduced moment coefficient for the design of supporting reinforced concrete beam.Following his work a number of investigators found interest to work on this new field and Wood didn't lose his interest either. He along with Simms[6] continued this study and in 1969, proposed a tentative design method which was based on the assumption that the vertical stresses in the vicinity of the supports formed a rectangular stress block which extended a distance 'X' into the span from each end of the beam as shown in Fig.2.2. This simplified loading diagram was conservative because it overestimates the bending moment in the beam and underestimates the actual peak stresses in the bricks locally.

In 60s' Burhouse[7] tested composite wall-beams supported over reinforced concrete beam until failure. The height to span ratio of the composite wall-beams were 0.33 to 0.81 having 12 ft. span in all cases. The overall dimensions of the beam in each test were 12 in. deep by 6 in. wide but the reinforcements were different. From the failure load he derived working load with a load factor of 5 in accordance with table 10 of B.S. code of practice CP 111:1970[8].

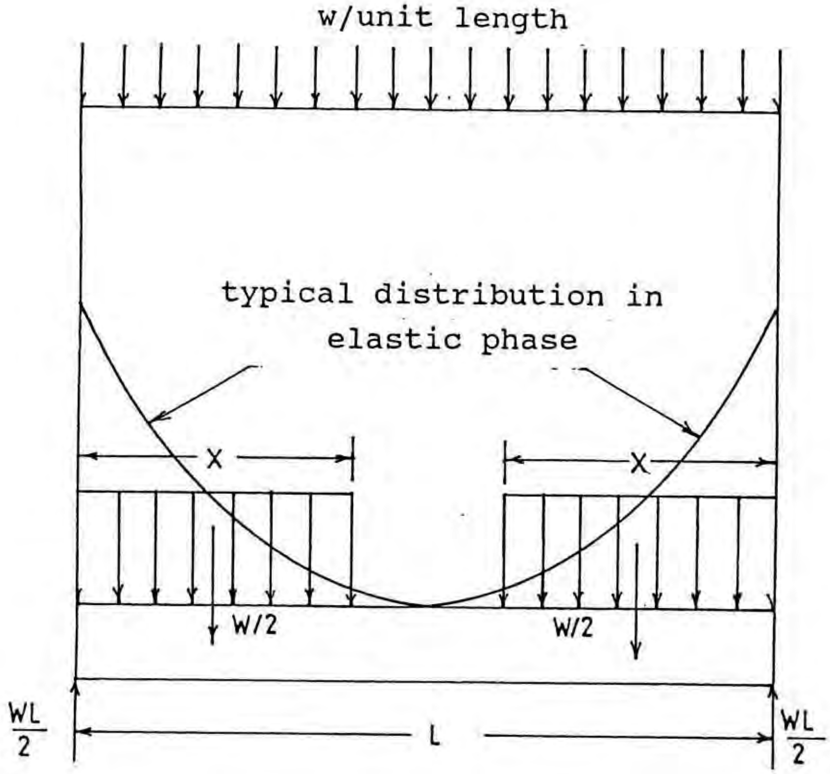


Fig.2.2 Assumed Equivalent Beam Loading (Wood & Simms)[6]

The ratio of working load derived with load factor and the permissible working load derived in accordance with CP 111 is about 0.29 to 0.66 which assumes the wall on a rigid foundation. It was found from his experiment that the failure load of composite beam-wall having H/L ratio of 0.33 (test specimen No.9) is higher than that of composite wall-beam having H/L ratio of 0.83 (test specimen No.8). He explained, that the latter beam failed at lower load due to the effect of higher slenderness ratio of the wall.

In the early 60s' Rosenhaupt[9] tested twelve masonry walls on point supported beams to failure under uniform distributed load. A typical test specimen along with the loading arrangement is shown in Fig.2.3.

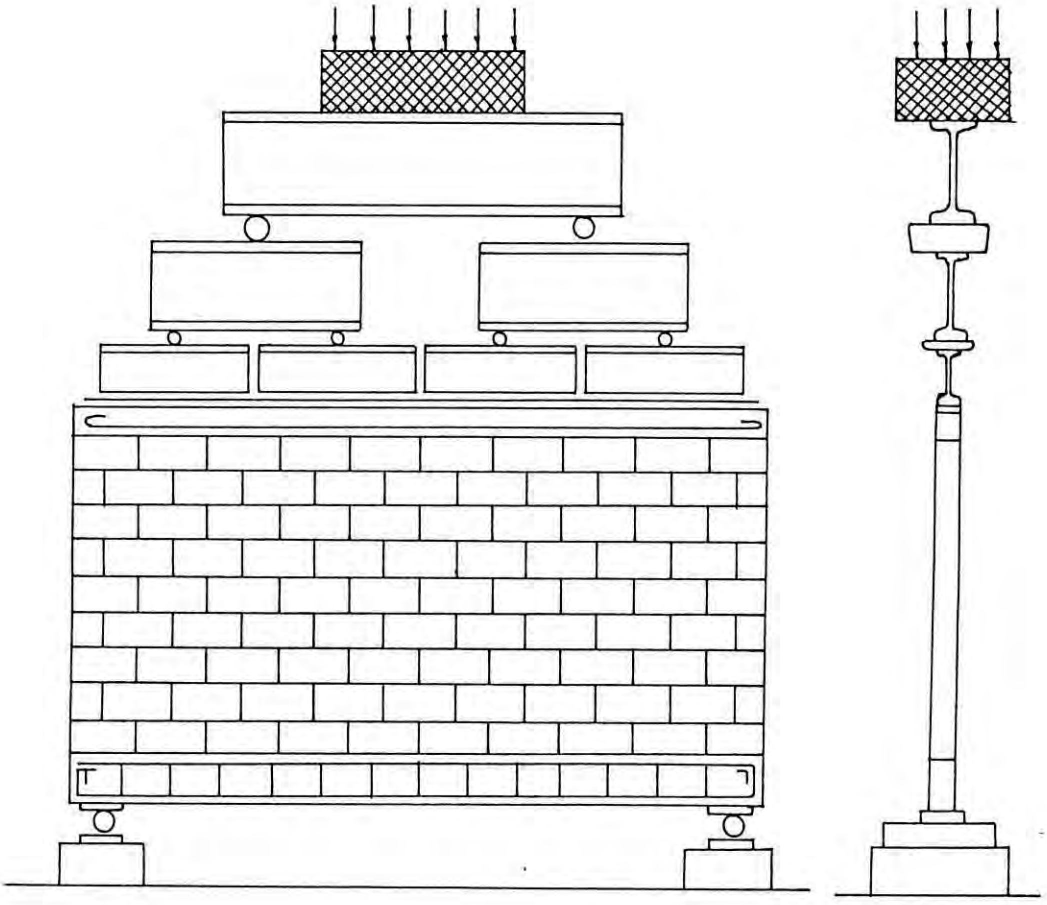


Fig.2.3 Loading Arrangement (Rosenhaupt)[9]

H/L ratio of tested specimen varied from 0.08 (without wall) to 0.80 (with wall) having span of 6 ft. and the depth of the foundation beams were 6 inch in each case. Masonry material of different strength (Ytong block = 171 psi and Hollow concrete block = 276 psi) was used and found that the rigidity of the structure depends on the modulus of elasticity of the masonry materials. He also suggested that the moment arm of composite wall-beam to be approximately equal to one-half the height. Composite action increases the internal moment arm, making possible a reduction in the quantities of concrete and steel in the beam. He recommended that effective exploitation of the composite action demands high strength of brick work or the extension of the foundation stanchions over the entire height of the wall.

Annamalai, Jayaraman and Madhava Rao[4] carried out experimental investigations on ten numbers of lintel-wall specimen having 4 ft. span with different types of beam and wall materials. The height of the masonry wall above the lintel was kept as eight courses to correspond to the normal height of the wall above the door and window openings. The thickness of the reinforced concrete lintel was 3 inch to correspond to a single course of brick work and the width was 9 inch to correspond to the thickness of a one-brick wall. The effect of compressive strength of brick work on composite action, vertical stress in wall, bending moment of beam and deflection were determined. They studied the effect of compressive strength of bricks on composite action by using wire cut bricks having a compressive strength 2175 psi and chamber bricks having 1015 psi. Among the two types of bricks wirecut brick walls are found to have contributed about 50 percent increase in the composite strength as compared to that of chamber brick walls.

In 1976 and in 1978, Stafford Smith and Riddington[10,11] analyzed the wall-beam problems considering beam on elastic foundation with varying values of the structural parameters by the finite element method of stress analysis. They proposed a non dimensional parameter R which is the relative stiffness of the wall to the beam to derive approximate expressions for the bending moment and the axial force in the beam, and also the maximum stresses in the wall which is shown in Fig.2.4.

The proposed relative stiffness is given by:-
$R = (E_w \, t \, L^3 / E_b I_b)^{1/4}$; where E_w = Modulus of elasticity of wall; t = thickness of wall; L = span length; $E_b I_b$ = flexural rigidity of beam.

The parameter R does not contain the variable 'h' since it was considered that the ratio of h/L was equal to 0.66 and this was representative of walls for h/L greater than 0.66.

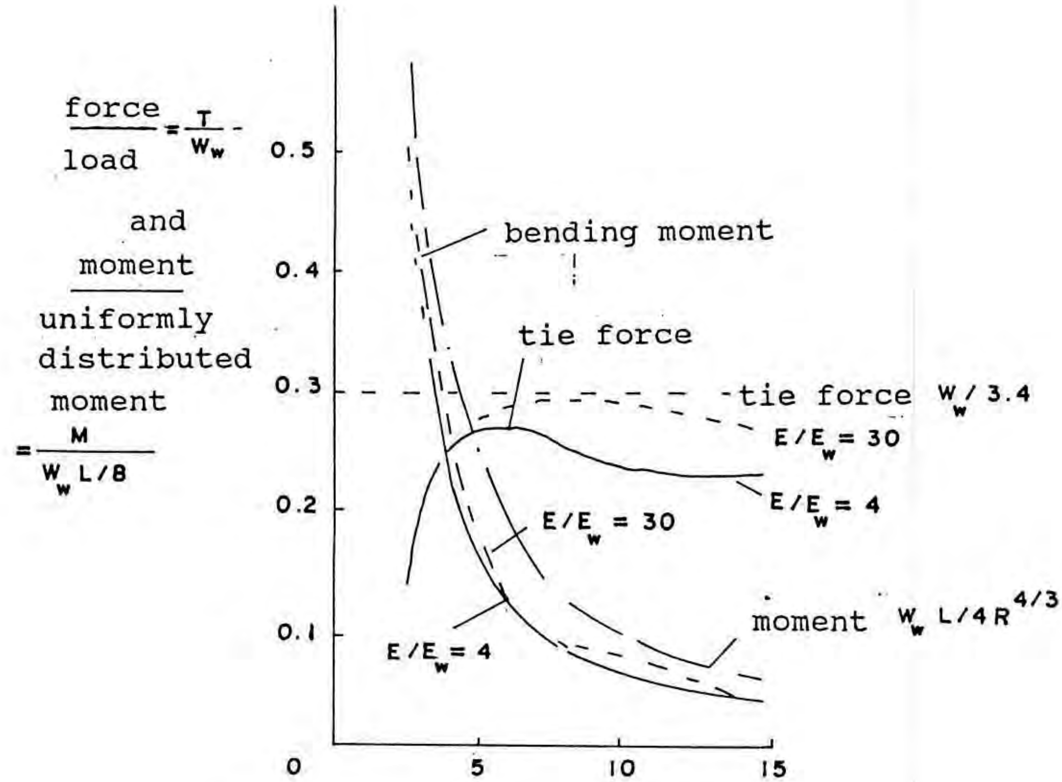


Fig.2.4 Maximum Bending Moments and Tie Force in Beam (Smith & Riddington)[10]

Davis and Ahmed[12,13,14] introduced two parameters one of them is relative flexural stiffness parameter given by: $R_f = (E_w t h^3/E_b I_b)^{1/4}$ and the other is axial stiffness parameter given by $K = E_w t h/E_b A$; where E_w = Modulus of elasticity of wall; t = thickness of wall; h = height of wall;

E_b = Modulus of elasticity of beam; I_b = Second moment of Area of beam (Moment of inertia); A = Cross-sectional area of beam. These suggested formulae for approximate calculation of the vertical stresses in the wall, the axial force in the beam, the maximum shear stress along the wall-beam interface, the distribution of bending moment in the beam and the central deflection in the beam using the finite element approach. Two assumptions were made by Davis and Ahmed[12,13,14]. The first one is the vertical stress distribution is related to the value of R_f. This distribution could be linear, parabolic or cubic as shown in Fig.2.5.

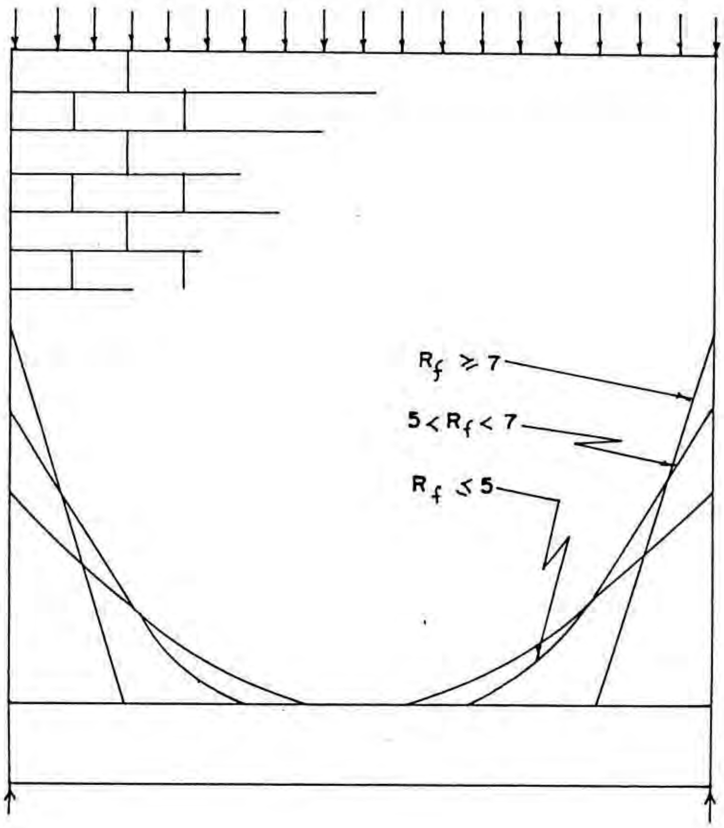


Fig.2.5 Vertical Stress Distribution (Davies and Ahmed)[14]

The second assumption is that the axial force in the beam is assumed to vary from zero at the supports to a maximum value at the center shown in Fig.2.11.

In 1969, Colbourne[15] represented the wall and the beam by a lattice analogy assuming the system elastic. The analogy is used to derive equilibrium equations to find stresses in the wall and stress resultants in the beam. The method has been made on the basis of a computer program, for which it is only necessary to provide geometric data, wall and beam elastic constants and load data.

Kamal,[16] carried out a parametric study based on elastic analysis by finite element method with different height to span ratios, sizes of beams, stiffness parameter, modular ratios. Particular emphasis given to the study were the variation of shear stress, vertical stress, and bending moment in the beam.

Gu Yisun et. al.[17] Developed formulae for design of composite wall-beam structure using finite element method and prepared a computer program to carry out an elastic analysis.

Green D. R.[18] developed equations for composite wall-beam structure with height to width ratio greater than 1.5 and without openings. The equations were developed from a parametric study using finite element technique considering beam on elastic foundation. The analysis of the wall-beam structure and the approximate force action is shown in Figs. 2.6 and 2.7.

Ramesh 'et al'[19] investigated the behavior of composite action between a brick panel wall or external facade wall in reinforced concrete framed structure, and the supporting concrete beam. The results of the study show that tensile connectors have to be provided between the wall and the beam, and that it is possible to effect a saving of about 20 percent in the steel if a composite deep beam design is adopted in place of the current semi-empirical approach to the problem.

Govindan and Santha Kumar[20] tested four brick in filled beams and one reinforced concrete conventional beam. They found that the behavior of the infilled beam is similar to that of solid beam at low-loads before cracking. The stiffness of the solid beam and infilled beam are same before cracking.

Riviere[21] the practice of composite construction of block wall and reinforced concrete beam in Mauritius has been illustrated. He found that economy achieved so far in using composite construction in the order to 20 to 25 percent.

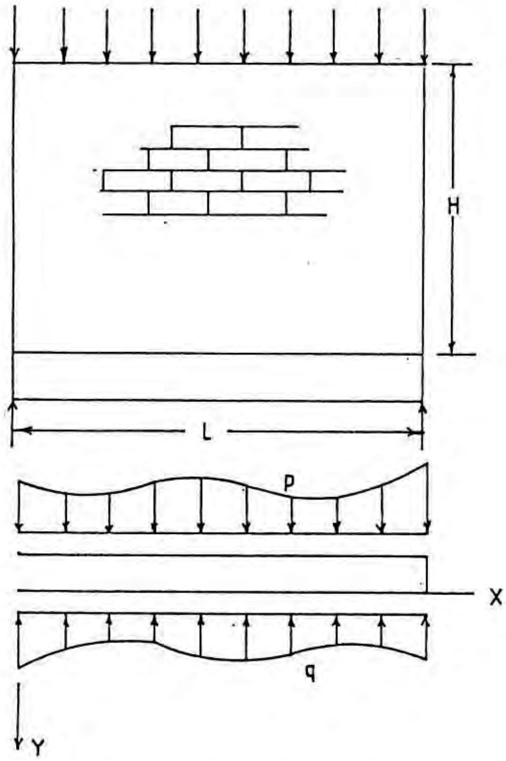


Fig.2.6 Wall-Beam as Beam on Elastic Foundations (Green)[18]

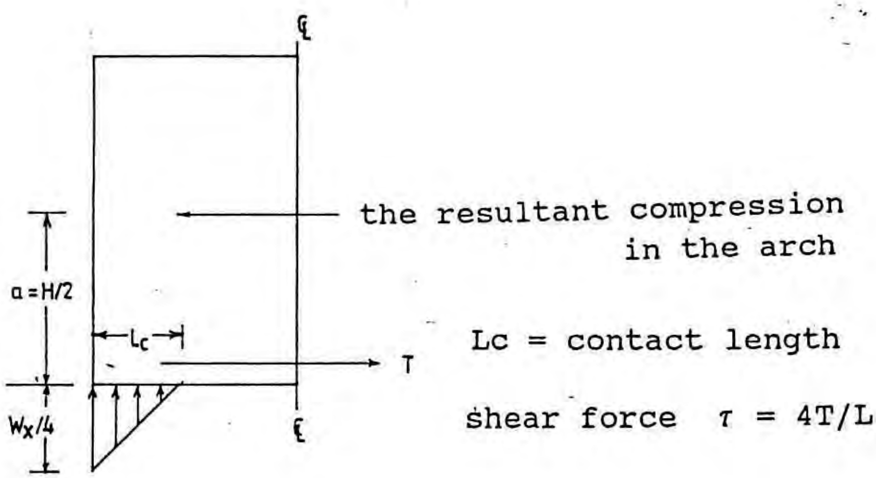


Fig.2.7 Approximate Force action in a wall beam (Green)[18]

Sundara Rao[22], described the development and use of composite brick masonry reinforced concrete load-bearing walls with thin ribbed in-situ slabs in five storied low cost residential buildings. Considerable savings has been reported as compared with the standard construction due to the adoption of new design.

2.3 Review on the Design Parameters

The previous investigators suggested various formulae for the calculation of the maximum bending moment and axial force developed in the beam. They also suggested formulae for evaluating the maximum vertical stress in the wall, and shear stress along the interface of wall and beam based on their theoretical and experimental study. However, some of them predicted maximum deflection from their experimental study. These are critically discussed in the following section.

2.3.1 Bending Moment of the Supporting Beam

Composite wall-beam structure when subjected to vertical loading, act as tied arch. Because of arch action, vertical load concentrated near to the supports and horizontal shear stress develop at the wall beam interface which is eccentric to the axis of beam with respect to beam centroid. This has the effect of reducing the bending moment in the beam which would be otherwise expected if the total load was uniformly distributed over the span.

Wood[5] suggested that the maximum bending moment of beam be taken as WL/100 for plain walls or walls with door or window openings occurring at midspan and WL/50 for wall with door or window opening occurring near the supports based on experimental work, where W = total uniformly distributed load; L = effective span length. He also introduced a moment arm based on elastic analysis of homogeneous deep walls of 0.67 times the depth of wall with the limit of 0.7 times the span for simply supported deep wall. He did not consider the modulus of elasticity of materials which is very important factor of composite wall-beam structure found by Annamalai 'et.al'[4] and Rosenhaupt[9] from experiment. G. Annamalai 'et. al'[4] found that the above moment coefficient vary from 1/30 to 1/50 instead of 1/100. Stafford smith and Riddington[10] proposed a formula for estimating the maximum beam bending moment of supporting beam as $[WL/4]\ [1/(E_w tL^3/EI)^{1/3}]$. It is to be noted that when this expression is simplified as $[W/4][1/(E_w t/E(I)^{1/3}]$, it becomes independent of span.

Davis and Ahmed[14] derived equations for maximum bending moment as $[WLr-2WD\ (\alpha - \gamma K)]\ /\ [4\ (1 + \beta R_f)]$.

They considered three cases according to the magnitude of the stiffness parameter R_f, which are as follows--

Case-1. $R_f \le 5$ Stiff beam ; $r = 0.2$ and $\lambda = 0.25$

$$M\ max^m = [WL - 10\ WD(\alpha - \gamma K)] / 5(1 + \beta R_f)$$

Case-2. $5 < R_f < 7$ flexible beam ; $r = 0.25$ and $\lambda = 0.33$

$$M\ max^m = [WL - 8WD(\alpha - \gamma K)\ (1 + \gamma R_f)] / [5.33(1 + \beta R_f)]$$

Case-3. $R > 7$ very flexible beam; $r = 0.33$ and $\lambda = 0.50$

$$M\ max^m = [WL - 6\ Wd\ (\alpha - \gamma K)] / [6(1 + \beta R_f)]$$

Where W = Applied uniform distributed load on wall; d = depth of beam; and α, β , γ coefficients depending on h/L ratio.

Kamal[16] in his study found that maximum moment occurs at a distance of about 1/15th the span from the support. The distribution of beam moments are shown in Fig.2.8. He also found that the maximum moment is dependent on the depth of bottom beam provided other parameters are constant.He concluded that the stiffer the beam, the more is bending moment developed in the beam shown in Fig.2.9.

He found that the maximum moment in the case of distributed and concentrated load is in close agreement with each other. This is possibly due to the depth of wall (H/L>0.6) which provides enough area for the dispersion of concentrated load through the wall. However, he suggested that both vertical and interface shear stresses be considered for computation of moment as shown in Fig. 2.10.

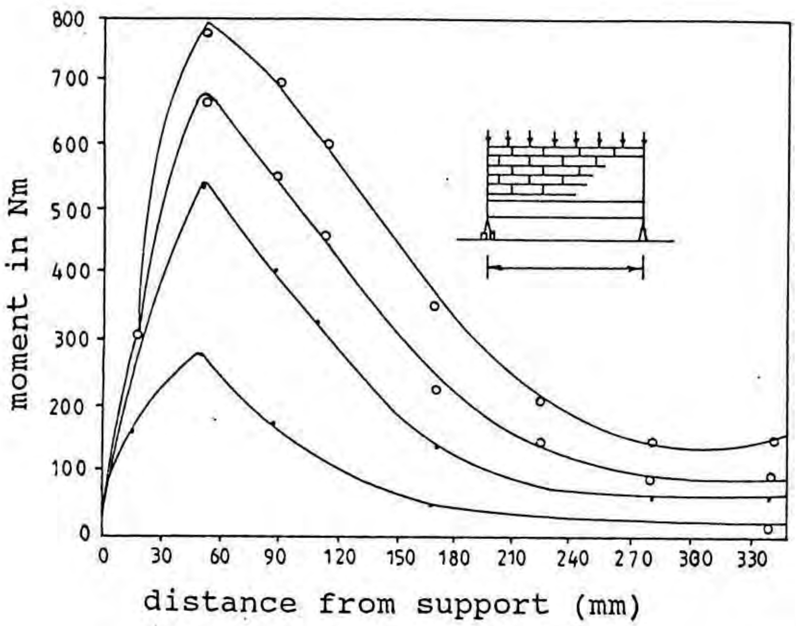


Fig.2.8 Distribution of Moment Along The Length of Beam (Kamal)[16]

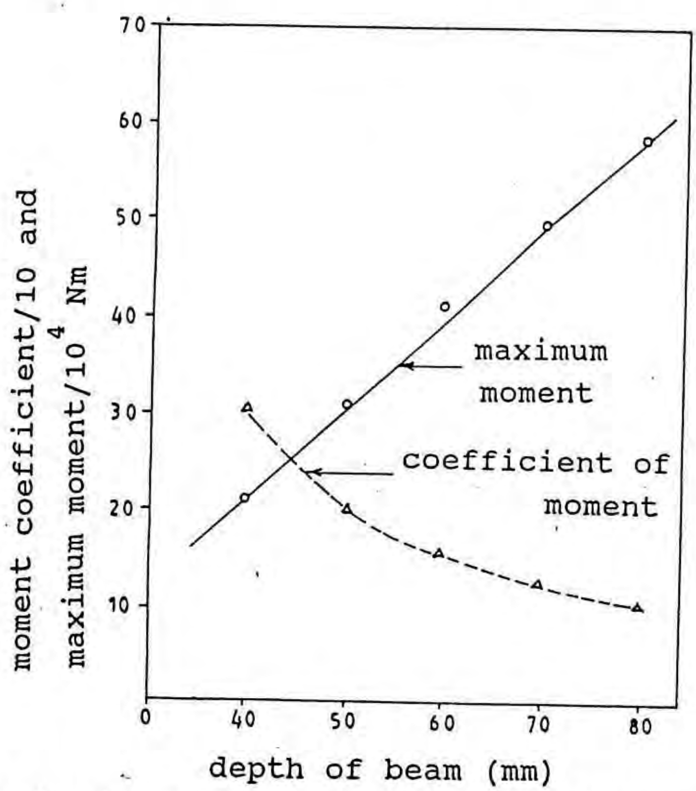


Fig.2.9 Maximum Moment Curve for Different Beam Sizes (Kamal)[16]

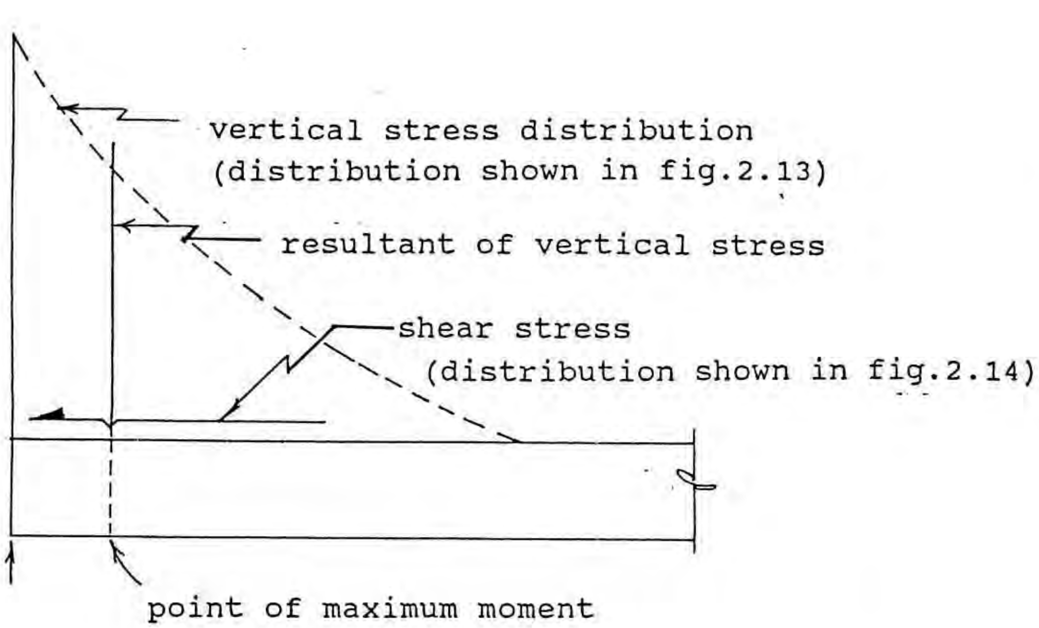


Fig.2.10 Stresses Contributing Moment (Kamal)[16]

2.3.2 The Axial Force in the Beam

Composite wall-beam of H/L greater than 0.60 when subjected to vertical loading act as tied arch. The bottom beam acts as a tie. According to Wood[5], tensile force in the beam may be calculated as T = 3 WL/16h; where W = total uniform distributed load; L = Span length, h = Total height of composite beam. This value is equal to T = W/3.4 as suggested by Stafford Smith and Riddington[10] assuming H/L = 0.60. This is in very good agreement with Wood's value. Annamalai 'et.al'[4] determined the tensile force in the beam experimentally and compared with the value calculated from the empirical formula T = W/3.4 and found good correlation. According to Davis and Ahmed the axial force in the beam varies from zero at the support and maximum at the center, variation being linear as shown in Fig.2.11.

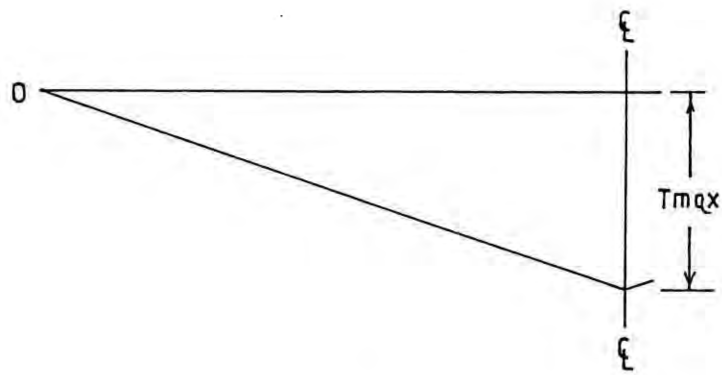


Fig.2.11 Assumed Tensile Force Distribution (Ahmed and Davies)[14]

This is in very good agreement with Kamal[16]. According to Davis and Ahmed maximum axial force developed in the beam is given by $T = W (\alpha - \gamma K)$ where α, γ are coefficients obtained from prescribed graphs. Mathematical expression of maximum tensile force as suggested by Kamal[16] is equal to T = W/3.7 where W = Applied load. It is also revealed from his study that upto a certain distance of about L/8 from the support, the magnitude is directly proportional to the distance from either support, then the curve flattens out gradually. The rate of flattening is found to depend on the value of H/L or R_f as shown in Fig.2.12.

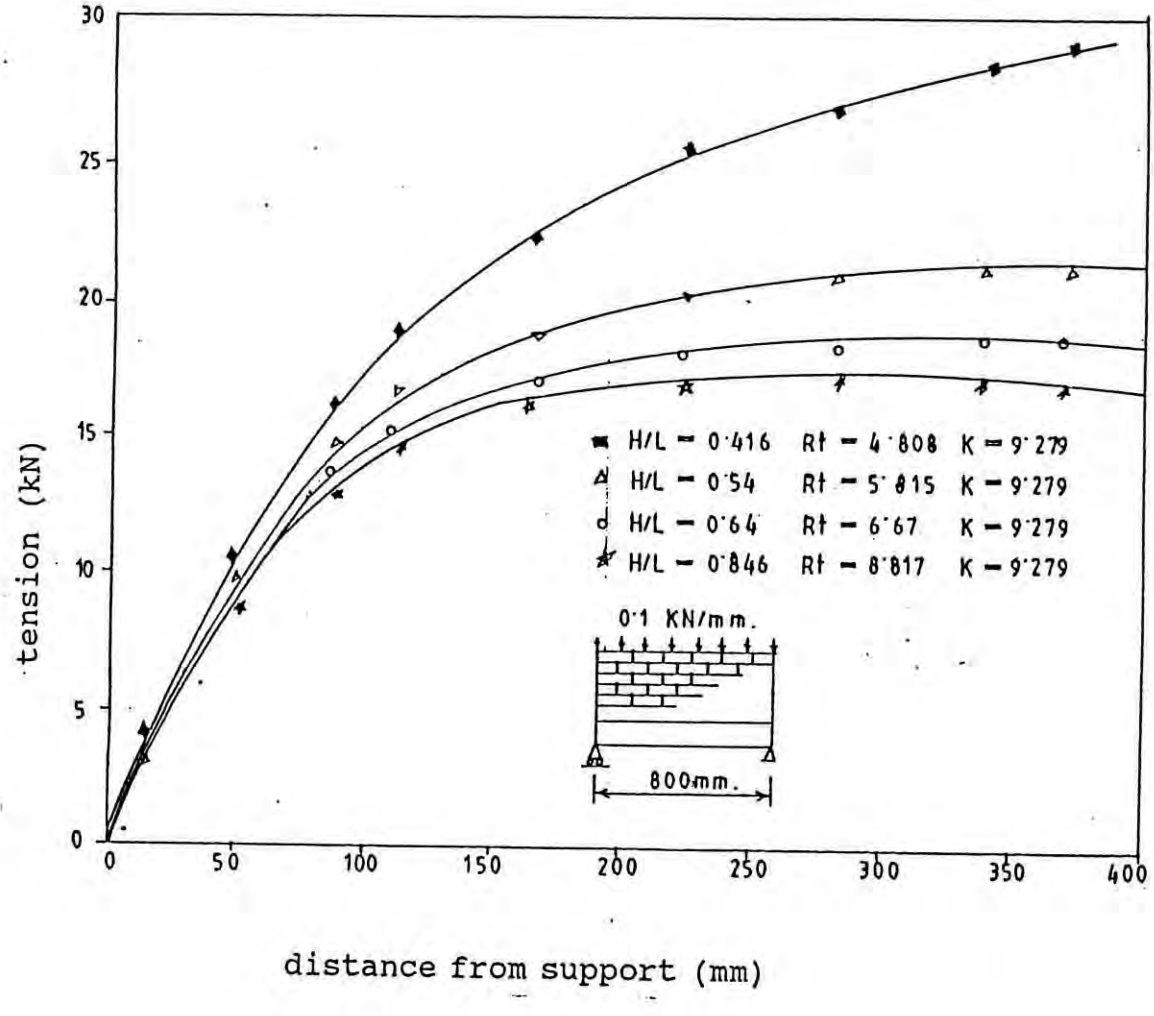


Fig.2.12 Tensile Forces Along the Length of the Beam for Different H/L ratio (Kamal)[16]

2.3.3 Maximum Vertical Stress in the Wall

It has already been recognized by the investigators of this subject that superimposed vertical loads on the wall of composite wall-beam structure are not transmitted vertically down to the beam below but are carried towards the support points by arching action as shown in Fig.2.1. The nature of distribution, method of calculation and magnitude of this stress one different as suggested by different investigators. For example, Wood and Simms suggested that the vertical stresses in the vicinity of the supports form a rectangular stress block (Fig.2.2) and is equal to 12.5 W/L per unit width.[10] Where W = Total uniform distributed load; L = effective span length. Whereas Davis and Ahmed[14] assumed that the vertical stress distribution along the contact surface is mainly governed by relative flexural stiffness parameters, R_f. (Fig.2.5). This distribution could be linear, parabolic or cubic depending on the value of 'R_f'. According to them maximum vertical stress is equal to [W/Lt] $(1 + \beta R)$ where β = is a co-efficient obtained from prescribed curve. The pattern of vertical load distribution of nearly parabolic nature is also found. Rosenhaupt[9] in his experimental study. Stafford Smith and Reddington[10] suggested that the maximum vertical stress in the wall is [1.63 W/Lt] $(E_W tL^3/E_b I_b)^{0.28}$ where W=Total uniformly distributed load, E_w = Modulus of Elasticity of wall, E_b = Modulus of Elasticity of beam; I_b = Moment of inertia of beam and L = Effective span length.

According to kamal[16] vertical stress is maximum over the supports and then the values decreases sharply for higher values of R_f (Relative stiffness parameter of Davis & Ahmed) and decrease gradually for lower values of R_f along the length of the beam. (Fig.2.13).

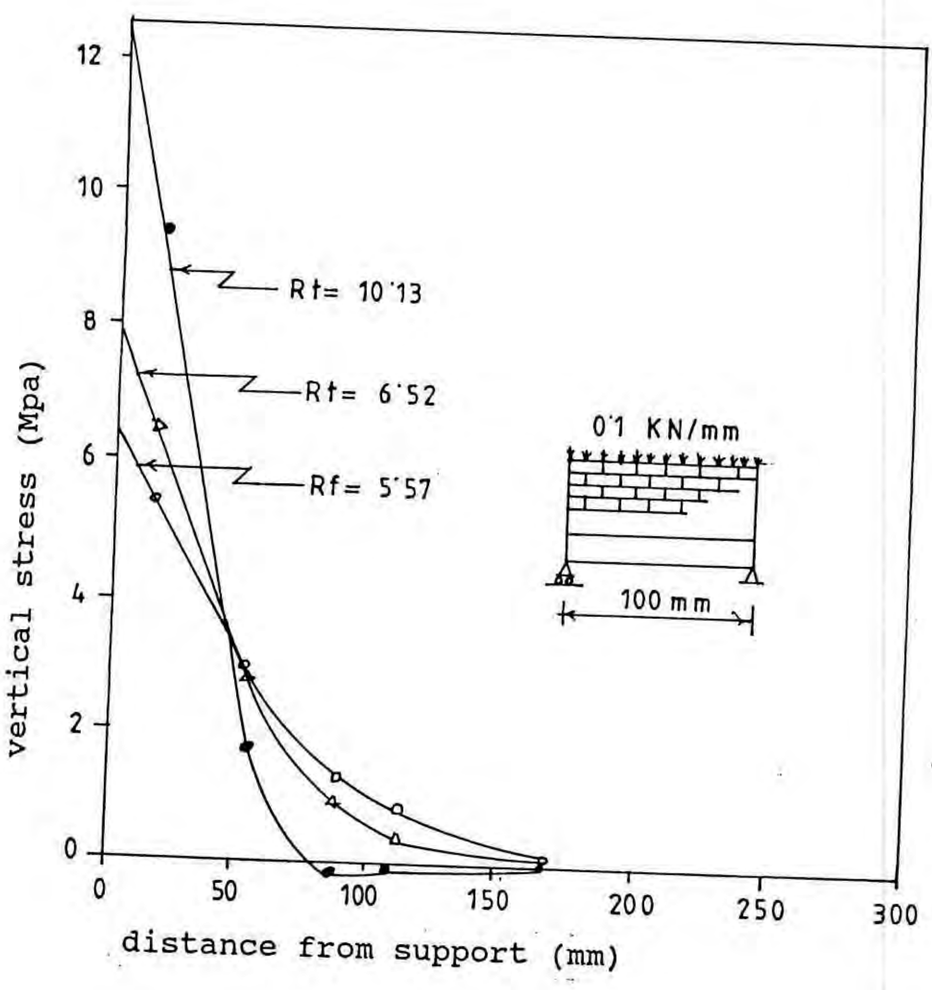


Fig.2.13 Distribution of Vertical Stress Along the Length of Beam (Kamal)[16]

2.3.4 Maximum Shear Stress Along the Interface

composite action can not be achieved unless there is sufficient bond between the wall and the beam to allow for the development of the required shearing forces. The large compressive stresses near the supports result in large frictional forces along the interface, and it has been shown that if the depth / span ratio of the wall is greater than 0.6, then the frictional forces developed are sufficient to supply the required shear capacity. Kamal[16] found in his study that shear stresses at supports are quite high and these values increase sharply attaining the peak at L/15 from the support and then decrease very slowly for uniform distributed load (Fig. 2.14).

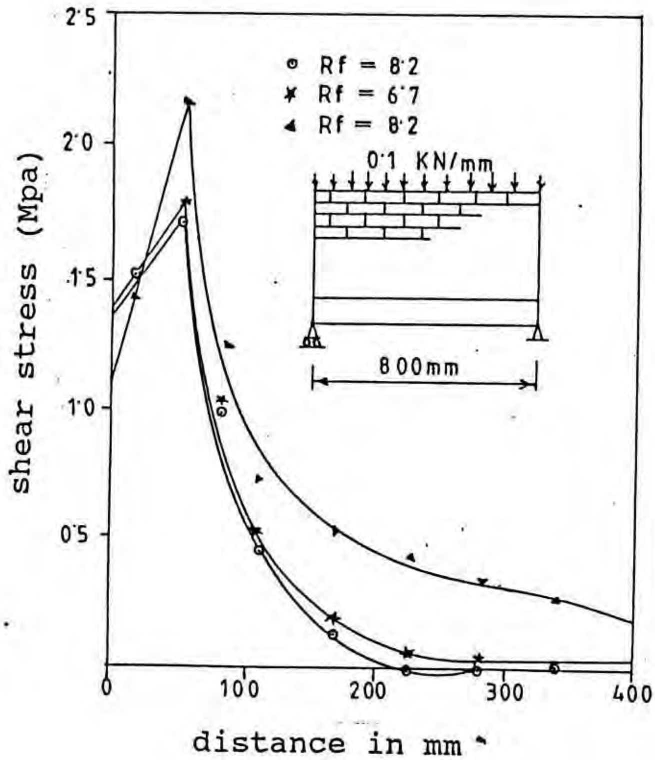


Fig.2.14 Shear Stress Along Interface (Kamal)[16]

He also found that the maximum shear stress caused by concentrated load is always higher than the shear stress caused by equivalent distributed load (Fig. 2.15).

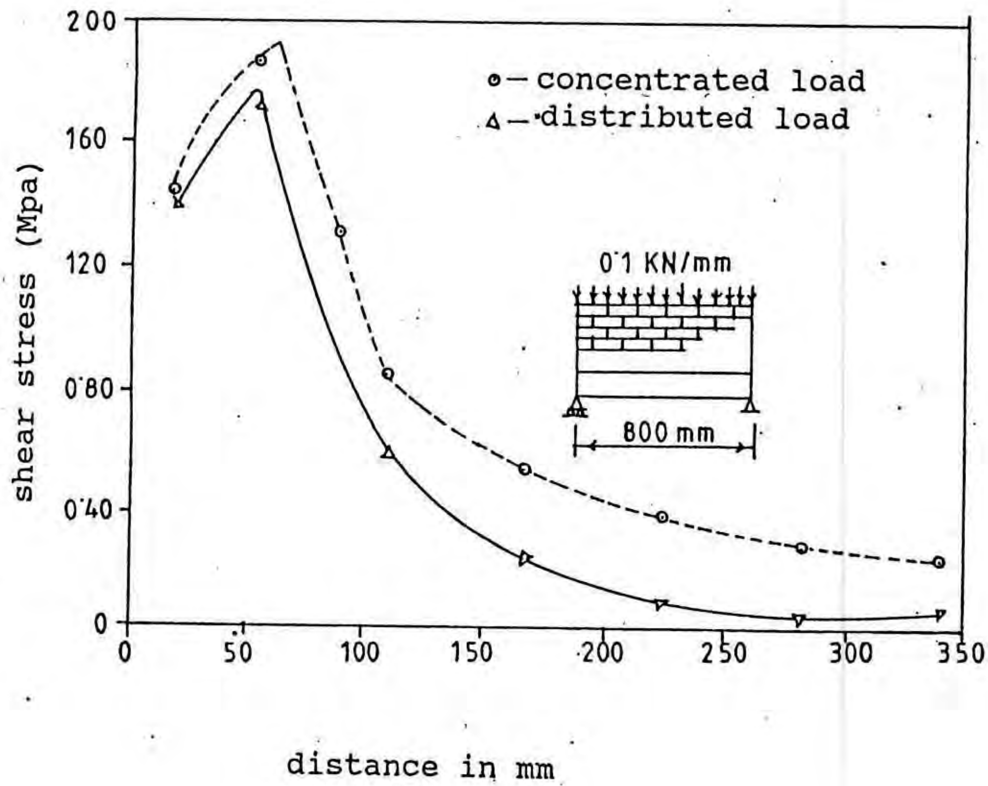


Fig.2.15 Comparison of Shear Stress for Different Type of Loading (Kamal)[16]

The stiffer the beam, the more is the distribution of shear stress along the interface. However, Davis and Ahmed[14] proposed a formula for calculating maximum shear stress along the interface which is given by

$\tau_m = W(\alpha - \gamma K)(1 + \beta R_f) / Lt$; where τ_m = maximum shear stress, α, β , γ are coefficient may be obtained from prescribed graphs and R is relative flexural stiffness.

Green[18] also emphasized that for full composite action to develop between the wall and its supporting beam, the shear strength at beam-wall interface should be adequate to transfer the shear stress as a result of the arch action. $\tau_m = 4T/L$ where τ_m = shear stress, T = maximum tensile force in the beam.

2.3.5 Maximum Deflection

Deflection of composite beam is found to be very less than the supporting beam if the total load is uniformly distributed over the span of beam. Annamalai et.al[4] found that actual midspan deflection at service loads are about span/1485 for specimen made with wirecut bricks having compressive strength of 2175 psi and about span/2380 for special chamber bricks having compressive strength of 1015 psi. H/L ratio of test specimen was about 0.60 having a span of 4 ft. The load deflection behavior indicates that the failure takes place by crushing of masonry walls rather than by flexure. Burhouse [7] found experimentally, the maximum deflection of composite beam (H/L = 0.33) as span/840 at failure load. He tested 4 more beams having H/L ratio 0.58 to 0.83 of span 12 ft. and found that the deflection was proportional to the load and less than span/840. Rosenhauft[9] found experimentally the load-deflection pattern of composite wall-beam and shown in Fig.2.16. The deflection of beam sofit at mid span under load 2kg/sq.cm were found 0.25mm, 0.43mm, 0.52mm and 0.86mm respectively for composite wall-beam having H/L ratio 0.8, 0.63, 0.46 and 0.29.

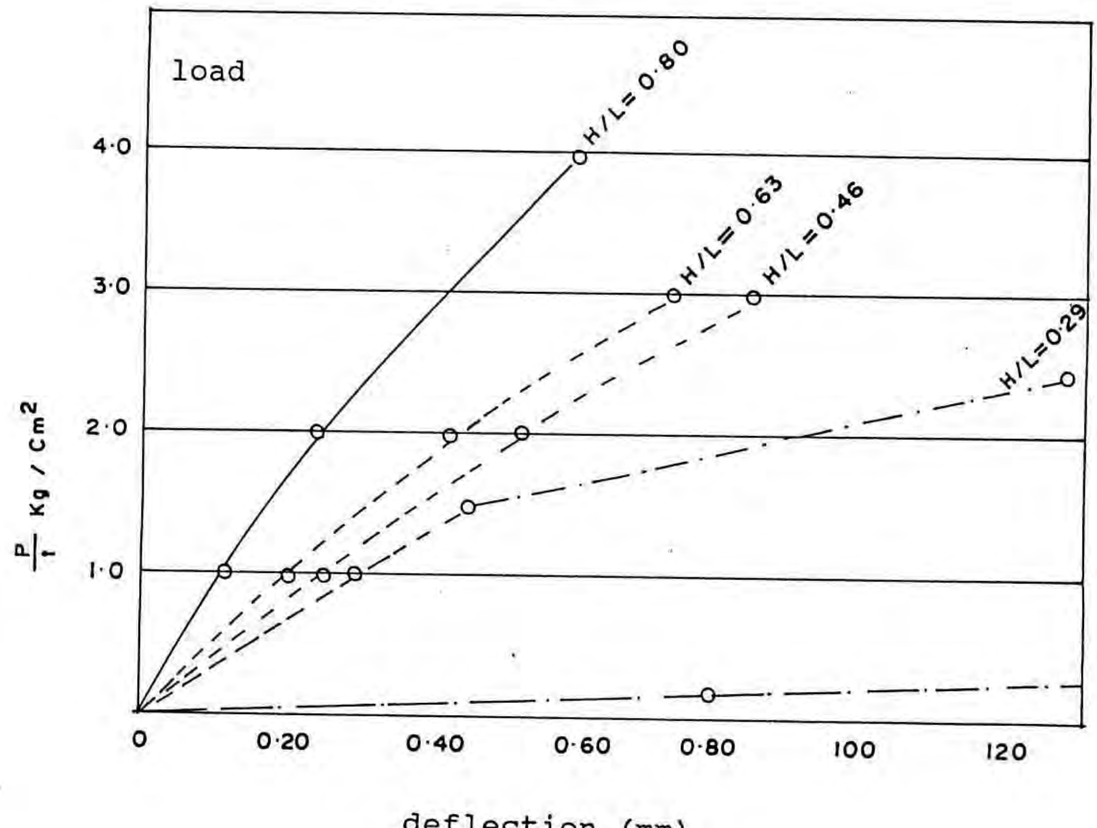


Fig.2.16 Load Deflection Curve for Different H/L ratio (Rosenhauft)[9]

2.4 Properties of Brick Masonry

The satisfactory performance of brick masonry depends upon the strength characteristics namely compressive strength, tensile strength and shear strength. Since the present investigation deals with the load bearing masonry, the deformation characteristics and the compressive strength under vertical compression is of prime importance.

2.4.1 Compressive Strength of Masonry

The compressive strength of masonry is an important parameter in the design of load bearing masonry structures. Under uniform load brick masonry fails due to vertical splitting. The essential mechanism of failure, which has been accepted is that the mortar is always weaker than the masonry units and it tends to be squeezed. This movement of mortar is restrained by the bricks which are then subjected to lateral tensile stress. The important factors influencing brick masonry strength are strength and geometry of brick unit, the strength of mortar, the joint thickness, the suction of the units and the water retention of mortar and the bonding pattern of brick work and the standard of workmanship.[23]

Most overseas codes have provision for determining brick masonry strength either from an approximate relationship between brick strength, mortar type and brick masonry strength or from compression test on stack bonded prisms. Based on a large number of tests on brick masonry and individual bricks the appropriate ratio of wall compression strength to brick compression strength is found to vary from 0.2 to 0.4, the lower values referring to the lower strength mortar and the higher value to the high strength mortar.[24] When a more exact estimate of compressive strength is required a prism test is done, as this nature of tests include the effects described above. Also the failure mode of the bricks in the prism is similar to that in the wall[25].

2.4.2. Other Properties of Brick Masonry

The allowable value for other properties such as, tensile, shear, modulus of elasticity, modulus of rigidity as per American National Building Code requirements for masonry. ANSI A41.1 and A41.2 has been tabulated in Table 2.1.[26]

2.5. Shear and Diagonal Tension in Reinforced Brick Masonry Beams

Shear and diagonal tension in reinforced brick masonry beams has been discussed considering beams without and with shear reinforcement in the following articles:

2.5.1. Beams Without Shear Reinforcement

Results of tests of fifty seven RBM beams without shear reinforcement developed by five research investigations were reviewed and analyzed by SCPI[27] to determine allowable shearing stresses in the 1969 SCPI standard for RBM beams without shear reinforcement.

The minimum shearing strengths of RBM beams without shear reinforcement:

$$\frac{V}{bd\sqrt{f_m'}} = 1.5 + \frac{3000pVd}{M\sqrt{f_m'}} \quad \ldots \quad 2.1$$

This formula was then divided by a factor of 3.0 to obtain the following design formula for computing allowable shear stresses (but not to exceed 50 psi);

$$v_m = 0.5\sqrt{f_m'} + \frac{1000pVd}{M} \quad \ldots \quad 2.2$$

The allowable shear stresses based on the above formula were computed for each beam and compared to the ultimate shearing stresses. With the exceptions of the beams having unfavorable brick bonding pattern already discussed, the ratios of ultimate to allowable shear stress were 2.1 and above.

2.5.2. With Shear Reinforcement

Withey, Schnaider[28]lch, and SCPI tested a total of twenty four RBM beams with varying amounts of shear reinforcement. The results of these tests were analyzed by SCPI[27] to investigate the strength of web reinforced RBM beams. Allowable shear stresses permitted in the masonry without shear reinforcement and allowable shear stresses based on the area of stirrups provided were computed and compared with the ultimate shear stresses obtained in these beam tests. The ratio of ultimate shear stress to allowable shear stress (based on stirrups provided) varied from 2.1 to 4.4 in eight of the ten beams which failed in diagonal tension. This ratio was less than 2.0 in the two other beams (belonging to Withey) which were reported to have been built with poor workmanship. In three of these beams flexural bars were terminated in the tension zone. Nevertheless, the ratio varied from 2.4 to 3.1 even though they did not contain additional stirrups as required by ACI Code.

2.6. Reinforced Concrete Deep Beams

Some concrete members have depth much greater than normal, in relation to their span, while the thickness in the perpendicular direction is much smaller than either span or depth. The main loads and reactions act in the plane of the member, and a state of plane stress in the concrete is approximated. Members of this type are called deep beams. They can be defined as beams having a ratio of span to depth L/D of about 5 or less or having a shear span a less than about twice the depth.[29,30,31,32]

Table 2.1 Allowable Stresses in Reinforced and Non-Reinforced Brick Masonry[26]

Description	Symbol	Non Reinforced		Reinforced
		Without inspection	With inspection	
1. Compressive, axial				
a. Walls h/t = 10 or less	f_m	$0.20\ f'_m$	$0.20\ f'_m$	$0.20\ f'_m$
h/t = 25 b. Column	f_m	$0.16\ f'_m$	$0.16\ f'_m$	$0.20\ f'_m$
2. Compressive, flexural				
a. Walls	f_m	$0.32\ f'_m$	$0.32\ f'_m$'	$0.33\ f'_m$
b. Column		$0.26\ f'_m$	$0.26\ f'_m$	
3. Tensile flexural				
a. Normal to bed joints				
M or S mortar	f_t	24	36	
N mortar	f_t	19	28	
b. Parallel to bed joints				
M or S mortar		48	72	
N Mortar		37	56	
4. Shear				50 psi beam with no web reinforcement
M or S mortar	V_m	$0.51\sqrt{f'_m} \ngtr 40$	$0.51\ f'_m \ngtr 80$	
N mortar		$0.51\sqrt{f'_m} \ngtr 28$	$0.51\ f'_m \ngtr 56$	150 psi beam with no web reinforcement
5. Modulus of elasticity	E_m	$1000\ f'_m \ngtr$ 2000,000 psi	$1000\ f'_m \ngtr$ 3000,000 psi	$1000\ f'_m$
Modulus of rigicity	E_v	$400\ f'_m \ngtr$ 8000,000 psi	$400\ f'_m \ngtr$ 1,2000,000 psi	$400\ f'_m$

Because of their proportions, they are likely to have strength controlled by shear. On the other hand, their shear strength is likely to be significantly greater than predicted by the usual equations, because of a special capacity to redistribute internal forces before failure, and to develop mechanisms of force transfer quite different from beams of common proportions.

In very deep beams, with span-depth ratios smaller than about 2, strains and stresses are no longer proportional to the distance from the neutral axis, even at low loads in the uncracked, elastic state. For this reason the flexural behavior of such beams, at low as well as high loads, is so different from that of ordinary beams that the normal strength design methods no longer apply. Because of their great depth, flexural strength itself rarely governs. It is important, however, to provide enough flexural reinforcement to prevent development of excessive tension cracks. Otherwise, appearance at service loads would be affected and the excessively long and wide cracks would also reduce the shear and bearing strength of the deep beam.

2.6.1 Code Provisions for Design of Reinforced Concrete Deep Beams

According to the ACI Code, special provisions for shear are to be applied to beams for which L/D is less than 5 and which are loaded at the top or compression face. If loads are applied at the sides or bottom of a member, design provision for ordinary beams apply.

The critical section for shear is to be taken a distance 0.15L from the face of supports for uniformly distributed loads and 0.5a for beams with concentrated loads, but not to exceed a distance d from the support face in either case. Shear reinforcement required by calculation or other Code provisions at the critical section is to be used throughout the span. For deep beams, the concrete contribution to shear strength can be computed from

$$V_c = (3.5-2.5\ M_u/V_ud)\ (1.9\ \sqrt{f'_c} + 2500pV_ud/M_u)bd\ < 6\ \sqrt{f'_c}$$

with the restrictions that the multiplier $(3.5-2.5\ M_u/V_ud)$ must not exceed 2.5 and that V_c must not be taken greater than $6\sqrt{f'_c}bd$.

The ACI code also provides that the minimum horizontal and vertical reinforcement distributed over the entire length and depth of the beam for control cracking and prevent the development of excessively wide and widely spaced cracks.

CHAPTER 3

LABORATORY INVESTIGATION

3.1 Introduction

An experimental program was designed to determine the behavior of wall-beam structure. Accordingly 9 wall-beams and 3 reinforced concrete beams have been made and tested in the laboratory. Since the wall-beam structure is a composite of brick masonry wall and reinforced concrete beam, the properties of bricks, cement mortar, mild steel bars have been tested to representative qualities. A Detail description of various investigations carried out are given here.

3.2 Experimental Scheme

The investigations in the laboratory were conducted in 3 phases. The first phase was concerned with general tests to determine the physical properties of the constituent materials viz. brick, cement, sand, coarse aggregate and reinforcing steel. Compressive strength of mortar, brick masonry prisms and concrete cylinder have also been done in this phase.

The second phase consisted of fabrication and casting of test specimens. This included cutting, bending and binding of mild steel bars to form the reinforcement cages, fabrication of steel and wooden form work, casting of reinforced concrete beam and laying of brick masonry wall on top of reinforced concrete beams along with standard concrete cylinders and brick masonry prisms. The test specimens (Wall-beam), concrete cylinder and brick masonry prisms have been cured for 28 days.

Twelve test beams were fabricated into four groups according to their H/L ratio, type of bond used in brickwork and presence of web reinforcement in brick work. The test beams of these four groups are designated as Group A through D and are shown in Figs. 3.1 to 3.4. Group A comprised of running bonded composite wall-beams whereas Group C and Group B are stack bonded composite wall-beams with or without vertical reinforcement in the wall. Group D included simple reinforced concrete beams, identical to the bottom beam of composite wall-beam. In each group of composite wall-beam there are three beams having different H/L ratio. In group D there are also three beams having identical section but different spans to match with the spans of composite wall-beams.

In the third phase all test beams were monotonically loaded without shock to failure and the relevant behavior of interest have been carefully observed and recorded.

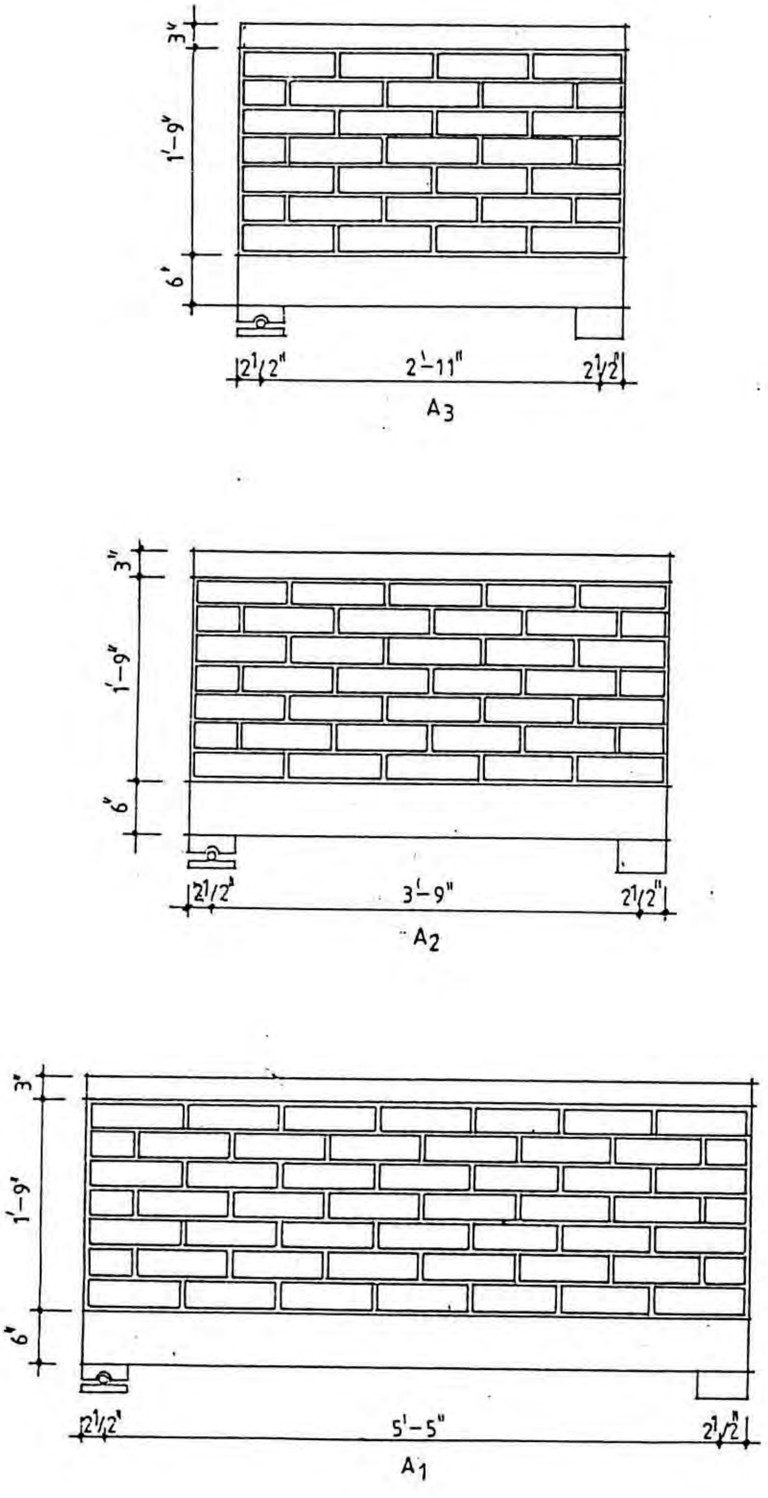
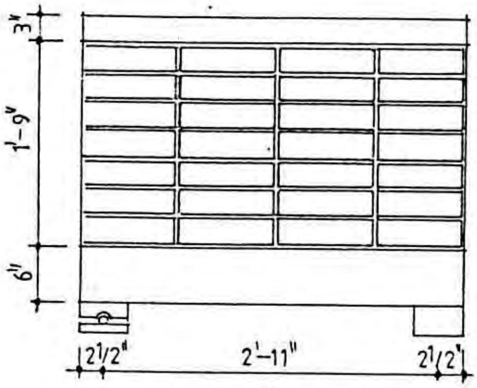
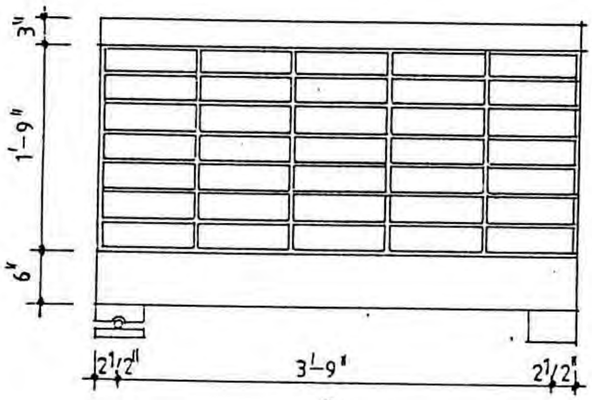


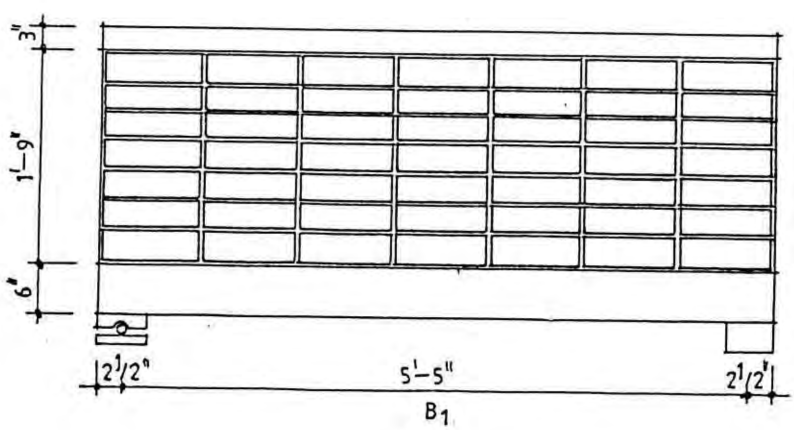
Fig.3.1 Running Bonded Composite Wall-Beam Group A



B3

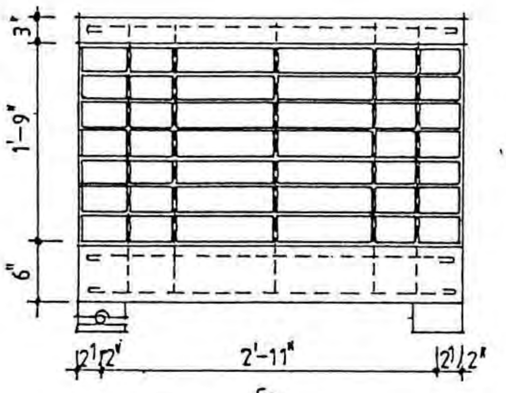


B2

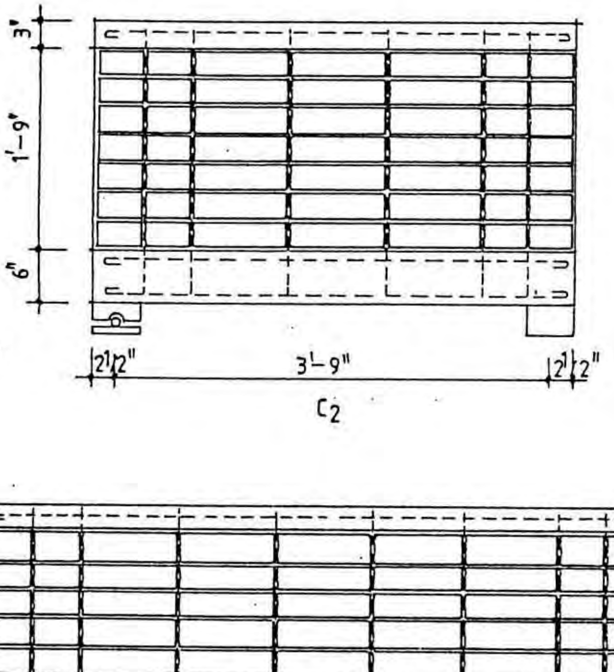


B1

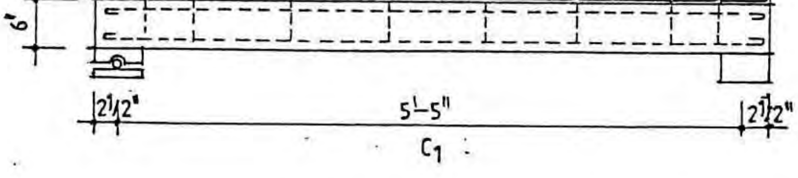
Fig.3.2 Stack Bonded Composite Wall-Beam Group B



C3



C2



C1

Fig.3.3 Stack Bonded Composite Wall-Beam Structure With Vertical Reinforcement in Wall Group C

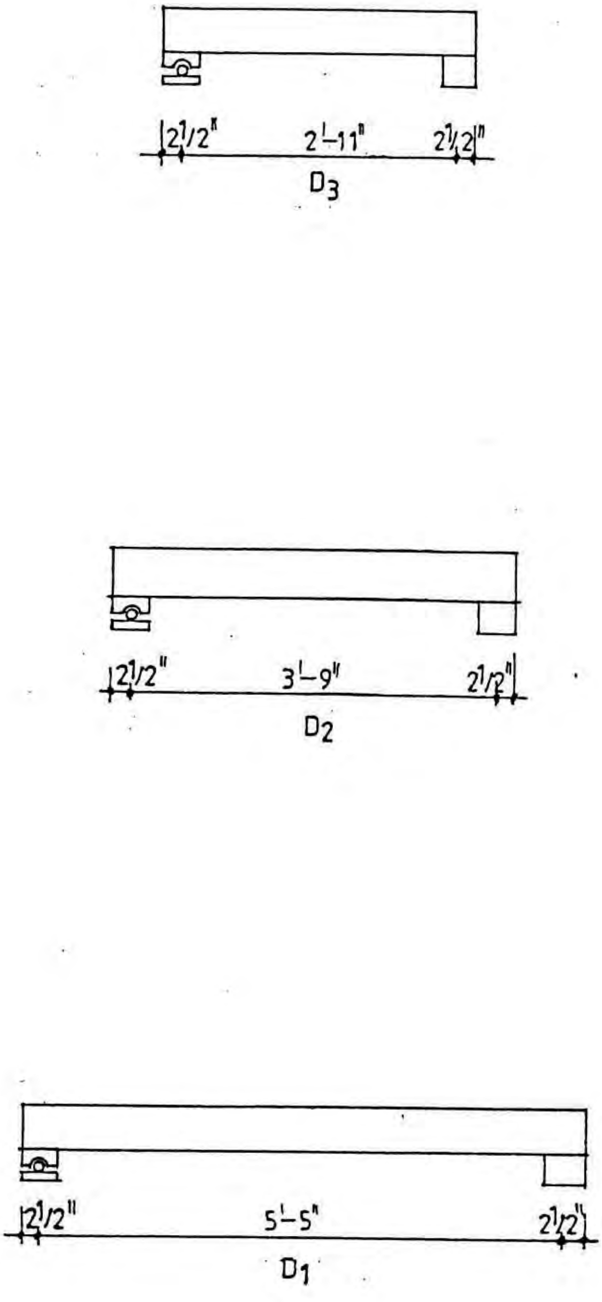


Fig.3.4 Reinforced Concrete Beam Group D

3.3 Properties of Brick

The clay burned bricks, used for all aspects of this investigation were procured from a local semi-automatic manufacturing plant. All bricks were from the same batch and stored in the laboratory till they were used in the preparation of the test specimen. Physical properties of bricks viz. size, weight, water absorption and crushing strength have been investigated according to ASTM designation C 67-77[33]. The findings are illustrated below.

3.3.1 Size

The nominal size of clay burnt brick used in this experiment was 9.5 in. X 4.5 in. X 2.75 in. For determination of actual size of brick, twelve bricks randomly collected from the stack of bricks in the laboratory procured for this purpose have been measured. The mean length, width and thickness have been found to be 9.54 in., 4.58 in. and 2.73 in. respectively. Mean observations along with standard deviation and the coefficient of variation are given in Table 3.1.

3.3.2 Weight

The bricks were dried at temperature 110° to 115° C in a ventilated oven for at least 24 hours or more until two consecutive weights taken at half an hour interval yielded almost identical values. Twelve bricks have been dried accordingly and the weight was measured. The average dry weight of the bricks have been found to be 7.64 lbs. The standard deviation and coefficient of variation are furnished in Table 3.1.

3.3.3 Water Absorption

Twelve bricks have been collected randomly from the stack in the laboratory for determination of water absorption of bricks used in this experiment as per ASTM standard[33]. The oven dry bricks were cooled and immersed in water for twenty four hours. The bricks were then weighed in a saturated surface dry condition to determine the water absorption of bricks. Mean water absorption have been found to be 14.7%.

3.3.4 Compressive Strength

Compressive strength of brick is an important property which has been traditionally used for brick quality as well as a parameter to define strength characteristics. The standard compression test involves loading the specimen between solid steel platen of the compression testing machine. For typical brick dimension this causes significant artificial strengthening due to aspect ratio effects. To obtain true compressive strength, the effect of platen should be accounted for. However for determination of compressive strength of brick the standard test method ASTM C67[33] have been followed. For the test, twelve bricks were selected at random from the stack. Each brick was divided into two halves with the help of cutting saw. One half of the brick was taken and neat cement paste was used on both faces to fill the frog mark and surface flaws. Thin sulfur capping was used on both the surfaces on top of neat cement finish. Accurate level of the capped surfaces was maintained using spirit level. Test was performed in the compression testing machine of the concrete laboratory. Test load was applied at a rate of 15 tons per minute. All the specimens failed by crushing. The mean compressive strength have been found to be 3195 psi with a coefficient of variation of 21.8 percent. (Table 3.1).

3.3.5 Tensile Strength

Tensile strength of brick is of great importance in defining the behavior of brick masonry as final failures originating in the brick are due to some form of biaxial tension developed in it. Direct tensile strength test are difficult to perform on brittle materials.

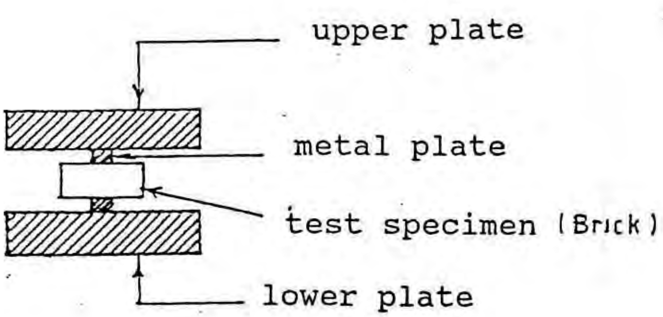
The indirect tensile strength of a homogeneous prism as suggested by Thomas and O' Leary[34] as a more convenient alternative to the use of cylinders can be obtained by the equation $T = 0.648P/dL$ where P = applied load, d = equivalent diameter, and L = length. This equation was verified by Ali[35] and the stress distribution pattern is shown in Fig.3.5(b). The test was modeled using a two dimensional linear elastic plane stress finite element analysis. A very fine mesh was provided near the loading point. The load was applied through a steel strip whose width was 10% of the width of the specimen as shown in Fig. 3.5(a).

A total of 12 randomly selected dry bricks from the stack were tested. The load was applied through a steel plate 0.45 in. wide and 0.2 in. high. The plate width was therefore 10% of the width of the specimen. The load was applied at a rate of 2 Tons per minute. Failure occurred by vertical splitting directly beneath the loading plate. The mean tensile strength have been furnished in Table 3.1.

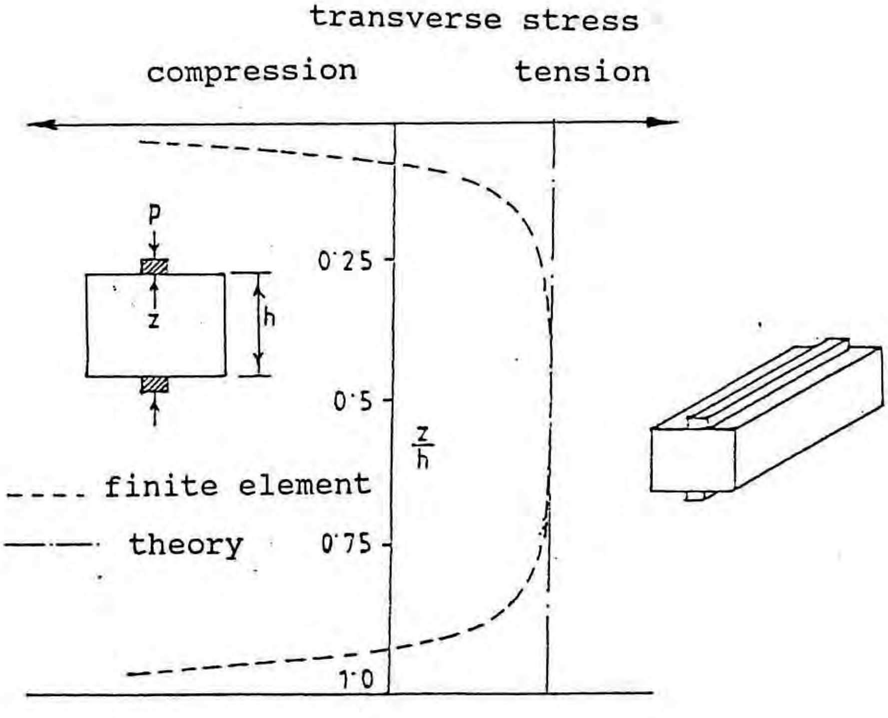
Table 3.1 Physical Properties of Brick

Physical properties		Statistical mean of 12 specimen	Standard deviation	COV in (%)
Size in inches	Length	9.53	0.10	1.04
	Width	4.58	0.10	2.18
	Thickness	2.74	0.08	2.93
Weight in lbs.		7.59	0.08	1.05
Water absorption in(%)		14.97	1.53	9.50
Crushing strength in psi		3195	669	21.8
Indirect tensile strength in psi		130	14.5	11.0

Note: COV = coefficient of varience



Testing Arrangement.



Transverse stress distribution along the centre line after S. Ali

Fig.3.5 Tensile Testing of Brick along with Stress Distribution

3.4 Properties of Cement

Cement in its broadest term means any substance which acts as a binding agent for materials. Cement is used as binding material in concrete and mortar. The properties of concrete or mortar largely depend upon the quality of cement used. So the test of quality of cement is of utmost importance. The quality of cement can be tested in the laboratory by determining -

i) Normal consistency

ii) Setting time.

iii) Compressive strength

iv) Tensile strength

v) Unit weight and specific gravity.

Twenty bags ordinary portland cement were collected from the university (BUET) store for this study. Out of these samples, 3 bags were chosen randomly and the samples were arbitrarily designated as 1, 2 and 3. Respective ASTM standard test procedures[36] were followed to determine the above mentioned properties of cement.

3.4.1 Normal Consistency

Determination of normal consistency of cement means the determination of amount of water required to prepare cement pastes for other tests. It is the amount of water when the 10 mm diameter Standard Plunger settles to a point 10 ± 1 mm below the original surface of cement paste in 30 seconds after being released. This tests has been performed as per ASTM designation C 187-83[36] and the result is given in Table 3.2.

3.4.2 Time of Setting

The principal aim of this test is to make a distinction between normal and quick setting cement and also detect the degree of deterioration in cement due to storage. Test specimen for determination of time of setting has been prepared with cement and water required for normal consistency. Both initial and final setting time have been determined by vicat needle as per ASTM designation C 191-82.[36] Initial time of setting is the time required to penetrate 25 mm by a 1 mm diameter needle and when the collar encircling the needle does not leave any impression on the cement paste determines the final time of setting. The results of setting time tests have been given in Table 3.2.

3.4.3 Compressive Strength

The main object of this test is to study the quality of cement about its compressive strength. Compressive strength of cement is determined by crushing standard cubes made of cement sand mortar. Test specimen (2 in. cube cement mortar) have been prepared with one part of cement to 2.75 parts of graded standard sand (Ottowa sand) by weight. Amount of water used to prepare the mix is such that a water-cement ratio of

0.485 is maintained. Preparation of the test specimen and also the tests have been performed as per ASTM designation C 109-80[36]. Test results of this investigation have been given in Table 3.2.

3.4.4 Tensile Strength

The tensile strength of cement is estimated by determining the tensile strength of cement sand mortar. Determination of the tensile strength of cement mortars employing the briquette specimens were done as per ASTM designation C 190-82[33]. Test specimens for tensile strength of cement have been prepared with one part of cement to 3 parts of standard sand (Ottowa sand) by weight. Preparation of test specimens and also the tests have been carried out as per ASTM standard. The results are given in Table 3.2.

3.5 Properties of Aggregate

Depending upon their size, the aggregates are classified as- i) Coarse aggregate, and ii) Fine aggregate.

3.5.1 Coarse Aggregate

Crushed hard stone and gravel are the common materials used as coarse aggregate. But in this investigation manually crushed first class bricks were used as the coarse aggregate for concrete. The brick chips passing through 3/4 in. sieve and retained on No.4 sieve were stored separately for use. Gradation, fineness modulus, water absorption, unit weight and Bulk specific gravity of the coarse Aggregates have been determined as per ASTM standard and shown in Tables 3.3 and 3.4.

Table 3.2 Physical Properties of Cement

Physical properties		Sample-1	Sample-2	Sample-3
Normal consistency		22%	21%	22%
Setting time	Initial	2 hr. 30 mins.	2 hr. 31 mins	2 hr. 30 mins
	Final	3 hr. 45 mins.	3 hr. 43 mins	3 hr. 46 mins
Average compressive strength	3 days curing	1292	1305	1280
	7 days curing	1729	1746	1702
	28 days curing	2563	2588	2523
Tensile strength	3 days curing	103	105	106
	7 days curing	140	145	140
	28 days curing	205	215	210

3.5.2 Fine Aggregate

Sand is commonly used as fine aggregate. Ordinary Sylhet sand passing No.4 sieve were used as fine aggregate in this experiment. Gradation, fineness modulus, water absorption specific gravity and unit weight were determined as per ASTM recommendations and given in Tables 3.3 and 3.4.

Table 3.3 Gradation of Aggregates

Sieve size	Cumulative percentage retained for		
	Coarse aggre (C)	Fine aggre. (F)	Combined aggre. (C:F=2:1)
1"	0	0	0
3/4"dia	0	0	0
3/8"dia	62.5	0	42.84
No. 4	95.0	0	65.12
No. 8	100.0	2.08	69.20
No. 16	100.0	12.88	72.60
No. 30	100.0	52.36	85.02
No. 50	100.0	91.40	97.30
No.100	100.0	97.70	98.28
Fineness modulus	6.58	2.56	5.31

3.6 Properties of Reinforcement (M.S. Bar)

Mild steel plain rods of 3/8 inch and 1/4 inch nominal diameter were used as the main flexural and web reinforcement respectively. The rods procured from the local market were slightly undersized and the actual area of these rods were used for computations. The rods were tested as per ASTM standard specification A 615 - 84a[37] to determine the yield strength, ultimate tensile strength, percent elongation and modulus of elasticity. Test results on the mechanical properties of the reinforcements are given in Table 3.5.

Table 3.4 Physical Properties of Aggregates

Physical property	Coarse aggregate (Brick chips)	Fine aggregate (Sylhet sand)
Fineness modulus	6.58	2.56
Unit weight (loose)	76.00	83.00
Unit weight (SSD, Compacted)	84.00	94.00
Bulk specific gravity	2.05	2.44
Absorption in % of dry weight	8.74	2.97

3.6.1 Yield Strength of M.S. Bar

Yield strength is the stress level in a material, less than the maximum obtainable stress at which significant increase in strain occurs without any appreciable increase in stress. Yield strength is intended for application only for materials like mild steel that exhibit the unique characteristic of yielding. Yield stress has been determined by dividing the load at yielding by the original cross-sectional area of the specimen. Test results are given in Table 3.5.

3.6.2 Tensile Strength

The tensile strength of M.S. rod was determined by dividing the maximum load the specimen sustained during a tension test by the original cross-sectional area of the specimen.

3.6.3 Elongation

The percent elongation is the increase in length of the gage length at failure, expressed as a percentage of the original gage length. The fractured specimen were fitted together carefully and the distance between gage marks were measured for gage lengths of 2 in. and 8 in. Test results are given in Table 3.5.

3.6.4 Modulus of Elasticity

The Modulus of Elasticity of mild steel was not determined in the laboratory. Because the variation of modulus of elasticity is very negligible for mild steel. It is assumed as 29 x 10^6 psi.

3.7 Properties of Masonry Prism

The most important property of the masonry prisms are its compressive strength. The standard procedure for determining this is described below.

Table 3.5 Physical properties of Mild Steel Reinforcement

Specimen Number	Actual		Yield	Ultimate	% elongation in	
	diameter (in.)	area $(in.)^2$	strength (Psi)	strenght (Psi)	2" G.L	8"G.L.
A. 3/8" inch nominal diameter M.S. bar.						
1	0.355	0.098	45,918	62,245	43	29.5
2	0.357	0.100	44,000	61,000	42	28.5
3	0.351	0.096	45,833	62,500	43	28.5
Average	0.354	0.098	45,250	61,915	42.66	28.83
B. 1/4 inch nominal diameter M.S. bar.						
1	0.2516	0.0497	42,640	74,420	32.00	-
2	0.2547	0.0510	43,137	73,405	33.00	-
3	0.2526	0.050	43,100	74,000	30.00	-
Average	0.2526	0.05	43,100	74,000	32.00	-

3.7.1 Compressive Strength

For determining the compressive strength of masonry, brick prisms are built at the job site with the same materials and workmanship that is to be used or being used in a particular structure. The specimens are short compression prisms. The thickness of prisms shall be the same as the thickness of the unit of the wall in the structure. The height to thickness ratio of prism shall neither be less than two nor more than five. The mortar joints shall be 10 mm thick and the mortar spread over full bed comprising each solid masonry unit and allow no furrowing of the mortar bed. The length of prisms shall be equal to or greater than the thickness of prism. The height of prism shall be at least twice the thickness containing at least two mortar joints and be a minimum of 380 mm (15 in.). Six brick prism specimens with 5 brick high (15 in.) and single Wyeth have been prepared and tested in first phase of investigation that is before fabrication of wall-beam structure. The result of these six specimens have been tabulated in Table 3.6 and marked as Group P1.

In the second phase three brick prisms in each group have been built at the time of construction of wall-beam structures with the same materials and workmanship. The prisms have been grouped as Group P2, P3, P4 corresponding to test beams of Groups A, B, C respectively. These prisms have been tested during the test of wall-beam structure after proper curing. The results have been tabulated in Table 3.6. Fabrication of masonry prisms and test were performed as per ASTM designation E447-84[38].

Table 3.6 Compressive Strength of Brick Prism

Specimen group	Number of specimen tested	Mean strength in psi	Standard deviation in psi	Coefficient of variation
At first visible crack				
P-1	6	866	90.36	10.90%
P-2	3	850	90.25	10.60%
P-3	3	830	87.50	10.54%
P-4	3	895	89.36	9.98%
At Ultimate stage				
P-1	6	1099	145.48	13.24%
P-2	3	1050	140.00	13.33%
P-3	3	1066	153.00	14.35%
P-4	3	1038	135.00	13.00%

The average ultimate compressive strength of brick prism (f"m) is found to be 1066 psi. The compressive strength of brick prism (f'm) according to the American Standard will be 0.9 x f"m which is equivalent to 959 psi. The ratio of compressive strength of brick prism to individual brick is found to be 0.30 from Table 3.1 and 3.6 which is in between the approximate ratio from 0.20 to 0.4.[24]. The testing arrangement for prism is shown in Fig. 3.6.

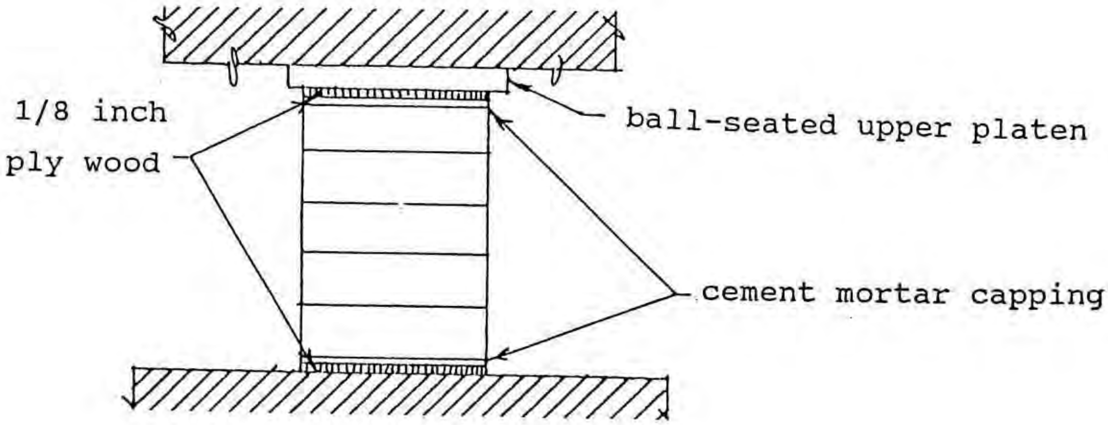


Fig.3.6 Testing Arrangement for Prism

3.8 Properties of cement mortar

The cement mortar used throughout the investigation was 1:4 (cement:sand) by volume. All mix components were stored in the laboratory for the duration of the testing program. The variation of bulk density in materials were eliminated by weighing measured volume of fresh materials and thereafter batching by weight. The cement used in this investigation was "Assam Bengal" and the sand was "Sylhet" sand. The properties of cement and sand were determined according to appropriate ASTM standards and are listed in Table 3.2 and 3.4

respectively. The compressive and tensile strength of cement mortar have also been determined in a way which is described here.

3.8.1 Compressive Strength

Standard test for compressive strength of cement mortar was performed in order to check the quality control of test specimens adopted in this investigation. The compressive strength of mortar has been determined from uniaxial compression test of 2 inch mortar cube prepared and tested according to ASTM C 109[36].

Initially, six 2 in. mortar cubes were prepared using cement and sand designated for use in the main test series. Compressive strength have been determined after curing for 28 days and are designated as Group MC-1. Three 2 in. cement mortar cubes were prepared during the construction of wall-beam structures of each Groups A, B, and C. These are designated as Group MC-2,MC-3, and MC-4 respectively. Average compressive strength of cement mortar for different groups have been shown in Table 3.7.

3.8.2 Tensile Strength

Tensile failure of masonry can occur either as a tensile bond failure at the brick-mortar interface, or as a tensile failure in the constituent materials. However, the second type of failure is more common in stack bonded prism. The tensile strength of mortar has been determined by making mortar briquette. Six briquettes designated as Group MT-1 in the first phase were prepared. In the second phase three briquettes accompanying the construction of wall-beam structures of Group A, B, and C have been prepared. These were grouped as Group MT-2, MT-3 and MT-4. Test results have been given in Table 3.7.

3.9 Properties of Concrete

The compressive strength (f_c'), Modulus of Elasticity (E_c) and modulus of rupture (f_r) are the most important properties of concrete relating to this study. The compressive strength of concrete has been determined in the laboratory as per ASTM standard. The other two properties were not determined experimentally in the laboratory rather the values were estimated using empirical relations as suggested by ACI.

The mix proportion of cement, sand and coarse aggregate in concrete used in this study were 1:2:4 by volume. 'Assam Bengal" brand ordinary portland cement type-1 was used throughout the test programe crushed brick aggregates and sand were used as coarse and fine aggregate in concrete respectively. Water cement ratio of the mix was 0.45 and slump of 1.5 in. was recorded. The physical properties of aggregate are tabulated in Tables 3.3 and 3.4.

Table 3.7 Strength of Cement Mortar

Specimen group	Number of specimen tested	Mean strength in psi	Standard deviation	Coefficient of variation
Compressive strength				
MC-1	6	1048 psi	93.90	8.96
MC-2	3	1065 psi	85.30	8.00
MC-3	3	1090 psi	92.53	8.49
MC-4	3	1010 psi	90.35	8.95
Tensile strength				
MT-1	6	135 psi	10.48	7.76
MT-2	3	143 psi	6.36	4.44
MT-3	3	140 psi	7.80	5.57
MT-4	3	137 psi	9.50	6.93

MC = Mortar in Compression.
MT = Mortar in Tension.

3.9.1 Compressive Strength

During the casting of bottom reinforced concrete beam of composite wall-beam structure, three concrete cylinders were cast as per ASTM standard with the same batch of concrete for determination of compressive strength. The cylinders were tested after 28 days curing and the test results are tabulated in the Table 3.8.

3.9.2 Modulus of Elasticity (E_c)

It is the slope of the initial straight portion of the stress-strain curve of concrete. ACI code suggested empirical equation for E_c by correlating the Weight (W_c) and the compressive strength of Concrete (f_c'). Modulus of elasticity of concrete E_c is assumed as 57000 $\sqrt{f_c'}$ as suggested by ACI for normal concrete with W_c equal to 145 Pcf.[30]

Table 3.8 Compressive strength of concrete

Test specimen Number	Proportion of Mix by volume	W/C ratio	Comp. strength in Psi	Av.Comp. strength in Psi
1	1:2:4	0.45	2200	
2	1:2:4	0.45	1950	2050
3	1:2:4	0.45	2000	

Note: comp. = compressive

3.9.3 Modulus of Rupture

The computed flexural tensile stress at which a test beam of plain concrete fractures, termed as modulus of rupture f_r. Better correlation is established between the modulus of rupture and the square root of compressive strength. The approximate range of modulus of rupture is 8 to 12 $\sqrt{f_c'}$. In this expression, compressive strength f_c' is expressed in psi units and the resulting modulus of rupture is obtained in psi. In this study we assume the value of modulus of rupture f_r as $10\sqrt{f_c'}$, the average value of the approximate range.

3.10 Workmanship

To maintain the uniform and satisfactory workmanship in brick masonry works and reinforced concrete beam casting, experienced masons and helpers were engaged who were not changed during the entire construction phase. Workmanship has considerable influence over the strength of brick work[24,39,40]. So special instructions were given to them to fill the mortar joints completely to maintain uniform and specified mortar bed thickness and not to attempt any correction of mistake in brick laying after the mortar had set. It may be mentioned here that the workmanship of the employed masons were found reasonably uniform and satisfactory.

3.11 The Test Specimens (Beams)

In total 12 beams were cast. The beams were divided into four Groups as A, B, C, and D. Each group consisted of 3 beams. In Group A, B, C, brick masonry wall were supported over reinforced concrete beam and are called composite wall-beam. Whereas, group D was reinforced concrete beams without brick wall. The size and reinforcement of reinforced concrete beams were same in all the groups. Three different effective spans were used in each group. Group A was the running bonded composite wall-beam, whereas Group-C and B, were the stack bonded composite wall-beam with or without vertical reinforcement in the wall respectively. The width of brick wall and supporting beams were 4.5 in. and 5 in. respectively for all the three groups. The depth of reinforced concrete beams were 6 in. over which 7 layer of bricks were layed and 3 in. concrete were cast on top of brick wall. The total height (H) of composite beams were 2 ft. 6 in. Three effective spans (L) of the beams were 5 ft. 5 in., 3 ft. 9 in. and 2 ft. 11 in. whereas the total length were 5 ft. 10 in., 4 ft. 2 in. and 3 ft. 4 in. Hence, H/L ratio were 0.46, 0.67 and 0.86 respectively for the 3 different effective spans just mentioned. Identification and physical dimensions of test beams has been shown in Table 3.9.

3.11.1 Fabrication

Preparing the steel mould, cutting and binding of mild steel rods, casting of reinforced concrete beam, laying bricks on reinforced concrete beam and curing were all included in fabrication of test beams. Steel mould of 5 in.x 6 in. x 5 ft. 10 in. was fabricated and the length of mould could be varied from 5 ft. 10 in. to 3 ft. 4 in. by shifting the end plate at appropriate locations with the help of nut and bolts. The longitudinal and web reinforcement have been fabricated as per design. The total longitudinal reinforcement of supporting beams were four 3/8 in. dia mild steel rods placed one at each corner. The stirrups were 1/4 in. dia mild steel wire spaced at 6 inch intervals. Vertical reinforcement in brick wall of Group C were 1/4 in. dia mild steel wire placed at each vertical joint and two 1/4 in. horizontal reinforcement used as hanger for the vertical steel was placed within top 3 inch concrete.

Table 3.9 Identification and Physical Dimensions of Test Beams

Gr. No.	Description of beams	Beam No.	Total ht.i/c RC beam (H)	Total depth of RC beam (D)	Eff. span of beam (L)	Wall thick. (t)	H/L ratio
A	Running bonded masonry wall without web reinforcement in wall over RC beam	A_1	2'-6"	6"	5'-5"	4.5"	0.46
		A_2	2'-6"	6"	3'-9"	4.5"	0.67
		A_3	2'-6"	6"	2'-11"	4.5"	0.86
B	Stack bonded masonry wall without web reinforcement in the wall over RC beam	B_1	2'-6"	6"	5'-5"	4.5"	0.46
		B_2	2'-6"	6"	3'-9"	4.5"	0.67
		B_3	2'-6"	6"	2'-11"	4.5"	0.86
C	Stack bonded masonry wall with web reinforcement in the wall over RC beam	C_1	2'-6"	6"	5'-5"	4.5"	0.46
		C_2	2'-6"	6"	3'-9"	4.5"	0.67
		C_3	2'-6"	6"	2'-11"	4.5"	0.86
D	Reinforcement concrete beam without wall over it	D_1	0'-6"	6"	5'-5"	4.5"	0.092
		D_2	0'-6"	6"	3'-9"	4.5"	0.133
		D_3	0'-6"	6"	2'-11"	4.5"	0.171

After fabrication and placing of mild steel bar on the mould maintaining proper clear cover in all sides, concrete were poured in the mould as per design proportion and compaction was done by a vibrator. Just after casting of reinforced concrete beam, one layer of brick was layed on top of reinforced concrete beam to get higher interface bond between beam and brickwork. These were embedded in to the concrete beam by about 1/2 in. to 3/4 in. by gentle hammering. It may be mentioned here that half bricks required in the work were obtained by cutting whole size bricks into two equal halves with electrically operated cutting saw. Bricks were immersed in water for about 30 minutes before they were layed.

The cement to sand proportion of mortar was 1:4 by volume. Water cement ratio was strictly maintained at fixed value both for concrete and mortar preparation throughout the investigation. Photographs of composite wall-beams are shown in Fig.3.7.

Three brick prisms were cast along with test specimen each group but only three concrete cylinders were cast for the whole lot using of the same materials used in the test specimen.

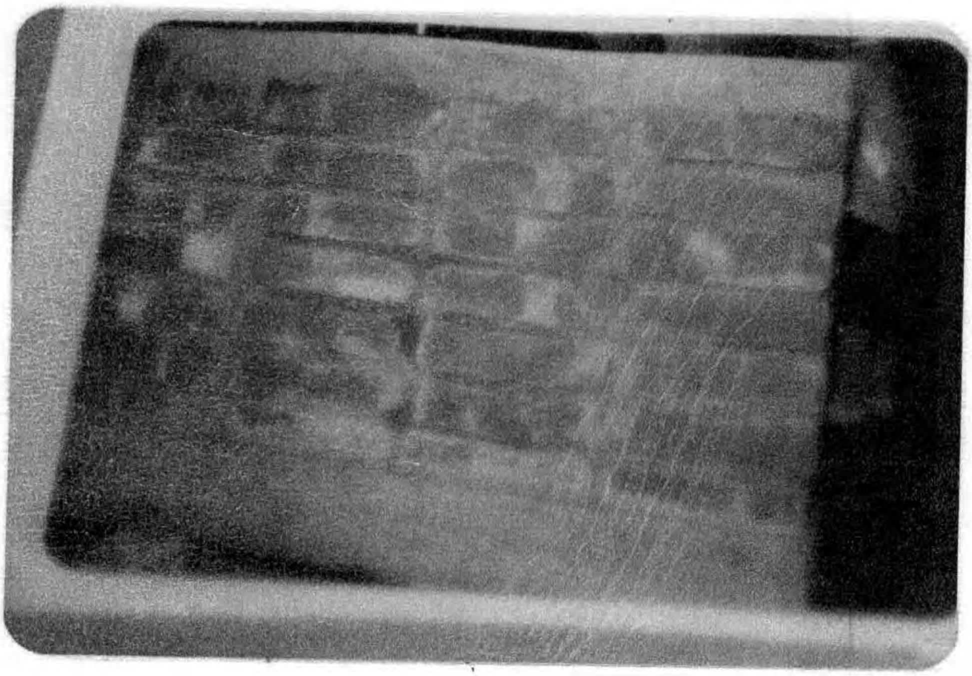


Fig.3.7 Composite Wall-Beam Before Test

3.11.2 Testing of Beams

Testing of beams were carried out in the structure laboratory, of the Department of Civil Engineering, BUET.

Arrangement of Testing Machine

The facilities of testing machine has been slightly modified for accommodating the test specimens. The size of platform and load transfer device of the machine could not accommodate the beam of span 5 ft. 10 in. Therefore, these facilities has been accomplished by attaching heavy steel I-joist both to the base platform and the load transfer end as shown in Fig.3.12. As a result, the effective gap between the two ends of the machine was reduced. This restricted the maximum permissible height of composite wall-beam structure to 2 ft. 6 in.

Carrying, Hoisting and Placing of Beam

The test beams were constructed at the concrete laboratory of the department of Civil Engineering, BUET, Dhaka. Beams were carried to the testing machine by trolley and then hoisted by chain pulley arrangement (Fig.3.8) on the testing platform. The hoisting tripod and the chain pulley systems have been designed and fabricated specially for this investigation. The beam was placed at one end on the steel joist over a number of steel round bar placed on top of the joist. The beam was then rolled in and placed at appropriate location by pushing it as shown in Fig.3.9.

Support Condition of Beam

One end of the beam was placed over a 5 in. square by 3 in. thick steel block. 1 in. dia guided steel rollers were placed under the supporting steel block. The other end rested on a steel block that was flatly placed on steel I-joist. The beams were placed over the supports by manual labor with shovel (flat ended large dia M.S. bar) shown in Fig.3.10.

Deflection Measurement

For determination of the maximum deflection of the beam, a deflectometer was placed at the center of the beam in between the bottom of beam and top of joist as shown in Figs. 3.11 and 3.12. Deflectometer constant was 0.001 in.

Loading Arrangement

The beams were tested under third point concentrated loading system. The load was applied on the top surface of the beams through two steel plates placed over the beam at a distance L/3 from the center of either supports as shown in Fig. 3.11. The size of the plate was 5 in. by 4 in. and 1.5 in. thick. Typical Testing arrangement for composite wall-beam and reinforced concrete beam is shown in Figs.3.12 and 3.13 respectively.

Fig.3.8 Hoisting of Composite Wall-Beam by Chain Pulley on Testing Platform

Fig.3.9 Placing of Wall-Beam on Testing Platform by Pushing

Fig.3.10 Placing of Composite Wall-Beam Over the
Support by Shovel

Fig.3.11 Composite Wall-Beam (B_1) is Ready for Testing

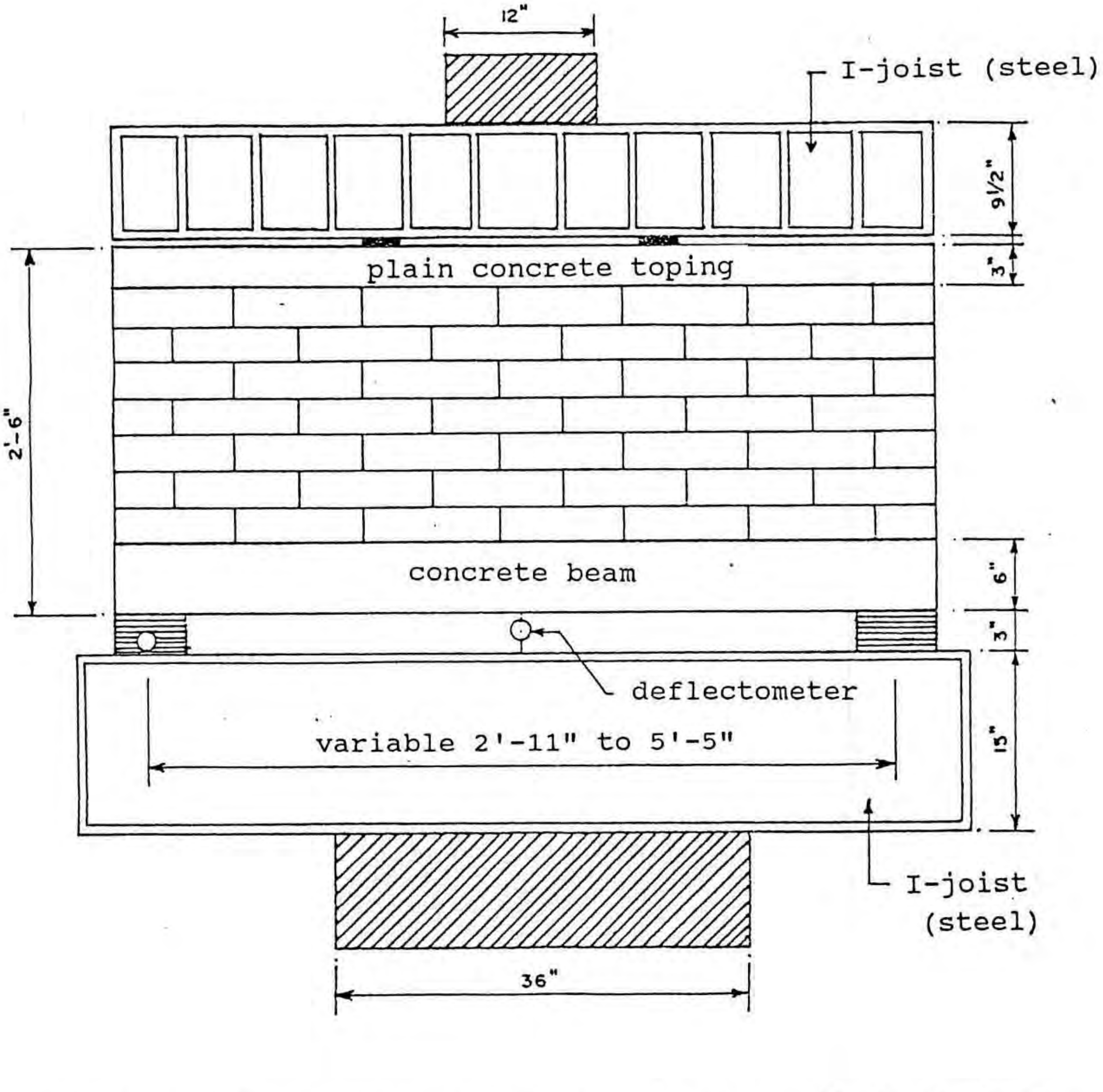


Fig.3.12 Typical Testing Arrangement for Composite Wall-Beam

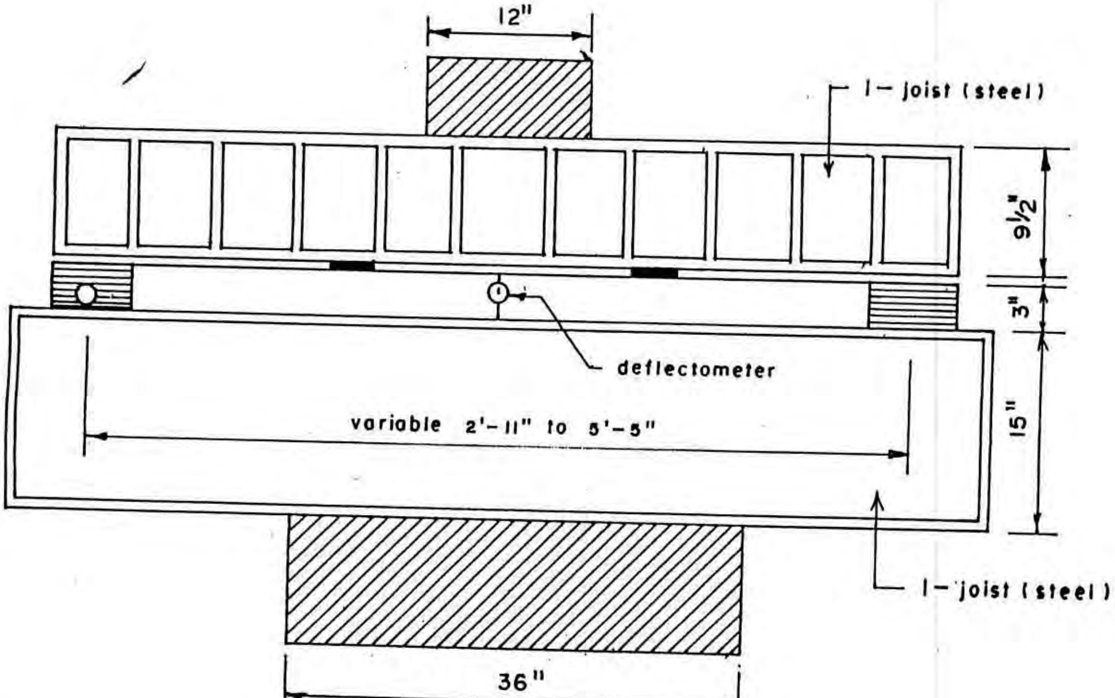


Fig.3.13 Typical Testing Arrangement for Reinforced Concrete Beam

CHAPTER 4

TEST RESULTS

4.1 Introduction

The main objective of this investigation is to study the behavior of wall-beam structure experimentally. An experimental scheme of testing wall-beams have been taken and accordingly carried out. The physical and mechanical properties of bricks, sand, cement, M.S. bar, coarse aggregate, brick prism and concrete cylinder have been investigated in the laboratory. These results have been provided in Chapter 3. In this chapter only test results of composite wall-beams and that of reinforced concrete beams are presented.

4.2 Testing program

According to the testing program nine composite wall-beams of Group A, B and C, and three reinforced concrete beams of Group D have been tested.These groups are -

Group A : running bonded brick masonry wall supported over reinforced concrete beam without vertical reinforcement in the wall shown in Fig.3.1.

Group B : stack bonded brick masonry wall supported over reinforced concrete beam without vertical reinforcement in the wall shown in Fig.3.2.

Group C : stack bonded masonry wall with vertical reinforcement in the wall shown in Fig.3.3.

Group D : there was no brick wall on the bottom RC beam. (Fig.3.4)

In each group, there were three beams with effective span 5 ft. 5 in, 3 ft. 9 in, and 2 ft. 11 in. The total height of all the composite wall-beams were 2 ft. 6 in. and the size of typical supporting (bottom) reinforced concrete beam was 5 in. by 6 in. All three beams of each group were tested on the same day. Typical arrangement of testing beams is shown in Figs. 3.12 and 3.13. During tests the deflection at mid span, mode of failure and failure loads have been carefully observed and recorded.

4.3 Load-Deflection Record

A deflectometer graduated in 0.001 in. per division has been placed at the center under the test beam. Deflections at mid span have been recorded at a regular interval of load both for the composite wall-beam and the bottom reinforced concrete beams as well.

4.3.1 Load-Deflection at Mid span of Composite Wall-Beam

Deflections of composite wall-beams have been recorded at an interval of 2 kips load and are given in Tables A_1 through A_9 in Appendix A. Load-deflection curves are plotted using these records and are shown in Figs.4.1, 4.2 and 4.3.

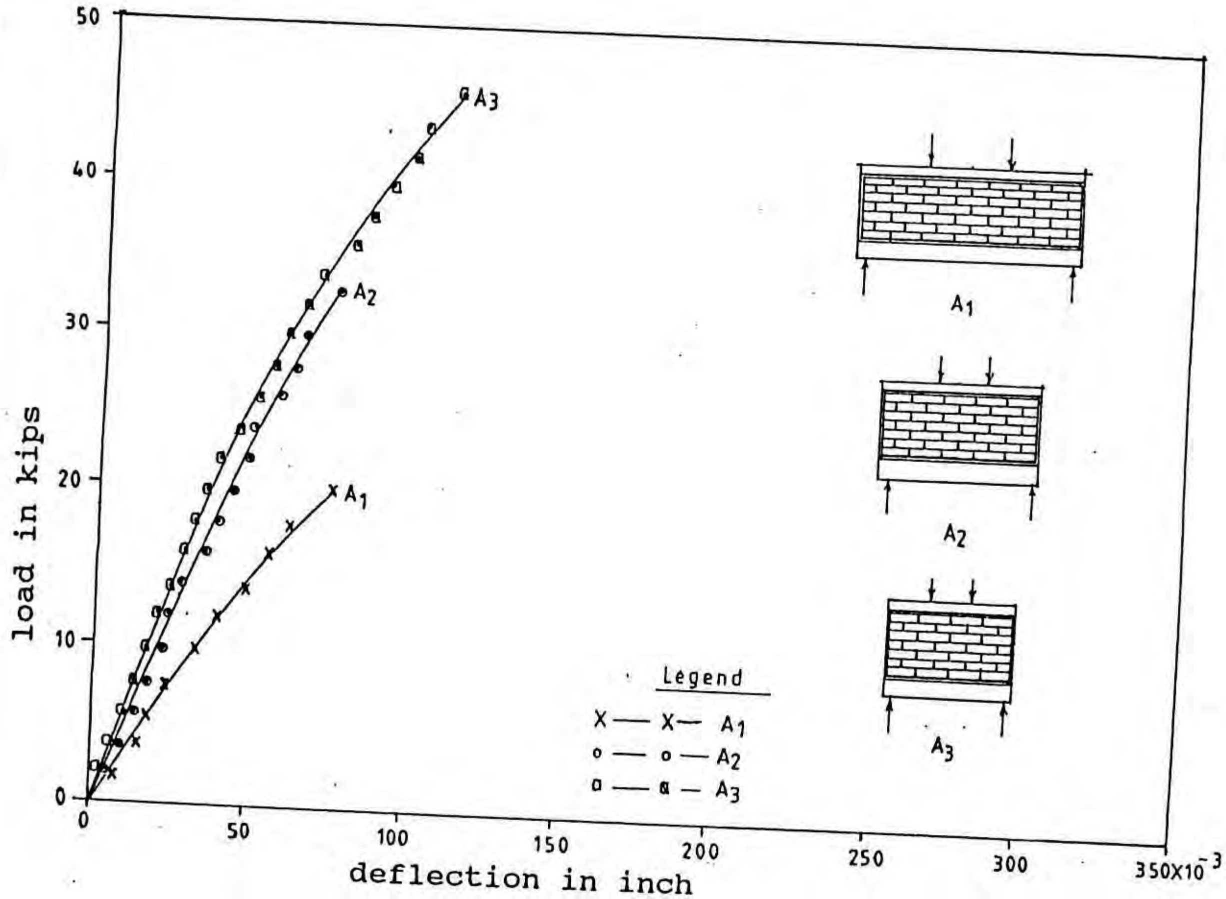


Fig.4.1 Load-Deflection Curve of Composite Wall-Beam Group A

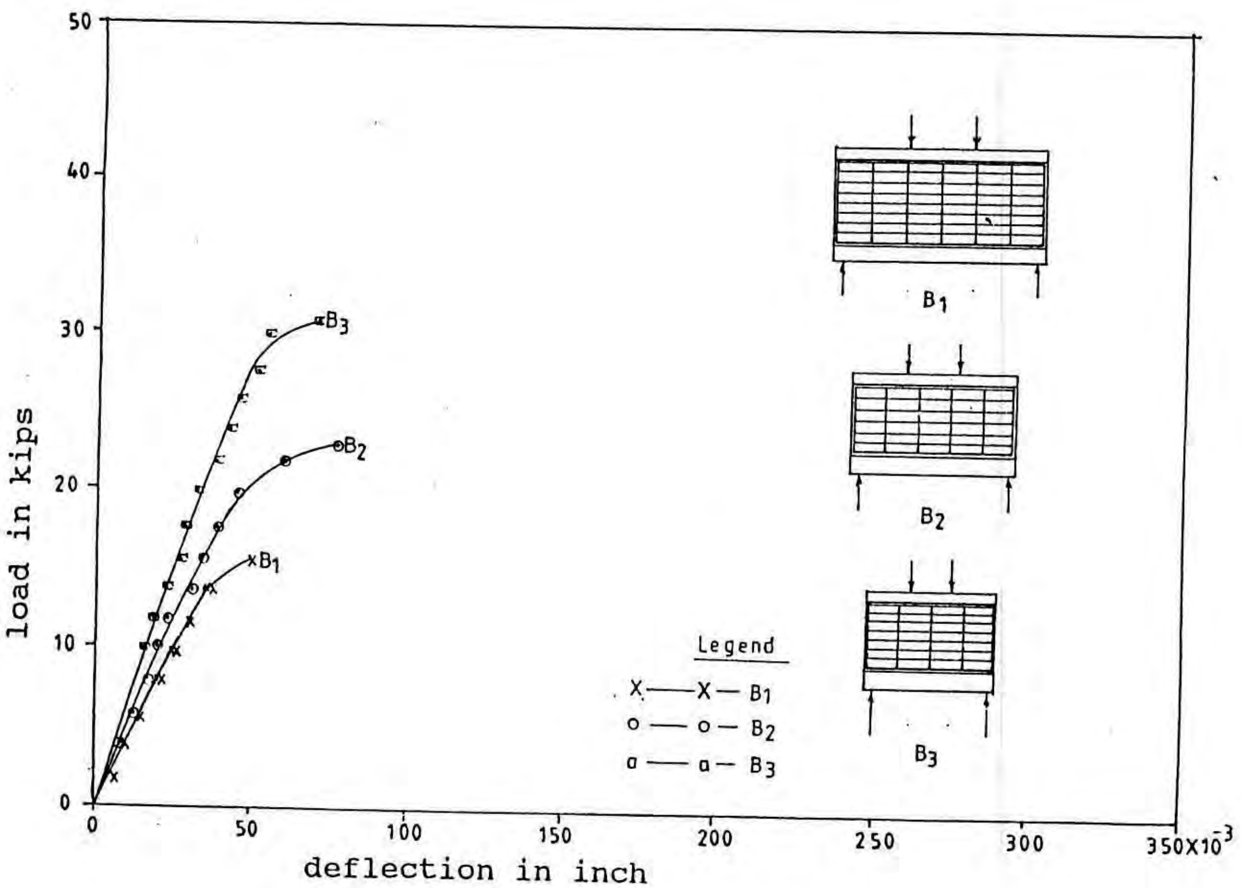


Fig.4.2 Load-Deflection Curve of Composite Wall-Beam Group B

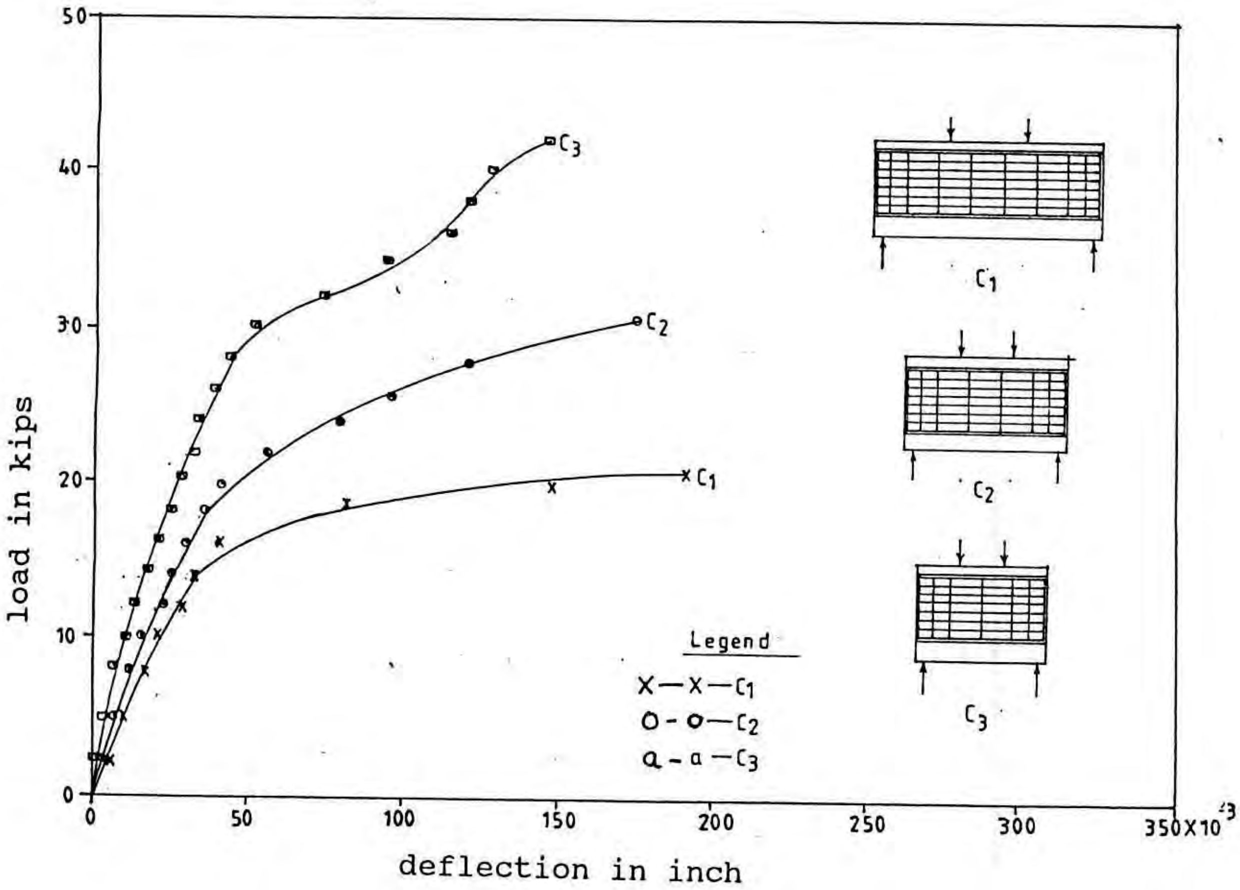


Fig.4.3 Load-Deflection Curve of Composite Wall-Beam Group C

The load-deflection pattern of running bonded composite wall-beams (Group A) shown in Fig.4.1. The load-deflection pattern of stack bonded composite wall-beam with or without vertical reinforcement in the wall (Group C and Group B respectively) are shown in Fig. 4.3 and 4.2 respectively.

From the above Figs.4.1, 4.2 and 4.3 it is found that -

i) The deflection of composite wall-beams decrease with increase of H/L ratio irrespective of bond pattern in wall.

ii) The load deflection pattern of running bonded composite wall-beam Group A and stack bonded composite wall-beam Group B were similar to brittle materials whereas the stack bonded composite wall-beams of Group C having vertical reinforcement behaves like a ductile material.

4.3.2 Load-Deflection at Mid Span of Reinforced Concrete Beam

The reinforced concrete beams D_1 , D_2 and D_3 having span 5 ft. 5 in.; 3 ft. 9 in. and 2 ft. 11 in. with D/L ratio 0.09; 0.13 and 0.17 respectively have been tested with the same support condition and same loading system used for composite wall-beams. Deflection of these beams have been recorded at an interval of 0.5^k load upto 3^k and then an interval of 1^k was used. The results of the test are given in Table A_{10} in

Appendix A. Load-deflection pattern of these beams are shown in Fig.4.4 which reveals that the beams behave like ductile material.

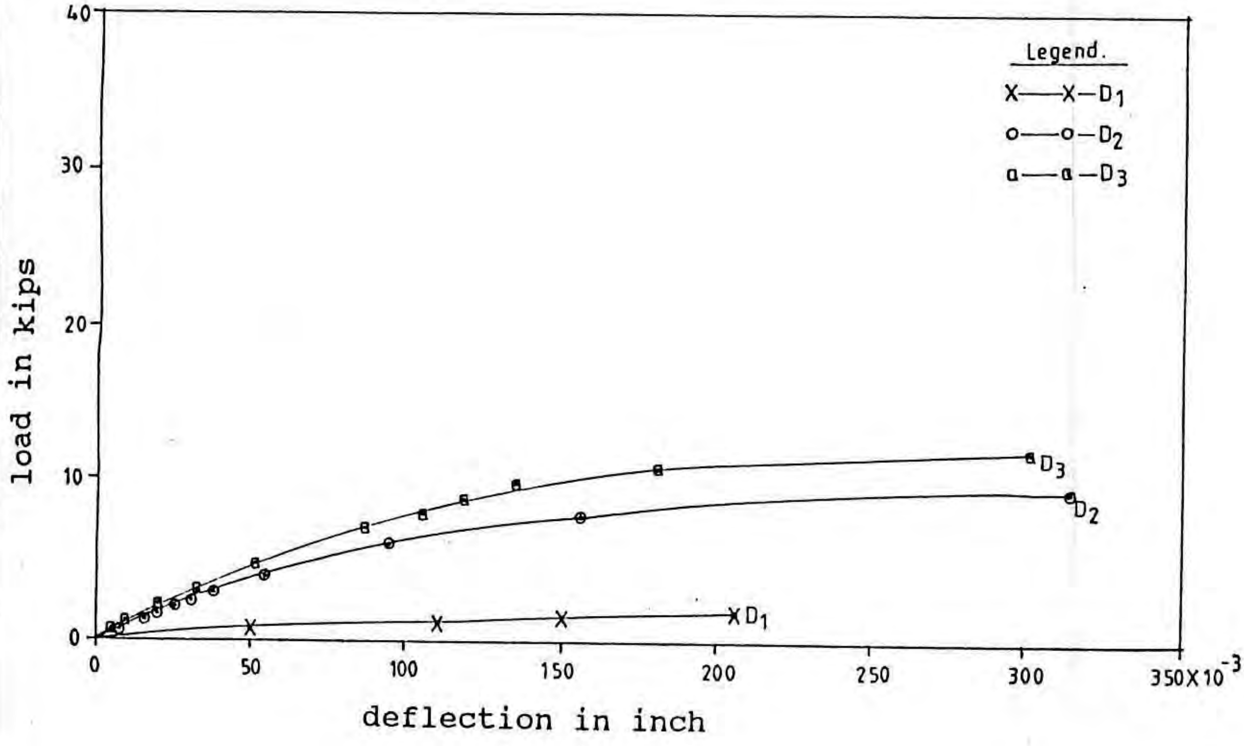


Fig.4.4 Load-Deflection Curve of Reinforced Concrete Beam Group D

4.4 Initiation of Cracking and the Failure Load

During test of beams the load at first visible crack and the failure load have been carefully observed and recorded. These loads were noted for both the composite wall-beam and bottom reinforced concrete beams.

4.4.1 Failure Load of Composite Wall-Beam

First visible cracking load and failure loads of composite wall-beam structure are shown in Table 4.1.

Table 4.1 Experimental Load of Beams

Beams			First visible cracking load (VCL)	Failure load (FL)	VCL/FL in (%)
Gr.	Name	H/L			
A	A_1	0.46	-	23.5^k	-
	A_2	0.66	-	33.2^k	-
	A_3	0.86	-	47.2^k	-
B	B_1	0.46	-	15.5^k	-
	B_2	0.66	20.0^k	23.0^k	86.96
	B_3	0.86	-	31.0^k	-
C	C_1	0.46	17.0^k	21.0^k	80.95
	C_2	0.66	21.0^k	32.0^k	65.63
	C_3	0.86	32.0^k	42.0^k	76.19
D	D_1	0.09	1.2^k	4.0^k	30.00
	D_2	0.13	7.6^k	9.0^k	84.44
	D_3	0.17	7.6^k	12.0^k	63.33

The Table 4.1 reveals that there was no significant difference between the first visible cracking load and failure load of running bonded composite wall-beams (Group A) and stack bonded composite wall-beams (Group B) except composite wall-beam B_2 (H/L = 0.67) of Group B. The load corresponding to first visible crack for this wall-beam was 20^k corresponding to the failure load 23^k.

It may be noted that Group C wall-beams having vertical reinforcement behave more like a ductile material. The first visible cracking load of C_1, C_2, and C_3 of Group C were 80.95%, 65.63% and 76.19% of failure load. The load at first visible crack load of Group C having vertical reinforcement in stack bonded composite wall-beam is very close to the failure load of composite wall-beam of Group B having same bond pattern without vertical reinforcements.

In all groups, the failure load was found to have increased with the increase of H/L ratio.

Failure load of composite wall-beams were found to be higher for Group A wall-beams with running bond than Group C and B stack bonded composite wall-beams with or without vertical reinforcement.

The failure load of composite wall-beams were much higher compared to the corresponding reinforced concrete beams supporting no brick walls (Group D).

4.4.2 Failure Load of Reinforced Concrete Beams

Third point vertical static concentrated load has been applied at a distance of L/3 from the center of each support of the reinforced concrete beams (Group D). The following observations were made from the test results of beams.

i) Load at First visible cracking for beam D_1, D_2 and D_3 was 30%, 84.44% and 63.33% of failure loads respectively.

ii) Failure load of beam D_3 having D/L ratio 0.17 was higher than that of beam having D/L ratio 0.13 and 0.09 respectively although the cross-sectional area of concrete and reinforcement provided was the same in all the three beams.

4.5 Mode of Failure of Beams

During tests of beams failure pattern and propagation of crack corresponding to the load for all the beams have been observed and recorded systematically. It is to be noted that the failure pattern of composite wall-beams and reinforced concrete beams were different in nature. Mode of failure of these beams have been described here.

4.5.1 Mode of Failure of Composite Wall-Beam

During tests, mode of failure of composite wall-beams have been recorded. These are shown in Figs.4.5 through 4.10.

The following observations were made on the mode of failure of composite wall-beams during test.

i) Flexural crack was not visible in reinforced concrete supporting beam during test.

ii) No visible crack was found in between two concentrated load except for the composite wall-beam C_3 of Group C having H/L ratio 0.6.

iii) All cracks initiated at the roller side of the beam.

iv) No crack was found at the interface of brick wall and reinforced concrete beam for most of the test beams. However, interface cracks were visible in beam B_1 and C_1.

v) For all the beams initially the cracks formed near the support and then propagated with the increase of loads. Nature of cracks was different in running bonded brick wall and stack bonded brick wall as shown in Figs.4.5, 4.7 and 4.9. In stack bonded brick wall the cracks (Figs. 4.7 and 4.8) were almost vertical and passed through the vertical mortar joint. Whereas in running bonded brick work, the crack was more or less diagonal passing through the mortar joints and sometimes through bricks as shown in Figs.4.5 and 4.6. The crack pattern of stack bonded brick wall with vertical reinforcement (Figs. 4.9 and 4.10) is found similar to the crack pattern of composite wall-beam (Group B).

4.5.2 Mode of Failure of Reinforced Concrete Beam

Mode of failure of reinforced concrete beams have been observed and drawn during test. These are shown in Figs.4.11 and 4.12. The following observations were prominent-

i) Flexural cracks were found in beam D_1 in between the two concentrated loads.

ii) Both shear and flexural cracks were visible in beam D_2 and D_3.

iii) Inclined cracks were observed along a line joining the support and the loading point in beam D_2 and D_3.

iv) In case of beam D_3, secondary cracks nearly parallel to the first inclined crack developed close to the support with the increase in load.

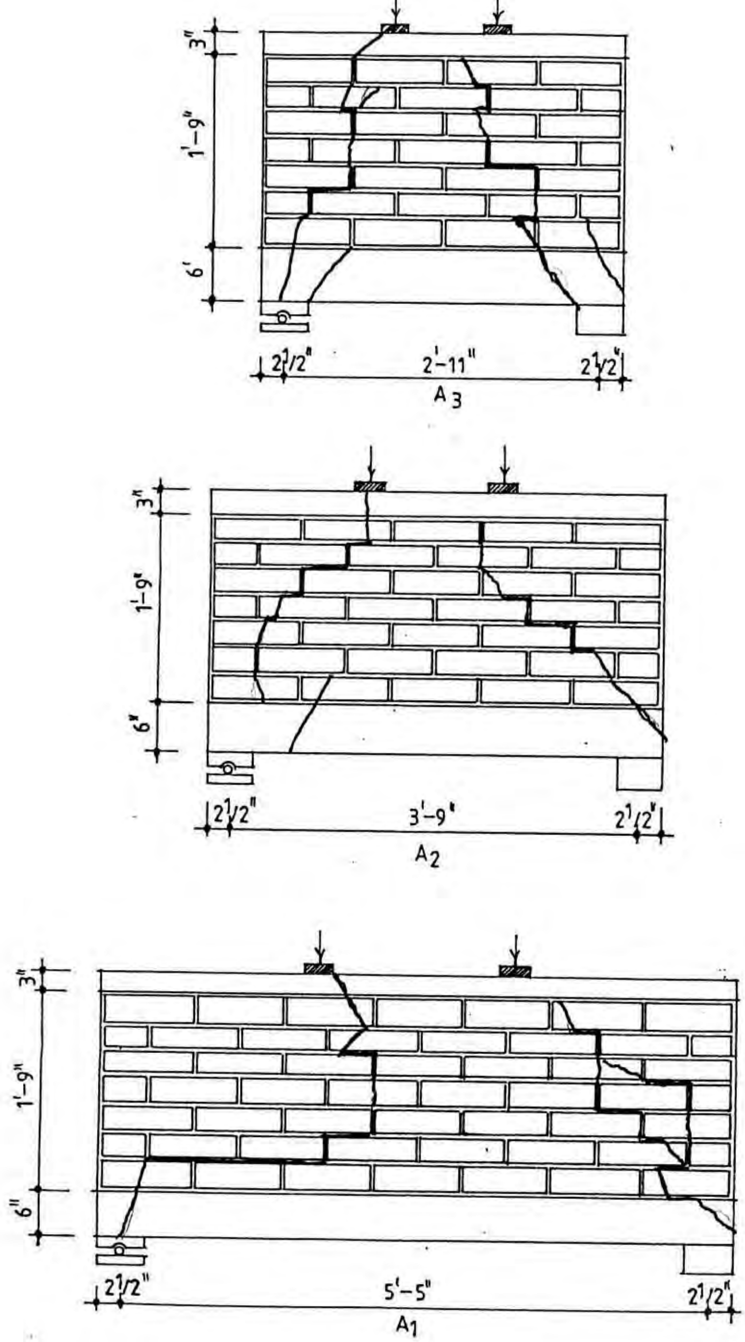


Fig.4.5 Crack Pattern of Running Bonded Composite Wall-Beam Group A

Fig.4.6 Running Bonded Composite Wall-Beam After Failure

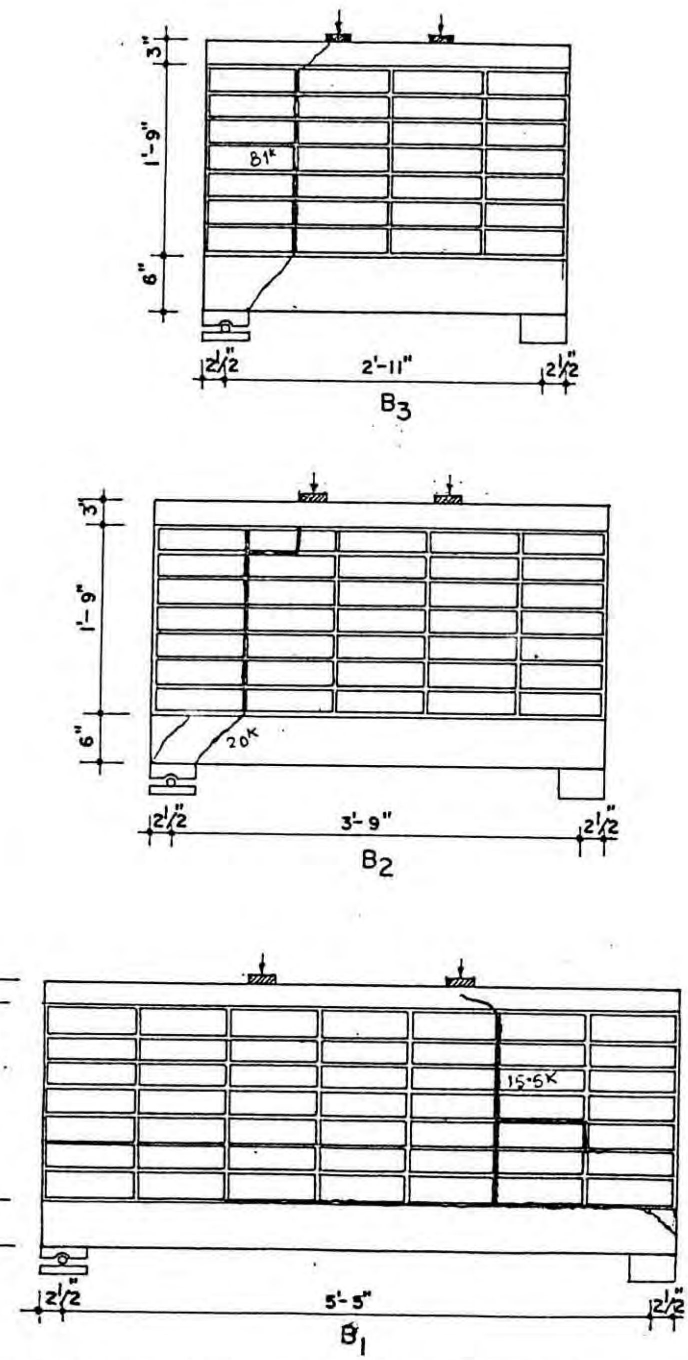


Fig.4.7 Crack Pattern of Stack Bonded Composite Wall-Beam Group B

Fig.4.8 Stack Bonded Composite Wall-Beam After Failure

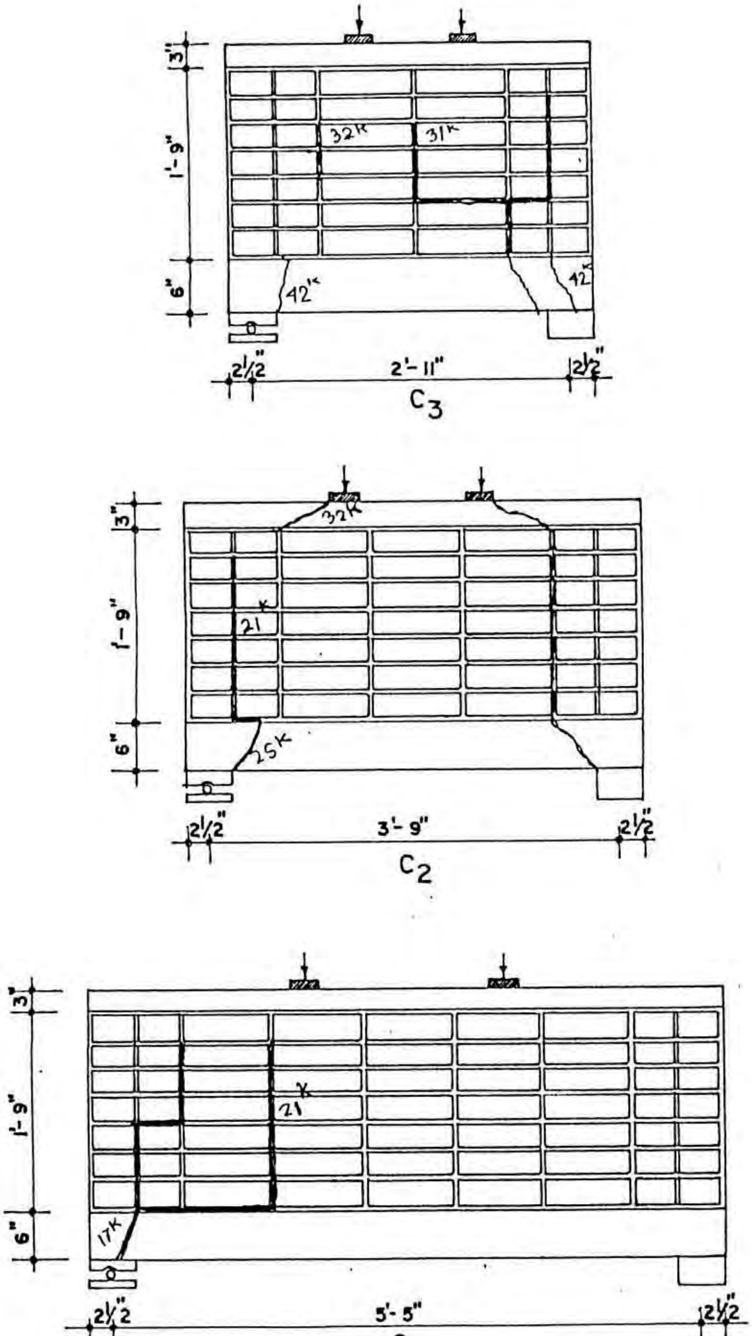
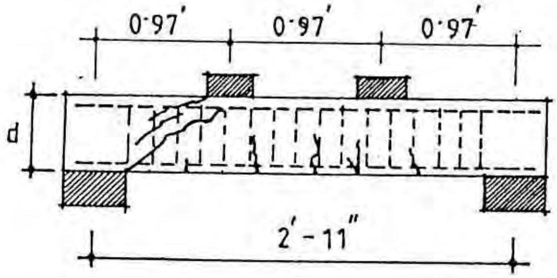
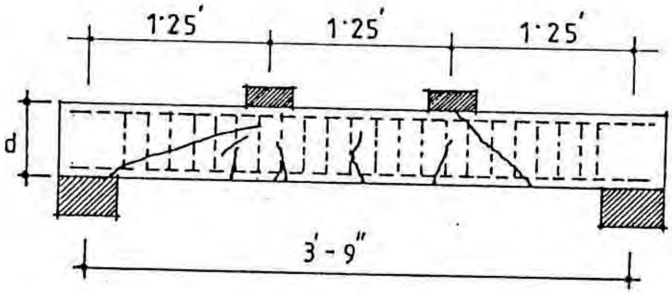


Fig.4.9 Crack Pattern of Stack Bonded Composite Wall-Beam with Vertical Reinforcement in Wall Group C

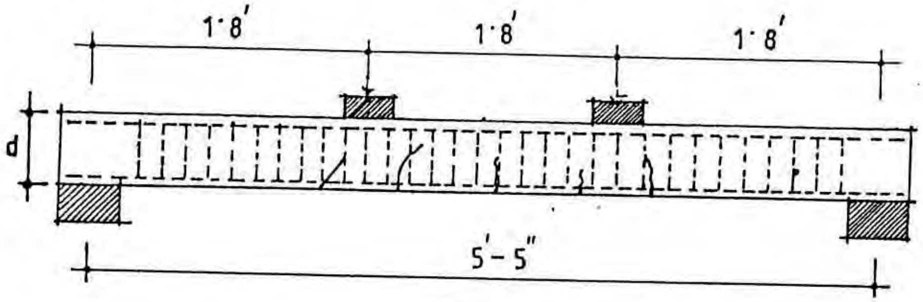
Fig.4.10 Stack Bonded Composite Wall-Beam with Vertical Reinforcement After Failure



D_3



D_2



D_1

Fig.4.11 Crack Pattern of Reinforced Concrete Beam Group D

Fig.4.12 Reinforced Concrete Beam After Failure Group D

CHAPTER 5

ANALYSIS OF TEST RESULTS AND DISCUSSIONS

5.1 Introduction

As per test scheme, nine composite wall-beams of Group A, B, C and three reinforced concrete beams of Group D have been tested. The results have been given in Chapter 4. Here the test results of these beams are analyzed and the findings are critically examined with observations of other researchers. The experimental load of reinforced concrete beams are compared with the predicted load from ultimate stress design method and also with the predicted load considering the section uncracked.

The experimental load, failure pattern and deflection of reinforced concrete (RC) beams are also compared with the failure load, mode of failure and deflection of composite wall-beam.

The effect of H/L ratio, bond pattern in brick wall and presence of vertical reinforcement in the wall on failure load, mode of failure and deflection of composite wall-beam are also discussed here.

The maximum bending moment and axial force of supporting RC beam, maximum vertical stress in the wall of the composite wall-beams have been calculated using the formulae suggested by Wood[5], Stafford Smith and Riddington[10], Davis and Ahmed[14] and compared with the test results of present study.

The test results of this study have also been compared with the results obtained by G.Annamalai[4], and P.Burhouse[7] in their similar experimental study.

5.2 Experimental Load and Theoretical Load of the Reinforced Concrete Beams

Three reinforced concrete beams of Group D, the effective span of which are 5 ft. 5 in., 3 ft. 9 in. and 2 ft. 11 in. have been tested in the laboratory by third point vertical concentrated load. All beams have been cast in the same day with same batch of concrete ($f_{c'}$ = 2050 psi) and reinforced with M.S. rods having same yield stress (fy = 45 ksi). The cross-sectional area of the beams were 5 in. by 6 in. The longitudinal reinforcements comprised of 4-3/8 in. dia M.S. rods, one at each corner and stirrups were 1/4 in. dia M.S. rods spaced 6 in. center to center. The theoretical moment capacity of the beams have been calculated considering the section (a) uncracked and (b) at ultimate state (USD). The corresponding computed values are $1.22^{\text{k-ft}}$ and $2.99^{\text{k-ft}}$ respectively. Relevant flexural loads were obtained by equating the moment expressions to the respective moment capacities. Calculations are shown in Appendix B and the results are given in Table 5.1.

Table 5.1 Comparison Between the Theoretical and Experimental Loads of Reinforced Concrete Beams (Group D)

Beam	Span	Theoretical flexural load in kips		Experimental loads in kips		Ratio of loads	
		Section uncrac-ked	Ultima-te state	First visible crack	Ultim-ate load	(5)/(2)	(5)/(3)
D_1	5'-5"	1.35	3.31	1.2	4.0	2.96	1.21
D_2	3'-9"	1.95	4.78	7.6	9.0	4.62	1.88
D_3	2'-11"	2.50	6.14	7.6	12.0	4.80	1.95

From Table 5.1 it is found that the experimental load of beam D_1 is about 1.21 times higher than the theoretical ultimate load from flexure consideration. The experimental load of beam D_2 and D_3 is differs widely (about 1.9 times higher) from the theoretical flexural load. It is to be noted that the span/depth ratio of beam D_1 (10.84) being greater than 8 is considered as shallow beam. The span depth ratio of beams D_2(7.5) and D_3(5.84) lie between 5 and 8 are considered as moderate deep beam[41]. Because of their proportions(D/L ratio), the internal forces redistribute before failure and develop mechanisms of force transfer quite different from beams of common proportions and their strength is mostly controlled by shear[30]. Table 5.1 also reveals that the first visible crack for RC beam D_1, D_2 and D_3 appeared at 30%, 84% and 63% of ultimate load respectively.

5.3 Experimental Load of Composite Wall-Beams

Nine composite wall-beams divided into three Groups A, B and C have been tested to failure by third point vertical concentrated load similar to the reinforced concrete beams. The test results are given in Table 4.1. Here the effect of H/L ratio, bond pattern in brick wall and also inclusion of vertical reinforcement in the wall on the failure load of composite wall-beam have been discussed and the relative comparisons are presented in Table 5.2.

5.3.1 Effect of H/L Ratio

The H/L ratio of composite wall-beam structure has a great effect on the failure loads. However, it does not mean that composite wall-beam structures having identical H/L ratio will fail at the same load. There are other parameters like modulus of Elasticity (E) of materials, moment of Inertia (I) of the section, the span length and slenderness ratio wall which influence the failure load as well. As an example, the failure load of running bonded composite wall-beam of present study having H/L ratio 0.67 is 33.2^k (Table 5.2) while wall-beam having similar H/L ratio (0.58) tested by Burhouse[7] and Annamalai 'et al'[4] failed at 132^k and 55.1^k respectively. It is an important observation to mention that the present test beams having H/L ratio of 0.46 behaved compositely with the supporting reinforced concrete beams. Whereas previous investigator[14] suggested not to consider composite actions for beams of H/L ratio less than 0.6. Further experimental work in this line is required to mark the lower limit of H/L ratio when composite action actually disappears.

Table 5.2 Comparison of Experimental Loads of Composite Wall-Beams Among the Groups (present study)

Composite wall-beam		Experimental load in kips	Ratio of loads of beams		
Group	Beam		Gr.A/ Gr.B	Gr.A/ Gr.B	Gr.A/ Gr.B
A	A_1 (H/L)= 0.46	23.5	1.52	-	-
	A_2 (H/L)= 0.67	33.2	1.44	-	-
	A_3 (H/L)= 0.86	47.2	1.52	-	-
B	B_1 (H/L)= 0.46	15.5	-	0.74	-
	B_2 (H/L)= 0.67	23.0	-	0.72	-
	B_3 (H/L)= 0.86	31.0	-	0.74	-
C	C_1 (H/L)= 0.46	21.0	-	-	0.89
	C_2 (H/L)= 0.67	32.0	-	-	0.96
	C_3 (H/L)= 0.86	42.0	-	-	0.89

5.3.2 Effect of Stiffness

Flexural stiffness (4EI/L) has a significant effect on the failure load of composite wall-beam structure. From the present investigation it is found that the ratio of failure load to stiffness is a constant number for respective group (Table 5.3) with coefficient of variation ranging between 3.9 to 5.16%. For the computation of flexural stiffness of composite wall-beam, the total section of composite wall-beam including longitudinal reinforcement of bottom beam transformed into equivalent concrete section was considered.

From the equivalent concrete transformed section, the moment of inertia (I_c) has been calculated for uncracked transformed section. The modulus of Elasticity (E_c) of concrete is considered as 57,000 times $\sqrt{f_c'}$ in psi unit. Detail calculation is shown in Appendix C. Table 5.3 reveals that the ratio of load to stiffness of composite wall-beam is different from group to group. This is possibly due to the consideration of equal value of modulus of Elasticity of brick work for all groups. Modulus of Elasticity of brick work may vary with the bond pattern of brick wall as well as with the inclusion of vertical reinforcement in the wall.

Table 5.3 Load to Stiffness Ratio of the Composite Wall-Beams (Test Beams)

Composite wall-beam		Experimental load in kips	Flexural* Stiffness (4 EI/L) in lb-inch	Ratio of Load to Stiffness	Mean Ratio
Group	Beam				
A	A_1 (H/L)= 0.46	23.5	9.74×10^{8}	2.41×10^{-5}	2.46×10^{-5}
	A_2 (H/L)=0.67	33.2	14.07×10^{8}	2.35×10^{-5}	
	A_3 (H/L)=0.86	47.2	18.09×10^{8}.	2.60×10^{-5}	
B	B_1 (H/L)= 0.46	15.5	9.74×10^{8}	1.59×10^{-5}	1.64×10^{-5}
	B_2 (H/L)=0.67	23.0	14.07×10^{8}	1.63×10^{-5}	
	B_3 (H/L)=0.86	31.0	18.09×10^{8}	1.71×10^{-5}	
C	C_1 (H/L)= 0.46	21.0	9.74×10^{8}	2.16×10^{-5}	2.25×10^{-5}
	C_2 (H/L)=0.67	32.0	14.07×10^{8}	2.27×10^{-5}	
	C_3 (H/L)=0.86	42.0	18.09×10^{9}	2.32×10^{-5}	

*Flexural stiffness of composite wall- beam based on uncracked section in concrete equivalent.

The load to stiffness ratio some of the composite wall-beams similar to the present study tested by Annamalai 'et al'[4] and Burhouse[7] are also calculated. Brief description of these beams are given in Table 5.4.

Table 5.4 Description of Some of the Composite Wall-Beam Tested by Annamalai[4] and Burhouse[7]

Parameters	Burhouse			Annamalai	
	B-6	B-8	B-9	RC-3	RC-4
L in inch	144.00	144.00	144.00	48.00	48.00
t in inch	6.00	6.00	6.00	9.20	9.20
D in inch	12.00	12.00	12.00	3.00	3.00
As in sq.in	0.614	0.614	1.277	0.16	0.16
H in inch	84	120.00	48.00	27.80	27.80
b in inch	4.5	4.50	4.50	9.20	9.20
H/L	0.58	0.83	0.33	0.56	0.56
f_c' in psi	3000.00	3000.00	3000.00	2300.00	2500.00
f_b' in psi	3000.00	3000.00	3000.00	2175.00	1050.00
f_y in psi	42420.00	42420.00	42420.00	60000.00	60000.00
E_s in psi	29.1×10^{6}	29.1×10^{6}	29.1×10^{6}	$29\ \times10^{6}$	$29\ \times10^{6}$
E_c in psi	3.15×10^{6}	3.15×10^{6}	3.15×10^{6}	2.76×10^{6}	2.87×10^{6}
E_m in psi	10^{6}	10^{6}	10^{6}	8.7×10^{5}	5.37×10^{5}
S_r	14	20	8	2.27	2.27
Load in kips	132	109.56	115.28	55.1	36.96

Note:
(a) Sr(Slenderness ratio ofbeam) = 0.75xtotal height/thickness
(b) other symbols has usual meaning

The calculations of load to stiffness ratio of beams are shown in Appendix C and the results are given in Table 5.5. The Table 5.5 reveals that the ratio of failure load to stiffness of running bonded composite wall-beam (Group A) of present study are close to the corresponding ratios of the composite wall-beam tested by Annamalai 'et al'[4] but it does not correlate well with the experimental result of Burhouse.[7] It may be mentioned here that the height of wall of composite wall-beam tested by Burhouse was 2 to 3 times higher than that of wall beams of present study and of Annamalai 'et al.'[4] Due to large height of the wall warping stress may develop in the wall resulting less experimental load than would be expected. As a result the ratio of load and stiffness of composite wall-beam tested by Burhouse was found less than that of present study and also the study of Annamalai 'et al.'[4]

Table 5.5 Comparison Between the Load to Stiffness Ratio of Composite Wall-Beams of Present Study with the Available Test Results

Research designation	Beam				Ratio of Load to Stiffness
	No	Load	Slenderness Ratio	H/L	
Present Study	A_1	23.5	5.0	0.46	2.41×10^{-5}
	A_2	33.2	5.0	0.67	2.36×10^{-5}
	A_3	47.2	5.0	0.86	2.60×10^{-5}
Experiment by G. Annamalai[4]	RC_3	55.1	2.27	0.58	2.80×10^{-5}
	RC_4	23.96	2.27	0.58	2.12×10^{-5}
Experiment by Burhouse[7]	B_6	132.0	14.0	0.58	1.32×10^{-5}
	B_8	109.56	20.0	0.83	3.98×10^{-6}
	B_9	115.28	8.0	0.33	5.65×10^{-5}

5.3.3 Effect of Bond Pattern in Brick Work

Bond pattern in brick work has been found to have significant influence on the strength of composite wall-beams. Running bonded wall-beams have exhibited higher failure loads compared to their corresponding stack bonded wall-beams. This may become apparent from Table 5.2 that the failure load of stack bonded (Group B) composite wall-beam is about 74% than that of running bonded composite wall-beam (Group A). This is possibly due to the crack arrest mechanism present in the bond pattern of the brick wall in group A. The presence of continuous vertical joint in stack bonded wall allows the early crack formation and rapid crack propagation which ultimately results in premature failure of the wall-beam.

5.3.4 Effect of Vertical Reinforcement in Wall

It has been observed from the experimental results that the inclusion of vertical reinforcement in the stack bonded composite wall-beam have substantially increased the failure load. Two groups of stack bonded composite wall-beam with or without vertical reinforcement in the wall (Group C and Group B respectively) have been tested in this study. It was found that the failure load of composite wall-beam of group C having vertical reinforcement in the stack bonded wall is about 1.36 times higher compared to the respective stack bonded composite wall-beam without vertical reinforcement (Group B) shown in Table 5.2. The first visible cracking load of group C is found similar to the failure load of Group B. The stack bonded composite wall-beam of Group B failed just after the initiation of first visible crack behaving like a brittle material whereas composite wall-beam of Group C failed at about 30 to 35% higher load after the formation of first visible crack. Therefore, it may be concluded that the inclusion of vertical reinforcement in the stack bonded composite wall-beam has increased the ductility. It is also observed that the failure load of stack bonded composite wall-beam having vertical reinforcement in the wall (Group C) is about 90% to the compare to the corresponding running bonded composite wall-beam.

5.4 Comparison of the Failure Loads of Composite Wall-Beam with that of Supporting Reinforced Concrete Beam

When a brick wall is built over the reinforced concrete beam and tested to failure, it is observed that the failure load becomes several times higher compared to the load at which the supporting beam would have failed. It should be mentioned here that the wall beams and corresponding reinforced concrete beams were tested under third point vertical concentrated load, i.e. in all the cases the loads were applied at a distance of 1/3rd span length from center of either supports. The position and magnitude of the failure load of composite wall-beam along with the failure load of their supporting reinforced concrete beams are shown in Figs. 5.1, 5.2 and 5.3. The failure loads of all composite wall-beams and that of corresponding reinforced concrete beams are also given in Table 5.6.

Table 5.6 Failure Load of Composite Wall-Beam and the Supporting Reinforced Concrete Beam

Span	Composite wall-beam in kips			RC beam	D/L ratio	Ratio of failure load		
	Gr.A	Gr.B	Gr.C	Gr.D		A/B	B/D	C/D
5'-5"	24.2	16.2	21.7	4.0	0.09	6.05	4.05	5.43
3'-9"	33.7	23.5	32.5	9.0	0.13	3.75	2.61	3.61
2'-11"	47.6	31.4	42.4	12.0	0.17	3.97	2.61	3.53

Note: A/B means load of group A / load of group B

Table 5.6 reveals that the failure load of composite wall-beam is much higher than the supporting RC beam. The ratio of failure load of running bonded composite wall-beam (Group A) to the supporting RC beam is higher than the stack bonded composite wall-beams(Group B). The failure loads of running bonded composite wall-beams vary from about 3.74 to 6.05 times and stack bonded composite wall-beam vary from 2.61 to 4.05 times higher compared to the corresponding reinforced concrete beams. This is possibly due to the crack arrest mechanism present in running bonded brick wall of composite wall-beam (Group A). The presence of continuous vertical joint in the stack bonded wall allows the early crack formation and rapid crack propagation which ultimately results in early failure of the wall-beam. It is also found that the ratio of failure load of stack bonded composite wall-beam having vertical reinforcement in the wall (Group C) to the corresponding reinforced concrete beam vary between 3.61 to

5.53. Due to the inclusion of vertical reinforcement in the wall of stack bonded composite wall-beam, the failure load have increased by about 35% compared to the stacked bonded composite wall-beam having no vertical reinforcement in the wall.It is observed in the Table 5.6 that the ratio of failure load of composite wall-beam to the corresponding reinforced concrete beam are not similar for all beams in a particular group. This is possibly due to the effect of D/L ratio of supporting reinforced concrete beam. The failure load of reinforced concrete beam D_1 of (D/L = 0.09) is very close to ultimate load predicted from flexure considerations whereas the failure load of reinforced concrete beam D_2 (D/L= 0.13) and D_3(D/L=0.17) is about 2 times higher than the ultimate load predicted from flexure consideration because of their proportions (D/L ratio). It is to be noted that when experimental failure load of composite wall-beams A_2 and A_3 are compared with the theoretical flexure load of corresponding reinforced concrete beam D_2 and D_3, the ratio of load is found to be about 7 which is similar to the load ratio of beam A_1/D_1. From the test results it may be concluded that the D/L ratio of supporting reinforced concrete beam within the range of shallow beam provides better composite action in wall-beam structure. This is a quite good resemblance with the finite element study on composite wall-beam carried out by Kamal[16]. He found that with increase in the depth of supporting beam, the composite action between brick work with bottom beam decreases. The pioneer investigator in this field Wood[5], also suggested that the depth of reinforced concrete beam (D) Should be between 0.05 to 0.067 times the effective span of the wall beams. Davis and Ahmed[14] also restricted the ratio of D/L = 0.1 in their graphs for computing design parameters of composite wall-beam structure.

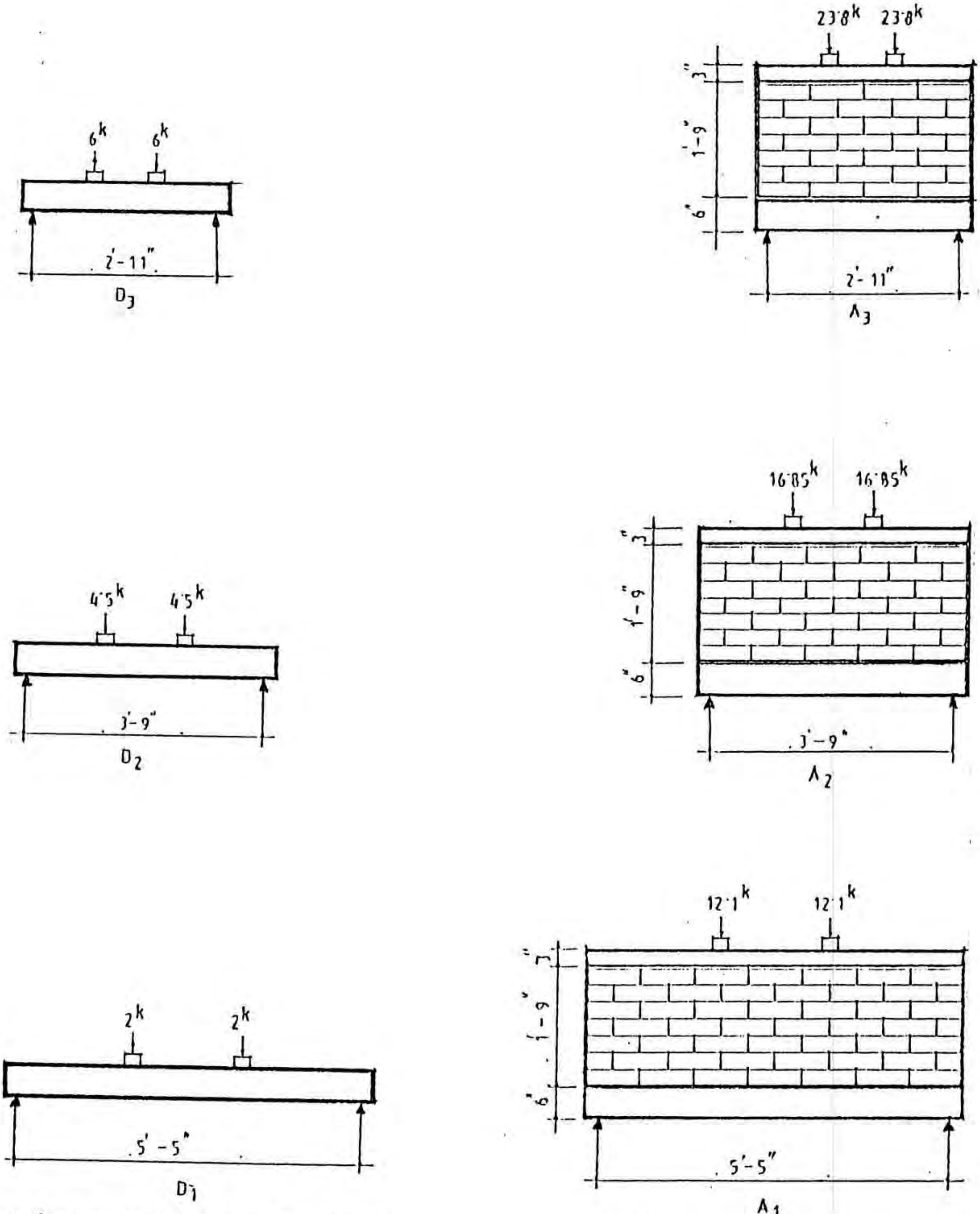
5.5 Discussion on the Mode of Failure of Beams

During tests first visible cracks and propagation of cracks both in reinforced concrete beam and composite wall-beam corresponding to the load have been recorded and furnished in Chapter 4. In this chapter mode of failure of these beams have been analyzed and compared among the groups and also within the groups.

5.5.1 Reinforced Concrete Beam (Group D)

The crack pattern of three reinforced concrete beams of Group D were observed during the tests and are shown in Fig.4.12.

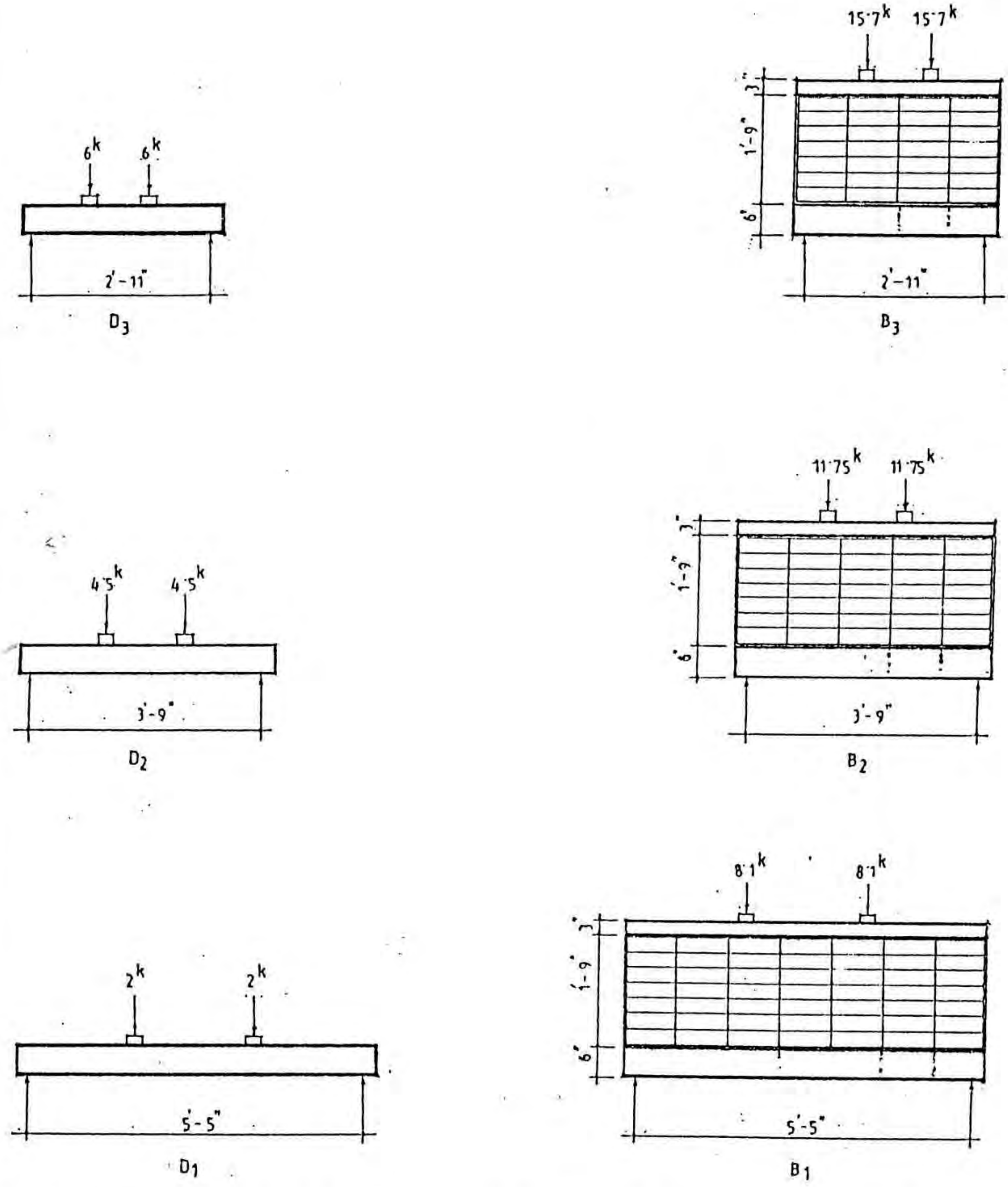
Flexural cracks were observed in the tension zone of beam D_1 (Fig.4.12a), that is in the middle third of the beam. No visible crack was found in the shear span. The beam was divided into a comb like structure in the tension zone. Flexural cracks extended on inclined planes after crossing the longitudinal reinforcements. It collapsed suddenly after the formation of inclined cracks.



reinforced concrete beam

running bonded composite wall-beam

Fig.5.1 Comparison of the Failure Load of Running Bonded Composite Wall-Beam and Corresponding Reinforced Concrete Beams



reinforced concrete beam   stack bonded composite wall-beam

Fig.5.2 Comparison of the Failure Load of Stack Bonded Composite Wall-Beam and Corresponding Reinforced Concrete Beams

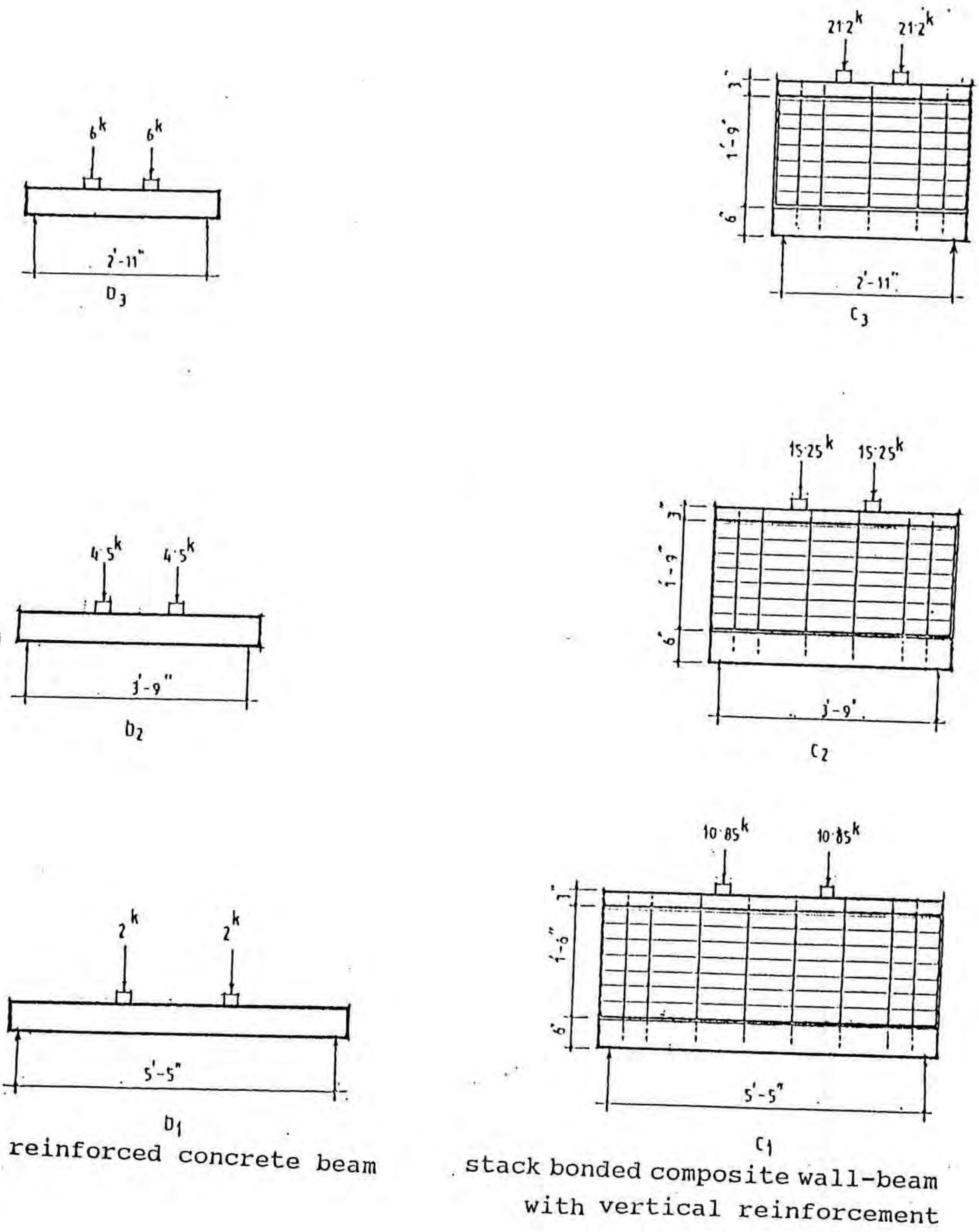


Fig.5.3 Comparison of the Failure Load of Stack Bonded Composite Wall-Beam With Vertical Reinforcement in Wall and Corresponding Reinforced Concrete Beams

From the failure pattern, it may be concluded that beam D_1 has failed in beam action mechanism. The a/d ratio of beam D_1 is 3.61 and this type of failure is common in beams with a/d ratio ranging between 3 to 7[41].

Cracks were observed both in shear span and in the middle third of the beams D_2 and D_3 (Fig.4.12b and 4.12c). Inclined cracks were observed in shear span along a line joining the support and the loading point in beam D_2 and D_3. But only in Beam D_3, secondary cracks were observed nearly parallel to the first inclined crack close to the support. From the failure pattern it may be concluded that the failure has occurred in beam D_2 and D_3 by diagonal tension and diagonal compression respectively. Failure by crushing of concrete in shear span of a reinforced concrete beam are frequent with a/d ratio below 2.5[41]. The a/d ratio of D_2 and D_3 is 2.5 and 1.94 respectively. In the diagonal tension mode of failure under concentrated load, a clear and sudden fracture is observed along a line joining either support with the loading point. This type of failure is similar to the splitting of a concrete cylinder under compressive load applied on diametral plane.

In diagonal compression failure, an inclined crack develops first nearly along a line joining the load and the support points. With the increase in load a second crack nearly parallel to the first develops closer to the support and extends with further increase in load. The final failure is due to the destruction of the concrete strut between these two cracks.

5.5.2 Composite Wall-Beams

Failure pattern of nine composite beams of group A, B and C are shown in Fig.4.5, through 4.10 and the discussion on the failure pattern are given here.

It was found during tests that flexural crack was not visible in the supporting reinforced concrete beam of composite wall-beam structure. This is possibly due to the composite action in wall-beam composite structure where the bottom reinforced concrete beam acts as a tie member[10, 14].

In between two concentrated load, no visible cracks were observed in the brick work of composite wall-beam structure. This is perhaps due to the vertical stress concentration near the support which is an established phenomenon accepted by all the investigators working on the composite wall-beam structures[9,14]. Distribution of vertical stress along the interface has been already shown in Fig.2.5.

Cracks were not visibly noticed in the interface of brick wall and bottom RC beam except partly in C_1(H/L= 0.46) of Group C and prominently in Beam B_1 (H/L = 0.46) of Group B. These horizontal cracks were initiated close to the support and then propagated towards the center of the beam. In composite wall-beams shear stresses along the interface of beams and wall are significantly high over the supports. These stresses increase sharply and attain the peak value at a distance of L/15 from the support and then decrease very slowly as shown in Fig.2.14. Interface cracks observed in beams B_1 and C_1 could be due to the fact that the high shear stresses developed at the interfaces exceeded the interface bond strength at failure load while for the other beams the interface bond strength were not exceeded. It may be mentioned that one layer of brick was laid on the green concrete of supporting beam for proper bonding between the interface of beam and wall which resists the interface shear stresses developed.

Failure pattern of running bonded composite wall-beam Group A and Group B stack bonded composite wall-beam was different from that of Group C having stack bonded composite wall-beam with vertical reinforcement. Group A and Group B failed just after initiation of first visible crack like brittle material. But the composite wall-beam C_1, C_2 and C_3 of Group C, sustained 23.5%, 52.38% and 31.25% more loads respectively after the appearance of the first visible crack. Obviously, the inclusion of vertical reinforcements increased the ductility of these beams. It was observed that the first visible cracking loads of Group C beams were similar to the corresponding cracking loads of Group B which are also the failure loads for the latter beams.

5.6 Load-Deflection Behavior of Composite wall-beam

Deflection of beams were recorded at a regular interval of loads and are plotted to get the load-deflection curves shown in Figs.4.1 through 4.3. From these curves it is observed that the H/L ratio, bond pattern in the brick wall and inclusion of vertical reinforcement in the wall have significant effect on deflection. These curves are superimposed between same groups and inter group beams to examine the effect of different parameters on deflection. These superimposed load-deflection curves are shown in Figs. 5.4, 5.6 and 5.6.

5.6.1 Effect of H/L Ratio

Deformation of composite wall-beam decrease with the increase of H/L ratio in all groups as shown in Figs.4.1, 4.2 and 4.3. From Fig.4.1, it is found that the deflection of composite wall-beams A_1, A_2 and A_3 at their respective service load (1/3rd of failure load) are 0.025 in.; 0.025 in.; and 0.03 in. which is equivalent to span/2600; span/1800; and span/1167 respectively. The service load of composite wall-beam A_1, A_2 and A_3 are 7.83^k, 11.06^k and 15.73^k. Maximum deflection recorded for beam A_1, A_2 and A_3 are span/867, span/600 and span/350 at 20^k, 31^k and 46^k load respectively.

The deflection of stack bonded composite wall-beam (Group B) is shown in Fig. 4.2 which reveals that the deflection of composite wall-beam B_1, B_2 and B_3 at their respective service load are span/5600, span/3200 and span/2060 at 5.86^k, 7.0^k and 10.83^k load.

The deflection of stack bonded composite wall-beam having vertical reinforcement shown in Fig. 4.3. reveals that the deflection at their respective service load are span/4640, span/2647 and span/1945 at 7^k, 10.66^k and 14^k respectively.

5.6.2 Effect of Bond Pattern in Brick Wall

Composite wall-beam of stack bonded brick work (Group B) is stiffer than composite beam of running bonded brick work (Group A) for all H/L ratio upto a certain limit and above this limit, the composite wall-beam of Group A is stiffer than composite wall-beam of Group B shown in load-deflection curve 5.4, 5.5 and 5.6. It is to be noted that Ali[35] found in his study that the deformation of brick work decrease when load is applied parallel to the bed joint. In stack bonded brick wall the vertical joints are parallel to the applied load and may be considered as bed joints. This may be the reason of getting stiffer behavior at the initial stage of loading for these wall-beams.

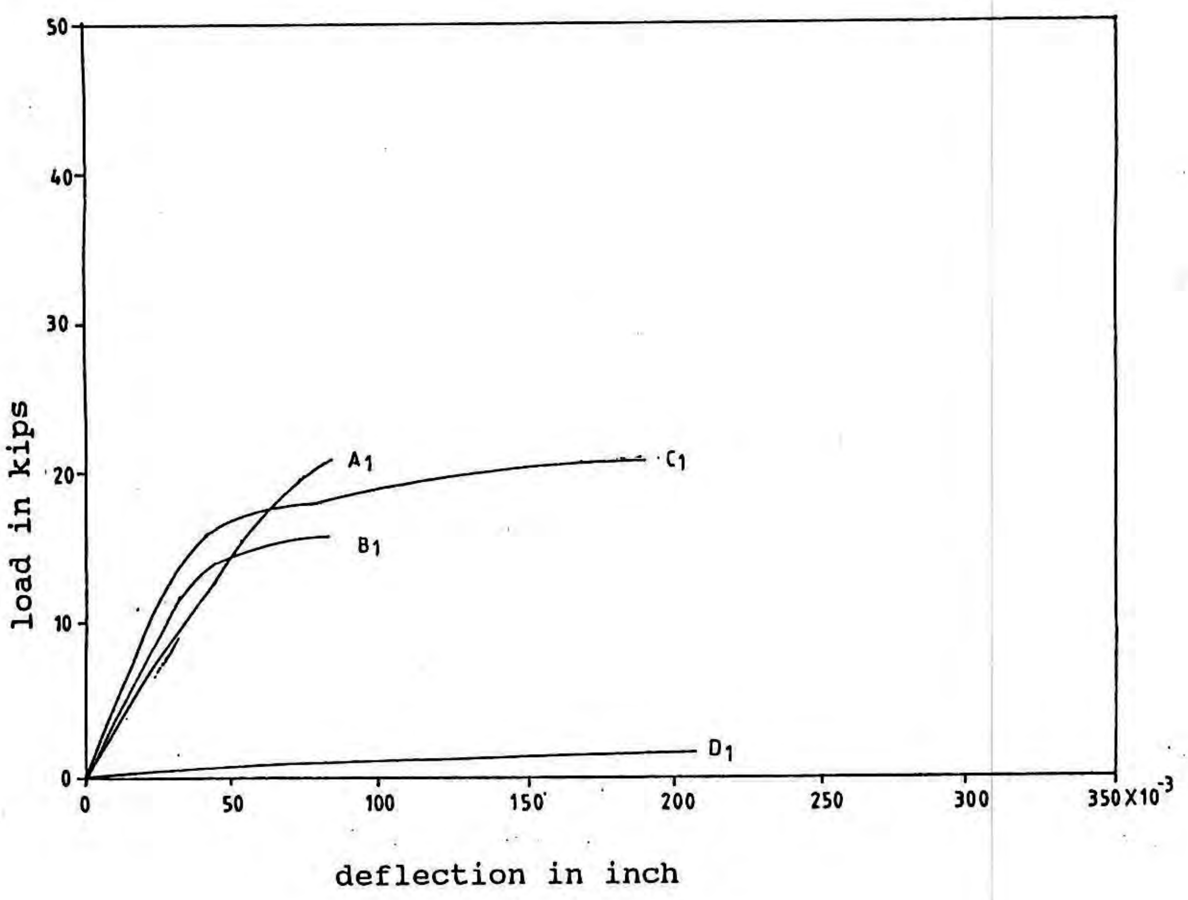


Fig.5.4 Load-Deflection Curve of Composite Wall-Beam (A_1, B_1, C_1, D_1)

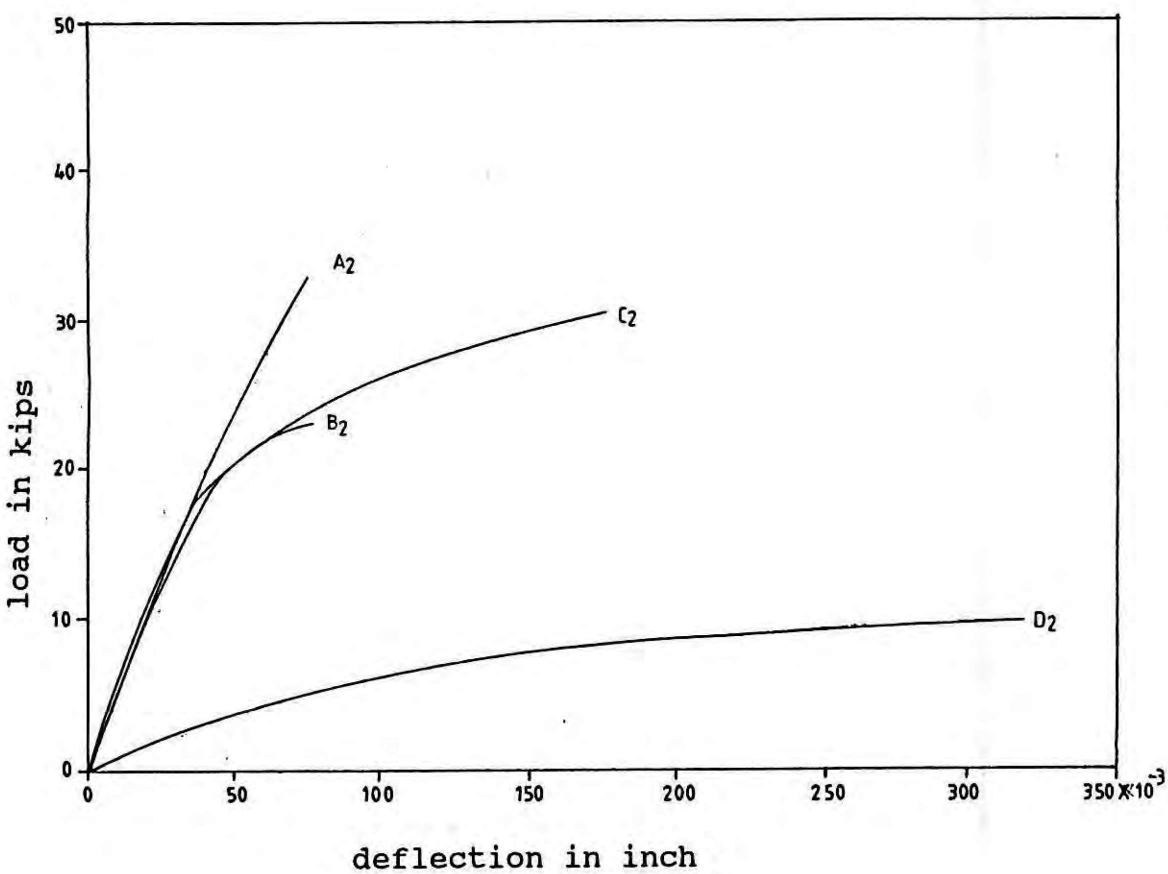


Fig.5.5 Load-Deflection Curve of Composite Wall-Beam (A_2, B_2, C_2, D_2)

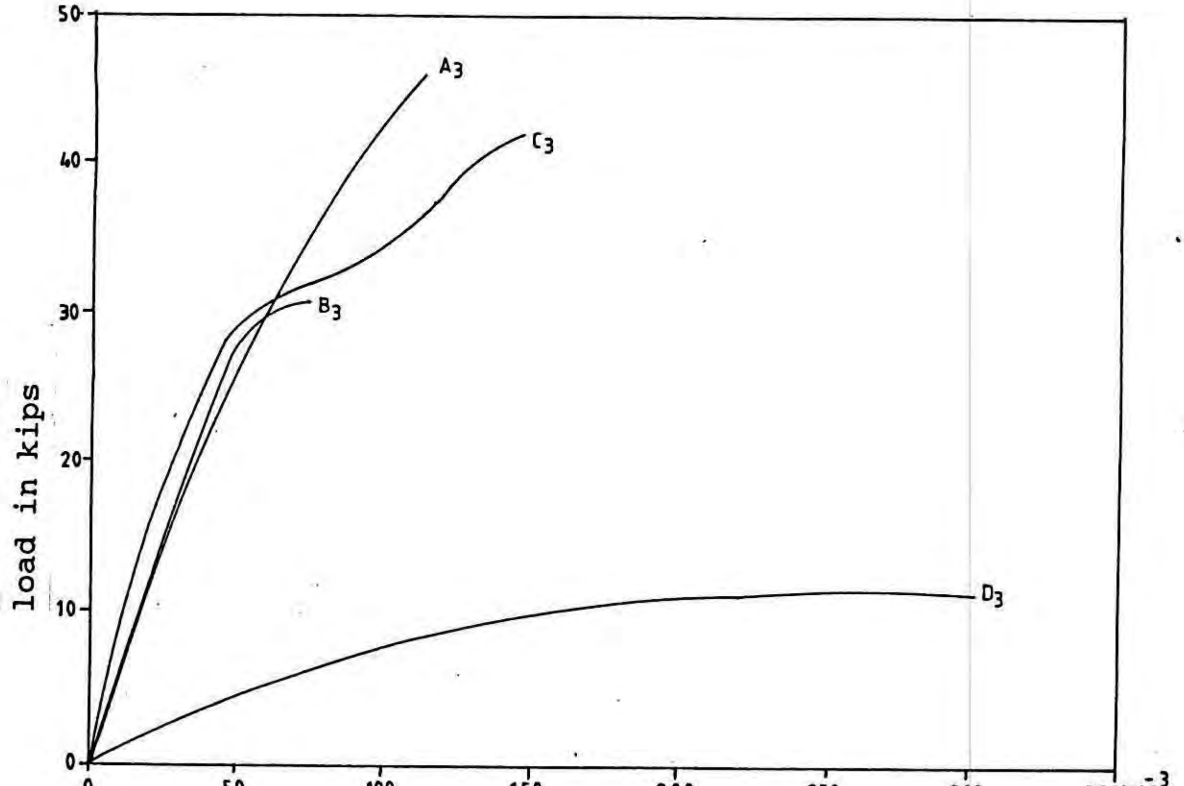


Fig.5.6 Load-Deflection Curve of Composite Wall-Beam (A_3, B_3, C_3, D_3)

5.6.3 Effect of Vertical Reinforcement in Wall

Composite wall-beam having stack bonded brick wall with vertical reinforcement in the wall (Group C) is stiffer than the composite wall-beam of Group A and Group B up to certain limit. It is found from the load deflection curve (Fig. 5.4, 5.5 and 5.6) that the composite wall-beam of Group A and Group B failed like brittle material whereas the composite wall-beam of Group C behaves more like ductile material due to the inclusion of vertical reinforcement in the wall.

5.7 Comparison of Load-Deflection Behavior of Composite Wall-Beams with Corresponding Supporting Beams

Experimental deflections of reinforced concrete beams up to first visible cracking load have been furnished in Table 5.7 along with the theoretically computed deflections. Deflections have been computed using the formula (derived from moment area theorems) in Appendix D. The modulus of Elasticity of concrete (E_c) is taken to be 2.58×10^6 psi and the gross moment of inertia (I_g) was computed as 90 in^4 considering uncracked section.

Experimental deflections have been recorded during tests at a regular interval of load. It is seen from the Table 5.6 that the experimental deflections are higher than the predicted deflections in all reinforced concrete beams. This may be partly due to not considering the deflection caused by shear deformation and reduction of moment of inertia with progressive cracking.

Table 5.7 Comparison of the Experimental and Computed Deflection of Reinforced Concrete Beams (Group D) up to First Visible Cracking Load.

Load in	Deflection of beam in inch x 10^{-3}					
	D_1		D_2		D_3	
	Expt.	Computed	Expt.	Computed	Expt.	Computed
0	0		0		0	
0.5	50	10.40	7.5	3.45	5	1.63
1.0	110	20.80	15.0	6.90	10	3.26
1.2 (a)	130	24.96				
1.5	150		20.0	10.35	15	4.89
2.0	205		25.0	13.81	20	6.52
2.5			31.0	17.26	27	8.15
3.0			37.5	20.71	32.5	9.78
4.0	Ultimate failure		55.0	27.61	45.0	13.13
5.0			77.5	34.51	61.0	16.29
6.0			95.0	41.42	75.0	19.55
7.0			112.5	48.32	88.0	22.80
7.5			142.5	51.77	97.5	24.44
7.6 (b)				52.46		24.76

Legend:
a. Appearance of the first cracks for beam D_1
b. Appearance of the first cracks for beam D_2 and D_3
c. Expt: Experiment.

5.7.1 Deflection of Composite Wall-Beam Corresponding to the First Visible Cracking Load of Reinforced Concrete Beam

The deflection of reinforced concrete beams and composite wall-beams at first visible cracking load of reinforced concrete beams may be compared from the load-deflection curves. Deflection of composite beams are much less than those of reinforced concrete beams. First visible cracking load of reinforced concrete beams of span 5ft. 5 in., 3 ft. 9 in. and 2 ft. 11 in. are 1.2^k, 7.6^k and 7.6^k respectively. It is found in Table 5.7 that the deflection of reinforced concrete beam D_1 having span 5 ft. 5 in. is 0.13 in. at 1.2^k load which is L/500 and at that load the deflection of all composite wall-beams are very negligible.

The deflection of reinforced concrete beam D_2 having span 3 ft. 9 inch is 0.145 in. which is L/311 at 7.6^k load whereas the deflection of composite wall-beam at that load is about 8 to 10 times less than that of reinforced concrete beam.

5.7.2 Deflection of the Composite Wall-Beam at Failure Load of Reinforced Concrete Beam

The failure load of reinforced concrete beams D_1, D_2 and D_3 were 4k, 9^k and 12^k. But the maximum deflection was recorded at 2^k, 8^k and 11^k respectively. The maximum deflection recorded for beam D_1, D_2 and D_3 were 0.205 in, 0.155 in. and 0.180 in. at 2_k, 8^k and 11^k load which were span/317, span/291 and span/195. At that load the deflection of composite wall-beam was 37, 10, 19 times less than the respective span of reinforced concrete beam.

5.8 Comparative Study of the Behavior of Composite Wall-Beam Using the Existing Formulae

It has been mentioned earlier that Wood(5) Stafford Smith and Riddington(10); and Davis and Ahmed (14) proposed various formulae for calculating maximum bending moment, and the axial tension developed in the supporting reinforced concrete beams. They have also suggested formulae for evaluating the maximum vertical stress in the wall of composite wall-beams. These formulae are critically examined for the composite wall-beams tested in this study. The maximum bending moment and axial tension of the supporting beam and maximum vertical stress in the wall are furnished in Table 5.8, 5.9 and 5.10 respectively. It is noteworthy that the existing formulae are for running bonded composite wall-beams. But the above mentioned formulae were also used for stack bonded composite wall-beams with and without vertical reinforcement (group-C and group-B respectively). It should also be noted that Wood, Stafford Smith and Riddington suggested their formulae for $H/L \geq 0.60$. But in this study, these formulae were also used for computing the forces and moment of some composite wall-beams having H/L ratio of 0.46.

5.8.1 Maximum Bending Moment of Supporting Beams

It is recognized that when external load is applied on a beam through a masonry wall, the distribution of the external load on the beam changes and concentrates the load near the supports. Thus, the maximum bending moment of the beam would be much less than the moment obtained considering the load to be uniformly distributed over the span.[14] On the basis of this conception, Wood;[5] Stafford Smith and Riddington;[10] and Davis and Ahmed[14] proposed formulae for calculating maximum bending moment of the supporting beam. Using their formulae, the maximum bending moment of the test beams are calculated in Appendix E for loads at which the composite wall-beam failed. The ultimate bending moment of reinforced concrete beams are also calculated for their respective failure loads. These moments are given in Table 5.8 for relative comparison.

Table 5.8 reveals that the maximum bending moment of supporting beam of composite wall-beams for their experimental failure loads predicted by Wood, is less than the ultimate moment corresponding to the failure load of reinforced concrete beam. The bending moment predicted by Wood is very close to the calculated moment capacity of reinforced concrete beam of uncracked section. Using Wood's formulae the bending moments are found to be lower than the ultimate moments. This may be due to the higher D/L ratio of supporting beam provided in the experimental beams.

Table 5.8 Maximum Bending Moment of Supporting Reinforced Concrete Beam Using Existing Formulae

Beam	Span	H/L ratio	TMC at USD	Maximum bending moment of beam in k-ft		
				Wood[5]	S and R[10]	D and A[14]
Running Bonded Composite Wall-Beam						
A1	5'-5"	0.46	2.99	1.27	1.86	1.95
A2	3'-9"	0.67	2.99	1.25	2.63	2.01
A3	2'-11"	0.86	2.99	1.38	3.73	2.54
Stack Bonded Composite Wall-Beam						
B1	5'-5"	0.46	2.99	0.84	1.23	1.29
B2	3'-9"	0.67	2.99	0.86	1.82	1.38
B3	2'-11"	0.86	2.99	0.91	2.45	1.67
Stack Bonded Composite Wall-Beam with vertical Reinforcement in Wall						
C1	5'-5"	0.46	2.99	1.14	1.66	1.74
C2	3'-9"	0.67	2.99	1.20	2.53	1.94
C3	2'-11"	0.86	2.99	1.23	3.33	2.27

Note:
TMC means Theoretical moment capacity
S and R = Smith and Riddington
D and A = Davis and Ahmed

Wood recommended D/L ratio in between L/15 to L/20 while the test beams had D/L ratio between L/10 to L/6. Kamal[16] also found in his finite element study that the more is the depth of beam, the more the beam behaves like a flexure member. However, Annamalai et. al[4] found the bending moment co-efficient in their experimental study as 1/33.75 to 1/38.67 instead of 1/100 proposed by Wood[5], although the D/L ratio of supporting beam was 0.0625. Rosenhaupt[9] tested composite wall-beam having hollow blocks in wall supported over reinforced concrete beam. He found the failure load of composite wall-beam (H/L = 0.63; test specimen wall-2) as 2.75 times higher than the failure load of reinforced concrete beam without wall (test specimen wall-5). Rosenhaupt[9] himself and Annamalai[4] also found that the modulus of Elasticity of wall is the most significant factor in composite wall-beam structure. They also found that the higher the modulus of Elasticity of wall, the higher is the failure load in composite wall-beam. However, test results of this investigation reveals that the maximum bending moment of beam was underestimated by Wood. It is recommended that further study is required to arrive at a more accurate value of the moment coefficient.

Stafford Smith and Riddington[10] proposed formulae for maximum bending moment of supporting beam for H/L = 0.60. The simplified form of the expression of maximum bending moment of beam is 0.95W k-in when the value of stiffness parameter is put in. (Appendix E). Where W is the total uniformly distributed load. The critical observation of their expression is that the maximum bending moment will be same for all span for a particular load if other parameters remain same. The maximum moment of supporting beam predicted by Smith and Riddington for running bonded composite wall-beam having H/L ratio 0.67 and 0.86 is found similar to the theoretical ultimate moment capacity as well as the experimental moment of beam D_1 ($D/L = 0.09$) which behaves like shallow beam. The moment coefficient can be found from the maximum bending moment of supporting beam of running bonded composite wall-beam A_1, A_2 and A_3 having H/L ratio 0.46, 0.67 and 0.86 predicted by Smith and Riddington (Table 5.8) to be 1/68, 1/47, and 1/37 at failure loads which is not similar to the moment coefficient suggested by Wood. The moment coefficient of stack bonded composite wall-beam with or without vertical reinforcement is also found similar to that of running bonded composite wall-beam. It should be noted here that the composite wall-beam A_1 having H/L ratio 0.46, which is less than their proposed H/L ratio of 0.6, the calculated moment is found to be less than the ultimate moment capacity of the beam.

Table 5.8 also reveals that the maximum bending moment calculated using the formulae proposed by Davis and Ahmed, is less than the ultimate moment capacity of beam at failure load. They introduced a flexural stiffness parameter R_f as $(E_w t h^3/E_b I_b)^{1/4}$ for their method. According to their formulae, the ultimate load is found to be 37.33^k, 59.8^k, and 56.41^k, for running bonded composite wall-beam of H/L ratio 0.46, 0.67 and 0.86 respectively. Thus, the loads predicted on the basis of theoretical ultimate flexural moment capacity are found to be much higher than the experimental load of 23.5^k, 33.2^k and

47.2^k for the same group. This load can be predicted close to the experimental value if the flexural stiffness parameter R_f is modified. The proposed flexural stiffness (R_p) instead of R_f is $(E_w th^3/E_b bd^3)^{1/4}$. The predicted load using R_p in their expression of moment is found to be 21.35^k, 29.9^k and 33.88^k against the experimental load 23.5^k, 33.2^k and 47.2^k respectively for running bonded composite wall-beams. It can be explained otherwise that the moment coefficient of supporting beams are found to be 1/66, 1/72, and 1/54 for running bonded composite wall-beam A_1, A_2, and A_3 respectively if flexural stiffness parameter R_f is used. Whereas these coefficient will be 1/39, 1/38, and 1/32 if R_p is used which brings the calculated bending moment more close to the experimental bending moment and the coefficients themselves become similar to the moment coefficient predicted from the moment given by Smith and Riddington formulae.

5.8.2 Axial Force in the Supporting Reinforced Concrete Beam

Axial force in the reinforced concrete supporting beam of composite wall-beams corresponding to the failure load found in this investigation are calculated (Appendix E) using the formulae suggested by Wood[5], Smith and Riddigton[10] and Davis and Ahmed.[14] The results are shown in Table 5.9. It should be noted that existing expressions are only for running bonded brick wall of composite wall-beam. Axial forces for stack bonded composite wall-beams are also calculated same formula here. The Table 5.9 reveals that the predicted axial force of supporting reinforced concrete beam of running bonded composite wall beam A_2 having H/L ratio 0.67 is found similar to the value from Wood's formula and Smith's formula. But these values are about 50% higher than the value given by

Table 5.9 Axial force in the supporting Reinforced Concrete Beam Using Different Formulae

Beam	Span	H/L	Depth of RC beam	Experimental load in kips	Maximum Axial force in bottom beam in kips		
					Wood[5]	S and R[10]	D and A[14]
Running Bonded Composite Wall-Beam							
A1	5'-5"	0.46	6"	23.50	9.58	6.91	6.35
A2	3'-9"	0.67	6"	33.20	9.29	9.76	9.30
A3	2'-11"	0.86	6"	47.20	10.29	13.88	11.33
Stack Bonded Composite Wall-Beam							
B1	5'-5"	0.46	6"	15.50	6.30	4.56	4.19
B2	3'-9"	0.67	6"	23.00	6.44	6.76	6.44
B3	2'-11"	0.86	6"	31.00	6.76	9.12	7.44
Stack Bonded Composite Wall-Beam with vertical Reinforcement in Wall							
C1	5'-5"	0.46	6"	21.00	8.56	6.18	5.67
C2	3'-9"	0.67	6"	30.00	8.40	8.82	8.40
C3	2'-11"	0.86	6"	42.00	9.16	12.35	10.08

Note: S and R = Smith and Riddington
D and A = Davis and Ahmed

Davis and Ahmed. In case of beam A_3, the running bonded composite wall-beam having H/L ratio of 0.86, the axial force calculated based on Davis and Ahmed is similar to that calculated by Wood while Smith and Riddington's formula over estimates by about 30%. Whereas the axial force calculated using Davis and Ahmed formula for composite wall-beam A_1 having H/L ratio 0.46 is about 48.30% higher than that based on Wood's formula but similar to the value obtained by using Smith and Riddington's method although they suggested their formula for H/L greater than 0.60.

Similar relation are found in stack bonded composite wall-beam with or without vertical reinforcement in the wall. But these values are about 1.1 and 1.5 times less than the respective running bonded composite wall-beams.

5.8.3 Maximum Vertical Stress in the Wall

Maximum vertical stress in the wall corresponding to the failure load found in this investigation of composite wall-beams are calculated using the formulae suggested by Wood[5], Stafford Smith and Riddington[10] and also Davis and Ahmed.[14] Detail calculation is shown in Appendix E and the results are shown in Table 5.10. They suggested these expressions for running bonded brick wall of composite wall-beams only. But the maximum vertical stress is also calculated for stack bonded composite wall-beam of group-B and Group C using the same formulae for running bonded brick wall. The vertical stress in the running bonded composite wall-beam A_1 having H/L ratio 0.46 can not be predicted well by the formula proposed by Wood, and Smith and Riddington as H/L ratio< 0.60. Yet, the vertical stress is calculated for this beam A_1 using the formula whose validity is said to be for H/L >0.60. Maximum vertical stress based on Wood, Smith and Riddington were found to be higher than that of Davis and Ahmed for all the test beams.

It should be mentioned here that, the vertical stress in the wall predicted from the formula proposed by Stafford Smith and Riddington; and Wood is about 4 times higher than compressive strength of brick masonry prism when H/L = 0.86. This ratio gradually decrease with the decrease of H/L ratio is shown in Table 5.10.

Davis and Ahmed introduced a flexural stiffness parameter as $R_f = (E_w th^3 / E_b I_b)^{1/4}$ for their method of vertical stress calculation. If this parameter is modified as $R_P = (E_w th^3/E_b bd^3)^{1/4}$ which is ratio of wall stiffness to beam stiffness, the maximum vertical stress in the wall is found to be 459 psi, 761 psi and 1166 psi for running bonded composite wall-beam of H/L ratio 0.46, 0.67 and 0.86 respectively.

Table 5.10 Maximum Vertical Stress in the Wall Using Existing Formulae

Beam	Span	H/L	Experimental load in kips	Maximum vertical stress in the wall in psi		
				S and R[10]	D and A[14]	Wood[5]
Running Bonded Composite Wall-Beam						
A1	5'-5"	0.46	23.50	1446	787	1004
A2	3'-9"	0.67	33.20	2167	1279	2049
A3	2'-11"	0.86	47.20	3207	1918	3746
Stack Bonded Composite Wall-Beam						
B1	5'-5"	0.46	15.50	954	519	662
B2	3'-9"	0.67	23.00	1501	886	1420
B3	2'-11"	0.86	31.00	2106	1260	2460
Stack Bonded Composite Wall-Beam with vertical Reinforcement in Wall						
C1	5'-5"	0.46	21.00	1292	704	897
C2	3'-9"	0.67	30.00	1958	1156	1852
C3	2'-11"	0.86	42.00	2854	1707	3333

Note:
S and R = Stafford Smith and Riddington
D and A = Davis and Ahmad

This result is very close to the max^{m} vertical stress predicted by approximate formula 5W/Lt derived from a finite element study on composite wall-beam by kamal[16]. The maximum vertical stress in the wall predicted from the approximate formula 5W/Lb is 401 psi,820 psi and 1498 psi as against 459 psi, 761 psi and 1166 psi of Davis and Ahmed using modified ratio of flexural stiffness R_p.

CHAPTER-6

PREDICTION OF LOAD FOR COMPOSITE WALL-BEAM

6.1 Introduction

Composite wall-beam structures as described in chapter 3 (Figs. 3.1 to 3.3) have been tested to failure and the experimental failure loads are given in Table 4.1. It may be mentioned here that the previous investigators[5,10,14] suggested formulae for calculations of design parameters such as maximum bending moment and axial force in the bottom reinforced concrete beam, maximum vertical stress in the wall, maximum horizontal shear stress at the wall-beam interface for a given load. Load carrying capacity of the composite wall-beam can not be found out using the existing formulae.

An attempt has been taken in this study to predict the load carrying capacity of a composite wall-beam. The load carrying capacity of composite wall beams have been determined by introducing the concept of reinforced concrete beams in flexure. Two different approaches have been considered. In the first approach, the brick wall has been transformed into equivalent concrete section having the same depth as that of the composite wall-beam. The ultimate load was calculated based on the moment capacity of this equivalent concrete beam considering the section to be uncracked i.e. when the maximum tensile stress reaches the modulus of rupture of concrete. The second approach is based on ultimate strength design concept of reinforced concrete beams. Here the compressive forces in the brick work above the neutral axis is equated to the tensile forces developed in the reinforcements considering steel to be yielding at failure. The effect of bond pattern in brick wall and vertical reinforcement in the wall are not considered in the calculation of loads. Therefore, predicted load for all types of composite wall-beams are found equal for the same span irrespective of bond pattern and vertical reinforcement in the wall, whereas the experimental loads are found to be different for different groups.

The loads of composite wall-beam have also been predicted using the concept of reinforced concrete deep beam with appropriate modification. The composite wall-beam with relatively low L/H ratios has been considered as Deep Beam.

Finally, maximum vertical stress in the wall of composite wall-beam has been calculated considering the findings of an elastic finite element study.

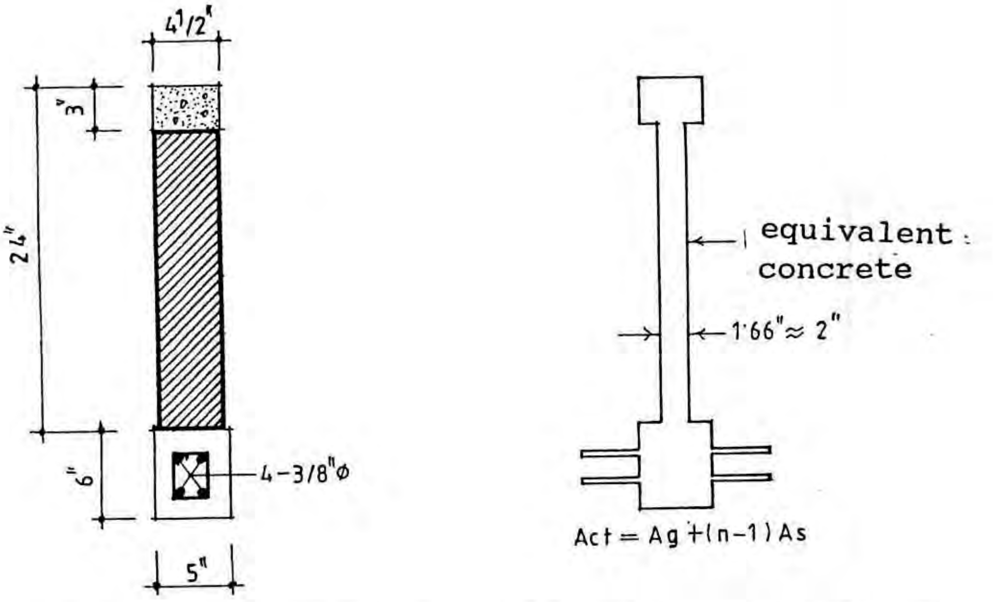
6.2 Section Uncracked Approach

The reinforced concrete beam and the brick wall above it is called composite wall-beam in this study. The reinforced concrete beam is also a composite of mild steel and concrete. The total beam is a composite of mild steel, concrete and brick wall. These three materials have been transformed into equivalent concrete and the section considered to remain uncracked. The bottom reinforced concrete beam is transformed into equivalent concrete and then the width of brick wall is transformed into the equivalent width of concrete wall using the modular ratio of these materials (Fig.6.1). The modulus of rupture of concrete (f_r') was found to be 450 psi considering $f_r' = 10\sqrt{f_c'}$. The moment of inertia (I_c) was determined to be 6165 in^4. Detail are shown in Appendix F. The predicted load and the experimental load of composite wall-beams are shown in Table 6.1.

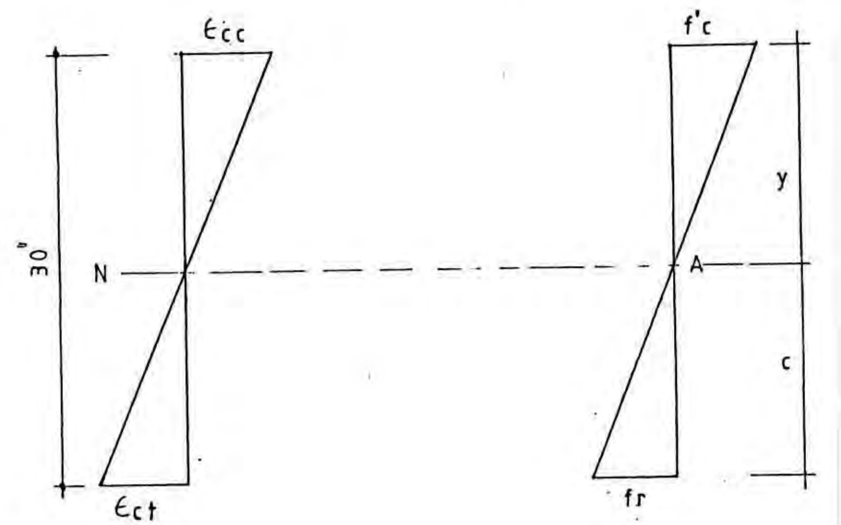
Table 6.1 Predicted Load and Experimental Load of Composite Wall-beams Based on Uncracked Section

Effective span	predicted load	Experimental load in kips			Ratio of predicted load to Experimental load		
		Group A	Group B	Group C	Group A	Group B	Group C
5'-5"	23.0	23.5	15.0	21.0	0.98	1.48	1.10
3'-9"	33.26	33.2	23.0	30.0	1.00	1.45	1.11
2'-11"	42.72k	47.2	31.0	42.0	0.91	1.38	1.02

From Table 6.1, it is found that the predicted loads are very close to the experimental load for Group A having running bonded brick walls whereas it is about 1.38 to 1.48 times the experimental load for Group B having stack bonded brick walls.



(a) actual section (b) transformed section (concrete equivalent)



(c) strain distribution through the section (d) stress diagram

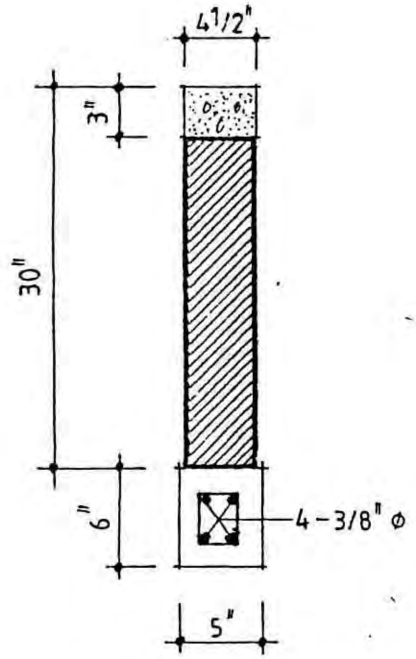
Fig.6.1 Stress-Strain Distribution of Composite Wall-Beam at Uncracked Section

It is to be noted that in case of Group C having stack bonded brick wall with vertical reinforcement in the wall, the predicted load is about 1.0 to 1.11 times the experimental failure load showing close correlation between experimental and predicted values. It may be mentioned here that Group A and Group B failed just after the initiation of first crack exhibiting brittle nature of failure.

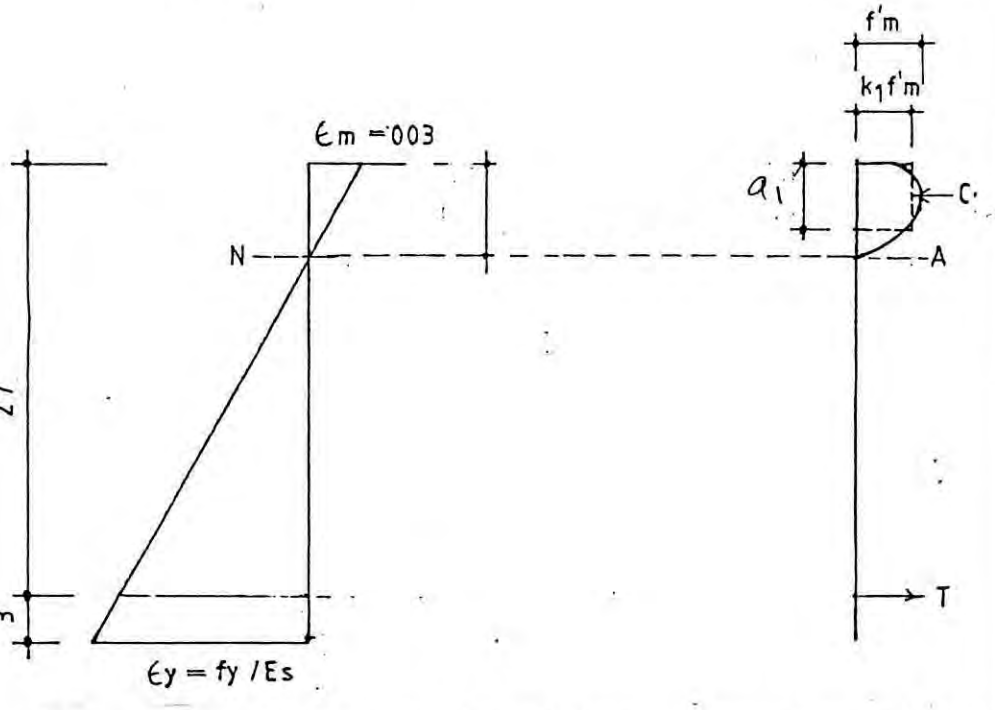
Though Group A and Group B beams exhibited brittle behavior the predicted loads of Group A are very close to the experimental loads. Experimental loads for Group B beams are lower as because, the presence of vertical joint across the whole depth of the beam. Thus, the beams failed prematurely with initiation of a crack. The situation is found to have improved with the inclusion of vertical reinforcement as depicted by Group C beams.

6.3 Ultimate Strength Design Approach

Ultimate load of composite wall-beam have been predicted neglecting the contribution of concrete of bottom reinforced concrete beam as shown in Fig. 6.2. Detail calculations are shown in Appendix F. From the calculations it is found that the composite wall-beams failed in tension. Predicted load and the experimental load of composite wall-beams are shown in Table 6.2. From Table 6.2, it is found that the predicted loads are about 1.65, 2.46 and 1.84 times the experimental



(a) actual section



(b) strain at balanced condition (c) stress block

Fig.6.2 Stress-Strain Distribution of Composite Wall-Beam at Ultimate State

load for Group A, Group B and Group C beams respectively. It is clear that the predicted loads using this method is quite high compared to the experimental loads. This is what may be expected because the concept of steel yielding is far from reality in composite wall beams if composite action really develops. Normally, the stress level in the steel of supporting beam remain much below yield value at failure. Even the compressive stress in the top brick work do not reach close to the ultimate strength of prisms. Rather the failure of composite beams under concentrate loads are more close to shear failure of reinforced concrete beams. However, under uniform loading along with low H/L ratio when composite action disappears this method can yield good results. Thus, it has been found that the beam of Burhouse having H/L ratio equal to 0.33 the load predicted using this USD method was close to within 8% of the experimental load.

Table 6.2 Predicted Load and Experimental Load of Composite Wall-Beam Based on U.S.D. method

Effective span	predicted load	Experimental load in kips			Ratio of predicted load to Experimental load		
		Group A	Group B	Group C	Group A	Group B	Group C
5'-5"	39.39	23.5	15.5	21.0	1.68	2.54	1.88
3'-9"	56.93	33.2	23.0	30.0	1.71	2.48	1.90
2'-11"	73.11	47.2	31.0	42.0	1.55	2.35	1.74

6.4 A Simplified Method for Prediction of Load of Composite Wall-Beam Structure

Load carrying capacity of composite wall-beam structures can be predicted closely considering the total section uncracked. This has been discussed in Art.6.2. The steps include the calculation of modulus of elasticity of materials, equivalent transformed area, moment of inertia (I), distance of bottom fibre from the neutral axis (y_b),distance of top fibre from the neutral axis (y_t) modulus of rupture of concrete (f_r') etc. of composite wall-beam structure for determination of moment capacity of uncracked section. Then, equating the moment capacity with the bending moment equation, the predicted load is obtained. For simplification of this procedure, a number of graphs and a simple formula is developed. Load of composite wall-beam structure can easily be predicted with the help of these graphs and formula.

6.4.1 Description of the Simplified Method

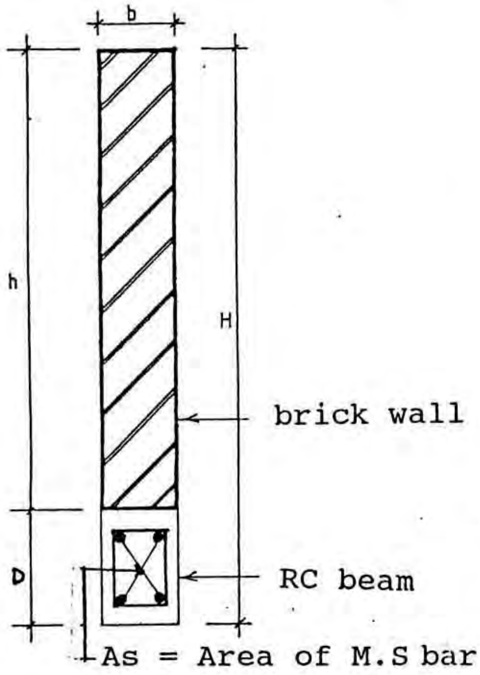
The strength of materials as well as the dimension of composite wall-beam structure has the influence on its load carrying capacity. Considering all the parameters, a simple formula is developed for the prediction of load.

This formula is given below

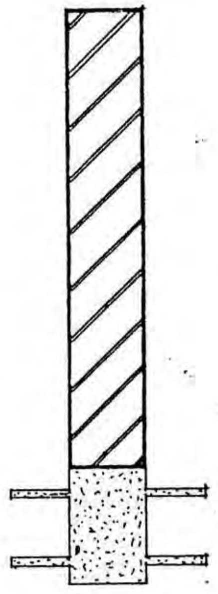
$$W = (bH^2/228) \times C_3/C_2 \times f_b' K \quad \ldots \quad (6.1)$$

Where, W = Total load of composite wall-beam structure in lbs, b = Width of wall in inch, H = Total height of composite wall-beam in inch, L = Effective span length in inch, f_b' = Compressive Strength of brick, k = Moment coefficient. The value of k for third point loading is 6 and for uniformly distributed loading is 8. C_3 is the ratio of I_m/I_g where I_g is the moment of inertia of gross area of composite wall-beam, I_m is the moment of inertia of composite wall-beam into masonry equivalent and C_2 is the ratio of distance of neutral axis from bottom fibre (y_b) to total height of composite wall-beam. Detail derivation is given in Appendix G. Dimensions of composite wall-beam and transformed area are shown in Fig. 6.3. The value of C_3 and C_2 depend on the value of C_1 and C_4. C_1 is the ratio of transformed area in equivalent brick masonry to the gross area of RC beam. The value of C_1 is depend on the compressive strength of brick, concrete and the ratio of longitudinal reinforcement in the beam. C_4 is the ratio of the total depth of bottom beam (D) to the total height of composite wall-beam (H).

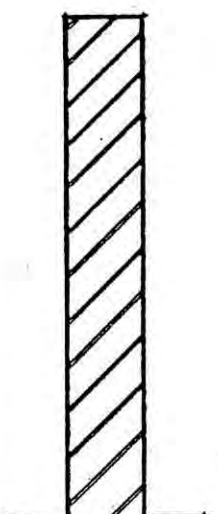
The value of C_1 can be obtained from the equation 6.2 or from the Figs. 6.4 to 6.6. The value of C_2 and C_3 can be computed from the equations 6.3 and 6.4 or from the Figs. 6.7 and 6.8 respectively.



(a) actual section



(b) Transformed area of RC beam into equivalent concrete



(c) Transformed area of RC beam into equivalent brick masonry

Fig.6.3 Transformed Area of Composite Wall-Beam

Equation 6.2, 6.3 and 6.4(a), 6.4(b), 6.4(c) are given below:

$$C_1 = [1+(509/\sqrt{f_c'} - 1)\ p]\ 190\ \sqrt{f_c'}/f_b' \qquad (6.2)$$

Where p = steel ratio

$$C_2 = (1 - C_4^2 + C_1C_4^2)/[2(1-C_4 + C_1C_4)] \qquad (6.3)$$

$$C_3 = (1.00 + 0.144\ C_1) \text{ when } C_4 = 0.1 \qquad (6.4a)$$

$$C_3 = (1.08 + 0.146\ C_1) \text{ when } C_4 = 0.15 \qquad (6.4b)$$

$$C_3 = (1.16 + 0.153\ C_1) \text{ when } C_4 = 0.20 \qquad (6.4c)$$

The analytical expressions of the equations C_1, C_2 and C_3 are given in Appendix G.

6.4.2 Procedure for the Prediction of Load

The following steps are to be followed for prediction of load of composite wall-beam structure using this simplified method.

i) The compressive strength of brick (f_b') used in the wall of composite wall-beam should be known.

ii) The compressive strength of concrete (f_c') used in the bottom beam should be known.

iii) The cross-sectional area of bottom beam and its longitudinal reinforcement to be found out from the given section.

iv) The percentage of longitudinal reinforcement in the bottom beam to be calculated.

v) The value of C_1 to be found out from the equation 6.2 or from the Figs. 6.3 to 6.5 corresponding to the compressive strength of brick and concrete and also percentage of reinforcement.

vi) The value of C_4 to be calculated from the given section. C_4 is the ratio of the total depth of bottom beam (D) to the total height of composite wall-beam structure (H).

vii) Coefficient of bending moment 'k' to be found out from the type of applied load.

viii) The value of C_2 and C_3 to be determined using equations 6.3 and 6.4 or from Figs. 6.7 and 6.8.

ix) Putting the value of b, H, L, C_2, C_3, f_b' and k in the equation 6.1, the load carrying capacity of composite wall-beam can be obtained.

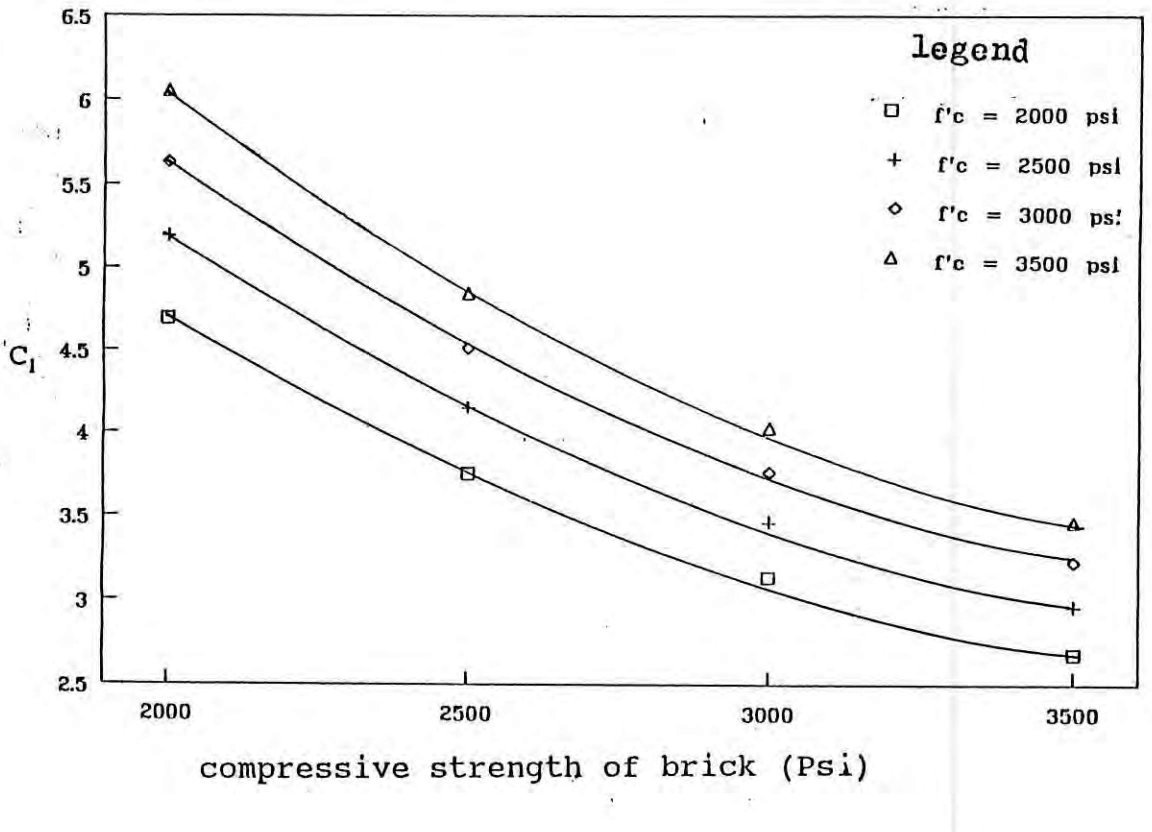


Fig.6.4 Value of C_1 at 1% Reinforcement

Note

C_1 is the ratio of transformed area in equivalent brick masonry to the gross area of reinforced concrete beam

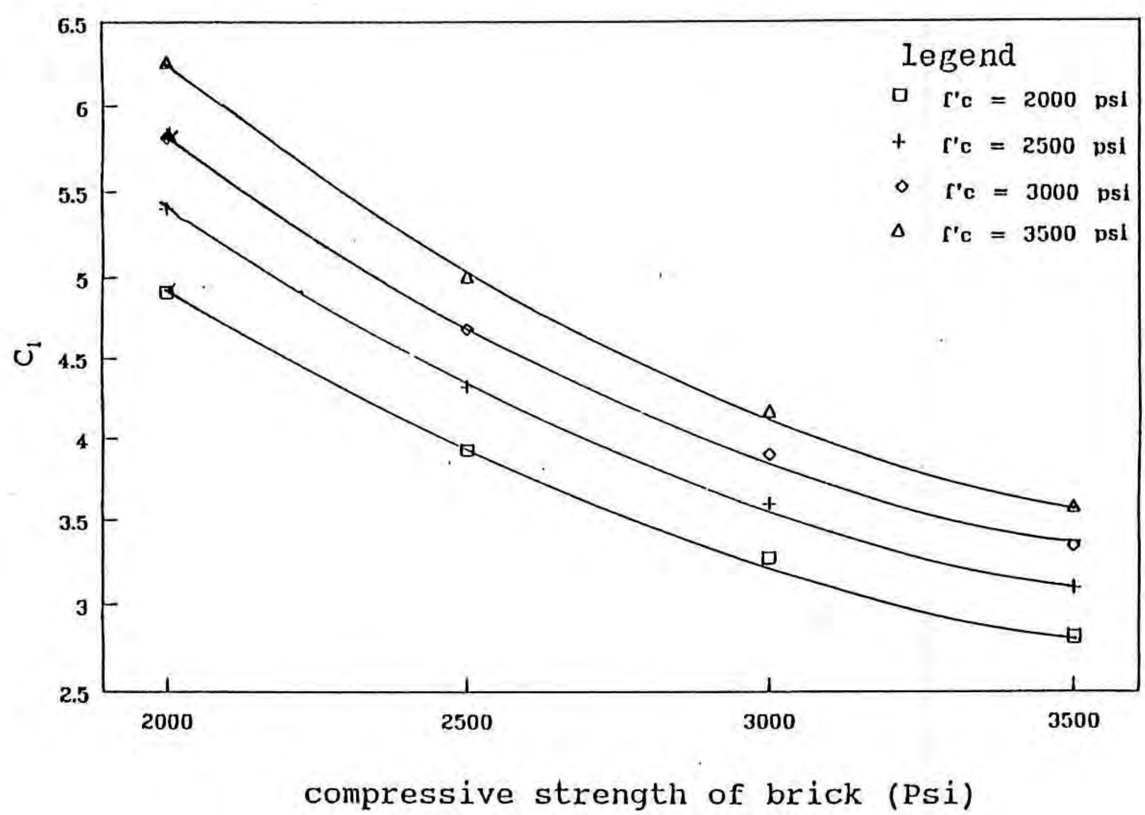


Fig.6.5 Value of C_1 at 1.5% Reinforcement

Note

C_1 is the ratio of transformed area in equivalent brick masonry to the gross area of reinforced concrete beam

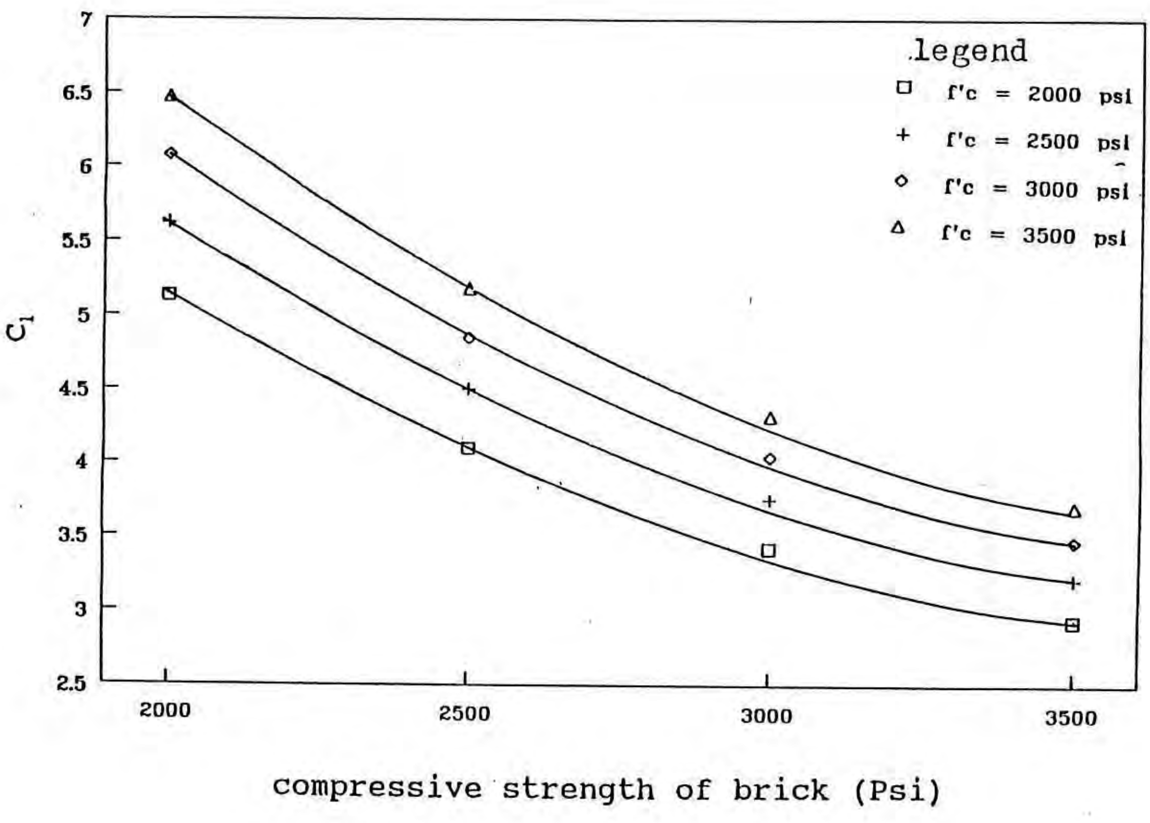


Fig.6.6 Value of C_1 at 2% Reinforcement

Note

C_1 is the ratio of transformed area in equivalent brick masonry to the gross area of reinforced concrete beam

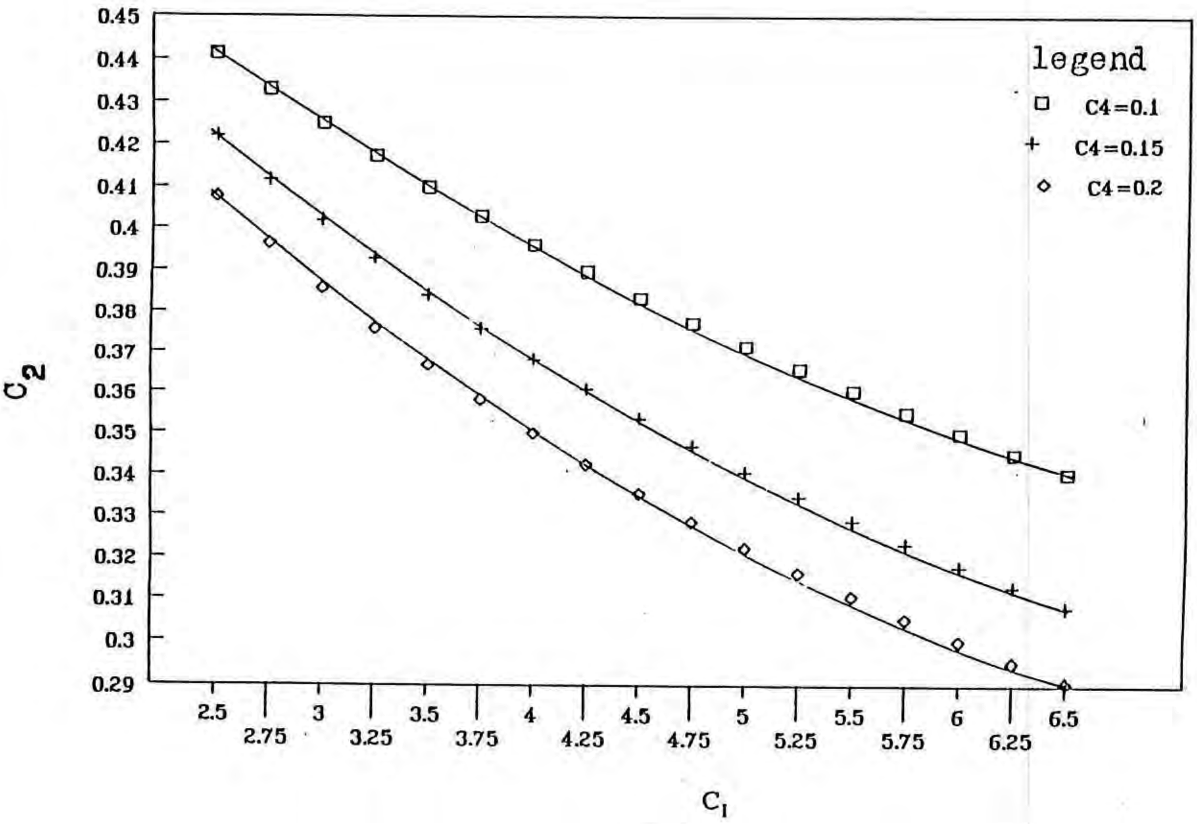


Fig.6.7 Value of C_2

Note

C_2 is the ratio of the total depth of bottom reinforced concrete beam (D) to the total height of composite wall-beam(H)

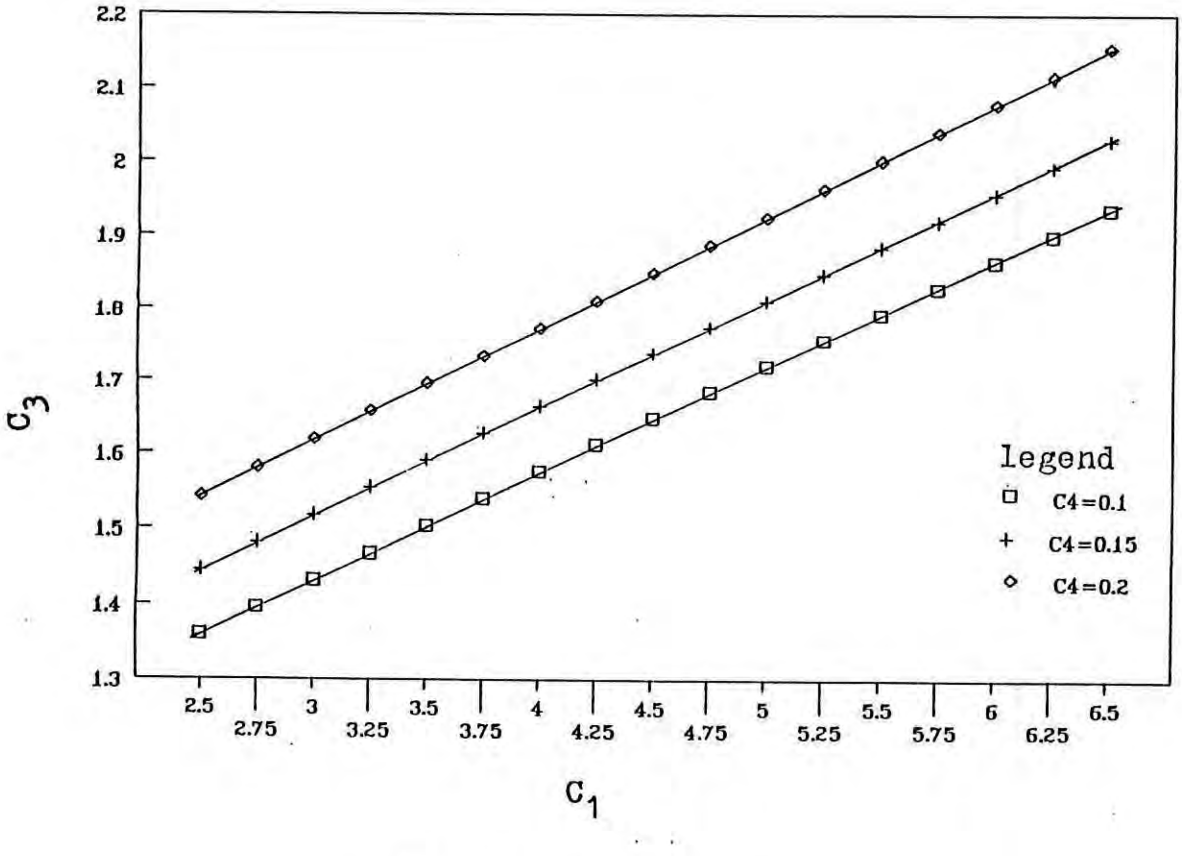


Fig.6.8 Value of C_3

Note

C_3 is the ratio of moment of inertia of composite wall-beam into masonry equivalent to the gross moment of inertia of composite wall-beam (I_m/I_g)

6.4.3 Verification of the Suggested Method

The performance of this method in predicting the load carrying capacity of composite wall-beams have been tested against the experimental loads of some composite beams available in the literature. The test results of previous works carried out by Annamalai 'et al[4]', and Burhouse[7] on composite wall-beam have been chosen for comparison. Experimental load of three running bonded composite wall-beams of present study (Group A), two composite wall-beams tested by Annamalai' et al'[4]; and three composite wall-beam tested by Burhouse[7] has been compared with the predicted load using this method. The results are shown in Table 6.3. The Table 6.3 reveals that the predicted load of composite wall-beam tested in this present study is about ninety five percent of their experimental loads. It is also evident from the Table 6.3 that the predicted load of beam RC_3 tested by Annamalai[4] is about eighty five percent of its experimental load whereas the predicted load of beam RC_4 is about ten percent highre than the experimental load.

The predicted load of composite wall-beam (Beam No.6) tested by Burhouse is about 75% of its experimental load. But the experimental loads of his other two beams (Beam No.8 and Beam No.9) are wide apart from their predicted load using this method. It may be mentioned here that in beam No.8, the slenderness ratio of the wall is 18. Due to the large height of wall warping stress may have developed in the wall resulting lower value of experimental load than could be expected. In beam No.9, the H/L ratio is 0.33, which is very low, due to which composite action between the wall and bottom beam could not have developed. Flexural action dominates the behavior of such beams with low H/L ratio. In fact the predicted ultimate load considering USD concept of design neglecting the contribution this beam was found to be 100^k which is quite close to the experimental load.

6.5 Deep Beam Design Approach

Ultimate shear capacity of composite wall-beams have been predicted using the concept of deep beam action. Due to non-availability of such method for predicting shear capacity of composite wall-beam, ACI method for reinforced concrete deep beam has been modified to some extent for predicting the ultimate shear capacity of composite wall-beams. Therefore, ACI code provisions for reinforced concrete deep beams have been discussed here first and the appropriate changes have been suggested later.

Table 6.3 Comparison Between the Predicted Load Using Simplified Method to the Experimental Load of omposite Wall-Beams with the Available Test Results.

Research Designation	Beam identification	Expremental load in kips	Predicted load in kips	PL/EL
Present Study	A_1 span= 5'-5" H/L = 0.46	23.50	22.47	0.96
	A_2 span= 3'-9" H/L = 0.67	33.20	32.46	0.98
	A_3 span= 2'-11" H/L = 0.86	47.20	41.74	0.88
Experiment by Annamalai[4]	RC_3 Span= 48" H/L = 0.58	55.10	48.83	0.89
	RC_4 Span= 48" H/L = 0.58	36.96	43.09	1.17
Experiment by Burhouse[7]	No.6 Span= 12'-0" H/L = 0.58	132.00	98.87	0.75
	No.8 Span=12'-0" H/L = 0.83	109.56	181.42	1.66
	No.9 Span= 12'-0" H/L = 0.33	115.28	39.47	0.34

6.5.1 Code Provisions for Reinforced Concrete Deep Beam

Concrete contribution to shear strength of reinforced concrete deep beam

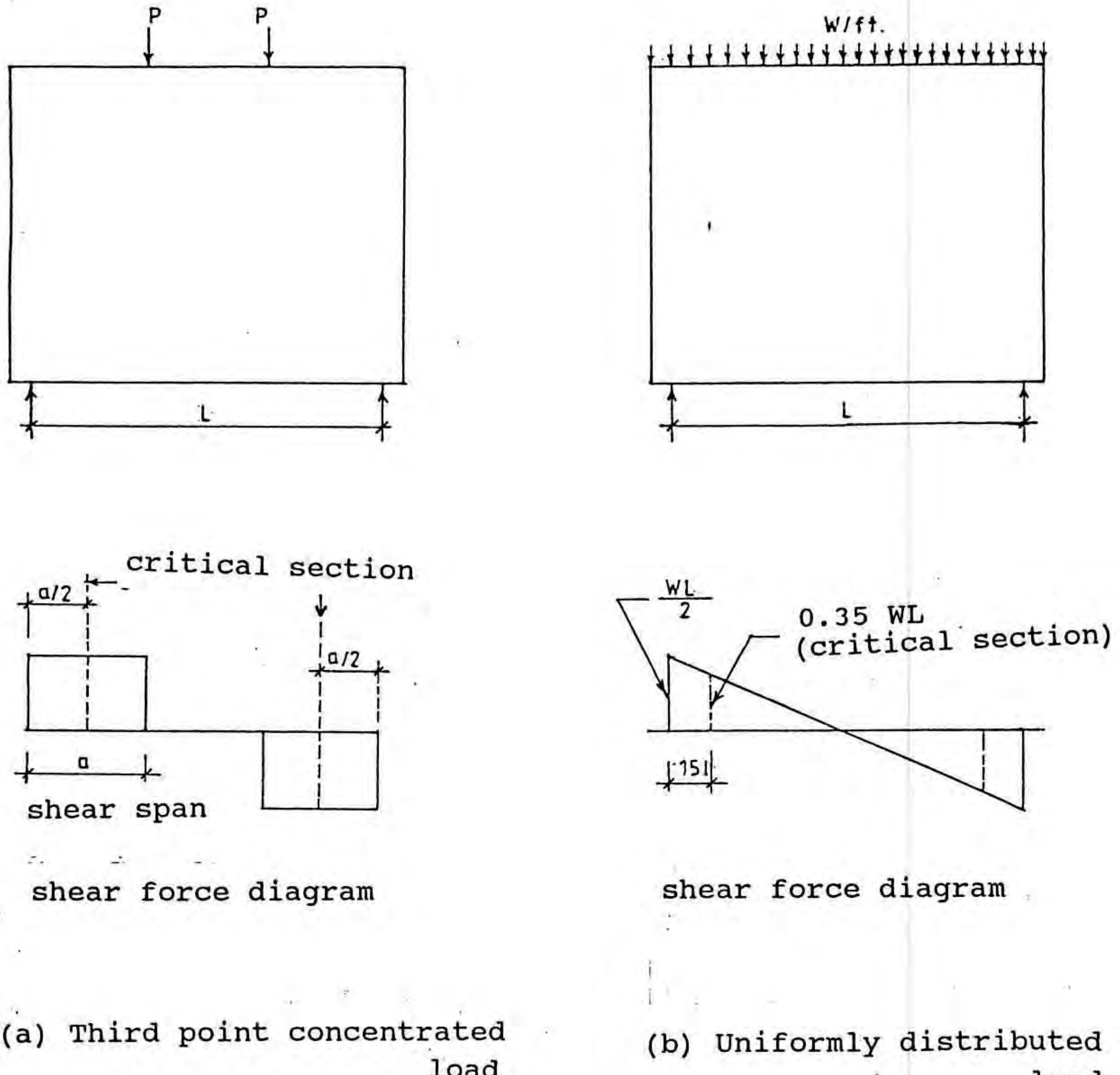
$V_c = (3.5-2.5\ M_u/V_u\ d)\ (1.9\ \sqrt{f_c'} + 2500\ p\ V_u d/M_u)\ bd$ ... (6.5)

Because of the strength increase attainable for deep beams due to tied arch action, code provisions permit the usual value of the concrete shear strength Vc, calculated by

$V_c = (1.9\ \sqrt{f_{c'}} + 2500\ p\ V_u d/M_u)\ bd$ 6.6

to be increased by a multiplier which depends upon the ratio $M_u/V_u d$. The multiplier $(3.5 - 2.5\ M_u/V_u d)$ must not exceed 2.5 and that V_c must not be greater than $6\ \sqrt{f_c'}bd$. Where M_u and V_u are the moment and shear force, at factored loads, occurring simultaneously at the critical section.

According to ACI code, the critical section for shear is to be taken a distance 0.15L from the face of the supports for uniformly distributed loads and 0.5a (where 'a' is the shear span) for beams with concentrated loads is shown in Fig. 6.9 but not to exceed a distance 'd' from the face of the support in either case. [28]



(a) Third point concentrated load

(b) Uniformly distributed load

Fig.6.9 Critical Section of Shear for Deep Beam

From Fig. 6.9(b), bending moment at critical section for uniformly distributed load can be found as

$M_u \quad = w\ L/2 \times 0.15\ L - w/2 \times (0.15\ L)^2$

$\quad\quad = 0.064\ w\ L^2$

Therefore,

$M_u/V_u d = (0.064\ w\ L^2)\ /\ (0.35\ w\ L \times d)$

$\quad\quad = 0.182\ L/d$

similarly, from Fig. 6.9(a) bending moment at critical section for third point concentrated load can be found as

$M_u \quad = P \times a/2$ and $V_u \ = P$

Therefore,

$M_u/V_u d = P \times a/2 \times 1/Pd = a/2d$

$\quad\quad = L/3 \times 1/2d = L/6d = 0.17\ (L/d)$

So, $M_u/V_u d$ is more or less similar for both uniformly distributed load and third point concentrated load for deep beams. Now the equation 6.5 yields -

For uniformly distributed load

$V_c = (3.5 - 0.455\ L/d)\ (1.9\ \sqrt{f_c'} + 13750\ pd/L)\ bd$ ...(6.7)

And for third point concentrated load

$V_c = (3.5 - 0.425\ L/d)\ (1.9\ \sqrt{f_c'} + 14706\ pd/L)\ bd$ .... (6.8)

6.5.2 Modification of ACI Code Provision of Reinforced Concrete Deep Beam for Composite Wall-Beam

It has been mentioned earlier that in the ACI code there is no provision for calculating the shear capacity of composite wall-beam . So, the ACI code provision for reinforced concrete deep beam is modified for predicting shear capacity of composite wall-beams. Two different approaches have been suggested here for the modification.

According to ACI code, the concrete contribution to shear strength is given in equation 6.5. The first part of this equation 6.5, is the magnifying factor which increase the usual value of concrete shear strength calculated by equation 6.6. Here the first approach is to introduce such a magnifying factor for composite wall-beam which will increase the ultimate shear strength of shallow composite wall- beam without shear reinforcement.

The second approach is to convert the cross-sectional area of deep composite wall-beam into a equivalent cross-sectional area of reinforced concrete deep beam. Thereafter, shear strength will be calculated according to equation 6.5. In this approach the width of brick wall will be changed using the modular ratio of concrete and brick masonry while the depth of beam will remain same. Detail of these approaches are described below.

a) Introducing a Magnification Factor

Ultimate shear capacity of composite wall-beam has been calculate according to the equation 6.5 considering its limitation and it was found that the calculated loads are much higher than the experimental loads. So, it needs some modifications. Ultimate shearing strength of reinforced brick beams without shear reinforcement[27] is

$$V_m = 1.5 \sqrt{f_m'} + 3000 \; p \; V_u d/M_u \qquad \ldots \quad (6.9)$$

where, V_m = Ultimate shear force of reinforced brick beams without shear reinforcement; f_m' = Compressive strength of brick masonry prism; p = steel ratio; V_u = Shear force at section at factored loads; M_u = Moment at section at factored loads.

For composite wall-beam like reinforced concrete deep beam a magnification factor may be introduced. Magnification factor for composite wall- beam has been predicted from experimental loads of beam (Group A) having running bonded brick wall. The total masonry contribution to shear strength of composite wall-beam without shear reinforcement is found to have provided three different equations comparing experimental loads of three beams of Group A. It should be noted that the preceding magnification factors are different.

Derivations are shown in Appendix H and the expressions are given below -

$$V_m = (2.35-2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m'} + 3000\ p\ V_ud/M_u)b\ d \quad .. (6.10)$$

$$V_m = (2.38-2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m'} + 3000\ p\ V_ud/M_u)b\ d \quad .. (6.11)$$

$$V_m = (2.68-2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m'} + 3000\ p\ V_ud/M_u)b\ d \quad .. (6.12)$$

From the equations 6.10, 6.11 and 6.12 it evident that only the first previously termed as magnification factor has been changed on the basis of the experimental loads of beam A_1, A_2 and A_3 respectively.

In order to select the best of these equations, that correlate well with all the beams, load has been predicted using each of these three equations and compared with experimental loads as shown in Table 6.4. From Table 6.4, it is seen that the predicted load considering equation 6.10 is very close to the experimental load of all the beams of Group A. Whereas the load predicted by using equations 6.11 and 6.12 shows greater scatter in result for the two beams other than the one from which it was derived. Therefore, it may be concluded that the first part of the magnification factor will be close to 2.35. However, more detail investigation is required in order to conclusively decide on the value of this factor or even the equation.

Table 6.4 Predicted Load of Composite Wall-Beam Considered Deep Beam Using Magnification Factor

Effective span	Beam	Experimental load in kips	predicted load using equation in kips		
			Equ.10	Equ.11	Equ.12
5'-5"	A1	23.5	22.54	28.54	28.15
	B1	15.5	-	-	-
	C1	21.0	-	-	-
3'-9"	A2	33.2	32.05	32.80	38.49
	B2	23.0	-	-	-
	C2	30.0	-	-	-
2'-11"	A3	47.2	39.32	40.00	46.57
	B3	31.0	-	-	-
	C3	42.0	-	-	-

It should be mentioned here that the loads have been predicted for composite wall-beams tested by Annamalai 'et al'[2] and Burhouse[18] using the proposed equation 6.10. The predicted and experimental loads have been compared in Table 6.5. From Table 6.5 it reveals that the predicted load considering composite wall-beam as reinforced brick work deep beam using magnification factor is about 0.74 (av. of two beams) times the experimental load of composite wall-beam tested by Annamalai 'et al'[4]. It is also found from the Table 6.5 that the predicted load is 0.51 and 0.25 of experimental load respectively. In beam No.9, the H/L ratio is 0.33 which is very low, due to which composite action between the wall and bottom beam could not have developed. Flexural action dominates the behavior of such beam with low H/L ratio.

b) Using Modular Ratio

In this approach the width of brick wall will be converted into equivalent width of the concrete using modular ratio of concrete and masonry. For example, the width of brick wall of composite wall-beam tested in this study is 4.5 inch and modular ratio of concrete and masonry wall is 2.71. Therefore, the equivalent width in concrete is 1.66 inch. Now, the ultimate shear capacity will be calculated according to ACI code of provisions for reinforced concrete deep beam given in equation 6.5. The predicted load in this approach is twice the ultimate shear capacity.

The loads have been predict for composite wall-beams tested in this study and also for beams tested by Annamalai 'et al'[4] and Burhouse[7]. The comparison between the predicted load and experimental load is shown in Table 6.6. From Table 6.6 it reveals that predicted load is about 0.88 to 1.30 times experimental load of composite wall-beam tested in the present study. Whereas the predicted load is about 0.74 times the experimental load ofcomposite wall-beam tested by Annamalai [4].

The Table 6.6 also reveals that, the predicted load of beam No.8 tested by Burhouse is very close to the experimental load. The ratio of predicted load and experimental load for beam No.6 is about 0.62 whereas in beam No. 9 this ratio is about 0.41. The H/L ratio of beam No.9 is 0.33 which is very low, due to which composite action between the wall and bottom beam could not have developed. Flexural action dominates the behavior of such beam with low H/L ratio.

6.6. Computation of Maximum Vertical Stress Based on Elastic Finite Element Study

Recently a finite element study on composite behavior of wall-beam structure was carried out by Kamal [16] in Civil Engineering Department, Bangladesh University of Engineering and Technology. From his analysis he found that the resultant of vertical stress passes through the point of maximum bending moment which occurs at a distance of about 1/15th span from the supports. He also concluded that for a very slender beam, that is with a higher values of R_f (relative stiffness), the stress distribution is almost triangular with large vertical stress over the supports. He also pointed out that the horizontal shear force develops at the wall-beam interface eccentric with respect to the beam centroid.

Table 6.5 Comparison Between the Experimental Load of Composite Wall-Beams of Previous Studies to the Predicted Load Considering Deep Beam Using Magnification Factor

Research Designation	Beam identification	Expremental load in kips	Predicted load in kips	PL/EL
Present Study	A_1 span= 5'-5" H/L = 0.46	23.50	22.54	0.96
	A_2 span= 3'-9" H/L = 0.67	33.20	32.05	0.98
	A_3 span= 2'-11" H/L = 0.86	47.20	39.32	0.83
Experiment by Annamalai[7]	RC_3 Span= 48" H/L = 0.58	55.10	36.82	0.67
	RC_4 Span= 48" H/L = 0.58	36.96	29.78	0.81
Experiment by Burhouse[4]	No.6 Span= 12'-0" H/L = 0.58	132.00	67.44	0.51
	No.8 Span=12'-0" H/L = 0.83	109.56	114.22	1.04
	No.9 Span= 12'-0" H/L = 0.33	115.28	28.90	0.25

Note:PL =Predicted load ; EL= Experimental load

Table 6.6 Comparison Between the Experimental load of Composite Wall-Beams of Previous Studies to the Predicted Load Considering Deep Beam Using Modular Ratio

Research Designation	Beam identification	Expremental load in kips	Predicted load in kips	PL/EL
Present Study	A_1 span= 5'-5" H/L = 0.46	23.50	30.50	1.30
	A_2 span= 3'-9" H/L = 0.67	33.20	36.48	1.1
	A_3 span= 2'-11" H/L = 0.86	47.20	41.40	0.88
Experiment by Annamalai[7]	RC_3 Span= 48" H/L = 0.58	55.10	40.64	0.74
	RC_4 Span= 48" H/L = 0.58	36.96	46.58	0.74
Experiment by Burhouse[4]	No.6 Span= 12'-0" H/L = 0.58	132.00	82.48	0.62
	No.8 Span=12'-0" H/L = 0.83	109.56	120.60	1.10
	No.9 Span= 12'-0" H/L = 0.33	115.28	47.64	0.41

Note:PL =Predicted load ; EL= Experimental load

This has the effect of causing substantial reduction in the beam bending moment produced by the vertical force shown in Fig. 2.10 . Fig.6.10 has been developed with slight modification of Fig. 2.10 i.e. the variation has been assumed linear.

From Fig.6.10 the following relation may be obtained

$$W/2 = 1/2\ f_v \times 0.2L \times b$$

$$W = f_v L/5b$$

$$f_v = 5W/Lb$$

Where f_v = Maximum vertical stress in the wall. Using this equation, the maximum vertical stress in Group A beams are calculated. Maximum vertical stresses are found to be 401 psi, 820 psi and 1498 psi for beam A_1, A_2 and A_3 respectively.

It may be mentioned here that the maximum vertical stress obtained, using expressions suggested by Davis and Ahmed[14] with proposed modification of relative flexural stiffness, are found to be in close agreement with these values.

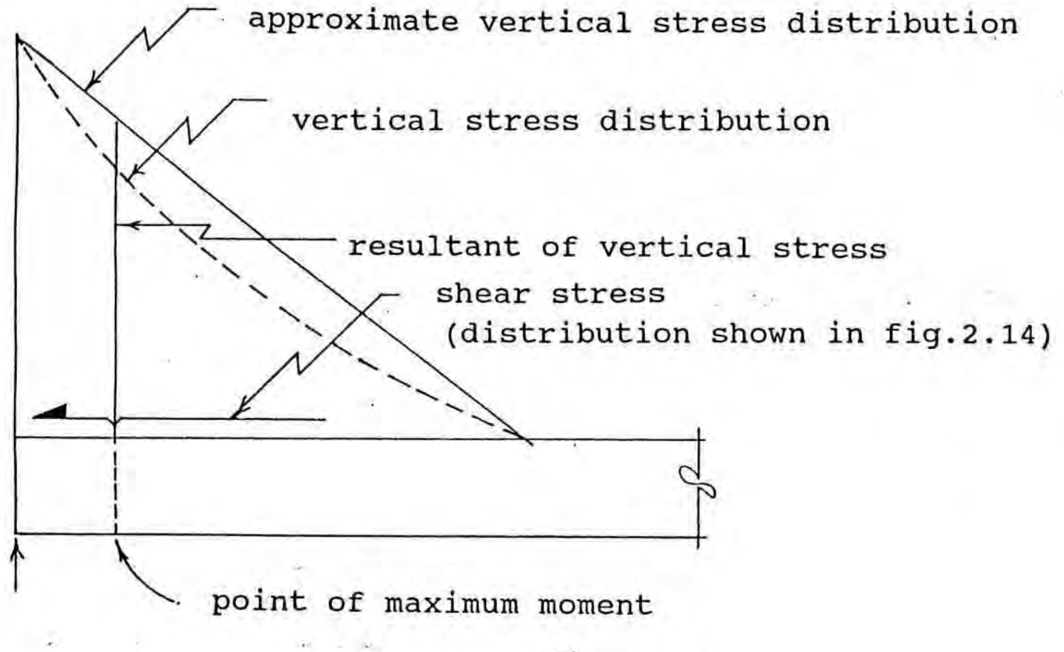


Fig.6.10 Approximate Stresses Contributing Moment

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

7.1 Conclusions

An experimental study has been performed on the composite behavior of wall-beam structures. Height to span ratio of wall-beam structure, bond pattern of the brickwork and inclusion of vertical reinforcement in the wall have been considered as the main parameters for this study. Only nine composite wall-beam and three reinforced concrete beam have been tested in this investigation. Conclusive remarks demand much larger number of tests results for statistical confidence. With the limitations and scope of this study, the following conclusions can be drawn.

i) To ensure proper composite action between brick wall and supporting beam, the height/span ratio of wall-beam should be at least 0.5.

ii) The failure load of running bonded composite wall beam is about six times the failure load of the respective reinforced concrete beam.

iii) The running bonded composite wall-beam takes about 1.5 times higher loads than the respective stack bonded composite wall beam.

iv) The stack bonded composite wall-beam with vertical reinforcement in the joints can carry more load than the corresponding stack bonded composite unreinforced wall- beam.

v) Both the running bonded and stack bonded composite wall- beam shows brittle fracture whereas the stack bonded brick work with vertical reinforcement in the wall shows ductile fracture.

vi) The maximum bending moment proposed by Wood[5] for supporting beam of wall-beam structure is found to be less than the experimental value of the present study.

vii) A relative flexural stiffness parameter is suggested with slight modification of relative flexural stiffness parameter proposed by Davis and Ahmed. Using this parameter, the moment coefficient is found to be more close to the experimental value.

viii) The calculated maximum vertical stress in the wall using the suggested relative flexural stiffness parameter is found to be similar to the maximum vertical stress predicted by approximate formula derived from the previous finite element study on composite wall-beam.

ix) A simplified method has been suggested to predict the load carrying capacity of composite wall-beam structure. The predicted load using this method is found to be about ninety five percent of experimental load for running bonded composite wall-beam and about 1.2 times higher than the experimental load of stack bonded composite wall-beam without vertical reinforcement in the wall.

x) The load carrying capacity of composite wall-beam has also been determined by using the deep beam theory of reinforced concrete beam with slight modification and found to be very close to the experimental load.

7.2 Recommendations for Future Study

It is believed that due to some limitations of this present study, a complete guidelines for the designers could not be developed here. The present study is a trigger off an extensive research work on composite wall-beam structure. Therefore, some guidelines for future theoretical and experimental study on this subject may be recommended for layingout a proper code of practice for designers.

The recommendations are:

i) Composite wall-beam structure with H/L ratio less than 0.4 and greater than 1 may be studied.

ii) Composite wall-beam structure with opening in the wall may be studied.

iii) Composite wall-beam with vertical edge ties may be studied for vertical as well as lateral load with or without opening in the wall.

iv) Effect of different bond pattern in brickwork of composite wall-beam may be studied.

v) Effect of horizontal reinforcement in the brickwork of composite wall-beam may be studied.

vi) Composite wall-beam may be studied by changing the depth of bottom beam and also thickness of the wall.

vii) Composite wall-beam may be studied for vertical concentrated and uniform distributed load.

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APPENDIX A

LOAD-DEFLECTION RECORD OF BEAMS

Table A.1 Load-Deflection Record of Composite Wall-Beam A_1

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Running bonded composite wall-beam.	0	0	
	2	8	
	4	15	
Effective span 5'-5"	6	19	
H/L ratio 0.46	8	25	
	10	34	
	12	40	
	14	48	
	16	55	
	18	63	
	20	75	Max^m recorded deflection
	22	-	
	23.5	-	Ultimate failure

Table A.2 Load-Deflection Record of Composite Wall-Beam A_2

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Running bonded composite wall-beam.	0	0	
	2	6	
	4	11	
Effective span 3'-9"	6	16	
H/L ratio 0.67	8	20	
	10	25	
	12	28	
	14	33	
	16	38	
	18	42	
	20	45	
	22	50	
	24	54	
	26	58	
	28	61	
	30	65	
	32	70	
	33.2	75	Ultimate failure.

Table A.3 Load-Deflection Record of Composite Wall-Beam A_3

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Running bonded composite wall-beam.	0	0	
	2	4	
Effective span 2'-11"			
H/L ratio 0.86	4	9	
	6	13	
	8	16	
	10	19	
	12	22	
	14	25	
	16	29	
	18	33	
	20	36	
	22	41	
	24	45	
	26	50	
	28	53	
	30	57	
	32	65	
	34	77	
	36	76	
	38	85	
	40	90	
	42	97	
	44	101	
	46	112	Max^m recorded deflection.
	47.2	-	Ultimate failure.

Table A.4 Load-Deflection Record of Composite Wall-Beam B_1

Description of Beam		Load in kips	Deflection in inchx10^{-3}	Remarks
Stack bonded		0	0	
composite wall-beam.		2	5	
		4	10	
Effective span	5'-5"	6	14	
		8	18	
H/L ratio	0.46	10	25	
		12	30	
		14	35	
		15.5	55	Ultimate failure.

Table A.5 Load-Deflection Record of Composite Wall-Beam B_2

Description of Beam		Load in kips	Deflection in inchx10^{-3}	Remarks
Stacked bonded		0	0	
composite wall-beam.		2	4	
		4	7.5	
Effective span	3'-9"	6	12	
		8	16	
H/L ratio	0.67	10	20	
		12	24	
		14	30	
		16	34	
		18	39	
		20	45	
		22	60	
		23	78	Ultimate failure.

Table A.6 Load-Deflection Record of Composite Wall-Beam B_3

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Stacked bonded	0	0	
composite wall-beam.	2	4	
	4	7	
Effective span 2'-11"	6	10	
	8	14	
H/L ratio 0.86	10	16	
	12	19	
	14	22	
	16	25	
	18	29	
	20	32	
	22	38	
	24	42	
	26	45	
	28	50	
	30	55	
	31	70	Ultimate failure

Table A.7 Load-Deflection Record of Composite Wall-Beam C_1

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Stacked bonded	0	0	
composite wall-beam.	2	4	
with vertical	4	8	
reinforcement.	6	12	
	8	17	
Effective span 5"-5"	10	21	
	12	26	
H/L ratio 0.46	14	33	
	16	42	First visible crack
	18	84	
	20	148	
	21	190	Ultimate failure.

Table A.8 Load-Deflection Record of Composite Wall-Beam C_2

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Stacked bonded	0	0	
composite wall-beam.	2	3	
with vertical	4	6	
reinforcement.	6	10	
	8	13	
Effective span 3'-9"	10	17	
	12	22	
H/L ratio 0.67	14	26	
	16	30	
	18	36	
	20	42	
	21	-	First visible crack
	22	56	
	24	80	
	26	96	
	28	120	
	30	174	
	32	-	Ultimate failure

Table A.9 Load-Deflection Record of Composite Wall-Beam C_3

Description of Beam	Load in kips	Deflection in inchx10^{-3}	Remarks
Stack bonded	0	0	
composite wall-beam.	2	2	
with vertical	4	5	
reinforcement.	6	7	
	8	9	
Effective span 2'-11"	10	11	
	12	14	
H/L ratio 0.86	14	18	
	16	22	
	18	25	
	20	28	
	22	32	
	24	37	
	26	40	
	28	46	
	30	54	
	32	76	
	34	95	
	36	111	
	38	120	
	40	125	
	42	146	

Table A.10 Load-Deflection Record of Reinforced Concrete Beam Group D

Load in kips	Deflection of beam in inch X 10^{-3}		
	D_1	D_2	D_3
0	0	0	0
0.5	50	7.5	5
1.0	110	15.0	10
1.5	150	20.0	15
2.0	205	25.0	20
2.5		31.0	27
3.0		37.5	32.5
4.0	Ultimate failure	55.0	45.0
5.0		77.5	61.0
6.0		95.0	75.0
7.0		112.5	88.0
7.5		142.5	97.5
8.0		155.0	105.0
9.0		Ultimate failure	117.5
10.0			135.0
11.0			180.0
12.0			Ultimate failure

Note:
1. First visible cracks at load 1.2^k for D_1; D/L = 0.09

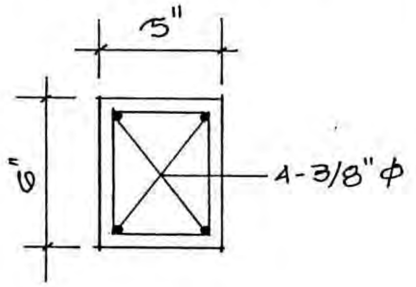
2. First visible cracks at load 7.6^k for D_2; D/L = 0.13

3. First visible cracks at load 7.6^k for D_3; D/L = 0.17

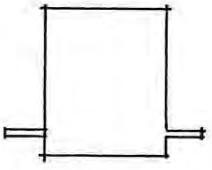
APPENDIX B

CALCULATION OF THEORETICAL LOAD OF BOTTOM REINFORCED CONCRETE BEAM OF COMPOSITE WALL-BEAM

B.1 Considering Section Uncracked



(a) Cross-Section of Beam



(b) Uncracked tranforcemed section

Fig.B.1 Cross-Section and Uncracked tranforcemed section of Beam

Compressive strength of concrete	:	2050 psi
Yield strength of M.S. bar	:	45,000 psi
Modulus of Elasticity of concrete(E_c)	:	57000 $\sqrt{2050}$ = 2580785psi
Modulus of Elasticity of M.S. bar(E_s)	:	29,000000psi
Modulus of rupture of concrete(f_r)	:	10 $\sqrt{2050}$ = 450psi
Modular ratio $n = E_s/E_c$	:	29,000000/2580785 = 11.24

Now, $(n - 1)$ As = 2.05

From Fig. B.1 (b) Location of neautral axis from top can be calculated as

$$y = (6 \times 5 \times 3 + 2.05 \times 4.5) / 32.05$$

$$= 3.10$$

Now, moment of inertia of beam at uncracked section

$$I = 5 \times 6^3 / 12 + 5 \times 6 \times (0.10)^2 + 2.05 \times (4.5-3.10)^2$$

$$= 94.32 \text{ in}^4.$$

Bending stress at bottom fibre of beam $f_r = M\, y_b / I$

$$450 = M \times 2.90 / 94.32$$

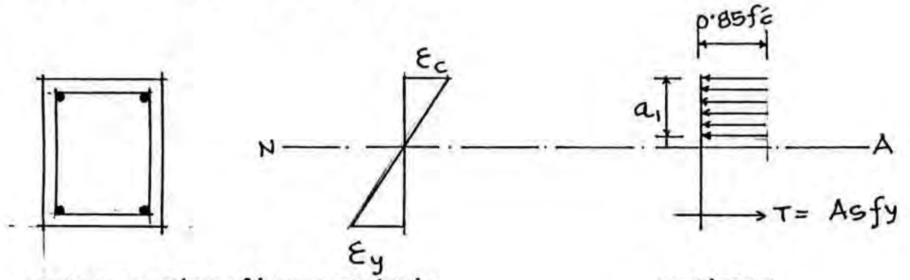
$$M = 450 \times 94.32 / 2.90 = 14636 \text{ lb-in} = 1.22 \text{ K-ft.}$$

Bending Moment of beam M =PL/6 =1.21 K-ft where P=Total load (for third point loading)

Now Load of beam (Uncracked section)

Span	Load
5'-5"	1.35^k
3'-9"	1.95^k
2'-11"	2.50^k.

B.2 Ultimate State



(a)cross-section of beam (b)strain (c)stress

Fig.B.2 Stress and Strain Distribution of Reinforced Concrete Beam at Ultimate Stage

Minimum reinforcement of beam section

A_s min = $(200/f_y)$ (bd)

= (200/45,000) x 5" x 4.5"

= 0.10 sq.in.

Maximum reinforcement of beam section

A_s max = 0.75 A_{sb}; Where A_{sb} = Balance steel ratio.

= 0.75 x [(0.722 x 87/(87+f_y) x f'_c / f_y)] x bd

= 0.37 sq.in.

A_s min < A_{sp} < A_{sb} = 0.105 < 0.2 < 0.3

Where A_{sp} = Cross-sectional area of steel provided in the beam.

Since the steel provided less than the balanced steel, therefore failure by yeilding is assured. Then depth of rectangular stress block

$a_1 = A_s f_y / 0.85\ f_c' b$

$= 0.20 \times 45{,}000 / 0.85 \times 2050 \times 5$

$= 1.03$ in.

Moment capacity of beam

$M_u = A_s\ f_y\ (d - a_1/2)$

$= 0.20 \times 45{,}000\ (4.5 - 1.03/2)$

$= 35865.0$ lb-in. $= 2.99$ k-ft.

Beam has been tested by third point loading system. So, Maximum Bending Moment is M = PL/6,

where P = Total load; L = Effective span length of beam.

Calculation of Ultimate Load P (Considering flexure)

Span	**Ultimate load**
5'-5"	3.3^k
3'-9"	4.78^k
2'-11"	6.14^k.

APPENDIX C

CALCULATIONS OF LOAD TO STIFFNESS RATIO OF COMPOSITE WALL-BEAM

Sample Calculation

A sample calculation of load to stiffness ratio of composite wall-beam is shown for Composite Wall-beam A_1 of Group A. (Present study).

From Art. F.1 in Appendix F, it is found that

Moment of inertia of composite wall-beam

I_c = 6165 in^4

Modulus of Elasticity of Concrete

E_c = 2580785 psi.

Now, Flexural Stiffness

$4E_cI_c/L$ = 4 x 2580785 x 6165/65

= 979110124.6

Experimental load was found for composite

wall-beam A_1 = 23500 lbs

Therefore, Load / Stiffness ratio

$P/[4E_cI_c/L]$ = 23500 /979110124.6

= 2.38×10^{-5}

Similarly Load to Stiffness ratio can be calculated for other beams also.

It is to be noted that the moment of inertia of composite wall-beam can also be calculated from Appendix G. A simplified method has been developed for calculating the moment of inertia of composite wall-beam in masonry equivalent, which can be converted into equivalent concrete by dividing the modular ratio (E_c/E_m). The steps of calculation of moment of inertia of composite wall-beam into concrete equivalent (I_c) is given below:

i) The gross moment of inertia of composite wall-beam is being calculated using the formulae $bH^3/12$.

ii) The value of C_3 which is equal to I_g/I_m is being calculated according to the procedure described in Appendix G.

iii) The value of I_g and C_3 is being multiplied for the value of I_m.

iv) For the value of moment of inertia of composite wall-beam into concrete equivalent (I_c), the value of I_m is being divided by the value E_c/E_m.

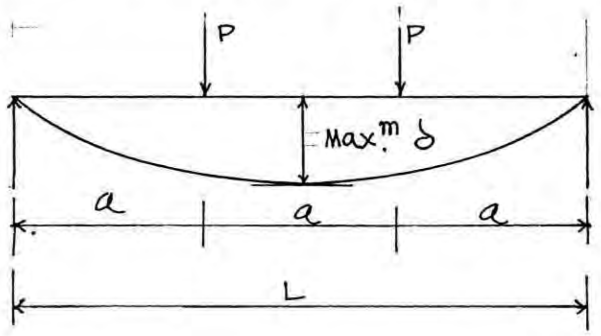
Calculation of flexural stiffness is given in Table C.1 and the load to stiffness ratio has been calculated for each of the following beams. The results are shown in Table 5.5.

Table C.1 Flexural Stiffness of Composite Wall-Beam

Beam	Moment of inertia (I_c) in in^4	E_c	L in in.	Flexural stiffness $4E_cI_c/L$
Present study				
A_1	$(4.5x30^3/12)x1.63/2.69$ = 6135	$2.58x10^6$	65	$9.74x10^8$
A_2	$(4.5x30^3/12)x1.63/2.69$ = 6135	$2.58x10^6$	45	$14.07x10^8$
A_3	$(4.5x30^3/12)x1.63/2.69$ = 6135	$2.58x10^6$	35	$18.09x10^8$
Annamalai[(4)]				
RC_3	$(9.2x27.8^3/12)x1.64/3.16$ = 8549	$2.76x10^6$	48	$19.66x10^8$
RC_4	$(9.2x27.8^3/12)x2.37/5.35$ = 7297	$2.87x10^6$	48	$17.45x10^8$
Burhouse[(7)]				
B_6	$(4.5x84^3/12)x1.62/3.15$ = 114307	$3.15x10^6$	144	$100x10^8$
B_8	$(4.5x120^3/12)x1.53/3.15$ = 314743	$3.15x10^6$	144	$275.4x10^8$
B_9	$(4.5x48^3/12)x1.77/3.15$ = 23303	$3.15x10^6$	144	$20.39x10^8$

APPENDIX D

CALCULATIONS OF DEFLECTION OF REINFORCED CONCRETE BEAMS



Maximum deflection of beam at midspan (Moment Area Method)

max δ = $Pa(3L^2-4a^2 / 24E_c I)$

= $[P \times L \times (3L^2 - 4 \times L^2 / 9) / 24E_c I \times 3]$

= $23 \times PL^3 / 24 \times 3 \times 9 \times E_c I$

= $PL^3 / 28.17 E_c I$ where $P = \frac{1}{2} \times$ Total load

Here $I = t \times D^3/12$

= $5 \times 6^3/12 = 90$ in^4

E_c = 57,000 $\sqrt{2050}$ psi.

= 2580785 psi.

Max δ = $PL^3 \times 1.53 \times 10^{-10}$ in

Sample calculation

When $P = 0.25^k$ and $L = 65$ in

Max δ = 0.0104 in = 10.04×10^{-3} in.

APPENDIX E

CALCULATIONS OF DESIGN PARAMETERS OF COMPOSITE WALL-BEAM USING EXISTING FORMULAE

E.1 Calculation of Bending Moment of Supporting Reinforced Concrete Beam

Bending moment of supporting reinforced concrete beam of composite wall-beam has been calculated according to the formulae suggested by Wood([5]), Smith and Riddington([10]), and Davis and Ahmed([14]).The results are given below.

E.1.1 According to Wood

Wood introduced coefficient for calculating bending moment of supporting reinforced concrete beam of composite wall-beam. This method is known as Moment Coefficient Method.

Moment Coefficient Method

Bending moment of supporting reinforced concrete beam suggested by Wood was WL/100 for simply supported composite beam of depth/span (H/L) ratio greater than 0.6. According to wood, this is valid for composite wall-beam without opening in the wall or oppening in the centre of walls.

Where W = Total uniform distributed load on wall panel

L = Effective span

H = Total height of wall

Sample Calculation

Calculation of bending moment of composite wall-beam A_2

Bending Moment M = WL/100

Where W i.e experimental failure load of composite

wall-beam A_2 = 33.2 k

Effective Span length L = 3.75'

Therefore, M = 33.2 x 3.75/100

= 1.25 K-ft.

Similarly bending moment of other composite wall-beams have been calculated and given in Table 5.8.

E.1.2 According to Stafford Smith and Riddington[10]

According to Smith and Riddington, the maximum bending moment of supporting reinforced concrete beam-

Maximum beam moment $M = (WL/4) \times [1/(R^4)^{1/3}]$ (1)

Where,

Relative Stiffness Parameter $R = 4\ (E_w b L^3/E_b I_b)^{1/4}$ .. (2)

Modulous of Elasticity of wall $E_w = 1000 \times f_m' = 1000 \times 959$

Thickness of Wall b = 4.5"

Effective Span L = From 5'-5" to 2'-11"

Modulous of Elasticity

of concrete beam E_b = $57000\ \sqrt{2050}$

= 2580785

Moment of inertia $I_b = 5 \times 6^3/12 = 90\ in^4$.

Putting the value of R from eq.(2) in eq.(1).

Max.^{m} Bending Moment $= WL/4 \times [1 / (E_w bL^3/E_b I_b)^{1/3}]$

$= W/4 \times [1 / (E_w b/E_b\ I_b)^{1/3}]$

This equation is independent of 'L' and H/L.

(They assumed H/L = 0.6 and derived this expression)

Now $(E_w b/E_b I_b)^{1/3} = (1000 \times 959 \times 4.5 / 2580785 \times 90)^{1/3}$

$= 0.264$

Max^m. Bending Moment $= (W / 4) \times 1 / 0.264$

$= 0.95 \times W^{\text{K-in.}}$

Now putting the value of load 'W', bending moment of beam can be calculated for the particular type of composite beams.

Sample Calculation

Calculation of bending moment of composite wall-beam A_2.

Bending moment of supporting beam $M = 0.95\ W^{\text{K-in}}$

Where W = Experimental Failure load of beam A_2 = 33.2 K.

$= 0.95 \times 33.2^{\text{k-in}}$.

$= 2.63^{\text{K-ft}}$.

Similarly Bending Moment have been calculated other beams also and given in Table 5.8.

E.1.3 According to Davis and Ahmed[14]

$$\text{Maximum bending Moment} = \frac{WLr - 2WD\ (\propto - \gamma k)}{4\ \lambda\ (1 + \beta R_f)}$$

Where,

Flexural stiffness R_f = $(E_w bh^3/E_b I_b)^{1/4}$

Axial stiffness K = $E_w bh/E_b A_b$.

Modulus of Elasticity of brick work E_w = 1000 x 959

Moment of inertia of bottom beam I_b = 90 in.^{4}

Only height of wall h = 24 in.

Therefore, $R_f = (1000 \times 959 \times 4.5 \times 24^3/2580785 \times 90)^{1/4}$
$= 4.00$

$K = 1000 \times 959 \times 4.5 \times 24 / 2580785 \times 30$
$= 1.34$

Sample Calculation

When $R \leq 5$, bending moment of beam to be calculatted as case 1 as they suggested

Case 1 $R < 5$, stiff beam.

$r = 0.2$ and $\lambda = 0.25$.

$M = [(WL - 10\ WD)(\propto - \gamma k)] / [5(1 + \beta R_f)$

$\propto$, β and γ were calculated from prescribed graph corresponding to h/L ratio and tabulated in the following table.

Values of α, β and γ according to Davis and Ahmed

Span	Wall height to span ratio	Beam depth/ span ratio	α	β	γ
5'-5"	0.37	0.09	0.46	2.2	0.14
3'-9"	0.53	0.13	0.39	1.7	0.084
2'-11"	0.69	0.17	0.32	1.35	0.06

Bending mement is calculated putting the values of α, β and γ in equation of bending moment.

Calculation of bending moment for beam A_2.

Beam	α	β	γ	Experimental load of beam	Bending Moment
A_2	0.39	1.70	0.084	33.2 K	2.01K-ft.

Similarly bending moments have been calculated for other beam also and given in Table 5.8.

E.2 Calculation of Axial Force in the Supporting R.C beam

E.2.1 According to Wood

Wood assumed lever arm of composite wall-beam = 2/3 H

where H is the total height of wall

Therefore,

$T \times (2/3\ H) = WL/8$ For uniform distributed load

$T = 3/16\ (WL/H)$

According to Wood, Axial force develope in the beam can be calculated using the above formula where $H/L > 0.60$

Sample Calculation

Calculation of Axial Force of beam A_2

Beam	Span	H/L	Experimental load	Axial force
A_2	3'-9"	0.67	33.2^k	9.29^k

E.2.2 According to Stafford Smith and Riddington

The axial force estimated by stafford Smith and Riddington $T = W/3.4$. They assumed $H/L = 0.6$

Sample Calculation

Calculation of axial force of beam A_2

Beam	Span	H/L	Experimental load	Axial force
A_2	3'-9"	0.67	33.2^k	9.76^k

Similarly axial force can be calculated for other beams also and given in Table 5.9.

E.2.3 According to Davis and Ahmed

The maximum axial force in a simply supported beam occurs at midspan and expressed as $T = W(\alpha - \gamma K)$

Sample Calculation

Calculation of axial force of beam A_2

α and γ were calculated from their prescribed graph corresponding to h/L ratio and given below:

Beam	Span	h/L	α	γ
A_1	5'-5"	.37	.46	.14
A_2	3'-9"	.53	.39	.084
A_3	2'-11"	.69	.32	0.06

Axial stiffness $K = E_w bh/E_b A_b = 1.34$

Now axial force for beam A_2 = 33.2(0.39-0.084x1.34) = 9.26 K.

Similarly axial force have been calculated for other beams also and given in Table 5.9.

E.3 Calculation of Maximum Vertical Stress in the Wall

E.3.1 According to Stafford Smith and Riddington

Maximum vertical stress in wall

$f_v \quad = (W/Lt) \times 1.63 \times (E_w t L^3/E_b I_b)^{.28}$

$= W/L\text{x}4.5 \times 1.63\ [1000 \times 959 \times 4.5L^3/2580785 \times 90]^{.28}$

$= 0.12\ W/L \times L^{0.84}$psi. where W = Load in lbs.

Maximum vertical stress of composite wall-beam A_2 corresponding to its experimental load was calculated using the above expression and given below:

$f_v \quad = 0.12 \times 33200 \times (45)^{0.84} / 45 = 2167$ psi.

where W = 33200 lbs and L = 45 in.

Similarly maximum vertical stress of other beams have been calculated and given in Table 5.10.

E.3.2 According to Davis and Ahmed

The maximum vertical stress in the wall is given by

$f_v = [W/Lb]\ (1 + \beta R_f)$

Maximum vertical stress corresponding to the experimental load was given below

Sample Calculation

The coefficient has been calculated from prescribed graph and shown in the calculation of axial force according to Davis and Ahmed in art E.1.3

Flaxural stiffness R_f has been found in the calculation of maximum bending moment according to Davis and Ahmed which is equal to 4.

Maximum vertical stress of beam A_2 corresponding to its experimental load was calculated using the given expression below:

Maximum vertical stress $f_v = [W / Lb] (1 + \beta R_f)$

$$= [33200 / 45 \times 4.5] (1+1.7\times4)$$

$$= 1279 \text{ psi.}$$

Similarly maximum vertical stress can be calculated for other composite wall-beams also and given in Table 5.10.

E.3.3 According to R. H. Wood

Maximum vertical stress in the wall according to Wood is

$f_v = 12.5 W / (Lb)$

Sample Calculation

Maximum vertical stress of composite wall-beam A_2 corresponding to its experimental load using the avobe formula is

$f_v = 12.5 \times 33200 /(45 \times 4.5)$

$= 2049.38$ psi

APPENDIX F

CALCULATIONS FOR PREDICTION OF LOAD OF COMPOSITE WALL-BEAM

F.1 Section Uncracked (Ref: Fig. 6.1)

1. Av. compressive strength of brick prism (From Art.3.7.1) f_m' $= 0.9 \times 1066$ psi $= 959$ psi

2. Av. compressive strength of concrete cylinder f_c $= 2050$ psi.

3. Modulus of elasticity of brick work E_m $= 1000\ f_m'$ (From Table 3.8) $= 959000$ psi.

4. Modulus of elasticity of concrete E_c $= 57000\ \sqrt{2050}$ psi $= 2580785$ psi .

5. Modulus of elsasticity of M.S. bar E_s $= 29 \times 10^6$ psi.

6. Ratio of modulus of elasticity of M.S bar and concrete n $= E_s/E_c$ $= 29000000/2580785$ $= 11.24$

7. Ratio of modulus of elasticity of concrete and masonry wall n_1 $= E_c/E_m$ $= 2580785/959000$ $= 2.69$

8. Equivalent width of brick wall $= 4.5/\ 2.69$ $= 1.67$in.

9. Equivalent transformed section of Reinforced concrete beam A_{tc} $= A_g+(n-1)A_s$ $= 5\text{x}6+(11.24-1)0.39$ $= 34.00$ sq. in.

10. Calculation of neutral axis from top

$$y_t = \frac{1.67 \times 24^2/2 + 34.00\ (30-3)}{1.67 \times 24 + 34.00}$$

$= 18.88''$

11. Moment of Inertia of composite wall-beam into concrete equivalent

I_c $= 1.67 \times 24^3/12 + 1.67 \times 24\ (18.88 - 12)^2$

$+ 34.00 \times 6^2/12 + 34.00\ (27 - 18.88)2$

$= 6164.77\ in_4. = 6165 in_4$ (say)

12. Modulus of rupter of Concrete $f_r = 10\ \sqrt{2050}$ psi= 450 psi

13. Maximum stress at bottom fibre $= M\ y_b\ /I_c$

where M = Bending moment of composite wall-beam ;

I_c = moment of inertia of composite wall-beam into equivalent concrete; y_b = distance of bottom fibre from the neutral axis.

Considering modulus of rupture of concrete = Maximum stress at bottom fibre

$450 = M \times (30 - 18.88)/6165$

$M = 249482.91 in. = 20.79^{\text{K-ft}}$

Third point load has been applied, so

Bending moment of beaam M = PL/6 and P = 6M/L

where P = Total vertical load and L = Effective span length

Calculation of Predicted Load

Span	Load
5'-5"	23.00 K
3'-9"	33.26 K
2'-11"	42.72 K

F.2 Ultimate State of Failure (U.S.D. method)

Basis of Calculations

Neglecting the contribution of concrete of bottom beam (Ref:Fig.6.2)

$\sum$(Horizontal force) = 0

or, Total compressive force = Total tensile force

$$C = T \qquad \ldots (1)$$

Eq(1) can be writren as

$K_1 f_{m'} b a_1 = P_b b d f_y$

or, $P_b = K_1 \ (f_{m'}/f_y)\ c/d \qquad \ldots(2)$

Modulus of elasticity E = Stress/Strain
= σ/ϵ

Strain in M.S bar at yield stress
$\epsilon_y = f_y/E_s$

= 45000/29000000

= 0.00155

Strain in masonry
ϵ_m = 0.003 (Ref.No.28)

Modulus of elasticity of brick masonry
E_m = 1000 x 959 psi

Modulus of elasticity of M.S bar
E_s = 29×10^6

From Fig.6.2

$c/(d-c) = 0.003/0.00155$

$c/d = 0.003/(0.003 + 0.00155)$

$= 0.66$

Putting the value of c/d and K_1 in equation (2), where $K_1 = 0.76$ (American code)

$P_b = K_1 \times (959/45000) \times 0.66$

$= 0.76 \times (959/45000) \times 0.66$

$= 0.0106$

Hence Balanced steel

$A_{sb} = P_b\, bd$

$= 0.0106 \times 4.5 \times 27$

$= 1.29$ sq. in.

4-3/8" dia is provided as reinforcement

$A_s = 4 \times .098 = 0.390$ sq.in.

$A_s < A_{sb}$, that is the section is underreinforced

So moment capacity will be

$M_u = A_s f_y (d - a_1/2)$

$a_1 = A_s f_y / K_1 f_m b'$

$$= \frac{0.39 \times 45000}{0.76 \times 959 \times 4.5}$$

$= 5.35$ in.

Hence $M_u = 0.39 \times 45{,}000\ (27 - 0.50 \times 5.35)$

$= 426903.75$ lb-in.

$= 35.58^{K\text{-ft}}$

Since third point load was applied , Maximum Moment

$M = P/2 \times L/3 = PL/6,$

$P = M6/L.$

Calculation of predicted load (P)

Effective span	Moment capacity of section in k-ft.	Max_m load carrying capacity in kips
5'-5"	35.88	39.39
3'-9"	35.88	56.93
2'-11"	35.88	73.11

APPENDIX G

DERIVATION OF SIMPLIFIED METHOD FOR PREDICTION OF LOAD OF COMPOSITE WALL-BEAM

G.1. Derivation of Simplified Method

This simplified method for prediction of load of composite wall-beam has been derived based on uncracked section.

Moment capacity of a reinforced concrete beam of uncracked section is

$M = f_r I_c / y_b$ ..... (1)

where,

f_r = Modulus of rupture of concrete

= 8 to 12 f_c' (Ref. 34)

= 10 $f_{c'}$ (taken as average value)

f_c' = Compressive strength of concrete cylinder.

I_c = Moment ofinertia of composite wall-beam at uncracked state into equivalent concrete beam.

y_b = Distance from the neutral axis to bottom of beam

E_c = Modulus of elasticty of concrete

= 57000 $\sqrt{f_c'}$ and $\sqrt{f_c'}$ = E_c / 57Q00 ....(2)

Putting the value of $\sqrt{f_c'}$ from Eq. (2) to Eq. (1), we get,

$$M = 10x \frac{E_c}{57000} x \frac{I_c}{y_b}$$

$$M = \frac{E_c}{5700} x \frac{I_c}{y_b} \quad \ldots . (3)$$

Bending moment of beam of effective span L for load W is

$$M = WL/k \quad \ldots (4)$$

Where K is a moment coefficient depend on the type of load; k = 6 for third point loading; and k = 8 for uniform distributed load

We know,for a perticular composite wall-beam

$$E_c I_c = E_m I_m \quad \ldots . (5)$$

where I_c = moment of inertia of composite wall-beam into equivalent concrete

I_m = moment of inertia of composite wall-beam into masonry equivalent

Modular ratio of concrete and masonry

$$n_1 = E_c / E_m$$

$$E_c = n_1 x E_m$$

$$I_c = I_m / n_1 \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Putting the value of E_c and I_c from Eq.5 and 6 in Eq.3,we get

$$M = \frac{n_1 E_m}{5700} \times \frac{I_m}{n_1} \times \frac{1}{y_b}$$

$$= \frac{Em\ Im}{5700} \times \frac{1}{y_b} \qquad \ldots (7)$$

Putting the value of M from Eq.4 in Eq.7,we get

$$W = \frac{E_m I_m}{5700\ L} \times \frac{k}{y_b}$$

$$= \frac{300\ f_b'\ I_m}{5700\ L\ y_b} \times k$$

where, E_m = Modulus of elasticity of brick masonry

$= 1000\ f_m'$ (From Table 2.1)

$= 1000 \times 0.3\ f_b'$ (Art.3.7.1)

$$\text{Now,}W = \frac{f_b' I_m}{19\ L\ y_b} \times k$$

putting the value of $I_m = I_g x C_3$; where I_g is the gross moment of inertia of composite wall-beam = $bH^3/12$ and $y_b = HC_2$ in the above equation, we get

$$W = \frac{f_b'\ I_g\ C_3}{19\ L\ HC_2} \times k$$

$$= \frac{f_b' bH^3}{19L \times 12} \times \frac{C_3}{HC_2} \times k$$

$$= \frac{bH^2\ f_b'}{228L} \times \frac{C_3}{C_2} \times K \qquad \ldots (6.1)$$

Where C_3 = The ratio of moment of inertia of composite wall-beam in masonry equivalent to the gross moment of inertia of composite wall-beam.

C_2 = The ratio of distance from the neutral axis to bottom fibre (y_b) of composite wall-beam to the total height of composite wall-beam (H).

L = Effective span in in.

b,H = Width and height of composite wall-beam in in.

k = Moment coefficient.

The expression of C_3 and C_2 are derived in the following articles.

G.2 Derivation of Transformed Area of Bottom R.C Beam

Transformed area of reinforced concrete beam into equivalent concrete

$A_{tc} = tD + (n-1)\ A_s$;where $n = E_s/E_c$

Transformed area of reinforced concrete beam into equivalent brick masonry $A_{tm} = [tD + (n-1)\ A_s]\ n_1$; where $n_1 = E_c/E_m$

This expression yields in the following form when value of n and n_1 is put

$$A_{tm} = [tD + (509\ /\sqrt{f_c'}-1)\ A_s]\ 190\ \sqrt{f_c}\ /\ f_b'$$

$$= tD\ [1 + (504.35/\sqrt{f_c'} - 1)\ p]\ 171.24\ \sqrt{f_c'}\ /f_b'$$

$$= tDC_1$$

$$C_1 = A_{tm}/tD$$

$$= A_{tm}/A_g$$

where,

$C_1 \quad = [1+(509 /\sqrt{f_c'}-1)p]\ 190\ \sqrt{f_c'}/f_b' \quad ....(6.2)$

And $A_s \quad = ptD$; where p = longitudinal steel ratio, t = width of RC beam, D = total depth of RC beam.

In the expression 6.2, it is found that the value of C_1 is changed with in the ratio of longitudinal reinforcement in the beam and the value of f_c' and f_b'. Therefore, expression 6.2 may be considered as a constant viz. C_1. for a particular amount of reinforcement and compressive strength of brick and concrete. C_1 may be defined as the ratio of transformed area of RC beam in equivalent brick masonry to the gross area of RC beam.

G.3 Derivation of Expression for the Distance of Bottom Fibre From the Neutral Axis (y_b)

Distance of neutral axis from the top fibre (ref. Fig. 6.3)

$$y_t = \frac{b(H-D)^2/2 + bDC_1\ (H-D/2)}{b(H-D) + bDC_1}$$

Distance of neutral axis from the bottom fibre

$$y_b = H - y_t$$

$$y_b = \frac{bH\ (H-D) + bDHC_1 - b(H-D)^2/2 - bDC_1(H-D/2)}{b(H - D) + bDC_1}$$

Putting the value of

$D = C_4H$; where $C_4 = D/H = 0.1, 0.15, 0.2$ etc.

$$y_b = \frac{bH(H-C_4H) + bC_4HHC_1 - b(H-C_4H)^2/2 - bC_4HC_1(H-C_4H/2)}{bH(1-C_4) + bC_1C_4H}$$

$$= \frac{H(1/2 - C_4^2/2 + C_1C_4^2/2)}{(1 - C_4 + C_1C_4)}$$

$$= \frac{H(1 - C_4^2 + C_1C_4^2)}{2(1 - C_4 + C_1C_4)}$$

$y_b = HC_2$; where C_2 is the ratio of y_b / H

$$C_2 = \frac{(1 - C_4^2 + C_1C_4^2)}{2(1 - C_4 + C_1C_4)}$$

C_2 may be defined as the ratio of the distance of neutral axis from the bottom fibre to the total height of composite wall-beam.

G.4 Derivation of Expression for the Moment of Inertia (I_m) of Composite Wall-Beam in Equivalent Brick Masonry at Uncracked Section

Moment of Inertia (Ref. Fig. 6.3)

$$I_m = bh^3/12 + bh(y_t - h/2)^2 + bDC_1xD^2/12 + bDC_1(h + D/2 - y_t)^2$$

$$= b(H-D)^3/12 + b(H-D)[(H-y_b) - (H-D)/2]^2 + bD^3C_1/12 + bDC_1(H-D + D/2 - H + y_b)^2$$

Putting $D = C_4H$ where $C_4 = D/H$; a constant like 0.1, 0.15, 0.2 etc.

$$I_m = b(H-C_4H)^3/12 + b(H-C_4H)[(H-y_b) - (H-C_4H)/2]^2$$

$$+ bC_4^3H^3C_1/12 + bC_4HC_1(y_b-C_4H/2)^2$$

$$= (bH^3/12) [(1-C_4)^3 + C_1C_4^3)] + bH[(1-C_4) 1/4(H-2y_b+C_4H)^2$$

$$+ (C_1C_4)(y_b-C_4H/2)^2]$$

$$I_m = (bH^3/12) \times C_3$$

$$I_m = I_gC_3$$

where,

$$C_3 = [(1-C_4)^3 + C_1C_4^3)] + bH[(1-C_4) 1/4(H-2y_b+C_4H)^2$$

$$+ (C_1C_4)(y_b-C_4H/2)^2] \quad \ldots\ldots \quad 6.4$$

When C_4 =0.1, mean value of y_b is found 0.39H. The value of y_b is depend on the value of C_1. Mean value of y_b is calcullated for the value of C_1 =2.5 to 6.5. Putting the value of y_b and C_4 in the above expression 6.4.

$$C_3 = (1.00 + 0.144C1) \quad \ldots. \quad (6.4a)$$

Therefore,

$$I_m = [bH^3/12] (1.00 + 0.144C1)$$

When C_4 = 0.15, mean value of y_b is found 0.36H, putting these value in the expression 6.4 we get

$$C_3 = [1.08 + 0.146 C_1] \quad \ldots \quad (6.4b)$$

Therefore,

$$I_m = [bH^3/12] (1.08 + 0.146 C_1)$$

When $C_4 = 0.20$ mean value of y_b is found 0.34H, putting these values in the expression 6.4, we get,

$C_3 = (1.16 + 0.153\ C_1)$ ... (6.4c)

Therefore,

$I_m = [bH^3/12]\ (1.16 + 0.153C_1)$

G.5 Sample Calculation for Prediction of Load of Composite Wall-Beams

A sample calculation of load to stiffness ratio of composite wall-beam is shown here for composite wall-beam RC_3 tested by Annamalai et al[2].

For Beam RC_3

Compressive Strength of brick f_b' = 2175 psi

Compressive Strength of Concrete f_c' = 2300 psi

Ratio of longitudinal reinforcement in the bottom beam (2-8 mm) = 0.16/9.2 x 3

= 0.0058

Value of C_1 from the equation 6.2 or from the relevent graphs = 4.45

Ratio of depth of bottom beam to the total height			
of composite wall-beam $C_2 = t/H$		=	3 / 27.36
		=	0.10
Value of C_3 from Eq.6.4		=	1.64
Value of C_2 from Eq.6.3		=	0.38
From the givben section			
Width of wall	b	=	9.2 in.
Height of Composite Wall-beam	H	=	27.80 in.
Effective Span	L	=	48.00 in.
Moment coefficient	K	=	8

From Eq.6.1

Predicted Load $W = [(bH^2/228L) / (C_3/C_2) \times f_b' \times k)]$

Now, the predicted load for the composite wall-beam RC_3 = 47.37^k

Similarly the calculations for prediction of load of composite wall-beam has been given in Table G.1 and the results are given in Table 6.3.

Table G.1 Calculation of Predicted Load of Composite Wall-Beam

Beam	L in in.	t in in.	D in in	b in in.	H in in.	f_c' in psi	f_b' in psi	p	k	C_1	C_4= D/H	C_2	C_3	C_3/C_3	PL in kip
Composite wall-beam tested in the Present study															
A_1	65	5	6	4.5	30	2050	3195	0.013	6	3.05	0.2	0.38	1.63	4.29	22.47
A_2	45	5	6	4.5	30	2050	3195	0.013	6	3.05	0.2	0.38	1.63	4.29	32.46
A_3	35	5	6	4.5	30	2050	3195	0.013	6	3.05	0.2	0.38	1.63	4.29	41.74
Composite wall-beam tested by Annamalai[4]															
RC_3	48	9.2	3	9.2	27.8	2300	2175	0.0058	8	4.42	0.11	0.38	1.64	4.32	48.83
RC_4	48	9.2	3	9.2	27.8	2500	1015	0.0058	8	9.48	0.11	0.29	2.37	8.17	43.09
Composite wall-beam tested by Burhouse[7]															
B_6	144	6	12	4.5	84	3000	3000	0.0085	8	3.71	0.14	0.38	1.62	4.26	98.87
B_8	144	6	12	4.5	120	3000	3000	0.0085	8	3.71	0.10	0.40	1.53	3.83	181.42
B_9	144	6	12	4.5	48	3000	3000	0.018	8	3.99	0.25	0.34	1.77	5.21	39.47

APPENDIX H

MODIFICATION OF FORMULA OF REINFORCED CONCRETE DEEP BEAM FOR COMPOSITE WALL-BEAM

H.1 Determination of Magnification Factor for Composite Wall-Beam

According to ACI code, ultimate shear force taken by concrete of reinforced concrete shallow beam is

$$V_c = (1.9 \sqrt{f_c'} + 2500\, p\, V_u d/M_u)bd \qquad \ldots. \quad (1)$$

Accordingly, Ultimate shear taken by masonry of reinforced brick shallow beam without shear reinforcement[27] is

$$V_m = (1.5 \sqrt{f_m'} + 3000\, p\, V_u\, d/M_u)bd$$
$$= (1.5 \sqrt{f_m'} + 3000\, p\, 2d/a)bd \qquad \ldots \quad (2)$$

For reinforced concrete deep beam, ACI code introduce a magnification factor which is equal to $(3.5-2.5\, M_u/V_u d) \leq 2.5$. For composite wall-beam like reinforced concrete deep beam a magnification factor may be introduced.
This magnification factor has been introduced from experimental load.
Let us assume the magnification factor = F

Therefore

$V_{(cb)} \quad = F\ V_m$ .... (3)

where $V_{(cb)}$ = Shear force taken by masonry of composite wall-beam

V_m = Shear force taken by masonry of reinforced brick shallow beam

For composite wall-beam A_1, span 5'-5"

Effective depth $d = 27"$

Shear span $a = (5.42'/3) \times 12 = 21.68"$

Steel ratio $P = 0.39 / (4.5 \times 27)$

$= 0.0032.$

Now,

$V_m = [1.5 \times \sqrt{959} + 3000 \times 0.0032 \times (2 \times 27/21.68)]\ 4.5 \times 27$

$V_m = 8549$ lbs $= 8.55^k$

Putting the value of V_m in equation ... (3)

$V_{cb} = 8.55\ F$

From the experiment it is found that for span 5'-5" the total load is found 23.5^k

Therefore,

$V_{(cb)} = 23.5/2 = 11.75^K$ and

$F = 11.75 / 8.55 = 1.37$

Similarly for beams A_2 and A_3, the value of F is found to be 1.68 and 2.13 respectively.

According to ACI code, magnification factor for reinforced concrete deep beam is $[3.5 - 2.5(M_u/V_ud)]$ ... (4)

Let us assume, the factor of the expression (4) may change for composite wall-beam . Let us find out this value in lieu of 3.5. Say F_1.

For beam A_1

$F_1 - 2.5\ a/2d = 1.37$, therefore $F_1 = 2.37 \approx 2.35$.

Now, the total expression for composite wall-beam is

$$V_{(cb)} = (2.35 - 2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m{}'} + 3000\ p\ V_ud/M_u)bd \quad ..(6.10)$$

$$= (2.35 - 0.425\ L/d)\ (1.5\ \sqrt{f_m{}'} + 17647\ p\ d/L)\ bd$$

Similarly, the values of F_1 are found to be 2.38 and 2.68 when the test results of beam A_2 and A_3 is considered and the expression given in equations 11 and 12.

$$V_{(cb)} = (2.38 - 2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m{}'} + 3000\ p\ V_ud/M_u)bd \quad ..(6.11)$$

It can be simplified as

$$V_{cb} = (2.38- 0.425\ L/d)\ [(1.5\ \sqrt{f_m{}'} + 17647pd/L)bd$$

$$V_{(cb)} = (2.68 - 2.5\ M_u/V_ud)\ (1.5\ \sqrt{f_m{}'} + 3000\ p\ V_ud/M_u)bd \quad ..(6.12)$$

It can be simplified as

$$V_{cb} = (2.68 - .425\ L/d)\ [(1.5\ \sqrt{f_m{}'} + 17647\ p\ (d/L)]\ bd$$

The equations 6.11 and 6.12 are found when the experimental load of beam A_2 and A_3 are considered respoectively.

Sample calculation

Ultimate Shear Capacity of Composite Wall-Beam Considered Using Magnification Factor Like Reinforced Concrete Deep Beam.

Beam	L in inch	b in inch	d in inch	A_s in sq.inch	p	Predicted shear force eq.6.10 in Kip	eq.6.11 in Kip	eq.6.12 in Kip
Composite Wall-Beam Tested in the Present Study								
A_1	65	4.5	27	0.39	0.0032	11.27	14.27	14.08
A_2	45	4.5	27	0.39	0.0032	16.03	16.40	19.20
A_3	35	4.5	27	0.39	0.0032	19.66	20.00	23.29
Composite Wall-Beam Tested by Annamalai et al[4]								
RC_3	48	9.2	25.86	0.16	0.000673	18.41	NC	NC
RC_4	48	9.2	25.86	0.16	0.000673	14.89	NC	NC
Composite Wall-Beam Tested by Burhouse[7]								
B-6	144	78	4.5	0.614	0.00175	33.72	NC	NC
B-8	144	114	4.5	0.614	0.0012	57.11	NC	NC
B-9	144	42	4.5	1.277	0.00676	14.45	NC	NC

Now, the predicted load is twice the ultimate shear capacity shown in Table 6.4 and 6.5.

Note: NC = Not Calculated.