BEHAVIOR OF DIAGONALLY LOADED BRICK INFILL PANELS

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ABSTRACT

Twenty-one brick infill panels surrounded by a reinforced concrete frame were tested under in-plane diagonal loading. Three other similar specimens were tested under racking load. The height-to-length ratio and the ratio of beam moment of inertia to column moment of inertia of the surrounding frame were varied within the specimen group. Experimental results indicated that the general behavior of a reinforced concrete frame with masonry panel could be divided into three distinct phases including a linear response up to the occurrence of the first crack, the post cracking and the post-ultimate phases. An analytical model based on the experimental results was developed to predict the ultimate strength of reinforced concrete infilled frames.

Key words: masonry panel; reinforced concrete frame; crack load; strength

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INTRODUCTION

Previous researchers (Benjamin and Williams, 1957, 1958; Stafford-Smith et al., 1966, 1967, 1968, 1969; Barua and Mallick, 1977; Brokken and Bertero 1981; Liauw and Kwan, 1983; and Dawe and Seah, 1989) investigated the contribution to the strength and stiffness of frames of infilled panels when these systems were subjected to in-plane lateral loads. Polyakov (1957, 1960) proposed that an infilled system could be idealized as a frame with diagonal struts to replace the infill. Based on experiments conducted on a wide range of frames infilled with brickwork or microconcrete panels, Mainstone (1971) developed empirical formulations to predict both in-plane strength and stiffness using idealized diagonal infill braces.

Considerable number of experimental and analytical studies have been conducted on infilled frames. Most of these have dealt with small models in the order of one-tenth to one-sixth scale. Few studies have been conducted on reinforced concrete infilled frames and there is no widely accepted design method for such structures. Therefore, it is important that research be conducted on a number of variables that are thought to markedly influence the behavior of reinforced concrete infilled frames. Several parameters of infilled frames were investigated at the University of New Brunswick. The main results of the experimental program were reported recently by Dukuze, 2000.

This paper reports test results for twenty-four, one-storey, one-bay reinforced concrete frames with brick masonry infill. The parameters studied were the frame aspect ratio of height to length, $\alpha = H/L$, the ratio of the beam moment of inertia to column moment of inertia, $\beta = I_b/I_c$.

The work reported serves to demonstrate how the strength of infilled frames is influenced by these parameters. Further, the work provides analytical modeling and the development of test-based formulations to predict the strength of reinforced concrete infilled frames.

EXPERIMENTAL PROGRAM

<u>Test specimens</u>

Twenty-four single storey, single bay specimens built to a one-third geometrical scale were made and tested to failure. Infills were fabricated of full solid brick units on edge. Typically, specimens are designated by a letter followed by a digit and then another letter followed by three more digits. The first letter is either S, for a square specimen, or R, for a rectangular specimen. The first digit indicates the ratio, $, of the beam moment of inertia to column moment of inertia of the enclosing frame. The second letter, P, refers to a continuous infill panel. Since up to three geometrically identical specimens were tested for each set of parameters, the second digit indicates the specimen rank in the series: 1, 2, or 3. Specimen frames cast using lightweight concrete are indicated by last letter, L, such as R2PL. A last letter, D, in a designation such as S1PD indicates that specimens were subjected to racking load rather than diagonal load. Specimens cast with normal density concrete are indicated by a last letter, R, such as S1PR. Overall specimen dimensions are shown in Figure 1.

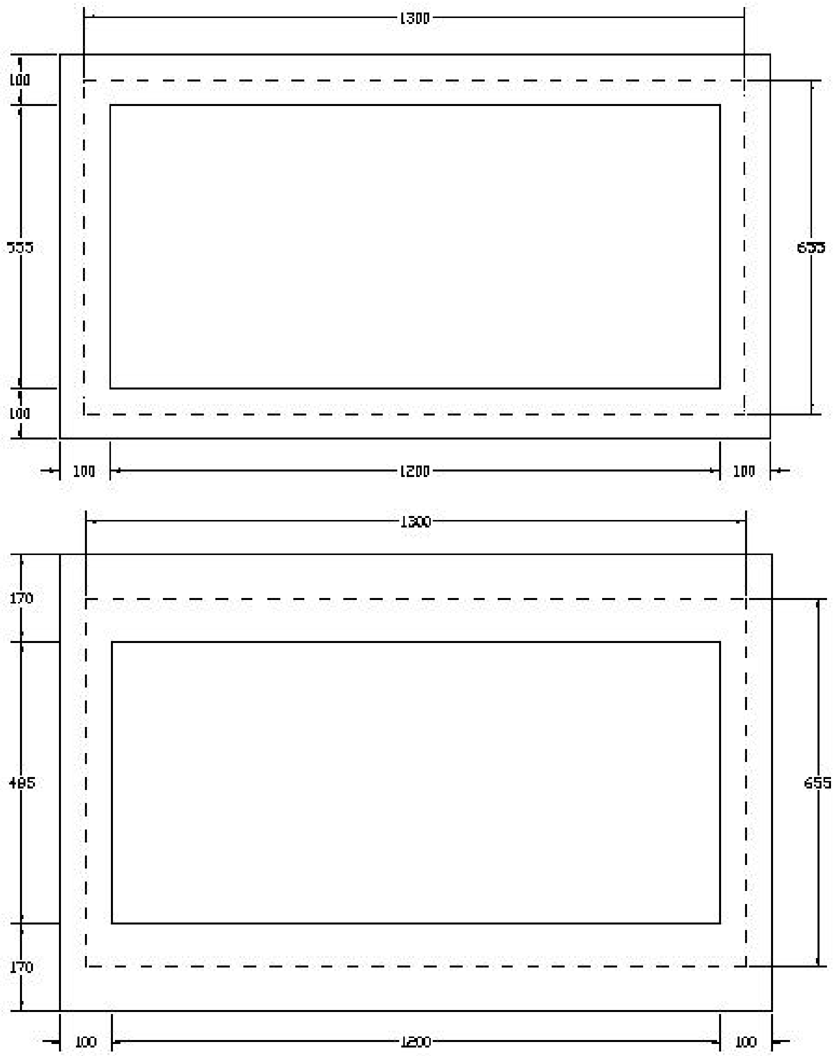

Figure 1(a): Typical Dimensions of Rectangular Specimens (mm)

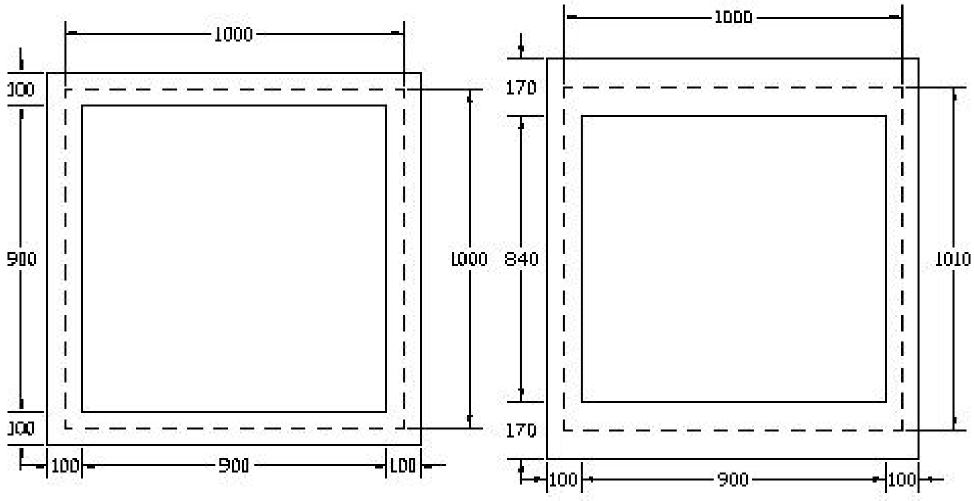

Figure 1(b): Typical Dimension of Square Specimens (mm)

Material Properties

Quality assurance tests were conducted on mortar, masonry prisms, wall panels, concrete, and reinforcing bars. The mechanical properties determined included concrete strength, f'_c, masonry wall strength, f'_m, masonry diagonal tensile strength σ_{dt}, and Young's moduli, E_c and E_m of both concrete and masonry, respectively.

Each batch of mortar used during construction of the infills was tested. Six 50x50x50 mm cubes and six tensile briquettes were sampled, cast in appropriate molds, and tested in accordance with ASTM C109-88 and ASTM C270-88. The curing conditions were as close as possible to that of related infill frames. Standard prisms were cured in the same environment as that of corresponding specimens and tested in accordance with CAN-A369-M84. Solid brick units were systematically sampled and tested according to CAN-A82.8-M78. Six single panel masonry specimens with the same aspect ratio as of that of corresponding infills were also built. They were tested under diagonal loading for their diagonal tensile strength in accordance with ASTM E519. For each concrete mix, six 50x100mm cylinders were cast and tested in accordance with ASTM C39. The mechanical properties applicable to each specimen are summarized in Table1. Since there were no infills for open frames, properties of panels are referred to as not applicable (NA) for these specimens.

Table 1 Specimen Material properties

Specimen	f'_c MP_a	f'_m MP_a	σ_{dt} MP_a	E_c GP_a	E_m GP_a
R1P104	14.2	18.0	0.6	8.4	4.8
R1P207	21.7	18.0	0.4	6.5	7.8
R1P318	21.6	18.0	0.8	8.65	6.5
R5P103	17.3	18.0	0.8	NA	4.8
R5P208	19.2	18.0	0.6	5.9	7.8
S1PD	20.0	18.0	1.8	9.1	9.0
S1PL	20.8	18.0	1.3	8.3	10.1
S1PR	45.6	21.0	1.4	17.7	18.3
S2PD	20.0	18.0	1.4	12.3	9.0
S2PL	20.8	18.0	1.8	8.3	10.1
S2PR	45.6	21.0	1.4	17.7	18.3
S5P101	21.8	18.0	1.1	NA	4.8
S5P211	26.1	18.0	1.3	7.6	7.8
R5P317	17.5	18.0	0.8	7.9	6.5
S1P102	16.1	18.0	1.4	NA	4.8
S1P212	25.2	18.0	1.1	8.9	7.8
S1P317	17.5	18.0	1.3	7.9	6.5
S5P318	21.6	18.0	1.3	8.7	6.5
S5PD	21.0	18.0	1.8	9.7	9.0
S5PR	45.6	21.0	1.4	17.7	18.3
R1PL	20.8	18.0	1.6	8.3	10.1
R1PR	45.6	21.0	1.4	17.7	18.3
R2PL	20.8	18.0	1.6	8.3	10.1
R2PR	45.6	21.0	1.4	17.7	18.3

Test Setup and Instrumentation

Specimens were tested in a Baldwin testing machine with load applied along the diagonal axis. Special steel caps were placed between the load heads of the testing machine and loaded corners of the specimens. Displacement transducers were secured in place. All instruments, including the testing machine output, were connected to a data acquisition system for continuous recording. The specimen instrumentation depended on the status of the infill panel. Since overall in-plane deformations were of interest, the specimen diagonals were instrumented to continuously monitor deformations by means of linear strain converters (LSC's).

Deformations were monitored along both panel diagonals and along the mortar bed joint. Furthermore, an attempt was made to assess the separation between masonry panels and surrounding frames. Thus, dial gauges were located at quarter points along beams and columns as measured from loaded corners.

Testing Procedure

Applied load was gradually increased in 4.5 kN increments. At each increment, a specimen was visually inspected for cracks which were documented on a specimen template. As loading progressed, a typical specimen underwent significant deformations accompanied by extensive damage of both masonry panel and surrounding frame. To protect the instruments from being damaged by a sudden specimen failure, they were removed prior to specimen collapse.

EXPERIMENTAL RESULTS

The observed overall behavior is described with respect to first cracking load H_c, ultimate strength H_u, and in-plane stiffness exhibited by the specimens.

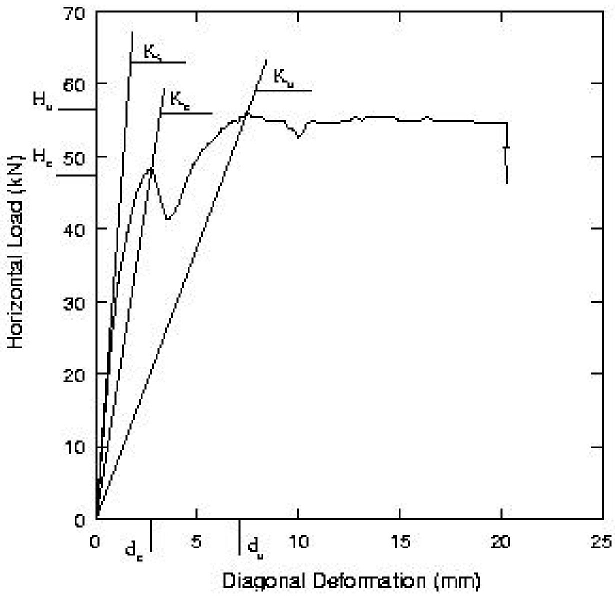

Figure 2 Stiffness Definition

With respect to load vs. deformation curves, three values of in-plane stiffness as defined in Figure 2 were derived corresponding to the three main states of behaviour of infilled frames. The stiffness in the initial response range is defined as the initial tangent stiffness, K_i. At the occurrence of the first crack in the masonry panel, the secant stiffness, K_c, is defined as the ratio between H_c and the corresponding diagonal deformation, d_c. Finally, the secant stiffness at the ultimate state, referred to as K_u, was obtained as the ratio of H_u and the diagonal deformation, d_u, at the ultimate state. Details regarding the in-plane stiffness of infilled frame have been reported in Dawe and Dukuze, 1998.

These diagonal deformations, d_c, at first crack and, d_u, at ultimate are reported in non-dimensional form, $,_c$ and $,_u$, respectively. The latter represent average strains along loaded diagonals and are defined as ratios of actual deformations to gauge lengths set during specimen instrumentation.

In-plane Behavior of Frames with Continuous Infill

Interest was directed towards horizontal load versus deformation along the compressed diagonal of a panel. For these curves, zones of pre-cracking, post-cracking and post-ultimate response can be identified as in Figure 3. Pre-cracking response extends from O to B. A lack of tight fit between panel and frame resulted in the gradual incline of segment OA. Between A and B, the curve is linear. During this phase, the loaded corners came into intimate contact and the other regions of the frame members deflected elastically away from the panel. This elastic behavior continued until a major diagonal crack occurred at which point a sudden temporary drop in load occurred.

Occurrence of the first major crack in the infill coincided with point B and initiated a nonlinear behavior. After the occurrence of the first major crack, additional cracks initiated and propagated in the panel. The crack orientation was related to the aspect ratio of the frame. While for a square specimen, cracks were diagonally oriented, slip along mortar bed joints was predominant in a rectangular specimen. Further loading led to extensive cracking of the enclosing frame due to a combination of shear and bending in the vicinity of loaded corners. Cracks in the infill and frame members led to substantial stiffness degradation of a specimen. Along with frequent load drops, a relative movement of adjacent brick courses was noticeable in the masonry panel. The slip along mortar joints was accompanied by load redistribution within the masonry panel.

As the load approached its peak, the masonry infill, cracked extensively along the compression diagonal and then re-contacted the frame causing a knee-brace effect resulting in continued lower strength and ductility (CD in Figure 3). Due to this wedging action, the infill came into full contact with the frame that restrained the panel from deforming, and eventually from falling out.

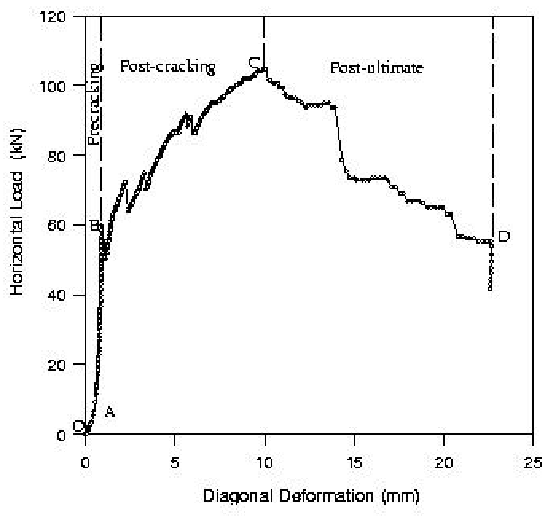

Figure 3 Load vs. Diagonal Deformation of a Square Frame with a Continuous Infill

Figure 4 Typical Rotation of Brick Units for Rectangular Specimens

In general, however, beyond ultimate (point C of Figure 3) the response is characterized by either a sudden drop, continuous deformation at a more or less constant load, or a gradual decrease in loading. The shape of the curve in this region depends on the geometrical and mechanical properties of the specimen. In general, rectangular specimens exhibit sudden drops in loading while square test units exhibit ductile and softening responses.

In this region, various deformations take place including extensive cracking, relative slip of adjacent brick courses, and marked rotation of masonry units especially for specimens with low values of β as shown in Figure 4. The slip and rotation mechanisms observed in the post-cracking phase are accompanied by localized fractures of brick units at points of high stress concentration near loaded corners. When β was increased, the frame had a strong restraining effect on the deformed infill which made the rotation mechanism less evident. In most cases, significant infill damage and shear failure of the frame members occurred before a full failure mechanism could develop. The testing procedure was stopped at this stage to avoid sudden failure of the specimen and damage to the instrumentation.

Failure mechanisms of square specimens underwent extensive damage to both masonry panels and surrounding frames. For frames that had strong columns, zones with extensive cracking were limited to beams in the vicinity of loaded corners. When specimens had a ratio of beam moment of inertia to column moment of inertia of 1, the damage distribution was approximately symmetrical and beams and columns failed in shear at loaded corners. For frames that had strong beams such as the S5P series, shear cracks extended along the loaded columns. This was accompanied with eventual crushing of brick units along the compressive diagonal strut.

For higher values of α, specimens failed predominantly by an overall bending mechanism. The masonry panel failed initially in diagonal tension while bed joint shear sliding was evident only in the upper section of the masonry panel. Flexural cracks of the windward column as well as the infill bed joint opened up due to high bending moment near the base.

ANALYTICAL MODELS

Infill/Frame Stiffness

A formulation was proposed by Mainstone (1971) based on replacing the infill by a diagonal strut whose relative stiffness between infill and frame is expressed through the parameter *8h* as follows:

$$\lambda h = \sqrt[4]{\frac{E_i t_i h^3 \sin 2\theta}{4 E_c I_c}} \qquad (1)$$

where E_i is Young's modulus of the infill, t_i is the infill thickness, 2 is the angle that the infill diagonal makes with the horizontal, h is the infill height, E_c is Young's modulus of the column, and I_c is the column moment of inertia. This value of relative stiffness is incorporated in the analysis below.

Ultimate Strength H_u and Cracking Load H_c

For analysis purposes, H_u and H_c are presented in the form of the non-dimensionalized load v_u and v_c. H_{ce} is the experimental result. The non-dimensional forms of H_u and H_c are given by:

$$\nu = \frac{H}{[2A_c\sqrt{\frac{f'_c}{f'_m}} + A_w]\sigma_{dt}} \qquad (2)$$

in which H is the horizontal component of the ultimate strength, F_{dt} denotes the diagonal tensile strength of the infill, f'_c and f'_m are the compressive strengths of concrete and masonry panel, respectively, and A_c and A_w are the cross sections areas of frame columns and the masonry wall.

To compare predicted and experimental results, values from current methods are reported as ratios of predicted values to corresponding experimental strengths. These ratios, $\nu_{uL} = H_{uI}/H_{ue}$ and $\nu_{uM} = H_{uM}/H_{ue}$, represent non-dimensionalized strengths derived from Liauw (1983) and Mainstone (1971) formulations, respectively.

A non-linear regression analysis conducted on non-dimensionalized strengths of the study reported herein yielded:

$$v_u = e^{\Omega_1} \qquad (3)$$

where S_1 = 0.16 - 0.51" + 0.07$ and " and $ represent the aspect ratio and the ratio of the beam moment of inertia to column moment of inertia, respectively. This formula, along with Equation 4, is used to calculate H_{uF} which is used as an assessment tool whose results are referred to as $\nu_{uF} = H_{uF}/H_{ue}$. From the above equation, the ultimate strength of an infill can be determined as follows:

$$H_u = v_u s_{dt}\left[2\sqrt{\frac{f'_c}{f'_m}}A_c + A_w\right] \qquad (4)$$

Using ultimate strength results of the present investigation and those reported by Barua and Mallick (1977) and Samai (1984), a nonlinear curve fit was conducted and yielded Equation 5. To find a general empirical formula applicable to reinforced concrete in filled frames, nonlinear curve fitting was conducted on results including those of the present study and respective data reported by Barua and Mallick (1977) and Samai (1984). The best fit is summarized by the following analytical equation:

$$v_u = 10.22(lh)^{-1.47} \qquad (5)$$

where $8h$ is the relative stiffness term from Equation 1. The results of the present study are summarized in Table 2 for frames with continuous infill. H_{ce} and H_{cu} are the values of cracking and ultimate strength obtained from experimental results, respectively. $\nu_{ce,}$ ν_{ue} represents the non-dimensionalized value of H_{ce} and H_{cu}, respectively. ν_{uL}, ν_{uM}, from Liauw and Mainstone formulations, respectively, generally overestimate the ultimate strength compared to ν_{uF} from Equation 3. It appears that ν_{uP} from Equation 5 generally provides better estimates of the ultimate strength. Although on the conservative side, ν_{uF} from Equation 3 provides an overall adequate prediction of the ultimate shear of infilled frames of the present study. ν_{uP} presents an advantage of being applicable over a range of aspect ratios α of 0.5 to 2 and ratios of the beam moment of inertia to column moment of inertia β of 0.2 to 5. The ratios of the predicted and experimental ultimate strengths of the results of the present study ranges from 0.66 to 1.42 while those of Liauw and Mainstone vary between 0.70 to 3.04 and 1.37 to 3.82, respectively. Those values of ν_{uP} range from 0.69 to 2.17 with an average of 1.22 and a coefficient of variation of 0.40. The methods developed by Liauw and Mainstone were derived from tests conducted on relatively small-scale steel frames infilled with various materials including plaster, brickwork, and micro-concrete. This may explain in part, the larger discrepancies associated with these results.

Table 2 Comparison of Ultimate Strength

Specimen	λh	H_{ce} KN	H_{ue} KN	ν_c	ν_u	ν_F	ν_{uL}	ν_{uM}	ν_{uP}
1	2	3	4	5	6	7	8	9	10
S2PD	3.72	51.60	108.30	0.44	0.92	0.78	1.05	1.37	1.6
S2PL	4.23	78.60	93.50	0.52	0.62	1.15	1.21	1.41	1.98
S2PR	4.06	59.11	100.00	0.62	0.73	0.98	1.56	1.60	1.78
R2PL	4.92	21.00	48.10	0.20	0.45	0.95	0.83	3.51	2.17
R2PR	4.72	20.10	57.90	0.19	0.54	0.8	0.81	3.82	1.93
R1P1	3.38	23.80	55.90	0.44	1.02	0.96	2.27	3.64	1.67
R1P2	4.07	26.30	48.40	0.80	1.47	0.66	3.04	3.58	0.88
R1P3	3.62	43.50	65.70	0.58	0.87	1.12	2.24	2.92	1.77
S1P1	5.74	31.20	58.50	0.31	0.59	1.29	1.71	3.36	1.34
S1P2	6.23	28.10	58.20	0.34	0.71	1.07	1.95	2.87	0.98
S1P3	6.12	36.40	84.40	0.39	0.90	0.84	1.18	2.21	0.79
S1PD	6.42	31.80	69.50	0.24	0.53	1.42	1.54	2.58	1.25
S1PL	6.76	56.60	66.30	0.59	0.69	1.09	1.61	2.60	0.89
S1PR	6.48	72.10	104.50	0.63	0.91	0.83	1.42	1.99	0.72
R1PL	7.33	48.70	55.50	0.32	0.66	0.69	0.72	3.19	0.83
R1PR	7.03	48.50	66.50	0.35	0.82	0.55	0.7	3.21	0.71
R5P1	3.47	28.80	73.10	0.39	1.00	1.28	1.82	2.72	1.65
R5P2	4.16	30.40	89.60	0.54	1.58	0.80	1.56	1.89	0.79
R5P3	3.70	57.50	90.30	0.78	1.23	1.03	1.55	2.08	1.21
S5P1	5.74	18.20	64.90	0.23	0.81	1.22	1.64	3.49	0.97
S5P2	6.47	42.80	83.10	0.43	0.84	1.17	1.37	2.48	0.78
S5P3	5.99	35.10	102.50	0.36	1.07	0.93	1.04	2.14	0.69
S5PD	6.32	38.10	117.50	0.29	0.90	1.10	0.91	1.79	0.76

For serviceability reasons, it is often important to determine the load at which the first crack occurs. Based on test results gathered in the present investigation and data collected from Barua and Mallick (1977) and Samai (1984), the load at first crack, H_c, can be expressed as a fraction of the ultimate strength expressed as follows:

$$H_c = 0.68 H_u \tag{6}$$

CONCLUSIONS

Twenty-four specimens were tested to failure. Using the test results, formulations were developed to predict the behaviour of this type of composite structure. As a result of this investigation, the following conclusions have been reached:

1. Frames with continuous infill exhibit three distinct stages of response, those being pre-cracking, post-cracking, and post-ultimate stages. In the pre-cracking stage, the infill and perimeter frame behave as a monolithic structure. Subsequently, separation of panel and frame occurs. For frames with an aspect ratio close to 1.0, intimate re-contact may occur resulting in significant post-peak ductility. In general, the post-peak strength and ductility depends largely on the frame aspect ratio;

2. The load at first crack of a reinforced concrete frame with a masonry panel could be estimated at about sixty-eight percent of the ultimate strength.

3. Failure mechanism depended on the aspect ratio. While for square specimens, the infill and surrounding frame underwent extensive damages, the rectangular specimens failed predominantly with bending mechanism.

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