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STRENGTHENING METHODOLOGY FOR LIGHTLY REINFORCED CONCRETE FRAMES

L.T. PHAN[1] and H.S. LEW[1]

[1]Structures Division, Building and Fire Research Laboratory,
National Institute of Standards and Technology, Gaithersburg, Maryland, 20899, USA

ABSTRACT

To develop an analytical tool for assessing seismic performance of strengthened lightly reinforced concrete (LRC) frames, hysteresis failure models were developed and incorporated into computer program IDARC (Kunnath *et al.*, 1987) for dynamic analysis of LRC frames. The models were developed by, first, using the system identification techniques to characterize the load-deformation histories of LRC frame tests in terms of the stiffness degradation parameter α, the strength degradation parameter β, and the pinching parameter γ. Next, multi-variable regressions were performed to relate α, β, γ as functions of the specimen's material and geometric properties and reinforcement parameters. The empirical expressions resulting from these regression analyses are the hysteresis failure models. The models were validated against experimental results and used in a parametric study to evaluate the influence of certain variables to the behavior of infilled LRC frames. The variables included the thickness of infilled wall, the amount of reinforcement in infilled walls, and the area of anchors which connects the infilled wall to the frame. General design guidelines are proposed.

KEYWORDS

Analytical; building technology; dynamic analysis; experimental; frames; hysteresis models; infilled walls; rehabilitation; reinforced concrete; seismic strengthening; system identification.

INTRODUCTION

It has been recognized that a large percentage of the existing RC buildings in the United States which were designed primarily for gravity loads and built prior to the 1970s' are not adequate for resisting seismic loading. These buildings are, in general, less than five stories in height and have characteristic reinforcement details that were acceptable based on the building codes used at the time. In recent U.S. earthquakes, these buildings sustained severe damage. Typical reinforcement details of such buildings included (1) low longitudinal reinforcement ratio for columns, (2) little or no transverse reinforcement within the beam-column joint region, and (3) large spacing of transverse reinforcement in columns.

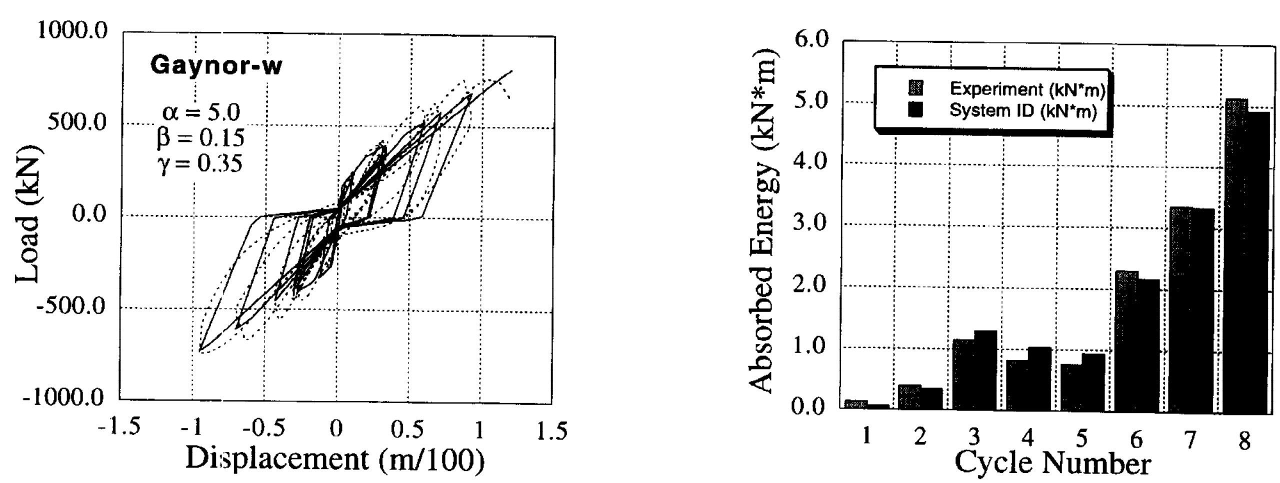
Research on the improvement of the seismic performance of lightly reinforced concrete (LRC) buildings have been conducted in recent years. Most are experimental studies which involved cyclic lateral load tests of scaled specimens representing subassemblages of LRC buildings. The strengthening techniques proposed and studied in the past studies can be group in four broad categories: (1) *infilled wall techniques*; (2) *beam-column joint upgrading techniques*; (3) *exterior steel bracing techniques*; and (4) *column strengthening techniques*. Within each individual test program, qualitative conclusions regarding the merits of strengthening techniques used were provided. However, since experimental work is very costly, none of the test programs had a broad enough scope to include all possible factors which might influence the performance of the LRC frames before and after strengthening. Thus, despite the knowledge obtained with regard to the effectiveness of the strengthening schemes studied. At present, quantitative assessment of the effectiveness of each of these strengthening schemes on the increase in strength and ductility of LRC buildings is not available.

In an effort to develop a quantitative assessment technique, a study was initiated at the National Institute of Standards and Technology (NIST). NIST's approach is to utilize available experimental research results to develop analytical techniques for evaluating quantitatively the effectiveness of common strengthening schemes.

DEVELOPMENT OF HYSTERESIS FAILURE MODELS FOR RC FRAMES

Hysteresis failure models, as referred to in this paper, are empirical expressions relating three hysteresis parameters: stiffness degradation α, strength degradation β, and pinching factor γ to materials and geometric properties of RC frames. These parameters (α, β, γ) characterize the hysteresis behavior of the structural components of RC frames up to failure and are required for use with program IDARC (Kunnath *et al.*, 1987) for analysis of LRC frames. Description of the procedure used in developing the hysteresis models and the selected experimental database are provided in a report (Phan *et al.*, 1993). The procedure involved the following two main tasks:

1. Estimate the values for three parameters α, β, and γ which best fit the experimental hysteresis loops. The method used for this process is referred to as the System Identification method. Principally, the system identification procedure performs a trial and error search for a set of initial values of α, β, and γ such that the cumulative error between the predicted and experimentally observed hysteretic energy is minimized. Interactive adjustments (by changing values of α, β, and γ graphically) can then be performed until a visual match between the hysteretic responses can be observed and an absorbed energy cumulative error of within a few percent can be attained. The values of α, β, and γ so determined constitute the estimated hysteresis parameters of the corresponding experiment. Example of the match between the estimated and experimental hysteresis responses is shown in Fig. 1. A total of fifty-five specimens were selected for system identification (Phan et. al., 1993).



(a) Load-Deformation History

(b) Absorbed Energy Histograms

Fig 1. System Identification of Experimental Result

2. Perform multiple-variable regression analysis to obtain empirical expressions for α, β, and γ in terms of the physical properties (material, geometric properties, and reinforcement details) of the test specimens. These expressions, which constitute the hysteresis failure models of the frame, are then used to calculate the parameters α, β, and γ for various frames for analysis. Typically, hysteresis failure models are linear functions of variables such as column thickness, column reinforcement ratios, column axial load, infilled wall thickness, infilled wall reinforcements, etc.. The hysteresis parameters have the following general forms:

$$\alpha = \sum a_i X_i$$
$$\beta = \sum b_i Y_i$$
$$\gamma = \sum g_i Z_i$$

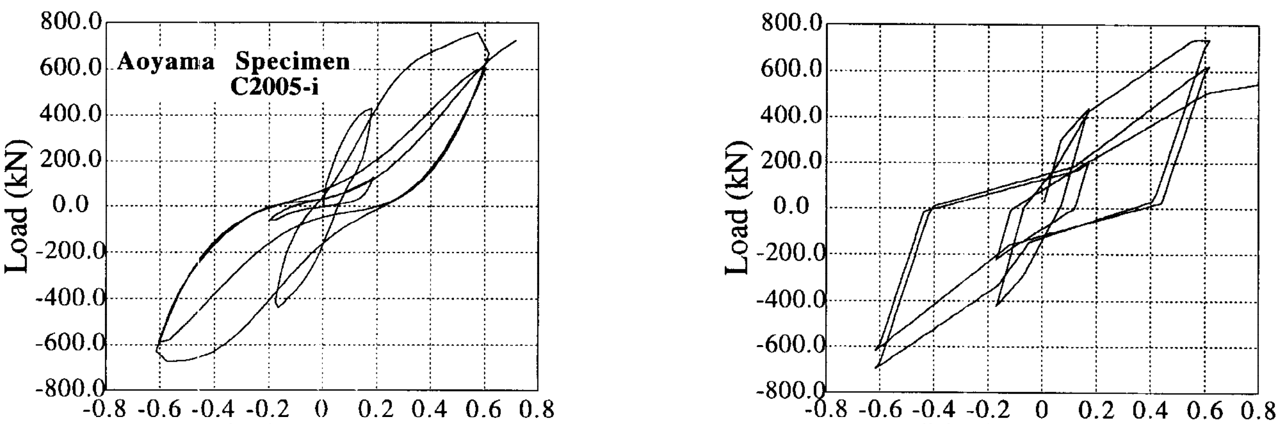
Where a_i, b_i, and g_i are regression coefficients and X_i, Y_i, and Z_i are the frame variables such as column effective depth, column axial stress, infilled wall cross sectional area, ratio of column reinforcement, etc.. Detailed expressions of these empirical hysteresis models may be found in a published report (Phan *et al.*, 1993).

VALIDATION OF HYSTERETIC MODELS FOR USE WITH IDARC

For validation, hysteresis failure models were used to compute parameters α, β, and γ in analysis of three tested frames. The analytical results, in terms of load-deformation characteristics and ultimate load capacity, obtained from IDARC analysis were then compared with results of the experiments. The analyzed frames include a one-story one-bay frame tested by Aoyama *et al.*, 1984, a three-story one-bay frame tested by Higashi *et al.*, 1981 and 1982, and a three-story two-bay frame tested by Yunfei *et al.*, 1986.

Analysis of One-Story One-Bay Frame

The 1:3 scaled one-story one-bay frame, tested by Aoyama *et al.*, 1984, was strengthened with a cast-in-place (CIP) infilled wall. The specimen was subjected to five cycles of statically reversing lateral load. Ultimate strength of 749 KN was attained in the second cycle. The experimental hysteresis behavior is shown in Fig. 2(a). The analytical hysteresis behavior of the frame corresponding to hysteresis parameters of α=11.0, β=0.35, and γ=0.25 is shown in Fig. 2(b).



(a) Experimental (b) Analytical

Fig. 2. Comparison of Hysteresis Behavior for Aoyama's One-Story, One-Bay Frame

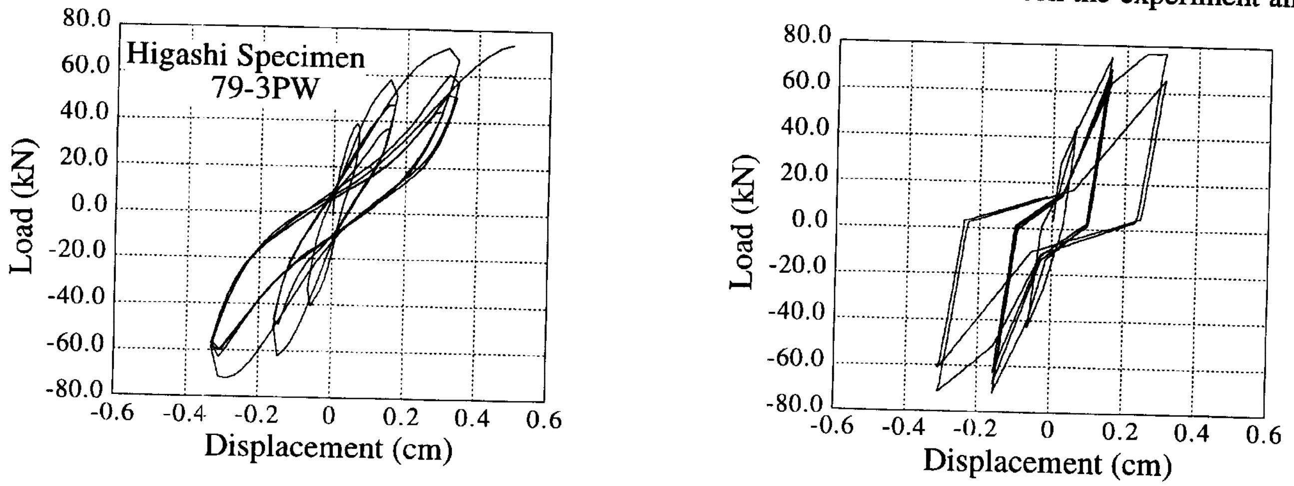
As can be seen in Fig. 2 (a) and (b), reasonable match in the ultimate load capacity, 749 KN for experimental vs. 750 KN for analytical was attained, even though the analytical result appears to lack the finesse to closely model the pinching action observed in the experimental behavior.

Analysis of a Three-Story One-Bay Frame

A 1:8 scaled three-story one-bay frame tested by Higashi *et al.*, 1981 and 1982, was analyzed. All three bays of the frame were infilled with CIP walls. The experimental and analytical load-deformation histories are shown in Fig. 3(a) and (b). Again, the analytical result compares reasonably well with the experimental behavior Again, a lack of finesse in terms of modeling the deformation characteristic (pinching) may be observed.

Analysis of a three-story two-bay Frame

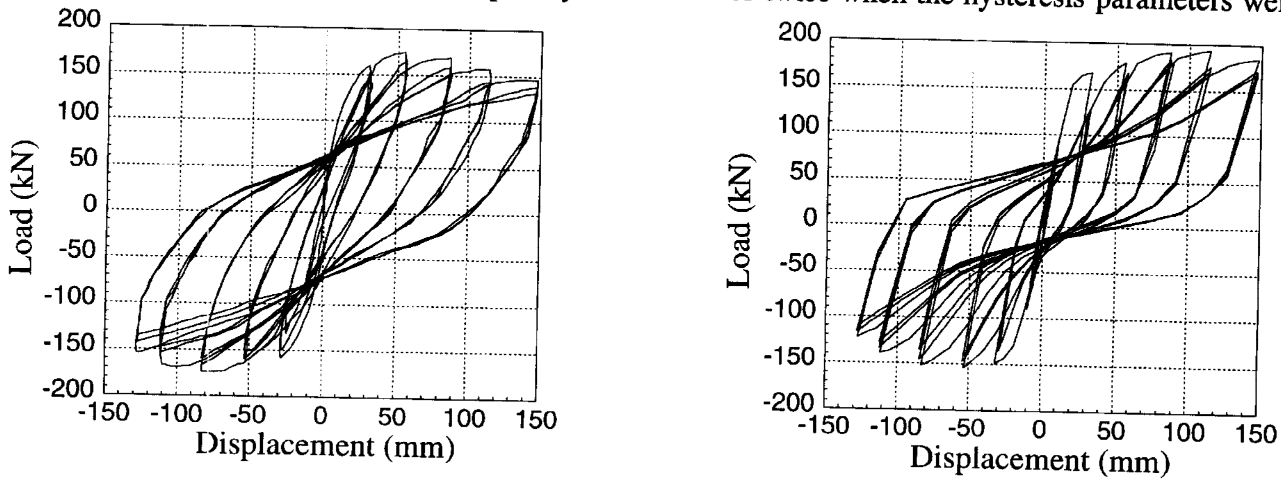
The experimental and analytical deformation histories of the 1:2 scaled three-story two-bay LRC frame, tested by Yunfei *et al.*, 1986,, are as shown in Fig. 4 (a) and (b). Reasonable match, in terms of structural stiffness and resulting shear forces, up to 1.25% drift may be observed. At this drift level, the frame has gone well into the inelastic range as shown in the experimental load-deformation history. Beyond this drift level, the analytical model predicted the shear capacity of 194 KN, while the experimental result showed 174 KN at 1.25% drift. The difference in shear capacity at higher drift levels between the experiment and



(a) Experimental (b) Analytical

Fig. 3. Comparison Hysteresis Behavior for Higashi's Three-Story, One-Bay Frame

the analytical model is probably due to the modeling technique used. In modeling the frame, each bay of the two-story three-bay frame was assigned a set of hysteresis parameters α, β, γ, computed using the hysteresis models derived from one-bay one-story frame tests. Thus, for the adjacent bays which shared the same column, the shared column is implicitly accounted for twice when the hysteresis parameters were



(a) Experimental (b) Analytical

Fig. 4. Comparison of Load-Deformation Histories of Yunfei's Frame

computed for each of the two bays. For this reason, the analytical model would be stronger than the actual test frame. There are no straightforward ways to model the shared column at this stage since the hysteresis models were fundamentally developed using one-bay one-story experiments. However, it should be noted that the model appeared to be adequate in predicting the load-deformation behavior well into the inelastic range.

PARAMETRIC STUDY

Parametric study was conducted to assess the sensitivity of three variables to the seismic behavior of LRC frames. The variables included the thickness of the infilled wall (t_w), the amount of wall reinforcement (ρ_w), and the area of connecting anchor (A_c). A 1:2 scaled one-bay one-story model of a prototype frame was used. The frame configuration and dimensions are as shown in Fig. 5. To study the influence of t_w, hysteresis parameters corresponding to wall thicknesses ranged from 0 mm (existing frame) to 250 mm (width of bounding columns) were computed and used in the analyses, while ρ_w and A_c were held constant. To study the influence of ρ_w, ρ_w was varied from 0.1% to 1.2% and the corresponding hysteresis parameters were computed while t_w and A_c were held constant. To study the influence of A_c, A_c ranged from 5 cm^2 to 15 cm^2 was used while t_w and ρ_w were held constant (Phan et al., 1995).

Both quasi-static and transient dynamic analyses were performed. For quasi-static analysis, the frame was subjected to prescribed displacement history as shown in Fig. 6. The quasi-static analyses examined predicted values of maximum shear forces and maximum story drifts experienced by the frames. For transient dynamic analyses, acceleration records obtained from free-field stations from various earthquakes were scaled and used as input motion. The transient dynamic analyses were performed to examine the hysteretic responses of the LRC frame to random excitation.

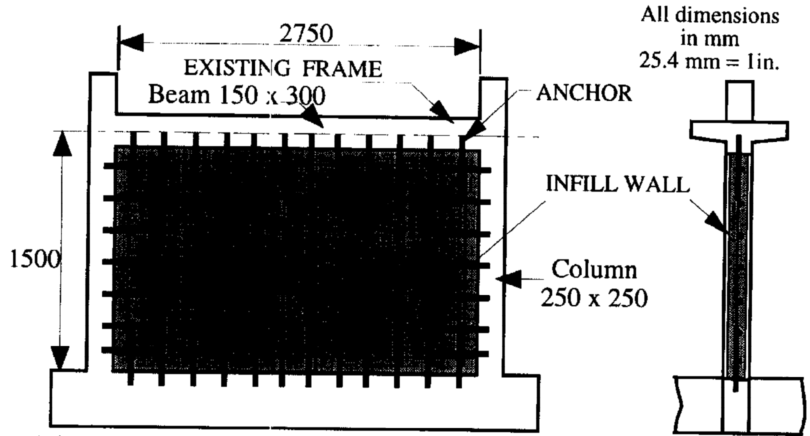


Fig. 5. Dimensions of Frame used in Parametric Study

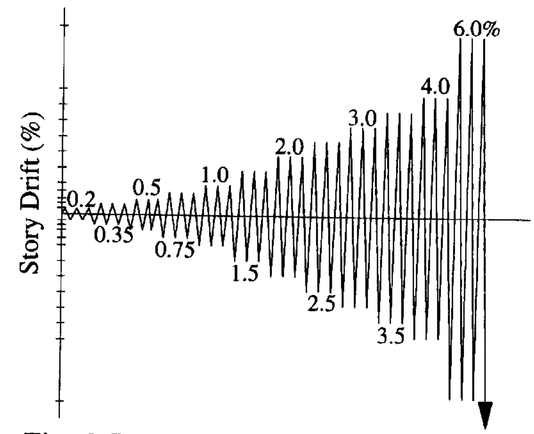


Fig. 6. Prescribed Displacement History

Results of Quasi-Static Analysis

In this paper, the maximum story drift is defined as the story drift that corresponds to the maximum shear force. The maximum story drifts and maximum shear forces obtained for different values of t_w are shown in Fig. 7(a) and (b), respectively. Those corresponding to different values of ρ_w are shown in Fig. 8(a) and (b), respectively. And those corresponding to different values of A_c are shown in Fig. 9(a) and (b), respectively.

As shown in Fig. 7(a) and (b), the maximum story drifts of infilled frames decreased with increasing the wall thickness. The decrease in story drift becomes less significant beyond t_w= 150 mm (t_w equals 3/5 of column width). For t_w = 75 mm to 250 mm, the maximum story drifts varied from 8% (t_w = 75 mm) to a minimum of 4% (t_w = 250 mm). The maximum story drift remained at 4% for wall thicknesses of 150 mm to 250 mm. The maximum shear strength increased parabolically with increasing t_w. A higher rate of increase in shear strength occurred at about t_w= 100 mm (t_w equals 2/5 of column width). These results indicate that for a CIP infilled wall, the optimal design in which highest increase in shear capacity can be achieved without substantial improvement in the maximum story drift should include wall with thickness t_w of at least 2/5 the thickness of the bounding columns.

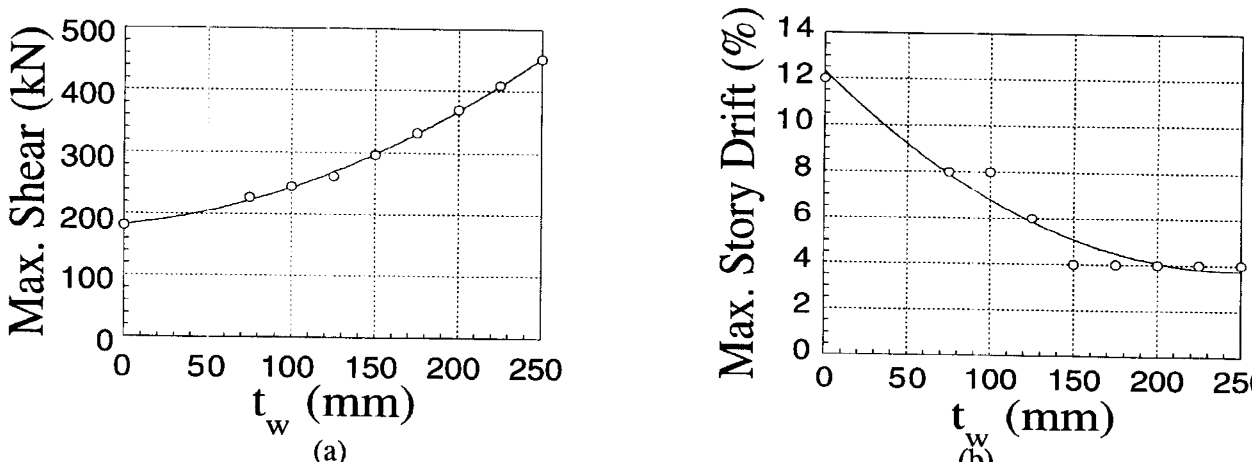


Fig. 7. Maximum Shear (a) and Story Drift (b) with Varying Infilled Wall Thicknesses

Neither the story drift nor shear strength are affected by increasing the infilled wall reinforcement ratios ρ_w, as shown in Fig. 8(a) and (b). There are no clear experimental data to corroborate with this analytical result since ρ_w was not a variable in all the experimental programs reviewed. In each experimental program reviewed (see Phan *et al.*, 1993), the ratios of ρ_w were selected to be equal in the vertical and horizontal directions and were the same for all specimens. Comparison between different experimental programs to isolate the effect of infilled wall reinforcement ratios is rather difficult and thus not conducted.

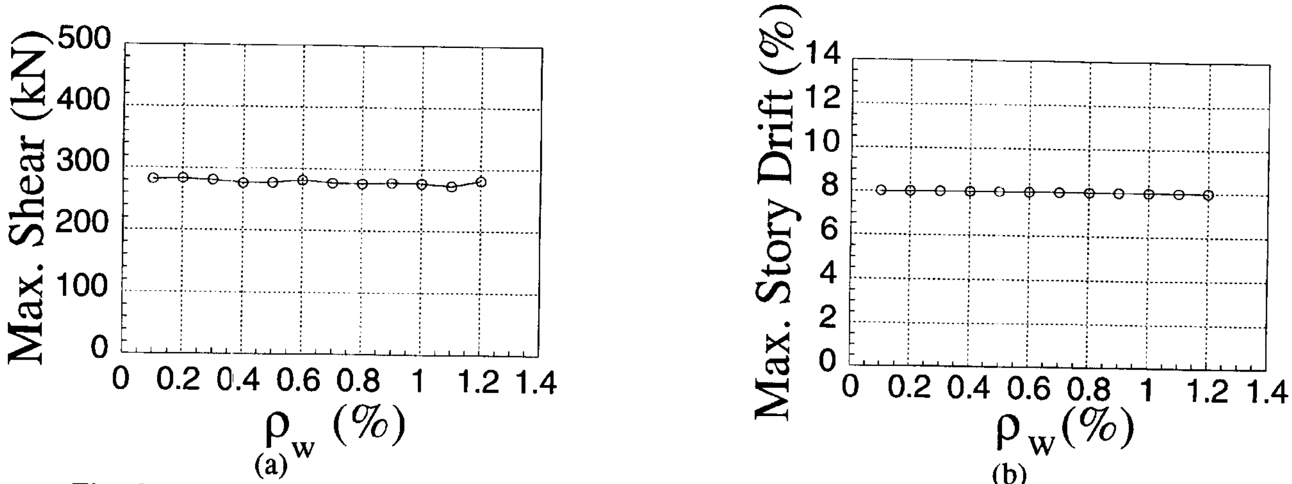


Fig. 8. Maximum Shear (a) and Story Drift (b) with Varying Wall Reinforcement Ratio ρ

Fig. 9(a) and (b) shows the effect of varying A_c between 5 cm^2 to 15 cm^2 (these corresponded with a ratio of A_c/A_w of 0.3% to 0.9%, where A_w is the area of the infilled wall on the wall/frame interface). The analytical results show the increase in story drift occurred with value of A_c greater than 7.5 cm^2 (A_c/A_w = 0.45%). This increase continues at different rates up to the value of A_c= 15 cm^2 (A_c/A_w = 0.9%). It appears that little increase in story drift will be obtained by increasing A_c beyond 15 cm^2 as the curve appears to level off at that point. The maximum shear force appears to increase slightly as the anchor area is increased. These results indicate that the desirable A_c/A_w ratios for appreciable gain in maximum story drift and some gain in shear strength should be greater than 0.45%. It should be noted that successful performance has been experimentally observed with A_c/A_w= 0.81% (Aoyama *et al.*, 1984).

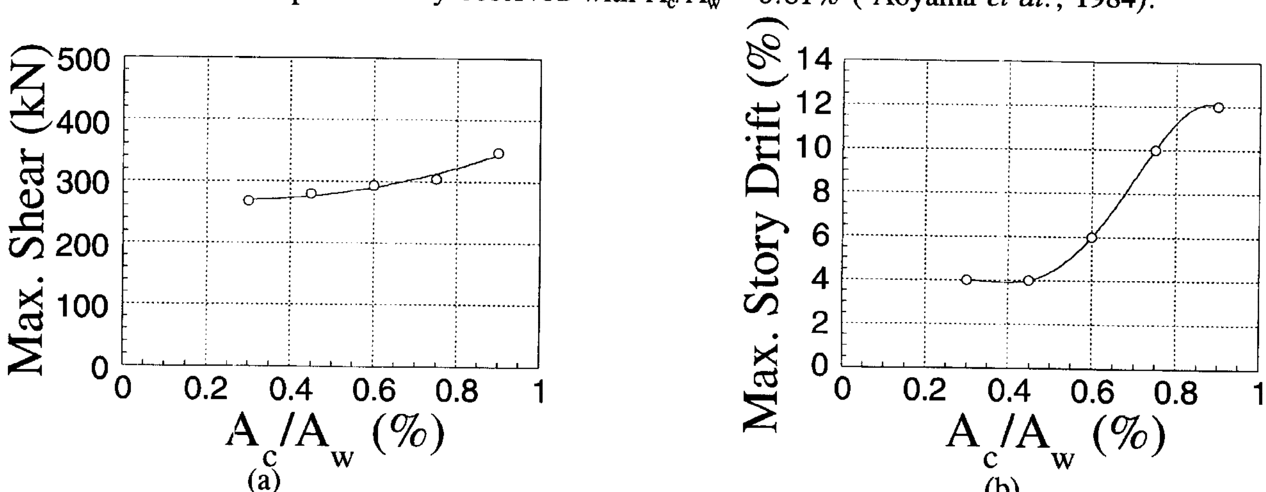
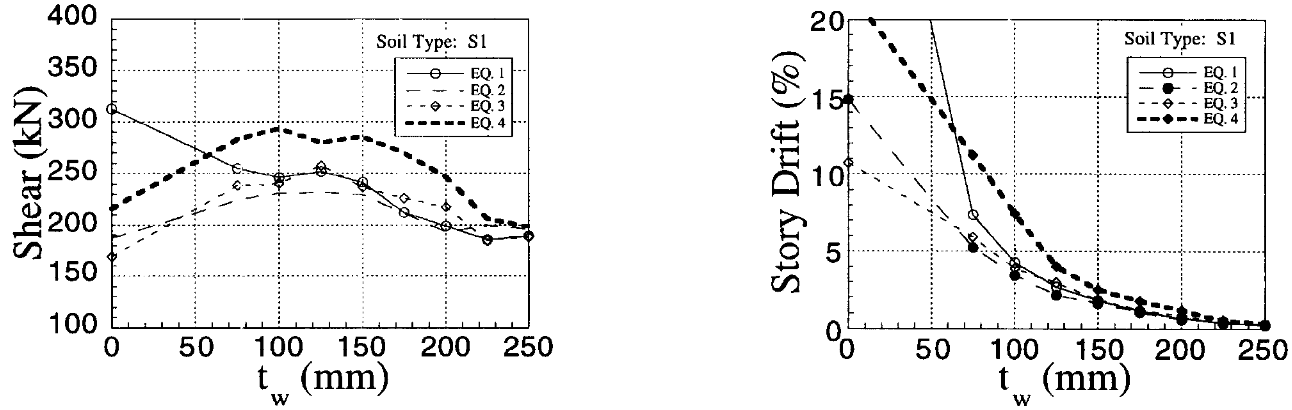


Fig. 9. Maximum Shear (a) and Story Drift with Varying A_c

Results of Dynamic Analysis

Due to the small aspect ratio (height to base length ratio) of the frame (one-bay one-story frame), it was necessary to scale the maximum horizontal acceleration used in the analysis to 3g in order to force the infilled frames into the failure ranges. The effects of t_w, ρ_w, and A_c on the maximum story drift and maximum shear strength of the frame were studied and shown in Fig. 10, 11, and 12, respectively.

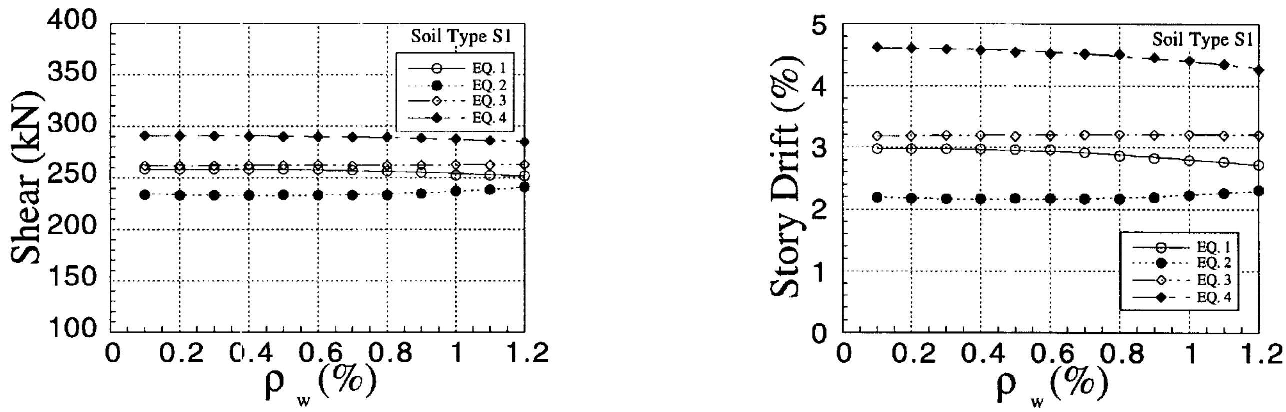
In Fig. 10(a) and (b), the maximum story drifts varied from 3.5% to 11.5% for 75 mm $\leq t_w \leq$ 100 mm,. For $t_w \geq$ 100 mm, the maximum story drifts varied from 0.2% to 4% with the higher end of the range corresponding to the models with lower t_w. Based on these plots, a minimum t_w of 100 mm (2/5 of column width) would be required to limit to maximum story drift to 3% or less. A wall thickness of 200 mm (4/5 of the column width) would limit the maximum story drifts to approximately 1% or less. The maximum shear remained approximately constant with increasing t_w for a given earthquake record, but a reduction in the shear force for t_w = 200 mm to 250 mm was observed for all input records.



(a) (b)

Fig. 10. Maximum Shear Strength (a) and Story Drift (b) with Varying t_w (Soil Type 1)

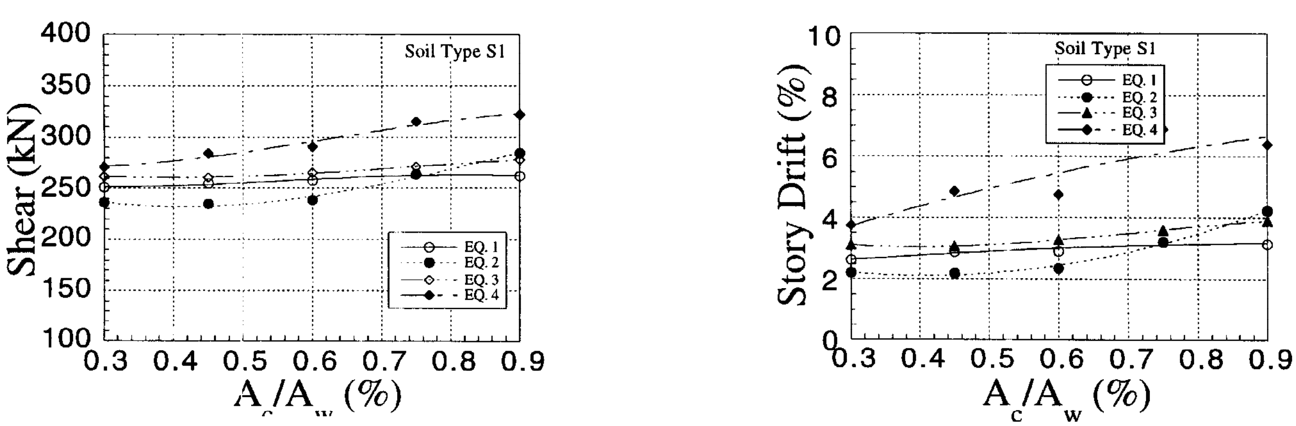
The effect of varying ρ_w on the maximum shear strength and story drift are shown in Fig. 11(a) and (b). There was insignificant variation in the maximum story drift and shear strength with increasing ρ_w values. The maximum story drifts varied from 2% to 5% (for soil type 1). This observation is similar to the case of quasi-static analysis.



(a) (b)

Fig. 11. Maximum Shear Strength (a) and Story Drift with Varying ρ_w

The plots for the maximum drifts and shears vs. A_c/A_w are given in Fig. 12(a) and (b). The trend for both the maximum drift and shear are increased drifts/shears for increasing anchor areas and the maximum drift and shear strength are independent of earthquake input motions. The story drifts ranged from 2% to 7% and the shear force ranged from 230 kN to 330 kN. This was also observed for the quasi-static analysis.



(a) (b)

Fig. 12. Maximum Shear Strength (a) and Story Drift (b) with Varying A_c

CONCLUSIONS

- For appreciable gain in shear strength and a reasonable reduction in story drift, the thickness of cast-in-place infilled wall, t_w, should be at least 40% the thickness of the bounding column.

- Based on experimental observation, the ratio of the total cross sectional area of the connecting anchors and the area of the infilled wall (A_c/A_w) should not be less than 0.8% for effective interaction between the infilled wall and the existing frame.

- While the parametric study indicated that the infilled wall reinforcement ratio did not have an effect on the ultimate strength and maximum story drift of infilled frame, it is recommended that infilled walls should have flexural reinforcement ratio, both in the vertical and horizontal directions, of not less than 0.75%. Lateral ties (confinement reinforcement through thickness of infilled wall) of 0.25 to 1.0% has been reported to be effective for confining concrete in the infilled walls.

The above recommendations were derived from both experimental and analytical observations. Below are recommendations that were extracted from experimental programs reviewed. These are details which are too complex to be studied by the hysteresis models.

- Either mechanical wedge anchors or epoxy dowels can be used to connect CIP infilled walls. The connectors should be placed on the inner surface of the frame and be located as close to the center line of the concrete infilled wall as possible so as to minimize the eccentricity of the transfer shear force.

- Where more than one line of connectors is required due to the required total area of connectors given above, the distance between the connector lines should be not less than 5 times the diameter of the connector.

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