STRENGTHENING METHODOLOGY FOR LIGHTLY REINFORCED CONCRETE FRAMES

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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 $\alpha$, the strength degradation parameter $\beta$, and the pinching parameter $\gamma$. Next, multi-variable regressions were performed to relate $\alpha$, $\beta$, $\gamma$ 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 subassemblies of LRC buildings. The strengthening techniques proposed and studied in the past studies can be grouped 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 $\alpha$, strength degradation $\beta$, and pinching factor $\gamma$ to materials and geometric properties of RC frames. These parameters ($\alpha$, $\beta$, $\gamma$) 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 $\alpha$, $\beta$, and $\gamma$ 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 $\alpha$, $\beta$, and $\gamma$ such that the cumulative error between the predicted and experimentally observed hysteretic energy is minimized. Interactive adjustments (by changing values of $\alpha$, $\beta$, and $\gamma$ 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 $\alpha$, $\beta$, and $\gamma$ so determined constitute the estimated hysteresis parameters of the corresponding experiment. Example of the match between the estimated and experimental hysteretic responses is shown in Fig. 1. A total of fifty-five specimens were selected for system identification (Phan et al., 1993).

2. Perform multiple-variable regression analysis to obtain empirical expressions for $\alpha$, $\beta$, and $\gamma$ 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 $\alpha$, $\beta$, and $\gamma$ 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:
\[ \alpha = \sum a_iX_i \]
\[ \beta = \sum b_iY_i \]
\[ \gamma = \sum g_iZ_i \]

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 \(\alpha, \beta,\) and \(\gamma\) 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 \(\alpha=11.0, \beta=0.35,\) and \(\gamma=0.25\) is shown in Fig. 2(b).

![Graph](image)

**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

![Graphs showing experimental and analytical hysteresis behavior](image)

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 $\alpha$, $\beta$, $\gamma$, 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

![Graphs showing experimental and analytical load-deformation histories](image)

Fig. 4. Comparison of Load-Deformation Histories of Yunfel’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 ($\rho_w$), 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 $\rho_w$ and $A_c$ were held constant. To study the influence of $\rho_w$, $\rho_w$ 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 $\rho_w$ 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 $\rho_w$ are shown in Fig. 8(a) and (b), respectively. And those corresponding to different values of $A_e$ 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 a 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 $\rho_w$, as shown in Fig. 8(a) and (b). There are no clear experimental data to corroborate with this analytical result since $\rho_w$ was not a variable in all the experimental programs reviewed. In each experimental program
reviewed (see Phan et al., 1993), the ratios of $\rho_w$ 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.

![Graphs showing maximum shear and story drift with varying wall reinforcement ratio $\rho$](image1)

**Fig. 8.** Maximum Shear (a) and Story Drift (b) with Varying Wall Reinforcement Ratio $\rho$

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).

![Graphs showing maximum shear and story drift with varying $A_c/A_w$ ratio](image2)

**Fig. 9.** Maximum Shear (a) and Story Drift with Varying $A_c/A_w$

*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 $t_w$, $\rho_w$, 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 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.

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

The effect of varying $\rho_w$ 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 $\rho_w$ 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.

Fig. 11. Maximum Shear Strength (a) and Story Drift with Varying $\rho_w$

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.

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.

REFERENCES


