BIBLIOGRAPHIC INFORMATION

PB97-110886

Report Nos: NCEER-95-0018

Title: Modeling of Masonry Infill Panels for Structural Analysis.

Date: 8 Dec 95

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Performing Organization: National Center for Earthquake Engineering Research, Buffalo, NY.

Sponsoring Organization: *National Science Foundation, Arlington, VA.*New York State Science and Technology Foundation, Albany.

Contract Nos: NSF-BCS-90-25010, NYSSTF-NEC-91029

Type of Report and Period Covered: Technical rept.

NTIS Field/Group Codes: 89D (Structural Analyses), 89G (Construction Materials, Components, & Equipment)

Price: PC A06/MF A01

Availability: Available from the National Technical Information Service, Springfield, VA. 22161

Number of Pages: 86p

Keywords: *Concrete structures, *Dynamic structural analysis, *Seismic effects, *Structural vibration, Dynamic response, Hysteresis, Stiffness, Ground motion, Seismic waves, Seismic loads, Mechanical properties, Static loads, Cyclic loads, Crackinq(Fracturing), Earthquake damage, Structural members, Nonlinear analysis, Mathematical models.

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

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This research was conducted at the State University of New York at Buffalo and was partially supported by the National Science Foundation under Grant No. BCS 90-25010 and the New York State Science and Technology Foundation under Grant No. NEC-91029.

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NCEER Task Number 93-3102B and 94-3103A

NSF Master Contract Number BCS 90-25010
and
NYSSTF Grant Number NEC-91029

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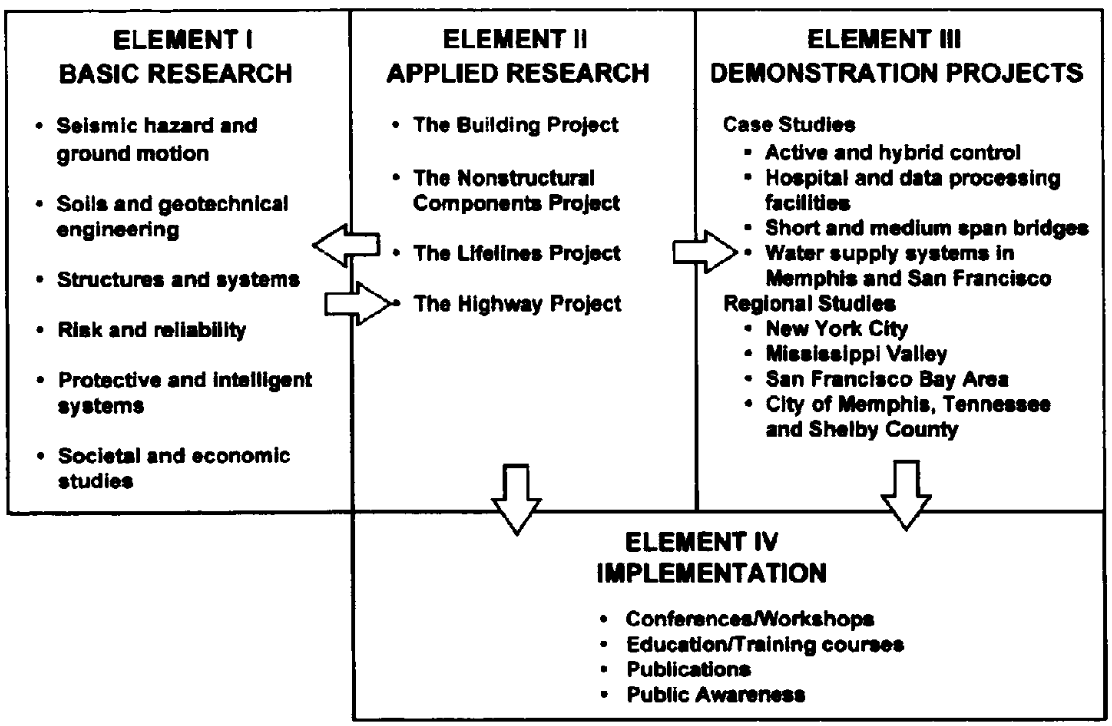
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

ABSTRACT

A smooth hysteretic model based on an equivalent strut approach is proposed for masonry infill panels to be used in non-linear analysis of building structures. The hysteretic model furnishes a versatile and robust simulation tool for representing masonry infill panels. The model, which is applicable for degrading 'pinching' elements in general, can be implemented to replicate a wide range of hysteretic force-displacement behavior resulting from different design and geometry by varying the control parameters of the model. The control parameters of the proposed hysteretic model can be determined using any suitable theoretical model for masonry infills. The report presents the development of the proposed hysteretic model. An available theoretical model for masonry infilled frames is recommended for estimating the control parameters of the proposed hysteretic rule. The methodology for calibrating the hysteretic model parameters is described. The hysteretic model is incorporated in the structural analysis program, IDARC2D Ver 4.0, for quasi-static cyclic and dynamic analysis of masonry infilled frames. Simulations of experimental force-deformation behavior of prototype infill frame subassemblages are performed to validate the proposed model and presented herein. A lightly reinforced concrete frame structure is analyzed for strong ground motions to evaluate the influence of masonry infill panels on the response.

ACKNOWLEDGEMENTS

Funding for this project was provided through the National Center for Earthquake Engineering Research (NCEER) Contract No. 933102 and 943103A under the National Science Foundation Master Contract No. ECE-86-07591 and the State of New York. The support is gratefully acknowledged.

The authors are grateful for the collaboration and assistance of Prof. Peter Gergely of Cornell University.

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INTRODUCTION

A large number of buildings are constructed with masonry infills. However, because of the absence of a realistic, yet simple analytical model, the contribution of masonry infill panels is often neglected in the non-linear analysis of building structures. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of the structure. The new design provisions dictate that the effect of structural and non-structural masonry infills be accounted for in the analysis of structures.

The importance of accounting for masonry infills in design of framed structures was recognized around four decades ago. The behavior of masonry infilled frames has been extensively studied since, in attempt to develop a rational approach for designing such frames. A limited review of the literature on the past research concerning modeling of masonry infills is presented in the next section. The literature survey is limited to issues directly related to the modeling presented in this report. A complete review of research on infilled frames through 1987 has been reported by Moghaddam and Dowling (1987). However, most of the studies provide semi-empirical formulations of design values suitable only for code implementations. A more rigorous force-displacement analysis of structures with masonry infilled frames requires a theoretical model of the force-deformation response of masonry infills. Furthermore, for seismic design and evaluation purposes, a complete dynamic time-history analysis of the structure may be required, which presents the need for a macro-model interpretation using a hysteretic model for masonry infills.

Theoretical micro-models such as finite element models offer a viable solution, but are inefficient computationally for analysis of large buildings with numerous components. Generalized macro-models are more suitable for representing the behavior of components in the analysis of such structures. A macro-model can be implemented to simulate varying response effects resulting from different design and geometry of structural components by controlling the parameters of the model. The control parameters can be calibrated using experimental data or micro-models to simulate real behavior [Madan, (1996)]. Thus, for analyses where the emphasis is on evaluating the overall structural response, macro-models can be substituted for micro models without substantial loss in accuracy and with significant gains in computational efficiency.

A hysteretic macro-model of the force-deformation behavior of masonry infill panels is proposed. The report presents the development of the hysteretic macro-model. An available "equivalent strut model" for masonry infilled frames was adapted for estimating the control parameters of the proposed hysteretic rules. The methodology for calibrating the hysteretic model parameters is described. The suitability of the model for implementation in time-history analyses of framed structures is assessed. The developed model is implemented in the non-linear structural analysis program IDARC2D Ver 4.0 for reinforced concrete structures. The model is used in a simulation study of the force-deformation response of a ductile steel frame infilled with brick masonry which was tested under severe cyclic loading. The potential contribution of masonry infills to the response of a lightly reinforced concrete framed building is evaluated for severe ground motions.

SECTION 2
STATE OF THE ART OF MODELING MASONRY INFILLS FOR STRUCTURAL ANALYSIS

A review of the literature indicates that there is a need for a generalized macro-model of the hysteretic force-deformation behavior of masonry infills suitable for implementation in time-history analysis of large building structures containing such infills. The resolutions of the NCEER Workshop on Seismic Response of Masonry Infills. [Abrams et.al (1994)] indicated the need for simplified models for use (i) in engineering design office and (ii) in advanced yet cost-efficient non-linear analyses. While a number of finite element models have been developed and used to predict the response of masonry infilled frames [Dhanasekar and Page (1986), Mosalam (1996), Shing et.al (1992)], such micro-modeling approaches are too cumbersome and time-consuming for the purposes of analyzing large or complex structures with a number of masonry infill panels.

An efficient yet accurate macro-modeling approach which accounts for the various factors that govern the infilled frame behavior is required. Holmes (1961) proposed replacing the infill by an equivalent pin-jointed diagonal strut of the same material with a width 1/3 of the infill diagonal length. Stafford Smith (1966) and Stafford Smith and Carter (1969) proposed a theoretical relation between the width of the diagonal strut and the infill-frame stiffness parameter λh. Mainstone (1971, 1974) provided empirical formulations in terms of λh for the same relation. Gergely et.al (1988) proposed a model based on equivalent strut approach for estimating the stiffness, strength and ductility of masonry infill panels. The infill panel is modeled as an equivalent compression strut with bilinear elastic-plastic behavior. A multi-strut model known as the compression-only six struts model was also investigated by Chrysotomou and Gergely et.al (1992). In this model, the in-plane load resisting mechanism is assumed to consist of three parallel struts (one diagonal and two off-diagonal struts) acting simultaneously in compression. The load carrying capacity of the diagonal strut may be reduced significantly due to crushing at the corners of the infills. At this point the load is transferred to the off-diagonal struts which transmit the forces directly to the frame members away from the corners. The multi-strut model can account for the post-crushing behavior of the infill panel and the formation of plastic hinges in the frame members. However, the model has several limitations, the foremost of which is that it cannot effectively model the force transfer and slip along the frame-panel interfaces [Gergely et.al (1994)]. A simplified model based on the equivalent strut approach that accounts for slip along the frame-panel interface was recently suggested by Mosalam (1996). The model uses empirically determined correction factors to determine the effective strut dimensions.

Mander et.al (1993) reported the results of cyclic pseudo-dynamic tests performed on masonry infilled frame subassemblages intended to provide a basis for developing hysteretic rules for this class of structural elements. The report presents the observed strength and deformation limit states as well the hysteretic behavior characteristics such as strength and stiffness degradation due to repeated load reversals. The load resisting mechanisms and failure modes of the masonry infill panels are also addressed. The load resisting mechanism of infilled frames is idealized as a combination of a moment resisting frame system formed by the frame and a pin-jointed truss system formed by the infill panel. The report summarizes the important in-plane failure modes of masonry infilled frames which include (a) Tension failure of the tension column due to overturning moments, (b) Flexural or shear failure of the columns, (c) Compression failure of the diagonal strut, (d) Diagonal tension cracking of the panel, and (e) Sliding shear failure of the masonry along horizontal mortar beds. Engineering formulations are provided for capacity values corresponding to the studied failure modes for the purposes of design. The suitability of the equivalent strut approach for design of masonry infills is discussed. A computational model of the hysteretic in-plane force-deformation behavior masonry infilled frames based on the multi-strut approach was proposed by Mander et.al (1994). The infill panel was modeled in the non-linear analysis program DRAIN-2DX [Prakash et.al 1992)] as a combination of three non-parallel struts (one diagonal and two off-diagonal) in each direction of loading. The force-deformation behavior of the struts was governed by generalized hysteretic rules available in the program. The proposed computational model provides a rational approach for complete non-linear analysis of masonry infilled frames. However, the analysis requires determination of the hysteretic rule parameters from theoretical or empirical models of the infill panel.

Recently, Saneinejad and Hobbs (1995) developed a method based on the equivalent diagonal strut approach for the analysis and design of steel frames with concrete or masonry infilling walls subjected to in-plane forces. The method takes into account the elastic and plastic behavior of infilled frames considering the limited ductility of infill materials. The method provides a rational basis for predicting the lateral strength and stiffness of infilled frames as well as the infill diagonal cracking load. Various governing factors such as the infill aspect ratio, the shear stresses at the infill-frame interface and relative beam and column strengths are accounted for in this development. However, the formulation furnishes only extreme or boundary values for design purposes.

In order to perform a step-by-step force-displacement response analysis or dynamic time-history analysis of large buildings with masonry infilled frames, a continuous force-deformation model for masonry infill panels is required. Such a model is suggested herein. The present formulation provides an integrated macro-model of the force-deformation hysteresis of masonry infilled frames. The theoretical model proposed by Saneinejad and Hobbs (1995) was used to develop formulations for estimating the control parameters of the proposed hysteretic model. A brief description of the model is provided in the following section. Details of the model are presented in the original reference.

SECTION 3
ANALYTICAL MODELING OF MASONRY INFILL PANEL

3.1 Equivalent Strut Model

The proposed analytical development assumes that the contribution of the masonry infill panel (Figure 3.1a) to the response of the infilled frame can be modeled by "replacing the panel" by a system of two diagonal masonry compression struts (Figure 3.1b). The stress-strain relationship for masonry in compression can be idealized as an increasing polynomial function [Mander et.al (1988)] until the peak stress (f'_m) is reached for a given strain. For higher strains, the stress is assumed to drop with increasing strains to a small fraction of the peak value where-after the stress remains almost constant at this value. The assumed constitutive model for the masonry struts is shown in Figure 3-2a. Since the tensile strength of masonry is negligible, the individual masonry struts are considered to be ineffective in tension. However, the combination of both diagonal struts provides a lateral load resisting mechanism for positive as well as negative directions of loading.

The lateral force-deformation relationship for the structural masonry infill panel is a smooth curve bounded by a bilinear strength envelope with an initial elastic stiffness till the yield force V_y and there on a post-yield degraded stiffness until the maximum force V_m. The corresponding lateral displacement values are denoted as u_y and u_m respectively. The monotonic lateral force-deformation relationship assumed for the system of diagonal compression struts is shown in Figure 3.2b. The analytical formulations for the envelope parameters were developed on the basis of the assumed masonry constitutive model and the aforementioned theoretical model for infilled masonry frames (Saneinejad et.al, 1995). The formulations and underlying theory are briefly summarized herein.

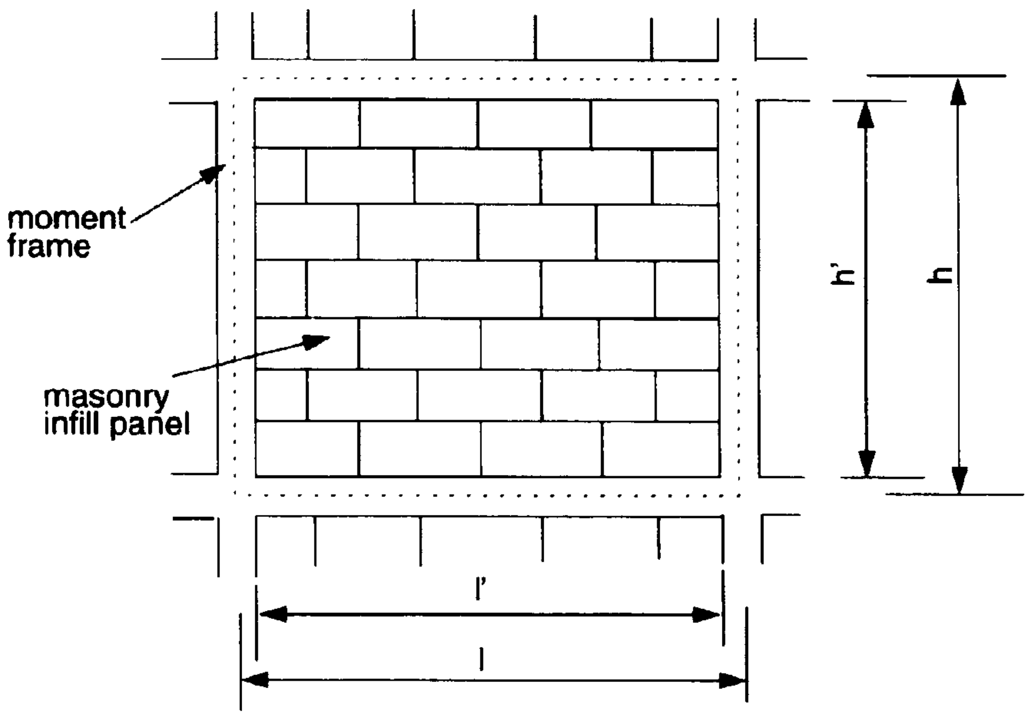


FIGURE 3.1a: Masonry Infill Frame Subassemblage

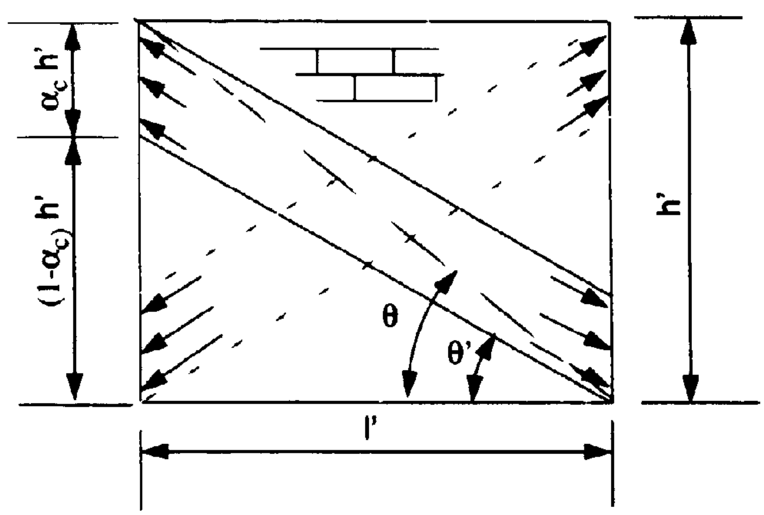


FIGURE 3.1b: Masonry Infill Panel

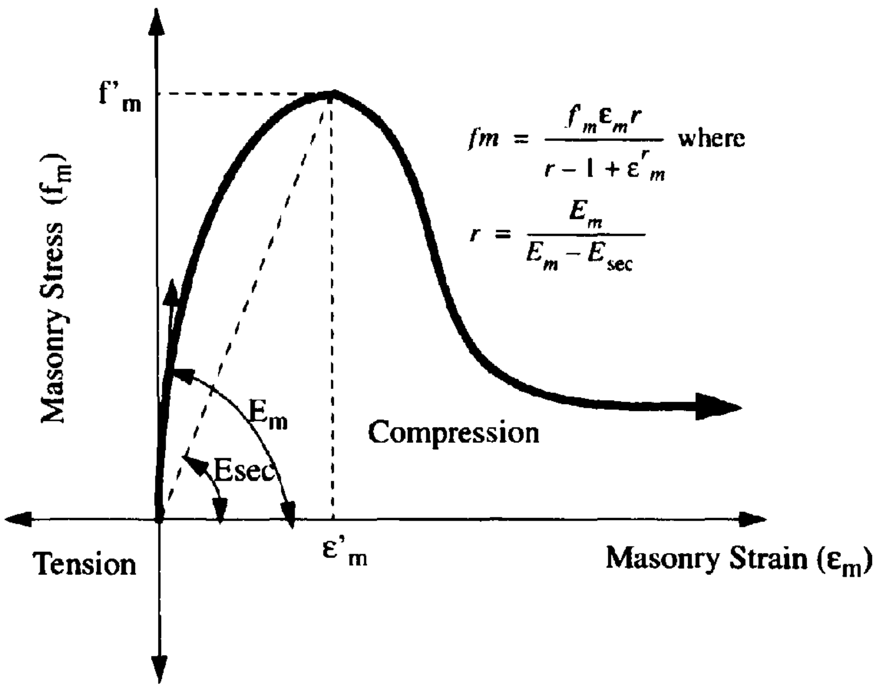


FIGURE 3.2a: Constitutive Model For Masonry

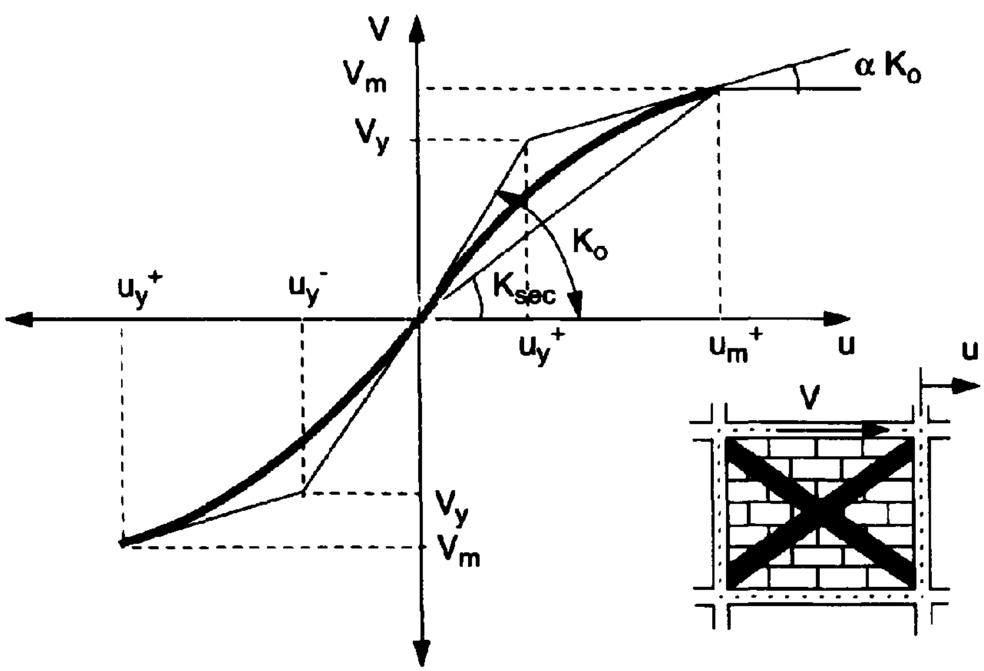


FIGURE 3.2b: Strength Envelope for Masonry Infill Panel

Considering the infilled masonry fram shown in Figure 3.1a, the maximum lateral force V_m and corresponding displacement u_m in the infill masonry panel [Saneinejad et.al (1995)] are calculated as:

$$
\begin{aligned}
V_m^+ (V_m^-) \; &\leq A_d f_m' \cos\theta \\
&\leq \frac{v t l'}{(1 - 0.45 \tan\theta') \cos\theta} \leq \frac{0.83(MPa) t l'}{\cos\theta}
\end{aligned}
\tag{3.1}
$$

$$
u_m^+ (u_m^-) \; = \frac{\varepsilon_m' L_d}{\cos\theta} \tag{3.2}
$$

in which t is the thickness or out-of-plane dimension of the infill panel, f_m' is the masonry prism strength, ε_m' is the corresponding strain, v is the basic shear strength or cohesion of masonry and A_d and L_d are the area and length of the equivalent diagonal struts respectively which are obtained as follows [Saneinejad et.al (1995)]:

$$
A_d = (1 - \alpha_c) \alpha_c t h' \frac{\sigma_c}{f_c} + \alpha_b t l' \frac{\tau_b}{f_c} \leq \frac{0.5 t h' \frac{f_a}{f_c}}{\cos\theta} \tag{3.3}
$$

$$
L_d = \sqrt{(1 - \alpha_c)^2 h'^2 + l'^2} \tag{3.4}
$$

where the quantities $\alpha_c, \alpha_b, \sigma_c, \tau_b, f_a$ and f_c depend on the geometric and material properties of the frame and infill panel. For the sake of completion, the relationships needed to compute these quantities are presented in Appendix A. A detailed description of the theoretical formulations is presented in Saneinejad et.al (1995).

The monotonic lateral force displacement curve is completely defined by the maximum force V_m, corresponding displacement u_m, the initial stiffness K_o and the ratio α of the post-yield to initial stiffness. The initial stiffness K_o of the infill masonry panel may be estimated using the following proposed formula:

$$K_o = 2\left(\frac{V_m}{u_m}\right) \qquad (3.5)$$

The lateral yield force V_y and displacement u_y of the infill panel may be calculated from geometry of the curve as follows:

$$V_y^+ (V_y^-) = \frac{V_m - \alpha K_0 u_m}{(1-\alpha)} \qquad (3.6)$$

$$u_y^+ (u_y^-) = \frac{V_m - \alpha K_0 u_m}{K_0 (1-\alpha)} \qquad (3.7)$$

A value of 0.1 is suggested for the ratio α of post-yield (degraded) stiffness to the initial stiffness K_o. The monotonic force deformation model presented in this section was extended to account for hysteretic behavior due to cyclic load reversals as well as strain softening effects.

SECTION 4
PROPOSED HYSTERETIC MODEL

A smooth hysteretic model is proposed for the structural masonry panel. The model takes into account hysteretic effects characteristic of structural masonry elements subjected to repeated loading reversals such as stiffness degradation, strength deterioration and 'pinching'. The control parameters of the model are derived using the theoretical formulations of the "equivalent strut model" presented in Section 3.

4.1 Time Independent Smooth Hysteresis Model

The development of the model is based on the well-known Bouc-Wen model for hysteretic behavior [Bouc (1967), Baber and Wen (1981)]. The model furnishes a smooth hysteretic force displacement relationship between force F and displacement u (Figure 4.1) which may be expressed as:

$$V_i = V_y\left[\alpha\mu_i + (1-\alpha)\, Z_i\right] \tag{4.1}$$

where μ_i is the normalized displacement calculated as

$$\mu_i = \frac{u_i}{u_y}, \tag{4.2}$$

the subscript i is used to refer to the instantaneous values, subscript y denotes yield values, α is the ratio of the post-yield to initial elastic stiffness and Z the hysteretic component determined from the following equations:

$$\dot{Z}_i = \dot{\mu}_i\left\{A - |Z_i|^n\left[\beta\,\mathrm{sgn}(\dot{\mu}_i Z) + \gamma\right]\right\} \tag{4.3}$$

$$\begin{aligned} \mathrm{sgn}(\ ) &= 1 \quad \textit{if}\ (\ ) > 0 \\ &= -1 \quad \textit{if}\ (\ ) < 0 \end{aligned}$$

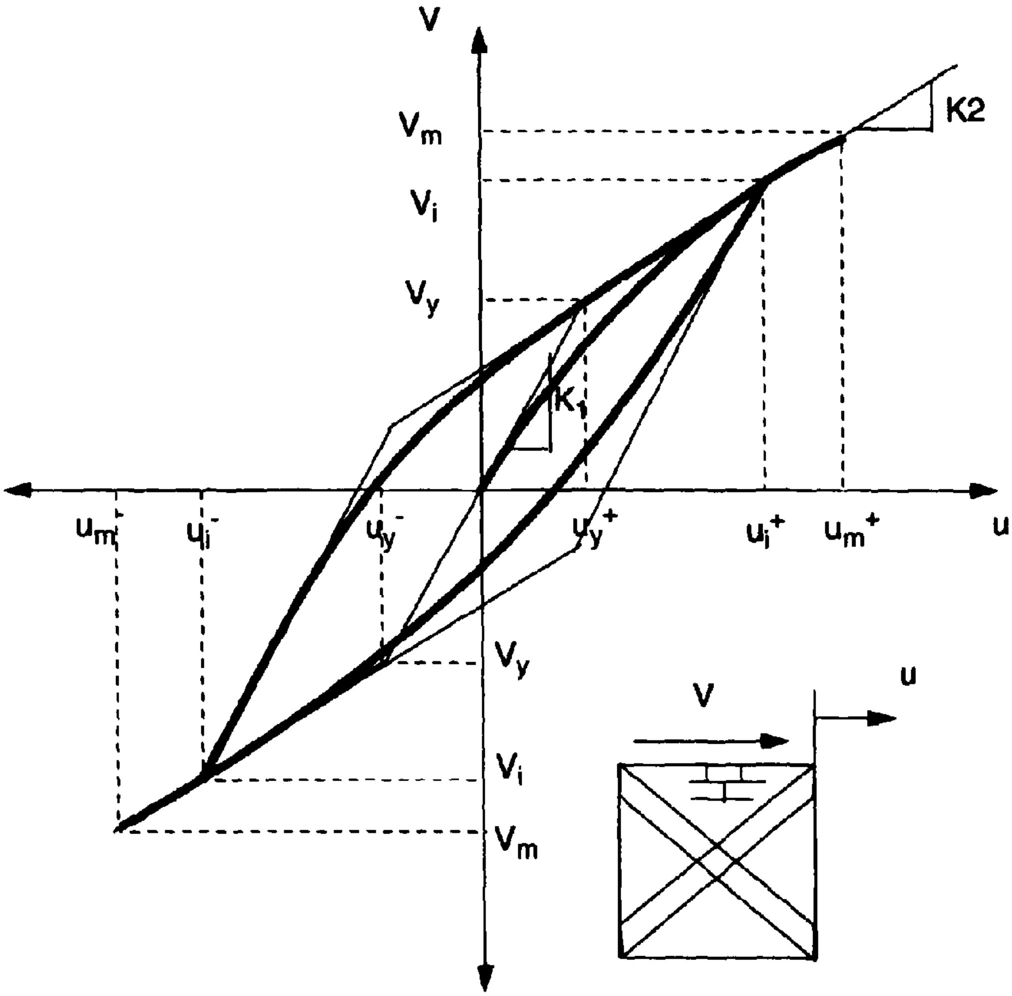


FIGURE 4.1: Bouc-Wen Model for Smooth Hysteresis

Eliminating the time differential dt, and noting that $\mathrm{sgn}(\dot{\mu}) = \mathrm{sgn}(d\mu)$, Equation 4.3 may be rewritten for quasi-statically loaded systems as:

$$dZ_i = d\mu_i \left\{ A - |Z_i|^n \left[\beta \,\mathrm{sgn}(d\mu_i Z) + \gamma \right] \right\} \tag{4.4}$$

In these equations A, β and γ are constants that control the shape of the generated hysteretic loops and n controls the rate of transition from the elastic to yielded state (Lobo, 1994). A large n approximates a bilinear hysteretic curve and lower values trace a smoother transition. Hysteretic shapes with variations in the magnitudes of β and γ can be found in Fang 1991. To satisfy viscoplastic conditions the present development assumes that $A = \beta + \gamma = 1.0$

4.2 Stiffness Decay

An important hysteretic property of structural masonry panels or yielding systems in general is the loss of stiffness due to deformation beyond yield (Figure 4.3). The stiffness deterioration due to plastic excursions of the infill masonry panel is expressed as a function of attained ductility (Reinhorn et.al, 1993) in the present model. The stiffness decay is incorporated directly in the hysteretic model by including a control parameter η. The differential equation for the hysteretic parameter Z (Equation 4.4) may be modified to generate stiffness deterioration as follows:

$$dZ_i = d\mu_i \frac{\left\{ A - |Z_i|^n \left[\beta \,\mathrm{sgn}(d\mu_i Z) + \gamma \right] \right\}}{\eta_i} \tag{4.5}$$

where

$$\eta_i = 1.0 + s_k \left(\frac{\mu_{max}^p + \mu_i}{2} \right); \tag{4.6}$$

$$DI = \frac{\mu_{max} - 1}{\mu_c - 1} \cdot \frac{1}{\left(1 - \frac{s_{p1} \int dE_h}{E_{hy}}\right)^{s_{p2}}} \tag{4.9}$$

in which μ_{max} is maximum attained ductility in the response history, μ_c is the ductility capacity of the masonry infill panel, the parameters s_{p1} and s_{p2} control the rate of strength deterioration, $\int dE_h$ represents the cyclic energy dissipated before the start of the current reloading cycle and E_{hy} is an energy parameter calculated as:

$$E_{hy} = 4\, V_y\, u_y\, (\mu_c - 1) \tag{4.10}$$

Thus, the damage index DI may also be expressed as:

$$DI = \frac{\mu_{max} - 1}{\mu_c - 1} \cdot \frac{1}{\left[1 - 0.25 s_{p1} \int \left(\frac{V}{V_y}\right) \frac{d\mu}{(\mu_c - 1)}\right]^{s_{p2}}} \tag{4.11}$$

The proposed damage index can reflect the cumulative effect of softening due to large monotonic inelastic excursions as well as strength degradation due to repeated cycling at moderate or small inelastic deformations.

4.4 Cracking Slip Model

'Pinching' of hysteresis loops due to opening and closing of cracks is a commonly observed phenomenon in concrete and masonry structural systems subjected to cyclic loading. Baber and Noori (1984) proposed a general degradation model to obtain the solution of the equations of motion of single degree of freedom degrading pinching systems. The model implements a smooth degrading element developed by Bouc and modified by Barber and Wen (1981) in series with a time-dependent slip-lock element (a non-linear hardening spring). A rate-dependent differential equation was proposed (Baber and Noori, 1984) relating the velocity contribution due to the slip-lock element with the hysteretic parameter Z, which was solved simultaneously with the equations of motion for the single degree-of-freedom system to obtain the response of dynamical degrading pinching systems.

The concept of slip-lock element proposed by Baber and Noori (1984) has been adapted in this study to formulate a generalized hysteretic rule for degrading pinching elements. The hysteretic rule is rate-independent and defines the force deformation response of the pinching element for any arbitrary displacement history independent of the system differential equations. The present formulation incorporates a slip-lock element in series with the smooth degrading element (Figure 4.2a) to develop a hysteretic model for the pinching response of masonry infill panels. Thus, intuitively, the normalized displacement of masonry infill panel μ is the sum of two components which are the normalized displacement of the smooth degrading element μ_1 and that of the slip-lock element μ_2 respectively. The relationship may be expressed in the incremental form as:

$$d\mu = d\mu_1 + d\mu_2 \tag{4.12}$$

in which $d\mu_1$ and $d\mu_2$ are the incremental normalized displacements of the the smooth degrading element and the slip-lock element respectively.

The smooth degrading element is based on the Bouc-Wen model discussed in earlier sections. Thus, the hysteretic parameter Z may be related to the displacement contribution μ_1 of the smooth degrading element by the Bouc-Wen model. Rewriting Equation 4.5, the following relationship may be obtained:

$$dZ = d\mu_1 \frac{\{A - |Z|^n \{\beta \operatorname{sgn}(d\mu_1 Z) + \gamma\}\}}{\eta} \tag{4.13}$$

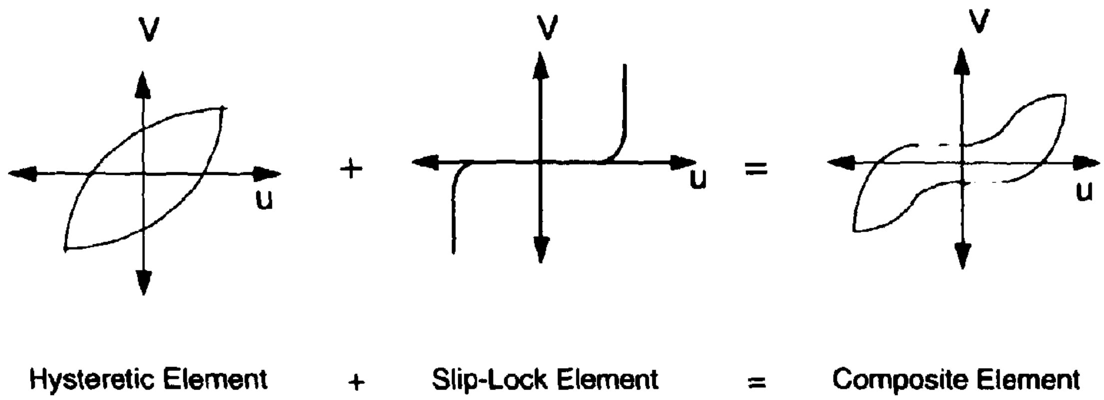


FIGURE 4.2a: Smooth Hysteretic Element in Series with Slip-Lock Element

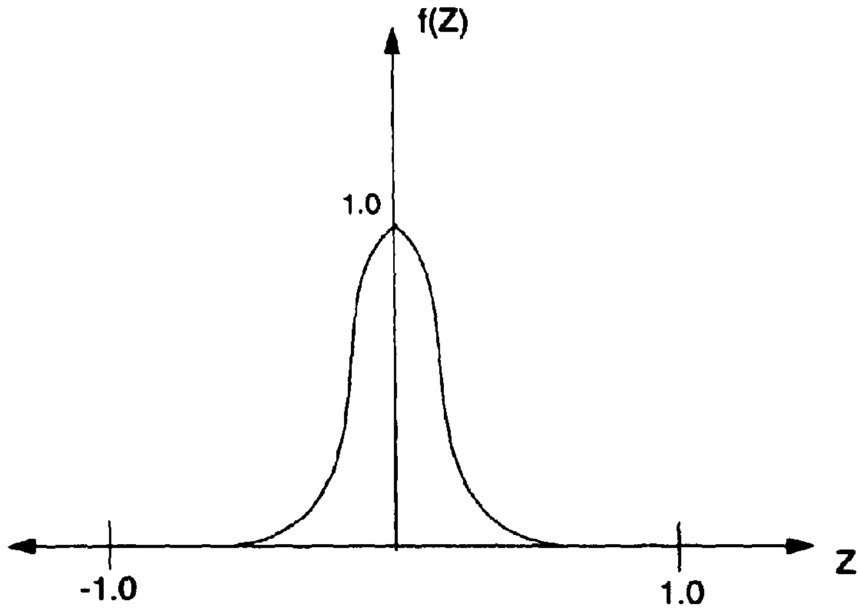


FIGURE 4.2b: Slip Lock Function

The following relationship is proposed for displacement component μ_2 due to the slip-lock element:

$$d\mu_2 = a f(Z) dZ \tag{4.14}$$

in which a is a constant defined as the slip length and the function $f(Z)$ is assumed as

$$f(Z) = \exp\left(-\frac{\{Z-\bar{Z}\}^2}{Z_s^2}\right); \quad -1 \le Z, \bar{Z} \le 1 \tag{4.15}$$

where, $\bar{Z}$ is the value of Z at which $f(Z)$ reaches its maxima i.e. the value of Z at which the maximum slip occurs. Z_s is the range of Z about $Z = \bar{Z}$ in which the slip occurs and thus controls the sharpness of the slip. The variation of $f(Z)$ for $\bar{Z} = 0$ is shown in Figure 4.2b. Upon substitution of Equations 4.12 and 4.14 into Equation 4.13, assuming that $\text{sgn}(d\mu_1) = \text{sgn}(d\mu)$, rearrangement of terms yields:

$$\frac{dZ}{d\mu} = \frac{\{A - |Z|^n\{\beta \,\text{sgn}(d\mu.Z) + \gamma\}\}}{\eta\left[1 + a\exp\left(-\frac{\{Z-\bar{Z}\}^2}{Z_s^2}\right)\left(A - |Z|^n\{\beta\,\text{sgn}(d\mu.Z) + \gamma\}\right)\right]} \tag{4.16}$$

Equations 4.16 and 4.1 together with Equation 4.7 furnish a modified Bouc-Wen's model (Figure 4.3) for hysteretic pinching elements subjected to dynamic or quasi-static loading. In this development, the slip length a was assumed to be a function of the attained ductility. The relationship may be expressed as:

$$a = A_s(\mu' - 1) \tag{4.17}$$

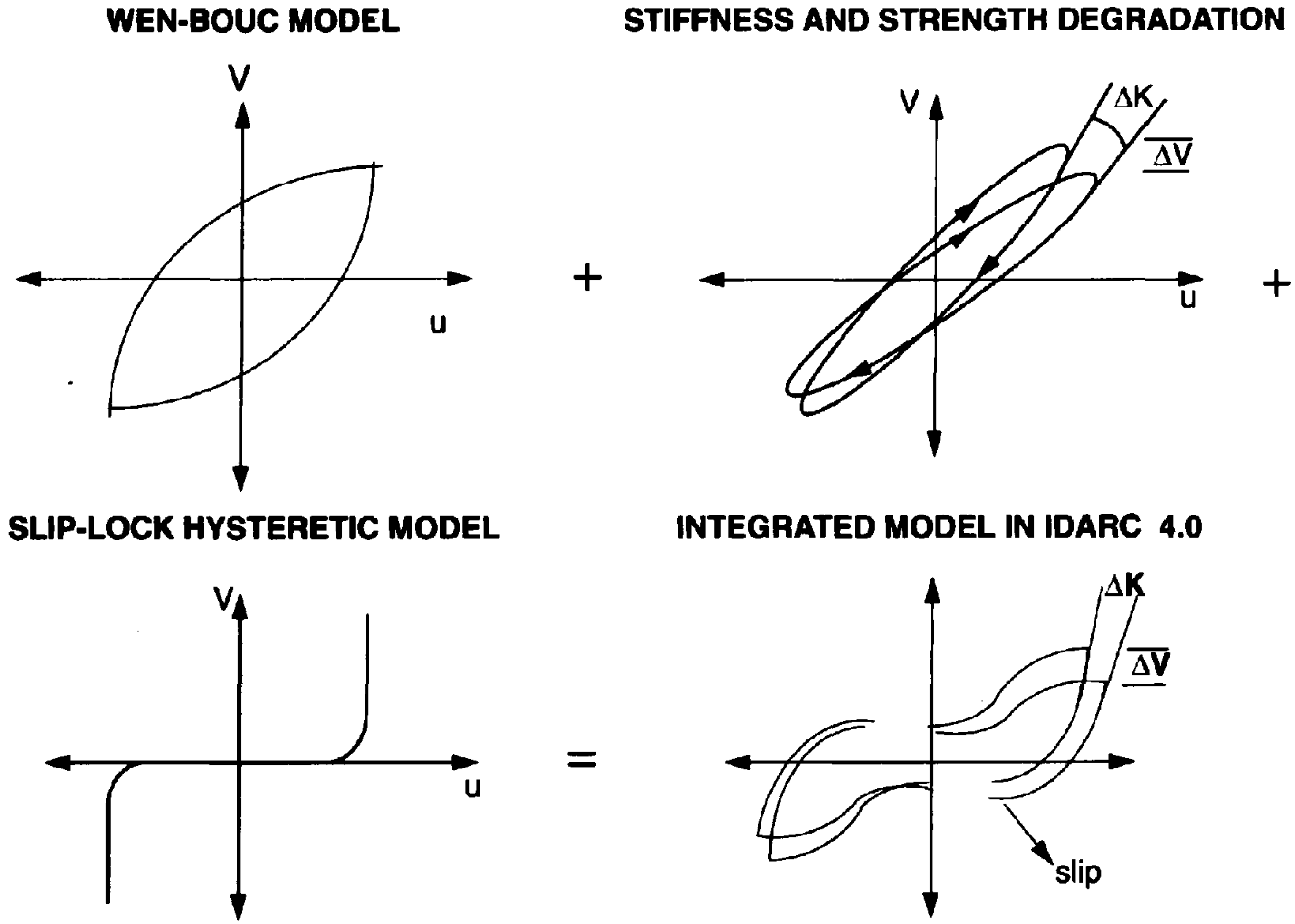
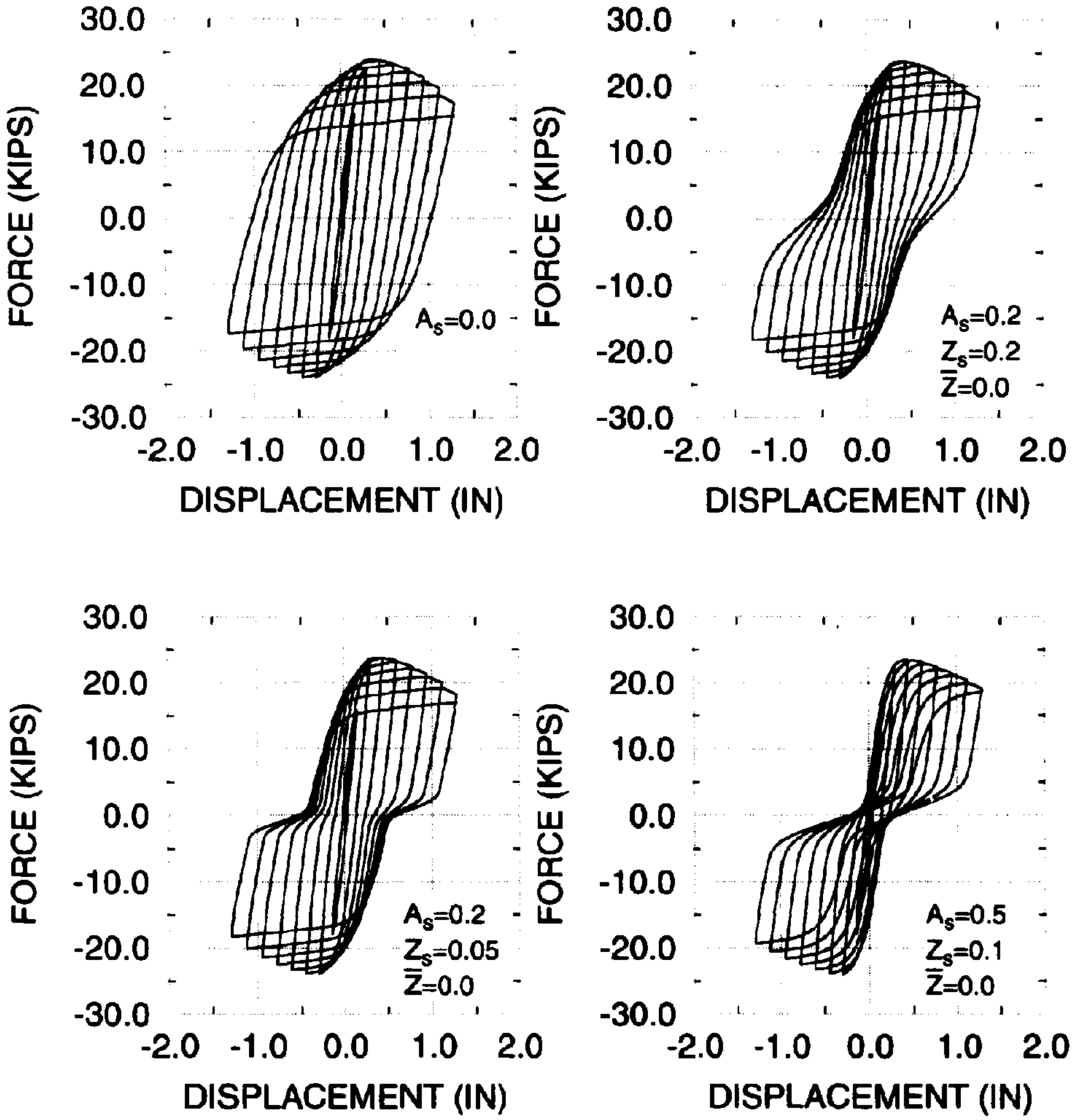


FIGURE 4.3: Integrated Hysteretic Model for Degrading Pinching Elements

where A_s is a control parameter to vary slip length and may be linked to the size of crack openings or reinforcement slip or both [Lobo et.al, (1994)], μ' is the normalized displacement attained at the load reversal prior to the current unloading or reloading cycle. The effect of varying the control parameters of the slip-lock element parameters on the pinching of hysteresis loops is illustrated in Figure 4.4. In this study, the parameter Z_s which controls the sharpness of the slip is assumed to be independent of response history. The slip occurs in the range of Z which is equal to Z_s and is symmetric about $Z = \bar{Z}$.



Constant Parameters		
$A = 1.0$	$\alpha = 0.01$	$s_k = 0.1$
$\beta = 0.1$	$V_y = 25$ Kips	$s_{p1} = 0.8$
$\gamma = 0.9$	K_0=125 K/in	s_{p2} =1.0
$n = 2$		μ_c=25

FIGURE 4.4: Influence of Varying Slip-Lock Parameters

SECTION 5
IMPLEMENTATION OF MODEL FOR NUMERICAL ANALYSIS

The proposed hysteretic model for masonry infill panels was implemented in IDARC 2D for the analysis of structural frames with infill masonry panels. IDARC 2D is a computer based analytical tool developed at the University of Buffalo (Park, Reinhorn and Kunnath, 1987) for the inelastic analysis and damage evaluation of flat slab reinforced concrete buildings and their components under combined dynamic, static and quasi-static loading. The program performs two-dimensional analysis of 3D structural systems in which a set of individual vertical column lines with groups of column lines in the same plane forming a frame. Each group of column lines which is out of the plane of previous frame, but lies in-plane with an axis parallel to the existing frame, generates a new frame. The frames parallel to the loading direction are interconnected by transverse frames to permit flexural torsional coupling (Kunnath, Reinhorn and Lobo, 1992). A number of element types are available to model a wide range of structural systems including beam column elements, shear walls, inelastic axial elements, transverse beams, discrete spring elements and various types of damping devices. Figure 5.1 illustrates a typical structural model with various components for analysis using IDARC.

The hysteretic pinching element developed in this study for masonry infill panels was incorporated in the computer program IDARC 2D version 4.0. The infill panel element may be specified in any bay of the principal frames i.e. between adjacent floors and neighboring column lines. Since the axial deformations of the beams and floor slabs are ignored in the analysis (beams and floor slabs are axially rigid, the horizontal degrees of freedom)of the floor level joints are slaved to give a unique horizontal displacement for a floor. The lateral force in the infill panel element is assumed to act on horizontal degree of freedom of the upper and lower confining floors. The element can be used for quasi-static cyclic or monotonic analysis in displacement or force control as well as dynamic time history analysis under earthquake excitations. The numerical solution schemes used in the implementation of the element are discussed in the present section.

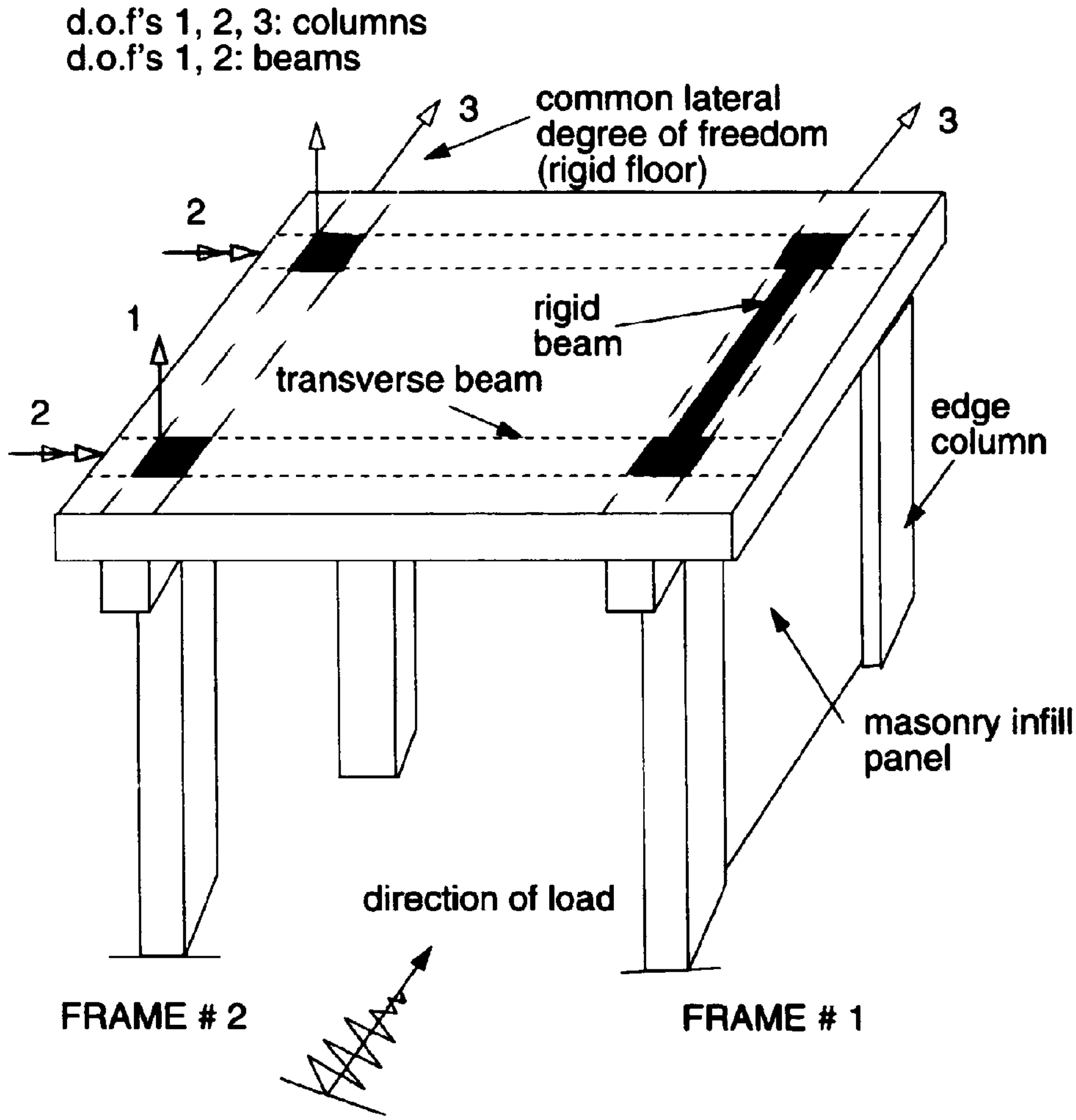


FIGURE 5.1: Component Modeling in IDARC 4.0

5.1 Numerical Solution of Hysteretic Model

The solution of the model for the hysteretic pinching element requires integration of the differential equation governing the hysteretic parameter Z (Equation 4.18). Rewriting Equation 4.18 as,

$$Z'(\mu)=\frac{\{A-|Z|^{n}(\beta\,\mathrm{sgn}(d\mu.Z)+\gamma)\}}{\eta\left[1+a\exp\left(-\frac{\{Z-\bar{Z}\}^{2}}{Z_{s}^{2}}\right)\left(A-|Z|^{n}(\beta\,\mathrm{sgn}(d\mu.Z)+\gamma)\right)\right]} \tag{5.1}$$

assuming that $\mathrm{sgn}(d\mu)$ is known as is generally the case for most applications, the equation may be reduced to the following general form:

$$F'(x)=f(F) \tag{5.2}$$

Differential equations of the aforementioned form can be incrementally integrated using the semi-implicit Runge-Kutta method (Rosenbrook, 1964). The increment ΔF or ΔZ in this case is given as:

$$\Delta F_{k}=F_{k+1}-F_{k}=R_{1}k_{k}+R_{2}l_{k} \tag{5.3}$$

in which the subscript k refers to the kth step. The quantities k_{k} and l_{k} are determined by solving the following coupled equations:

$$k_{k}=\left[1-a_{1}\Delta x\frac{\partial f(F_{k})}{\partial F}\right]^{-1}f(F_{k})\Delta x \tag{5.4}$$

$$l_{k}=\left[1-a_{2}\Delta x\frac{\partial f(F_{k}+c_{1}k_{k})}{\partial F}\right]^{-1}f(F_{k}+c_{1}k_{k})\Delta x \tag{5.5}$$

In the foregoing equations the constants R_1, R_2, a_1, a_2, c_1 and c_2 are obtained from the solution of the following equations:

$$R_1 + R_2 = 1 \tag{5.6 a}$$

$$R_1 a_1 + R_2(a_2 + b_1) = \frac{1}{6} \tag{5.6 b}$$

$$R_1 a_1^2 + R_2\left[a_2^2 + (a_1 + a_2)b_1\right] = \frac{1}{6} \tag{5.6 c}$$

$$R_2\left(a_2 c_1 + \frac{1}{2}b_1^2\right) = \frac{1}{6} \tag{5.6 d}$$

A series of coefficients were recommended (Reinhorn et.al, 1994) to obtain a fourth order truncation error $O(\Delta t^4)$ that satisfy Equations xx which are $R_1 = 0.75; R_2 = 0.25;$ $a_1 = a_2 = 0.7886751; b_1 = -1.1547005$ and $c_1 = 0$. It should be noted that the solution of F at k+1th step requires the knowledge of F, x and sgn(dx) at the kth step.

5.2 Displacement Controlled Quasi-static Analysis

The quasi-static analysis in displacement control proceeds by increasing the displacements of specified global degrees of freedom by specified increments. The lateral deformation of the infill panel element are computed from the relative displacements of the upper and lower confining floors. The change in the hysteretic parameter Z is obtained from the value of Z at the previous step and the displacement increment using the semi-implicit Runge-Kutta integration technique discussed in the foregoing section. The lateral force increment in the infill panel element is computed from Equation 4.1 and transformed into the global force for output. The numerical integration scheme of the hysteretic pinching model is based on the assumption that the initial deformation and force (thus the hysteretic parameter Z) of the infill panel are zero. To ensure accuracy of the numerical solution, the specified interpolation interval for analysis between the applied global displacement steps should be small. If the interpolation interval is too large, the program subdivides the element displacement increments into suitably small intervals for integration. However, in this case, it may not be possible to trace the non-linear response accurately. Details of the quasi-static analysis algorithm of IDARC is available in Park et.al (1987) and Kunnath et.al (1992).

5.3 Force Controlled Quasi-static Analysis

An iterative pseudo-force method was employed for implementing force controlled quasi-static analysis of structures with masonry infill panels. The pseudo-force method has been used for non-linear dynamic analysis of shells by Stricklin et.al (1971). The pseudo-force method is an iterative procedure which proceeds by estimating the incremental forces in the elements (infill panels in this case) and considering them as incremental forces on the structure. Thus, the incremental non-linear element force vector is brought to the right hand side of the global equilibrium equation along with the load vector and treated as a pseudo-force vector to compute structural displacements. The structural displacements thus obtained are used to compute a corrected estimate of the incremental element force vector. The iterative procedure is performed at each step until the estimated element force increments agree with the computed element force increments within a specified tolerance thus implying equilibrium. For faster convergence, the equilibrium incremental element forces are taken as estimated element force increments for first iteration in the next step. It is recommended to apply the external load in small increments to ensure convergence.

5.4 Dynamic Time History Analysis

For implementation in dynamic analysis, the governing differential equation for the hysteretic parameter Z (Equation 4.18) can be written in the rate-dependent form as:

$$\dot{Z} = \dot{\mu}\frac{[A - |Z|^n(\beta\,\mathrm{sgn}(\dot{\mu}.Z) + \gamma)]}{\eta\left[1 + a\exp\left(-\frac{\{Z - \bar{Z}\}^2}{Z_s^2}\right)\left(A - |Z|^n(\beta\,\mathrm{sgn}(\dot{\mu}.Z) + \gamma)\right)\right]} \tag{5.7}$$

The foregoing equation is of the general form

$$\dot{F} = f(F, x, \dot{x}) \tag{5.8}$$

and can be integrated using the semi-implicit Runge-Kutta method (Rosenbrook,1964). A detailed description of the integration scheme is available in Reinhorn and Li et.al (1995). The rate-dependent form of the modified Bouc-Wen model presented above for hysteretic pinching elements was incorporated for dynamic time history analysis of multistory frames with masonry infill panels under earthquake loads. A step-by-step integration procedure using the Newmark-β method [Clough and Penzien et.al (1993)] is used to solve the equations of motion of the system. A non-iterative pseudo-force method with one-step correction similar to the one used for supplemental damping devices (Reinhorn and Constantinou et.al, 1995) is employed to solve for forces in the infill panel elements. Details of the solution algorithm are presented in the same reference.

SECTION 6
SIMULATION AND EXPERIMENTAL DATA

6.1 Test Method

Three infill frame sub-assemblages were tested as part of the research program to obtain experimental data on the hysteretic force deformation behavior of masonry infilled frames [Mander et.al, (1994)]. The sub-assemblages constructed from bolted steel frames and infilled with clay brick masonry were tested under in-plane quasi-static cyclic loading. Two of the specimens were retrofitted with different types of ferrocement overlays. Details of the experimental study are presented elsewhere [Mander et.al, (1994)]. For sake of completion, a brief description of the test setup and method is presented in this section. The important geometric and material properties of the infill frame sub-assemblages are included in Appendix II.

The test specimens consisted of three story steel frames with the central bay infilled with brick masonry (Figure 6.1). The outer half-bays were provided with pin-jointed diagonal braces whose stiffness was similar to that of the infill. The specimens were anchored by bolting the bottom beam to the strong floor and loaded by applying a concentrated lateral force at the top beam. Semi-rigid bolted connections with top and bottom angle seats were used to connect beams to the columns. The connections were designed to have half the strength capacity of the connecting members in order to achieve concentrated yielding in the connections thus preserving the principal members from being damaged. Thus the test setup was designed to replicate boundary conditions that produce a load resisting mechanism shown in Figure 6.2a in which plastic hinges form at beam ends and a diagonal compression strut forms in the infill panel. The purpose was to reproduce field conditions that exist in such frames during lateral earthquake loading wherein high story shears may cause the infill panel to be the critical region [Mander et.al, (1994)]. A bare frame (without infill) specimen was also tested as part of the test program for comparison with the infilled frames.

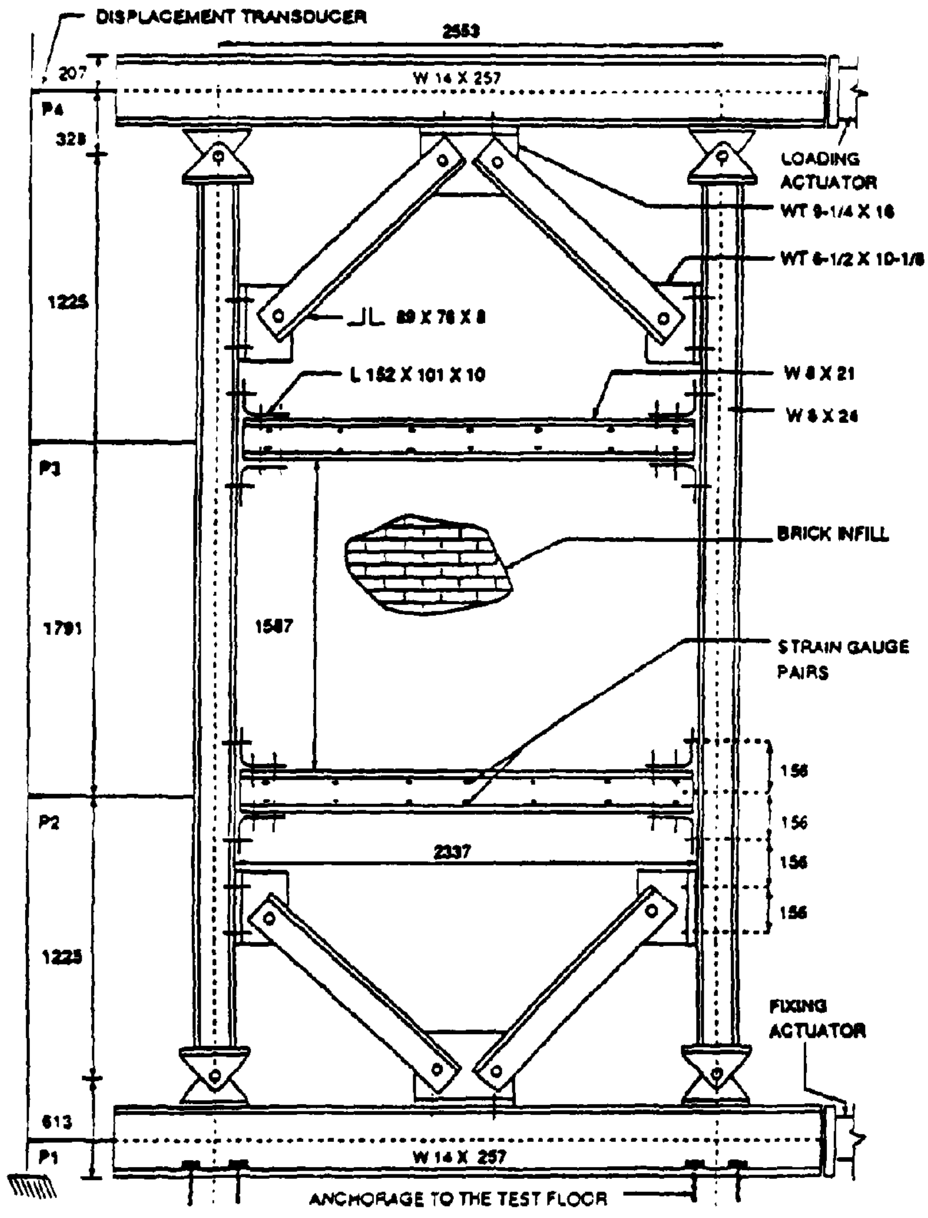


FIGURE 6.1: Infilled Frame Test Specimen

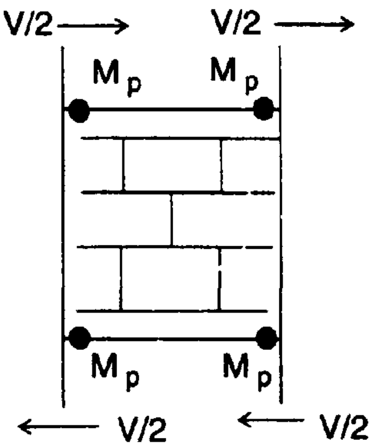


FIGURE 6.2a: Boundary Conditions of Infill Frame Subassemblage

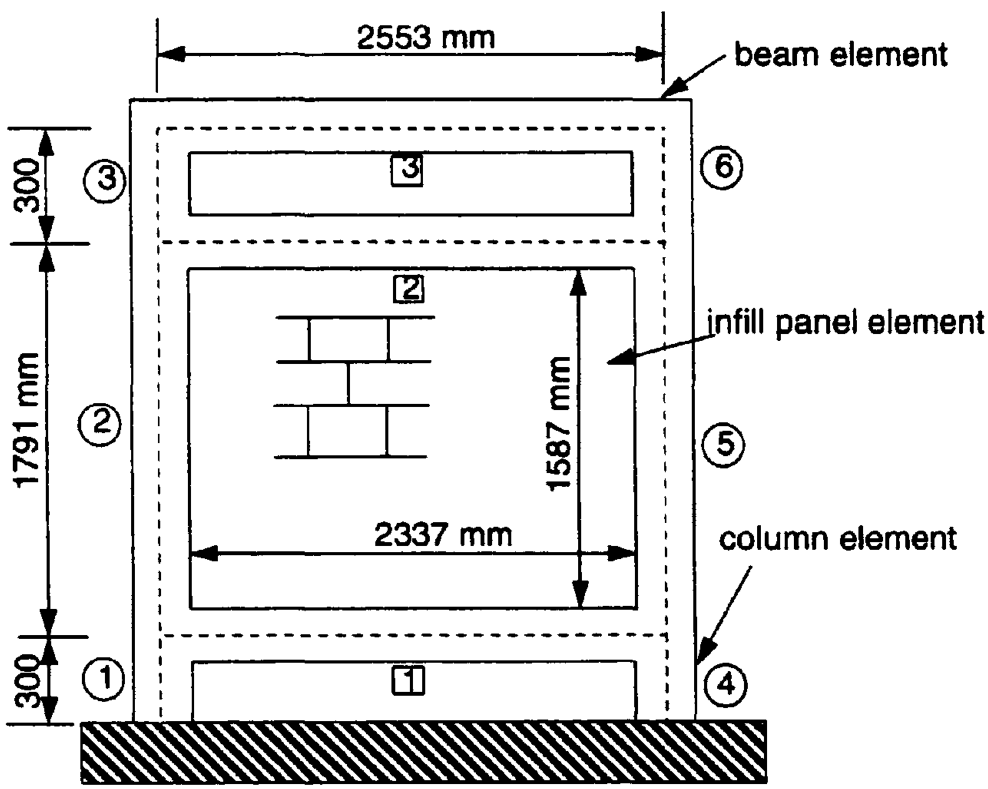


FIGURE 6.2b: Idealized Structural Model For Analysis

The test specimens were subjected to cyclic lateral load in drift control using a servo-controlled hydraulic actuator. The lateral story displacements were measured using displacement transducers. A sinusoidal drift history with the sine wave frequency of 0.01 Hz and increasing amplitudes was prescribed for the tests. Two loading cycles were applied for each displacement amplitude. The interstory drift of central (infilled) bay was specified as the control displacement.

6.2 Simulation

A simulation study was performed to evaluate the response of the masonry infilled ductile steel frame with semi-rigid connections which was tested under cyclic loading. The analytical evaluation was done using IDARC2D Ver 4.0. It may be noted that the simulation was performed to verify the analytical approach. The idealized structural model used for the analysis of the test specimens is shown in Figure 6.2b. Quasi-static cyclic analysis of the idealized structure was performed in displacement control to generate the experimental hysteresis loops of the bare and infilled frame specimens. The displacement of the second story was specified as the control variable. A cyclic displacement history was input with same amplitudes as prescribed in the test. A typical prescribed displacement history is shown in Figure 6.3.

The experimental force-deformation response of the bare frame test specimen was simulated by analyzing structural model without the infill panel element. An elastic-plastic moment curvature envelope was specified for the beams and columns in the frame. Figure 6.4 shows the comparison of the experimental and simulated response of the bare frame specimen. Simulations for the masonry infilled frame test specimens were performed by specifying an infill panel element in an identical frame model. The moment-curvature parameters of the frame elements (beams and columns) were specified the same as those used for simulating the response of the bare frame specimen.

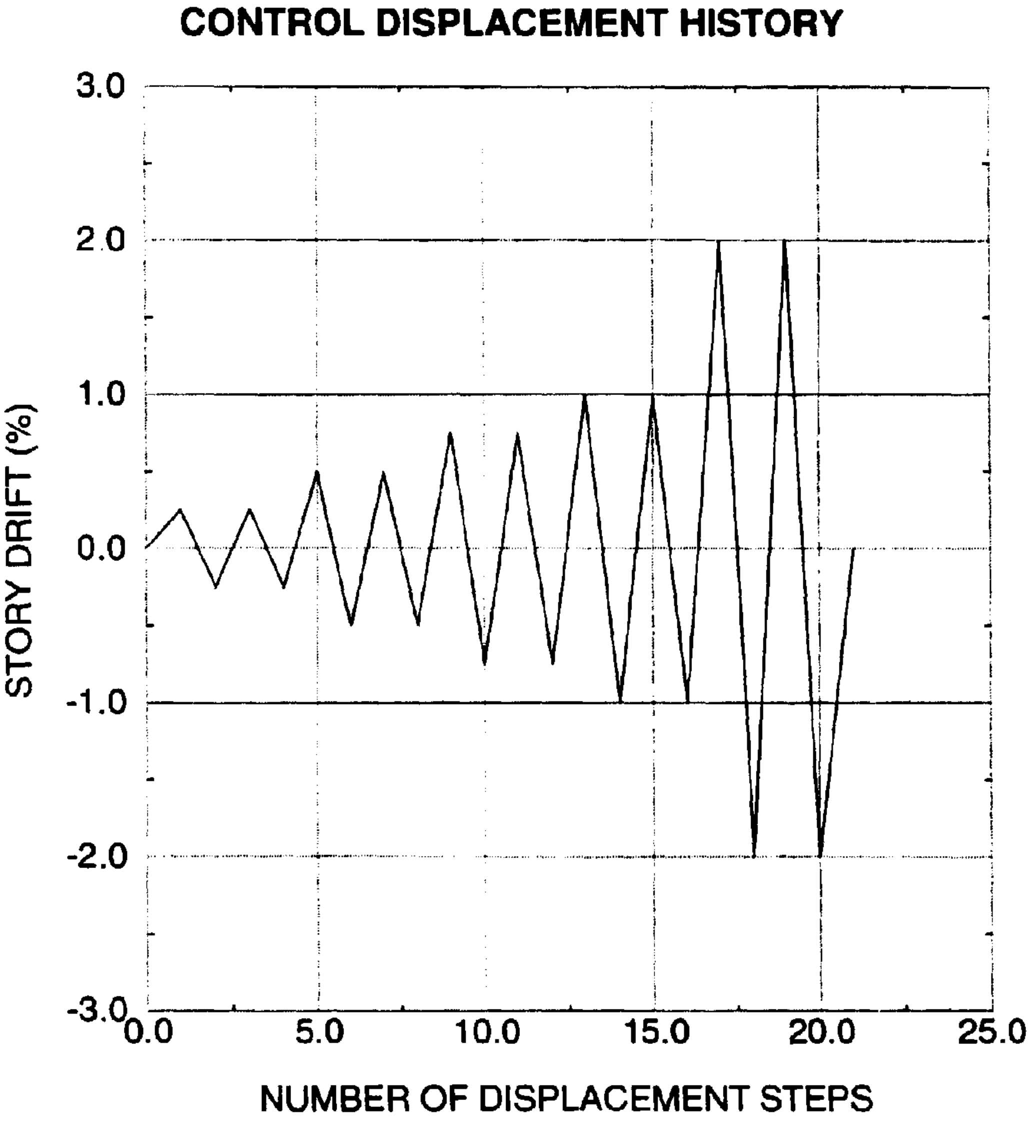


FIGURE 6.3: Typical Prescibed Drift History for Simulation

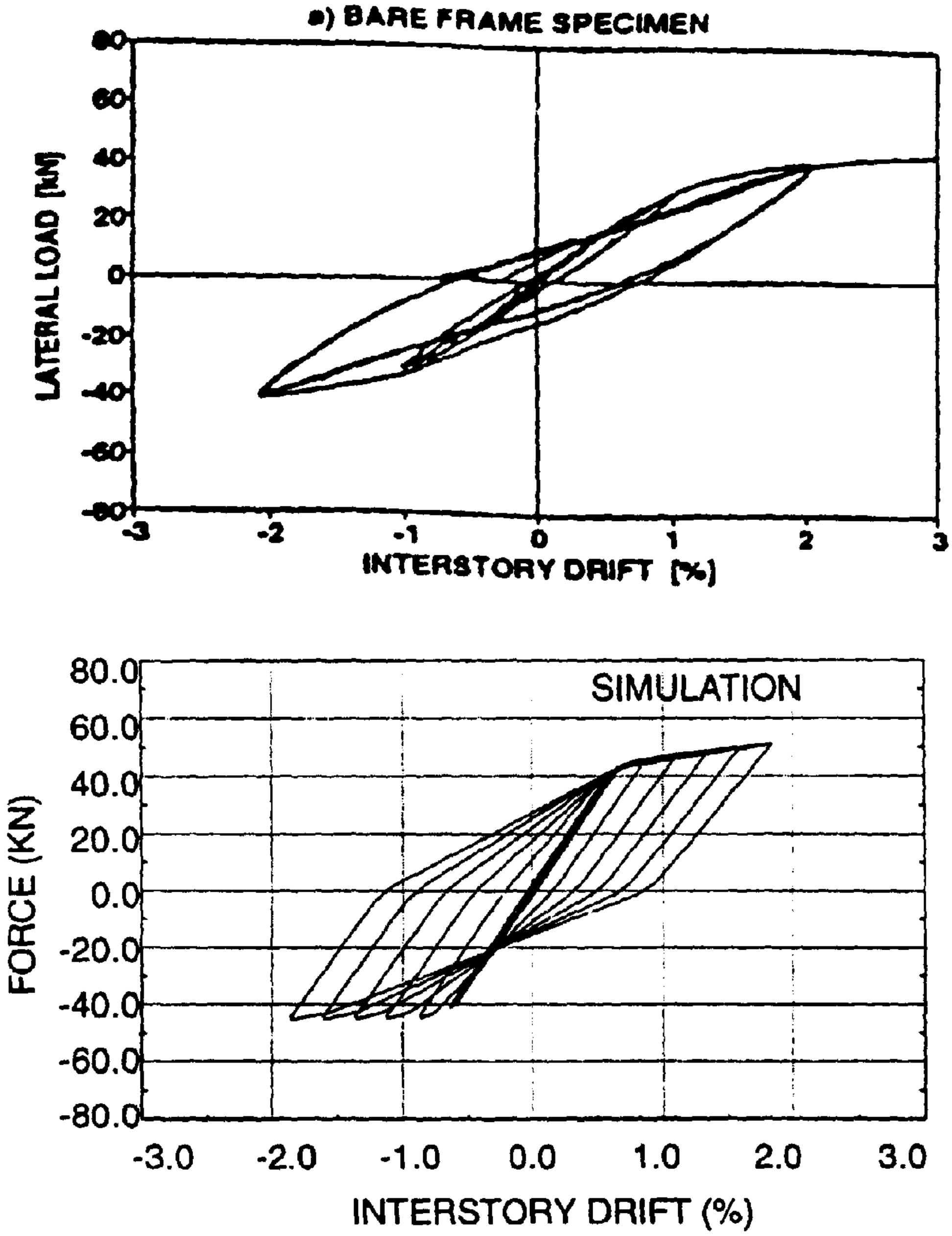


FIGURE 6.4: Comparison of Experimental vs. Simulated Force Displacement Response of Tested Bare Frame Specimen

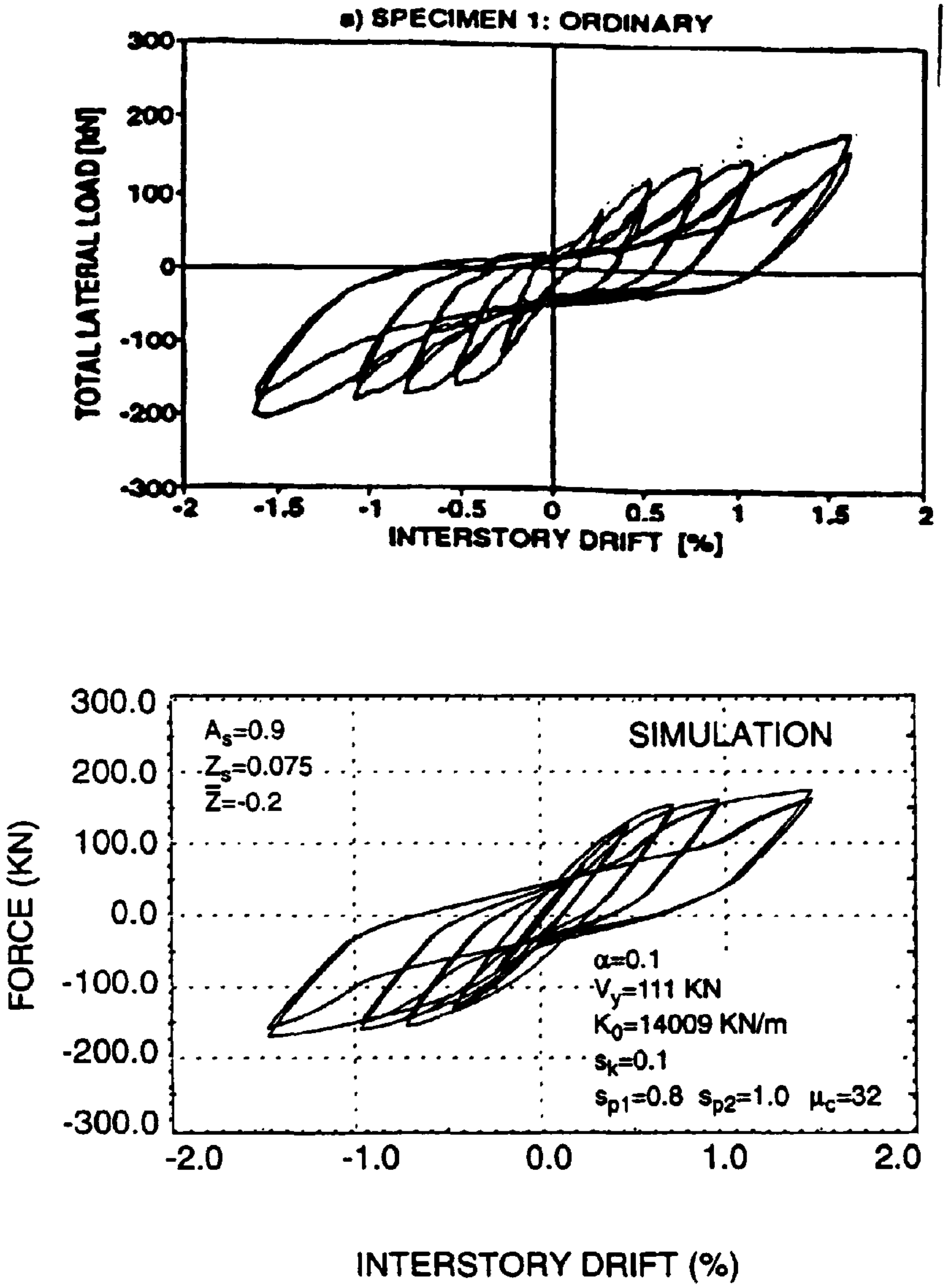


FIGURE 6.5: Comparison of Experimental vs. Simulated Force Displacement Response of Tested Masonry Infill Frame Specimen 1

The hysteretic model parameters for the infill panel were calculated using the analytical formulations presented in the earlier sections. Sample calculations for the infill panel in Test Specimen 1 are included in Appendix A. The comparison of the simulated and experimental response for infilled frame Test Specimen 1 is presented in Figure 6.5. The figure shows the lateral force vs. interstory drift hysteresis loops obtained from the experiment and simulation. The hysteretic model parameters prescribed for the infill panel element are also included in the figure. Similar comparison for infilled frame Test Specimen 2 is illustrated in Figure 6.6.

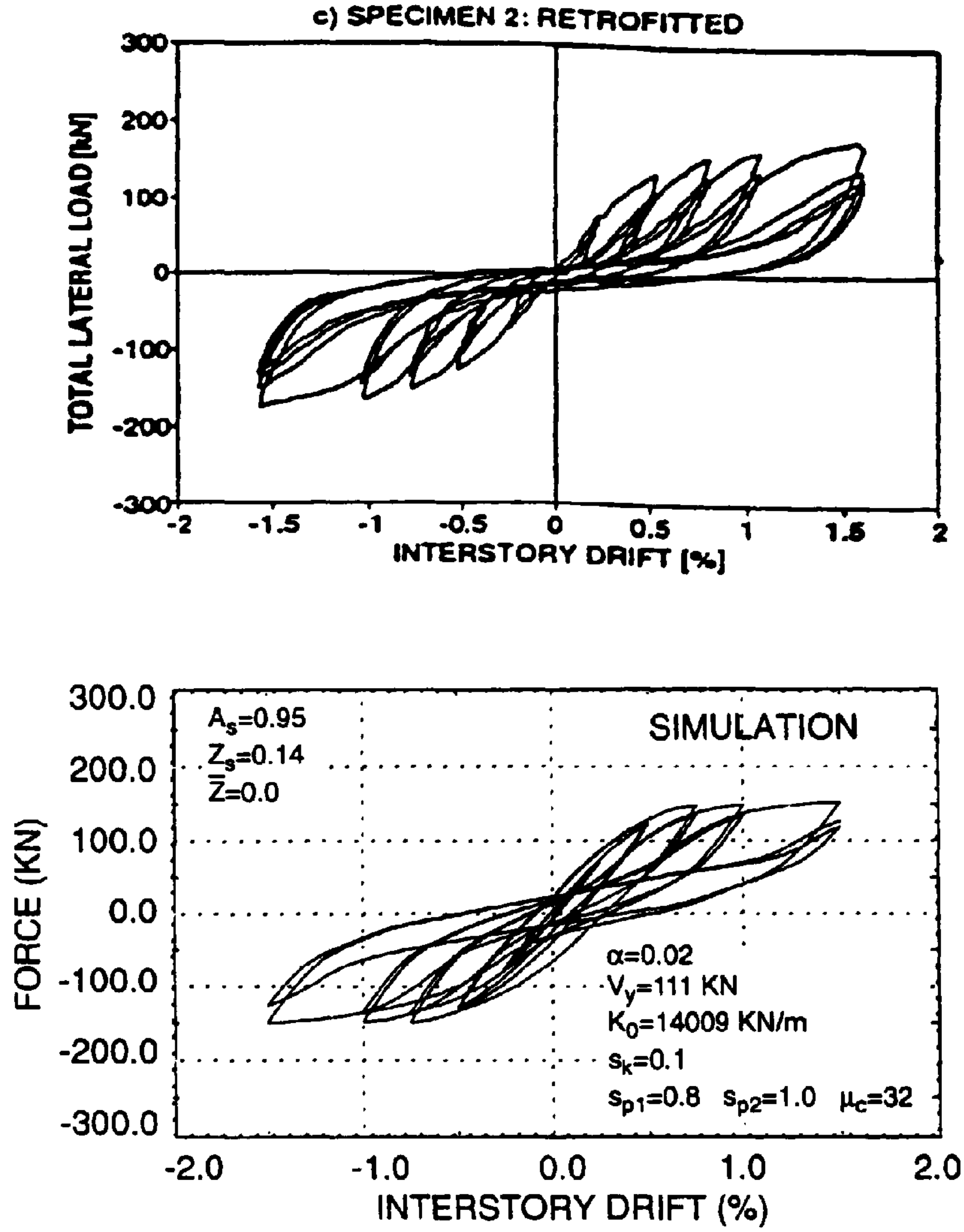


FIGURE 6.6: Comparison of Experimental vs. Simulated Force Displacement Response of Tested Masonry Infill Frame Specimen 2

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SECTION 7

DYNAMIC ANALYSIS OF MULTISTORY MASONRY INFILLED FRAMES

As discussed in the earlier sections, the proposed hysteretic macro-model was implemented in the program IDARC 2D Version 4.0 for nonlinear dynamic analysis of reinforced concrete framed structures with structural masonry infill panels. The program was employed to analyze a lightly reinforced concrete frame structural model subjected to representative ground motions for the purpose of evaluating the contribution of masonry infill panels to the dynamic response. Results of the dynamic analysis are presented in this section of the report.

7.1 Structural Model

A one-third scale model of a three story lightly reinforced concrete frame building (Figure 7-1) was analyzed for representative ground motions to study the modification of the structural response due to the addition of masonry infill panels. The structural model has been extensively tested and modeled at the University at Buffalo to evaluate its seismic response with and without a variety of supplemental damping systems and retrofit techniques [Bracci et.al (1992), Lobo et.al (1994), Reinhorn et.al (1993), Pekcan et.al (1995)]. In order to ensure common grounds for comparison with the results of these studies, the structural model was selected as the subject of study. The design and construction details of the structural model have been presented by Bracci et.al (1992). Time history analysis of the R/C frame building model was performed without and with infill panels for the purposes of comparative evaluation. Infill panels were specified in the interior bay of all the frames in the latter case.

7.2 Dynamic Analysis

The structural model was analyzed for an input base motion of the Elcentro 1940 accelerogram scaled to a PGA of 0.68g Comparison of the force-deformation response at

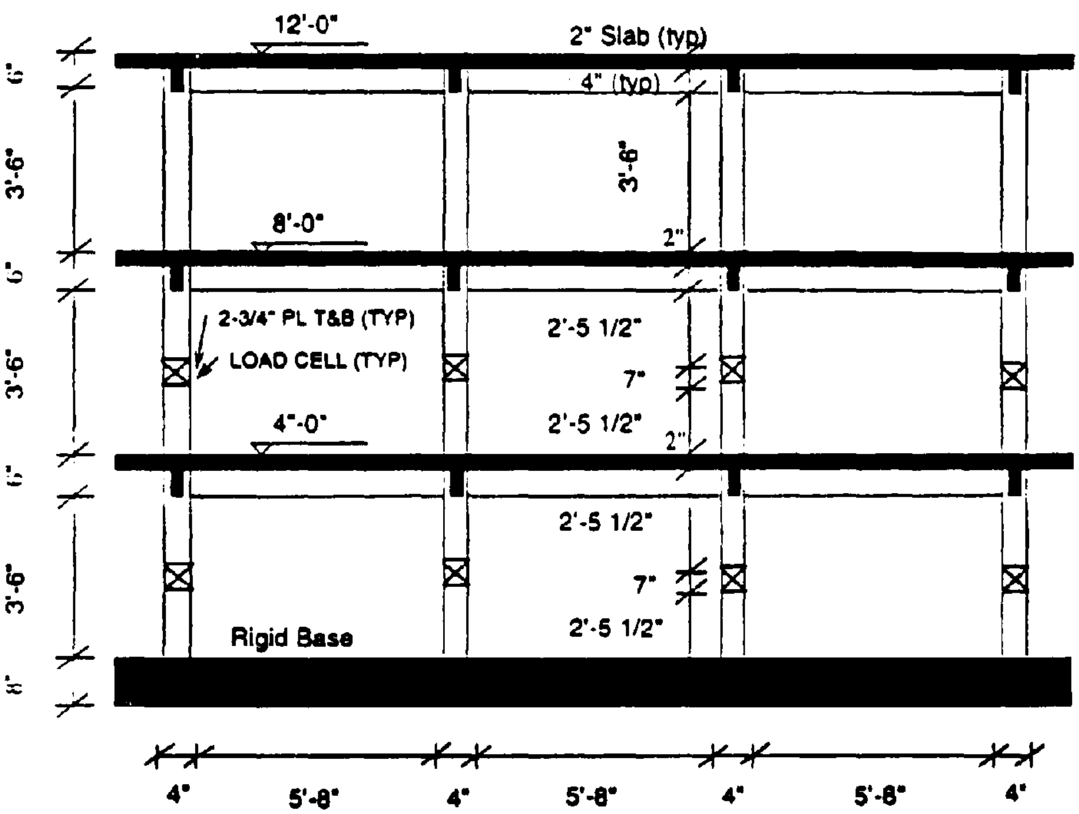

FIGURE 7.1a: Front Elevation of 1/3 Scale Reinforced Concrete Frame Model

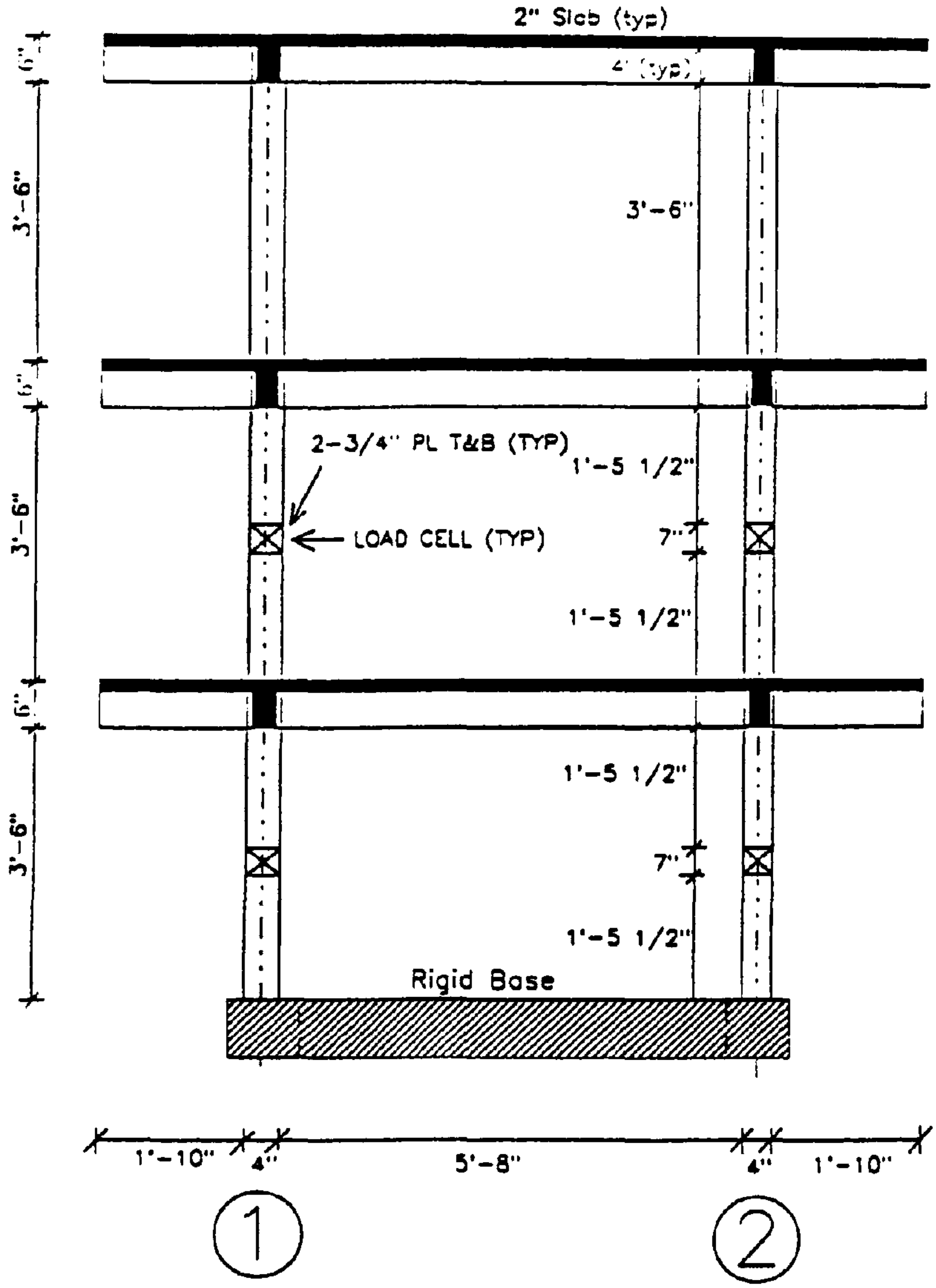

FIGURE 7.1b: Side Elevation of 1/3 Scale Reinforced Concrete Frame Model

the first story without and with the masonry infill panels is shown in Figure 7-2. A single wythe masonry infill panel was assumed. The dynamic analysis was repeated for infill panels with double wythe masonry. The results of analysis for infill panels with double wythe masonry are shown in Figure 7-3. It is evident from the figures that while the column shear forces remain the same, the maximum story drift is reduced approximately by half due to the addition of infills. As a result the hysteretic energy dissipation in the columns is substantially decreased. The final damaged state of the multistory R/C frame with double wythe infill panels is compared to that of the bare frame in Figure 7-4. The figure also illustrates the load-resisting mechanism of the first story in each case. A summary of the results of dynamic analysis is presented in Table 7-1. It should be noted that the local damage to the frame at the frame-infill interface or the infills at the corners cannot be precisely assessed using this modeling approach. The results are further discussed in the next section.

TABLE 7-I: Summary of Dynamic Analysis Results

Analysis Case	Story No.	Maximum Drift Ratio %	Maximum Story Shear (Kips)	Beam-Slab Damage	Column Damage	Overall Structural Frame Damage
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Bare Frame	3	1.24	12.59	0.007	0.727	1.107
	2	5.55	18.51	0.140	1.426	
	1	3.24	20.77	0.443	0.234	
Frame with single wythe infill panels	3	0.31	19.00	0.016	0.113	0.488
	2	1.28	31.83	0.011	0.565	
	1	1.99	38.07	0.241	0.201	
Frame with double wythe infill panels	3	0.21	25.83	0.020	0.072	0.247
	2	0.65	43.73	0.024	0.178	
	1	1.54	57.51	0.152	0.149	

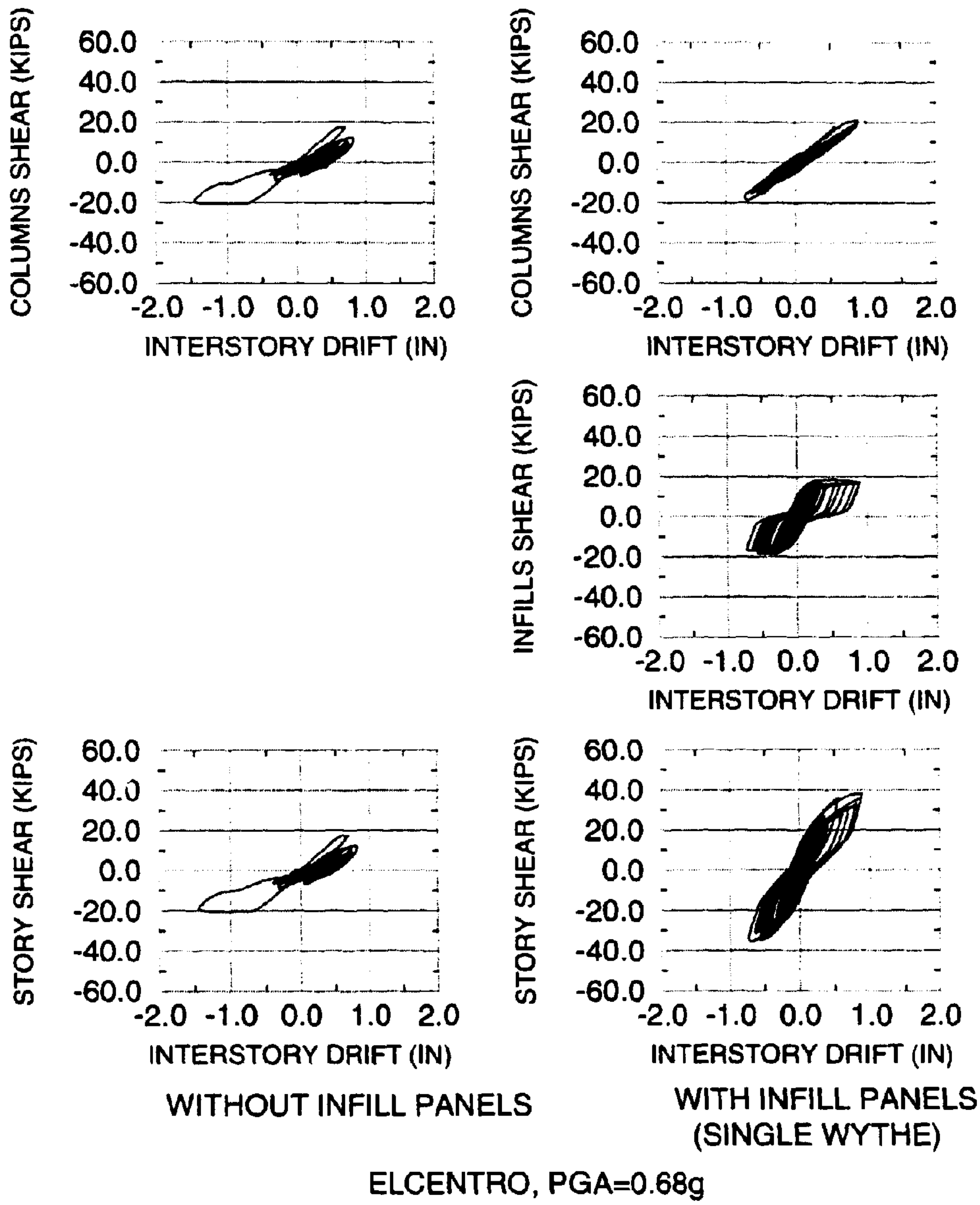

FIGURE 7.2: First Floor Response with Single Wythe Masonry Infills

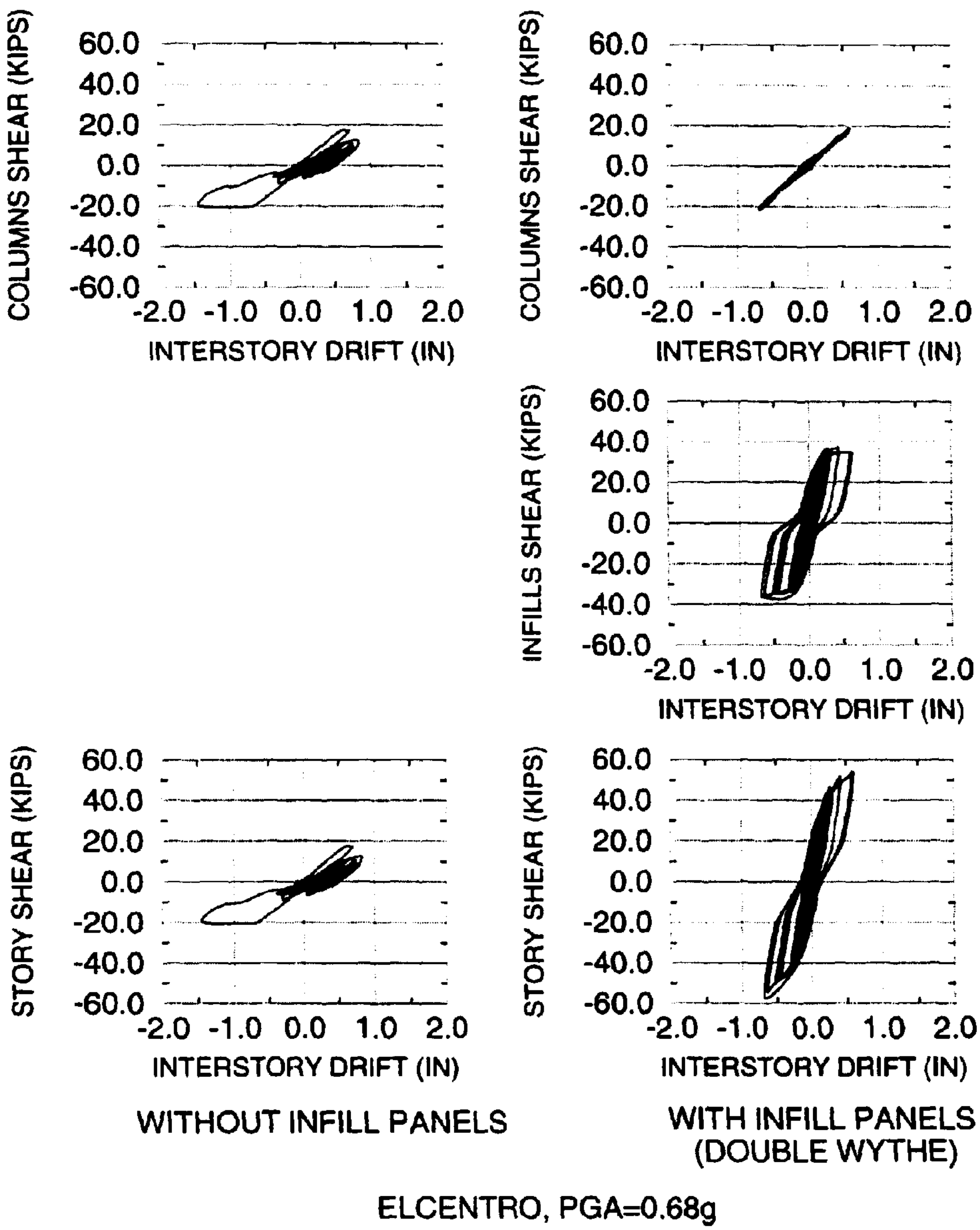

FIGURE 7.3: First Floor Response with Double Wythe Masonry Infills

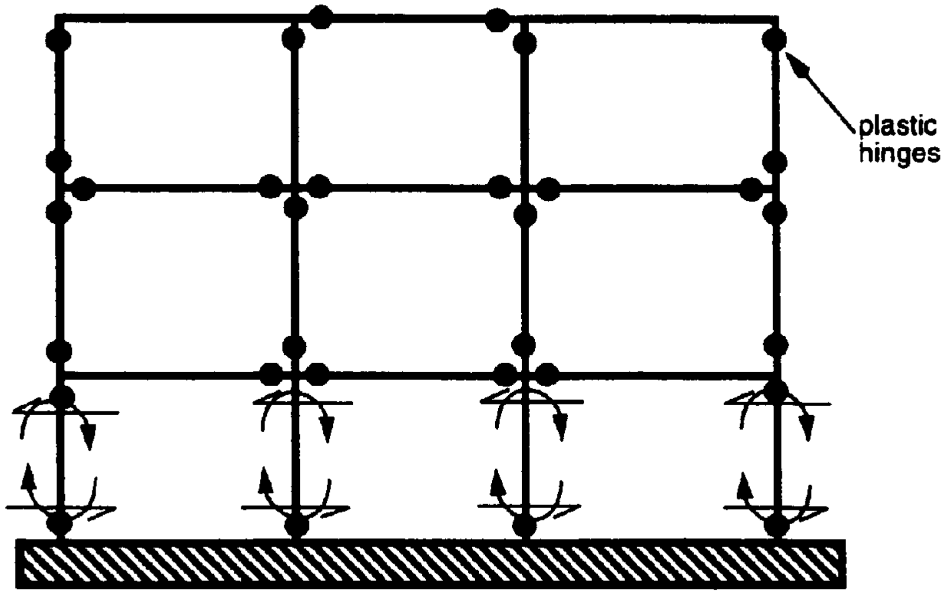

FIGURE 7.4a: Load Resisting Mechanism of Multistory Bare Frame

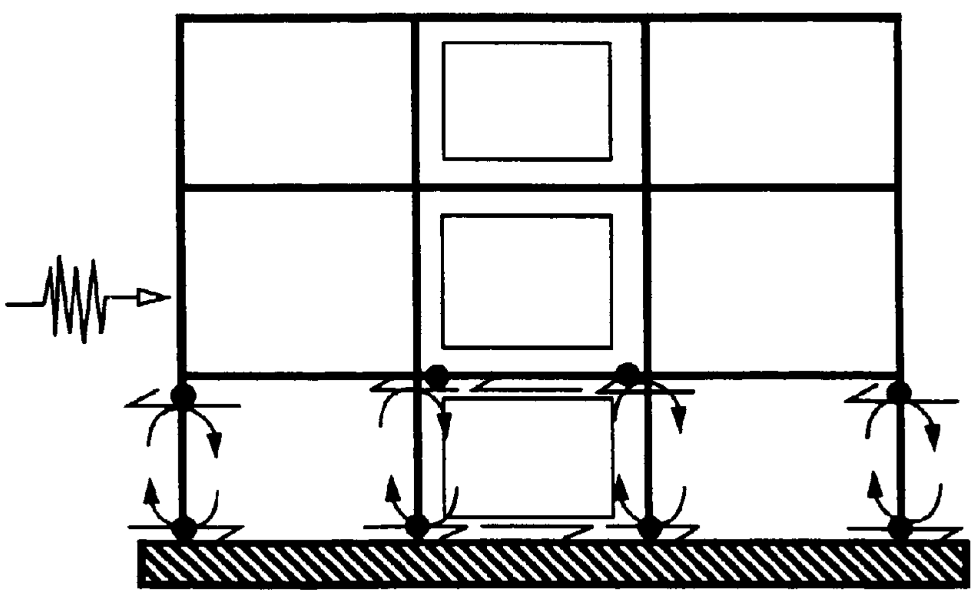

FIGURE 7.4b: Load Resisting Mechanism of Multistory Infilled Frame

SECTION 8
DISCUSSION AND CONCLUSIONS

The report presents an alternative non-linear hysteretic model for masonry infill walls. The model is based on the commonly used equivalent tie and strut approach in which the envelope properties of the strut are determined based on mechanics of infill-frame interaction [Saneinejad et.al (1995)]. The model is complemented by a proposed "smooth" hysteretic model which accounts for energy dissipation and gradual stiffness degradation, strength reduction and crack slippage. The hysteretic model along with the base envelope model constitute a more efficient analytical alternative to the micro-models (i.e. finite element based) for analysis of complex structures in which the infill is just one component.

The envelope model determines the equivalent properties of the strut i.e. the stiffness and control force-deformation points based on behavior of infill panel and its interaction with the enclosing frame. The deterioration parameters are determined from the analysis of experimental data, however, such parameters can also be determined from micro-models such as finite element models [Dhanasekar and Page (1986), Mosalam and Gergely et.al (1994), Mosalam (1996), Shing et.al (1992)]. The resulting hysteretic strut model is suitable for use in non-linear analysis - monotonic static "push-over" or time-history analysis of complex frame systems. The hysteretic model provides a convenient and versatile analytical tool for simulating and predicting the response of framed structures with masonry infill panels.

The proposed hysteretic model was implemented in the computer program IDARC Version 4.0 for quasi-static analysis in displacement or force control as well as dynamic analysis under earthquake excitations. The model was subsequently used to simulate experimental behavior of tested masonry infill frame sub-assemblages under quasi-static displacement controlled cyclic loading (Section 6). The model was also used to perform dynamic time-history analysis of a one-third scale model of a 3 story R/C frame for the dynamic analysis presented in Table 7-1 reveals that the damage in the structural frame as well as the individual components is the highest in case of the bare frame and the damage decreases with increasing strength of the infill panels. Whereas the frame was prone to collapse at the second floor (see Table 7-1, column 6), the presence of the double wythe masonry wall panels reduces the potential damage to a level similar to that in the other stories [Reinhorn et.al, (1995)]. At the same time the damage in elements (columns) is reduced to serviceable levels (i.e., only cosmetic repairs will be required in case of a severe earthquake).

(d) The final mechanisms in the bare frame and in the masonry infilled frame (with double wythe masonry) at the end of the dynamic analysis are presented in Figure 7.4. The figure suggests (based on quantification) that the lateral load resisting mechanism of the masonry infilled frame is essentially different from that of the bare frame. Under dynamic lateral loads, the bare frame acts primarily as a moment-resisting frame with the formation of plastic hinges at the joints in the inelastic range. In contrast, the infilled frame behaves like a braced frame in which the lateral loads are resisted by a truss mechanism (tie-strut mechanism) formed by the compression in the masonry infill panel and tension in the columns. The plastic hinges are confined to the joints in contact with the infill panels and the ductility demand is considerably lower than in case of the bare frame.

(e) The macro-modeling approach presented herein considers the entire infill panel as a single unit and takes into account only the equivalent global behavior of the infill in the analysis. As a result, the approach does not permit study of local effects such as frame-infill interaction within the individual infilled frame subassemblies. More detailed micro-modeling approaches such as the finite element models need to be used to capture the spatial and temporal variations of local conditions within the infilled frames by multiplicity of small elements each satisfying equilibrium and compatibility. However, the approach allows for adequate evaluation of the non-linear force-deformation response of the structure and individual components under seismic loading. The computed force-deformation response may be used to assess the overall structure damage and its distribution to a sufficient degree of accuracy. Thus, the proposed macro-model is better suited for representing the behavior of infills in the time-history analysis of large or complex structures with multiple components particularly in cases wherein the focus is on evaluating the structural response.

SECTION 9
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APPENDIX A

THEORETICAL BACKGROUND

A-1 Theoretical Formulation

The permissible stress f_a for the masonry strut in compression may be calculated as

$$f_a = f_c\left[1-\left(\frac{l_{eff}}{40t}\right)^2\right] \text{ where } f_c = 0.6\phi f'_m \text{ and } \phi = 0.65 \tag{A-1}$$

The upperbound or failure normal uniform contact stresses at the column-infill interface σ_{c0} and beam-infill interface σ_{b0} are calculated from the Tresca hexagonal yield criterion as:

$$\sigma_{c0} = \frac{f_c}{\sqrt{1+3\mu_f^2 r^4}} \quad ; \qquad \sigma_{b0} = \frac{f_c}{\sqrt{1+3\mu_f^2}} \tag{A-2}$$

where r is the aspect ratio of the infill i.e. $r = h/l$ and μ_f is the coefficient of friction of the frame-infill surface. The contact lengths at the column-infill interface $\alpha_c h$ and beam-infill interface $\alpha_b l$ are calculated from equilibirium as follows:

$$\alpha_c h = \sqrt{\frac{2M_{pj}+2\beta_c M_{pc}}{\sigma_{c0} t}} \le 0.4h' \tag{A-3}$$

$$\alpha_b l = \sqrt{\frac{2M_{pj}+2\beta_c M_{pb}}{\sigma_{b0} t}} \le 0.4l' \tag{A-4}$$

in which $\beta_o = 0.2$

The actual normal contact stresses σ_c and σ_b are calculated from the rotational equilibirium of the infill panel using the following methodology:

If $A_c \geq A_b$ then

$$\sigma_b = \sigma_{b0} \text{ and } \sigma_c = \sigma_{c0}\left(\frac{A_b}{A_c}\right) \tag{A-5}$$

If $A_b \geq A_c$ then

$$\sigma_c = \sigma_{c0} \text{ and } \sigma_b = \sigma_{b0}\left(\frac{A_c}{A_b}\right) \tag{A-6}$$

where,

$$A_c = r^2 \sigma_{c0} \alpha_c (1 - \alpha_c - \mu_f r) \tag{A-7}$$

$$A_b = \sigma_{b0} \alpha_b (1 - \alpha_b - \mu_f r) \tag{A-8}$$

The contact shear stresses at the column-infill interface τ_c and beam-infill interface τ_b are given as:

$$\tau_c = \mu_f r^2 \sigma_c \tag{A-9}$$

$$\tau_b = \mu_f \sigma_b \tag{A-10}$$

The sloping angle θ' of masonry diagonal strut at shear failure is given as:

$$\theta' = \tan^{-1}\left[(1 - \alpha_c)h' / l'\right] \tag{A-11}$$

A-2 Sample Calculations for Test Infill Frame Subassemblage:

Subassemblage Geometry:

$h = 1791\,mm,\ h' = 1587\,mm,\ l = 2553\,mm,\ l' = 2337\,mm,\ t = 89\,mm$

$\theta = \tan^{-1}(h'/l') = 34.18\deg$

$r = h/l = 0.7$

Material Properties:

$f'_m = 23.5\,Mpa,\ \varepsilon'_m = 0.003,\ f_t = 0.83\,Mpa,\ \nu = 0.79\,Mpa,\ \mu_f = 0.3$

$f_c = 0.6 f'_m = 14.1\,Mpa$

$$\sigma_{c0} = \frac{f_c}{\sqrt{1 + 3\mu_f^2 r^4}} = 13.251\,Mpa$$

$$\sigma_{b0} = \frac{f_c}{\sqrt{1 + 3\mu_f^2}} = 12.512\,Mpa$$

Frame element properties:

$M_{pc} = 8.501\,KNm,\ M_{pb} = 7.402\,KNm$ (from section properties),

$M_{pj} = 1.438\,KNm$ (from simulation)

$$\alpha_c h = \sqrt{\frac{2M_{pj} + 2\beta_c M_{pc}}{\sigma_{c0}\, t}} = 0.0730\,m \le 0.4h' \ (= 0.635\,m)$$

$\therefore \alpha_c h = 0.0730\,m \Rightarrow \alpha_c = 0.0408\,m$

$$\alpha_b l = \sqrt{\frac{2M_{pj} + 2\beta_b M_{pb}}{\sigma_{b0}\, t}} = 0.0723\,m \le 0.4l' \ (= 0.935\,m)$$

$\therefore \alpha_b l = 0.0723\,m \Rightarrow \alpha_b = 0.0283\,m$

$\theta' = \tan^{-1}\left[(1 - \alpha_c)h'/l'\right] = 33.08\deg$

Rotational Equilibirium of panel:

$A_c = r^2 \sigma_{c0} \alpha_c (1 - \alpha_c - \mu_f r) = 0.1982\,Mpa$

$A_b = \sigma_{b0} \alpha_b (1 - \alpha_b - \mu_f r) = 0.2699\,Mpa$

$$A_b \geq A_c \Rightarrow \sigma_c = \sigma_{c0} = 13.251\, Mpa, \quad \sigma_b = \sigma_{b0}\left(\frac{A_c}{A_b}\right) = 9.188\, Mpa$$

$$\tau_c = \mu_f\, r^2\, \sigma_c = 1.948\, Mpa$$

$$\tau_b = \mu_f\, \sigma_b = 2.756\, Mpa$$

Strut Dimensions:

$$L_d = \sqrt{(1-\alpha_c)^2 h'^2 + l'^2} = 2.789\ m$$

$$f_a = f_c\left[1 - \left(\frac{l_d}{40t}\right)^2\right] = 0.386 f_c = 5.446\ Mpa$$

$$A_d = (1-\alpha_c)\alpha_c\, t h' \frac{\sigma_c}{f_c} + \alpha_b\, t l' \frac{\tau_b}{f_c} = 0.00634\ m^2 \leq \frac{0.5 t h' \frac{f_a}{f_c}}{\cos\theta} \quad (= 0.0330\ m^2)$$

Hysteretic Model parameters:

$$V_m^+ (V_m^-) \quad \leq A_d f_m' \cos\theta \quad (= 123.75\ KN)$$

$$\leq \frac{v t l'}{(1 - 0.45 \tan\theta')\cos\theta} \quad (= 277.42\ KN) \leq \frac{0.83(MPa) t l'}{\cos\theta} \quad (= 208.7\ KN)$$

$$\therefore V_m^+ (V_m^-) = 123.75\ KN$$

$$u_m^+ (u_m^-) \quad = \frac{\varepsilon_m' L_d}{\cos\theta} = 0.010\, m$$

$$K_o = 2\frac{V_m}{u_m} = 24655.0\ KN / m$$

$$V_y^+ (V_y^-) = \frac{V_m - \alpha K_0 u_m}{(1-\alpha)} = 109.578\, KN$$

$$u_y^+ (u_y^-) = \frac{V_m - \alpha K_0 u_m}{K_0 (1-\alpha)} = 0.00444\, m$$

APPENDIX B
NOTATION

θ' sloping angle of masonry diagonal strut at shear failure $= \tan^{-1}\left[(1-\alpha_s)h'/l'\right]$

u lateral displacement of infill panel

u_1 displacement of smooth hysteretic element described by Bouc-Wen model

u_2 displacement of slip-lock element

μ_f coefficient of friction of frame-infill surface

μ normalized lateral displacement of infill panel

μ_1 normalized displacement of smooth hysteretic element

μ_2 normalized displacement of slip-lock element

V_m maximum lateral force of infill panel

V_y yield lateral force of infill panel

v basic shear strength (cohesion) of masonry bed joints

σ frame-infill uniform contact normal stress

τ frame-infill uniform contact shear stress

Z parameter for smooth hysteresis

Z_s parameter defining range of Z in which slip occurs

$\bar{Z}$ value of Z about which the slip is distributed

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