RESPONSE OF REINFORCED CONCRETE FRAMES WITH INFILLED PANELS UNDER EARTHQUAKE EXCITATION

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Dedicated to

Manbir

&

Gagandeep

CERTIFICATE

Certified that the work presented in the thesis entitled "**RESPONSE OF REINFORCED CONCRETE FRAMES WITH INFILLED PANELS UNDER EARTHQUAKE EXCITATION**" which is being submitted by Mr. Harpal Singh in fulfillment of the requirement for the award of the Degree of **Doctor of Philosophy** in the **Department of Civil Engineering, Thapar Institute of Engineering and Technology (Deemed University), Patiala** is an authentic record of candidate's own work carried out under our supervision and guidance. The matter presented in the thesis has not been submitted for the award of any other degree in any University.

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ABSTRACT

The studies reported herein focus on the three-dimensional linear elastic and nonlinear inelastic seismic response of reinforced concrete frame structures with non integral masonry infill panels. The infills provide significant stiffness and strength to the framing structure characterised by its interaction with the frame. Under the loads, the mortar may crack causing sliding and separation at the interface between the infill and the frame. Further the infill may get cracked and/or crushed which change its structural behaviour and may render the infill ineffective leaving the bare frame to take all the load which may lead to the failure of the framing system itself. A mathematical model for the inelastic static and dynamic analyses of the infilled frame systems has been presented to simulate the above behaviour.

In the investigation, eight-noded panel elements, six-noded interface elements and three-noded frame elements have been used to idealise the infill panel, frame-infill interface and the bounding frame, respectively. For the infill, smeared crack model has been adopted, while for the frame, elasto-plastic behaviour with associated flow rule has been assumed. At the interface between the frame and the infill the tangential stress-strain relationship has been assumed to be elastic perfectly plastic. Based on the above model a computer program has been developed for the elastic and inelastic static and dynamic analyses of three-dimensional infilled reinforced concrete frame buildings. The model predicts the sequence of formation of plastic hinges in the frame and cracks in the infill, and the entire time history response of the infilled frame systems to the earthquake excitation.

The investigation has shown that the stiffness and strength of an infilled frame system are extremely under estimated on the exclusion of the effect of infill from the analysis. The seismic response of both the two and three dimensional infilled frame systems has been discussed in detail. The computer program has successfully simulated the post earthquake failure/damage analyses of infilled reinforced concrete framed buildings. A simplified model has been proposed for the analysis

of the real sized infilled concrete frame systems. In the simplified model the infill has been idealised as a bilinear diagonal member acting both as a strut and a tie. The model provides reasonably close results to the reported experimental results.

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NOTATIONS AND SYMBOLS

The notations used in this study are listed below. Small bold letter represents a vector and capital bold letter represents a matrix. Some times, a symbol may have an alternate meaning but in such a case, the context is sufficient to avoid confusion.

A	Cross-sectional area.
A_s	Shear area.
$\mathbf{a}$	Flow vector.
a_o, a_1	Coefficients.
$\mathbf{B}$	Strain-displacement matrix : general.
$\mathbf{B}^a$, $\mathbf{B}^s$, $\mathbf{B}^t$, $\mathbf{B}^b$	Strain-displacement matrix : axial, shear, torsional, and bending, respectively, for beam element
$\mathbf{C}$	Damping matrix of the system.
$\mathbf{D}$, $\mathbf{D}^a$, $\mathbf{D}^s$, $\mathbf{D}^t$, $\mathbf{D}^b$	Material property matrix : general, axial, shear, torsional and bending, respectively.
$\mathbf{D}_{ep}$	Elasto-plastic material property matrix.
E	Modulus of elasticity.
F_x, F_{xu}	Axial force and axial yield force, respectively.
$\mathbf{f}$	External applied load vector on structure.
$\mathbf{f}^e$	Vector of applied nodal forces for an element.
$\mathbf{f}_p^e$	Vector of equivalent nodal forces for an element.
f	Yield function.
F_j	Applied force per unit length on beam element along jth translational degree of freedom.
G	Shear modulus.
H	Hardening parameter.
$\mathbf{I}$	Identity matrix.
I_{xx}, I_{yy}, I_{zz}	Moment of inertias of beam element about X, Y and Z axes, respectively.

I_x, I_y, I_z	Rotatory inertias of beam element about the X, Y and Z axes, respectively.
K^e, $\underline{K}^e$	Stiffness matrix of element in local and global coordinates, respectively.
K, K^*	Stiffness and effective stiffness matrix of the structure, respectively.
k_s, k_n	Shear and normal stiffness coefficients for interface element, respectively.
K^e_a, K^e_s, K^e_t, K^e_b	Axial, shear, torsional and bending stiffness matrices of beam element, respectively.
κ	Material parameter.
M^e, $\underline{M}^e$	Mass matrix of element in local and global coordinates, respectively.
M	Mass matrix of the system.
M_x, M_y, M_z	Moments for beam element about X, Y and Z axes, respectively.
M_{xu}, M_{yu}, M_{zu}	Yield moments for beam element about X, Y and Z axes, respectvely.
N	Matrix of shape functions.
n	Exponent
p	Set of body forces.
q	Internal resisting force vector.
R	Transformation sub-matrix.
r	Residual force vector.
S_{xy}, S_{xz}	Shear stiffness coefficients for beam element.
T	Transformation matrix.
t	Time
∇t	Time increment
u, v, w	Translational degrees of freedom with respect to X, Y and Z axes, respectively.
u, $\dot{u}$, $\ddot{u}$	Displacement, velocity and acceleration vectors, respectively.

$\underline{u}$, $\dot{\underline{u}}$	Newmark's prediction vector of displacement and velocity, respectively.
dv	Incremental volume.
dx, dy	Incremental length along X and Y axes.
Q_i	Total load corresponding to the ith degree of freedom.
X, Y, Z	Global coordinate axes.
x_m, y_m, z_m	Local coordinate axes.
α	Twist, the angle between the normal of crack surface and the X-axis.
β, ϕ	Coefficients.
θ_x, θ_y, θ_z	Rotational degrees of freedom about X, Y and Z axes, respectively.
σ_u, σ_v	Tangential and normal stresses at a Gauss point of interface element.
σ, σ_x, σ_y, τ_{xy}	Stress vectors : general, along X and Y axes and stress, respectively.
ε, ε_x, ε_y, γ_{xy}	Strain vectors : general, along X and Y axes and strain, respectively.
ν	Poission's ratio.
ρ	Mass density.
r_i	Unbalanced load corresponding to ith degree of freedom.
δ^e, δ	Displacement vectors : elemental and general.
ε_{xx}, ϕ_{xy}, ϕ_{xz}	Axial strain and shear strains for beam element.
k_{xz}, k_{xy}	Curvatures or bending strains for beam element.
ε_{to}	Tolerance
μ	Coefficient of friction for interface element.

CHAPTER 1

INTRODUCTION

1.1 Prelude

Infill panels are commonly used all over the world even in the regions of high seismicity to meet architectural and functional requirements, particularly where resources and skilled labour are scarce and masonry construction continues to be economical. Documented reports about earthquake damage to structures in different regions highlight the poor performance of structures with infilled frames when subjected to strong ground motions [Hansen *et al.*(1967), Uzsoy (1972), Golagau (1974), Shephered *et al.* (1983) and Paul *et al.*(1991)]. Some of these reports describe the extensive failure of exterior masonry infills which resulted in hazard to life and property. Further some instances of the reinforced concrete frame buildings in India [Paul *et al.*(1991)] have been quoted in which improper infill disposition resulted in damage to the framing system. Thus there is an urgent need for acquiring further knowledge regarding seismic response of the infilled frames to reduce the loss of life and property associated with the failure of masonry infills.

The composite behaviour of infilled frames under static and dynamic loads is highly indeterminate in nature. Even though the rigid frame directly carries some of the load, it primarily serves to transfer and distribute major portion of the load to the infill. The stiffness of the composite frame depends to a large extent on the mode of its deformation and as a result on the response of the infill to the deformation. On the other hand the strength of the infilled frame is influenced by the interaction of the frame and the infill which may be monolithic or separated at some points with the frame. The degree of interaction controls the stress distribution in the infill and therefore affects the strength and modes of failure of the composite frame.

Generally the designers tend to ignore the structural contribution of infill panels to avoid complexity in the analysis presuming such an omission to be safe and conservative. Besides being unrealistic, such an approach is also likely to lead to lesser safe designs due to improper distribution of lateral shears among the frames, induction of unintended shear and axial forces in frame members, induction of torsional response, increased inertial forces etc. [Thiruvengandam (1985)].

IS: 1893-1984, recommends a use of performance factor of 1.6, if the designer wishes to include the effect of masonry infills in stiffness and strength calculations. This indicates that the infilled frames are believed to possess very low ductility.

1.2 Statement of the Problem

Post earthquake damage inspection of framed buildings has helped to identify the reasons of observed damage and to assess the reliability of various analytical models and techniques for predicting structural response to earthquake ground motions. The observations have strongly advocated further investigations into the role played by the infills in framed buildings [Bertero (1980)]. A number of simplified 2-D nonlinear finite element analyses of the infilled frames under monotonic and cyclic loading have been carried out. As a matter of fact, the problem is not that simple as it involves an infill behaviour with several complexities. The behaviour of infills is primarily of 3-D in nature even in otherwise symmetrical frames depending upon their disposition in space. The nonlinear behaviour of the reinforced concrete frame, the cracking and crushing of the infill , separation and closing of gaps between the frame and the infill, and slipping are to be accounted for in an investigation. The following properties of the frame and the infill need to be suitably modelled:

(i) initial monolithic character of the frame and the infill,

(ii) slipping between the frame and the infill,

(iii) separation of the infill from the frame,

(iv) closing of gap between the frame and the infill, and

(v) cracking and crushing of the infill.

In the investigations reported in this thesis all the above complexities are included in the numerical procedure adopted by making suitable assumptions and simplifications.

The infilled frame has been modelled using isoparametric finite elements. Eight-noded panel elements, 6-noded interface elements and 3-noded frame elements have been used to idealize the infill panel, the frame-infill interface and the surrounding frame respectively. The frame element has six degrees of freedom per node. Suitable yield criteria for the frame member sections have been adopted to incorporate the elasto-plastic behaviour. Only in-plane stiffness of infill panels have been considered, since out of plane stiffness of unreinforced masonry panel is negligible as compared to the in-plane stiffness. To model the interface conditions at the interface between the frame and the infill, the interface element has been used.

A simplified model for the analysis of infilled reinforced concrete frame systems has been proposed for the use in design office. The infill has been modelled with bilinear diagonal member acting both as a strut and a tie.

1.3 Scope and Objective of the Study

The objective of the study is to investigate nonlinear behaviour of the reinforced concrete frames with non-integral infilled panels under earthquake excitations. The investigations deal exclusively with ductile reinforced concrete frames, infilled with unreinforced masonry panels without any integral connections with the frame. Following are the main objectives of the present investigation.

(i) to carry out literature survey on the behaviour of the infilled frame buildings, and identify the areas where research is required,

(ii) to use the 2-D and 3-D nonlinear analyses for the infilled reinforced concrete frames, in order to model their behaviour more precisely,

(iii) to develop a finite element analysis procedure for the nonlinear analysis of reinforced concrete frames with non-integral infill panels,

(iv) to develop a computer software for nonlinear analysis of the infilled reinforced concrete frame building and to check its validity by comparing the computed response with the published analytical and experimental results,

(v) to study the various stages of frame-infill interaction such as the initial loss of bond at the interface of the frame and the infill, slipping, separation, cracking, yielding, crushing, and numerical modelling of these phenomena,

(vi) to carry out the post-earthquake damage/failure analyses to demonstrate the utility of the proposed model in predicting the earthquake damage/failure of the buildings and to establish the reliability of the proposed model,

(vi) and finally, to develop a simplified model which can be used in design office for the design of the infilled reinforced concrete frame systems.

1.4 Layout of the Thesis

The work reported in the thesis is organized in the following manner.

The need and importance of the investigation have been highlighted in Chapter 1 along with the scope of the study.

In Chapter 2, a brief review of the literature is presented. The review covers the work done on linear and nonlinear, static and dynamic behaviour of the frames with infills. Some investigations on nonlinear solution techniques are also presented.

In Chapter 3, finite element formulations of the frame, the infill panel and the interface elements are discussed. Nonlinear modelling and solution of dynamic equilibrium equations are presented. The behaviour of a number of structures analysed by using the proposed finite element procedure have been compared with those reported in the literature to establish the validity of the formulation.

In Chapter 4, nonlinear behaviour of 2-D infilled frames is presented. Whereas Chapter 5, presents the nonlinear response studies of 3-D frames with the infills.

A simplified model for the analysis of the infilled reinforced concrete frame systems has been presented in Chapter 6 for the use in design office.

Chapter 7 deals with major conclusions drawn from the investigations reported in the thesis along with suggestions for further research in the area.

1.5 Concluding Remarks

In this Chapter, current status as regards to the evaluation of the response of the reinforced concrete frames with the infill panels under earthquake excitation has been described. The need and importance of the study along with the significance of modelling the infilled frames have been discussed briefly. The underlying principles of the proposed formulations are also stated.

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CHAPTER 2

REVIEW OF LITERATURE

2.1 General

A comprehensive review of literature related to the performance of the infilled frames has been carried out. The literature review examines the concepts behind the major experimental and analytical studies on linear and nonlinear, static and dynamic behaviour of infilled frames and other related structural components. The review is made under the following sub-systems:

(i) bare (unbounded) walls,
(ii) bare frames (without infilled panels), and
(iii) infilled frames.

2.2 Experimental Investigations

In this section experimental investigations related to the linear and nonlinear behaviour of bare walls, bare frames and effects of infill on the frames subjected to both static and dynamic loads have been reviewed.

2.2.1 Bare Walls under Static Loads

Benjamin *et al.* (1958) investigated the behaviour of unreinforced brick masonry walls under pure horizontal racking load. The aspect ratios of the walls varied from 0.8 to 3.0. The walls were observed to fail in flexure. They concluded that the pre-cracked stiffness and failure load could be predicted by the Simple Beam Theory.

Scrivener (1967) tested a series of reinforced concrete masonry panels subjected to lateral shear and vertical compression. Provision of peripheral reinforcement along with horizontal and vertical reinforcements was found to improve ductility and cracking resistance. As a consequence, the failure load was observed to increase considerably.

Schneider (1969) conducted a series of failure strength tests on full scale I-shaped concrete block masonry piers with nominal horizontal and vertical reinforcements. The failure load was found to increase with decreasing aspect ratios of the piers. An increase in the strength and ductility of the piers was observed with the addition of horizontal reinforcement. However, the vertical reinforcement was found to be comparatively ineffective.

Borchelt (1970) carried out laboratory studies on clay brick and high strength mortar panels in diagonal compression and found that the failure by diagonal tension occurred when the principal tensile stress in the panel reached its critical value.

Negoita (1972) tested plain and reinforced brick masonry wall models with aspect ratios ranging from 0.23 to 0.7 under lateral shear. The plain walls were found to fail by cracking along the diagonal. However, reinforced brick masonry walls subjected to both horizontal and vertical loads were found to fail due to diagonal cracking and horizontal rupture.

2.2.2 Bare Walls under Dynamic Loads

Mallick (1962) noticed that in linear elastic range , masonry walls developed a damping ranging from 2 to 5 per cent of the critical damping.

Williams (1971) conducted a series of cyclic and dynamic load tests. The type of walls exhibiting shear behaviour under cyclic static loads continue to do so under dynamic loads, but they undergo a high degree of structural deterioration after one or two cycles. However, the type of walls exhibiting flexural behaviour under cyclic static loading suffered a severe loss of carrying capacity with the load repetition.

Scrivener and Williams (1971) performed a series of cyclic quasi-static load tests on reinforced brick and concrete block walls. Two distinct types of behaviours were observed: (a) flexure characterized by initial cracking in the horizontal mortar joints and (b) walls with low aspect ratios carrying high bearing load and having considerable vertical reinforcement were found to display 'shear' behaviour and to possess comparatively low ductility.

Shing *et al.*(1989) found that reinforced masonry wall panels exhibit a certain amount of ductility and energy dissipation capability under cyclic displacement reversal. They recommended use of shear panels for seismic resistance design, provided proper reinforcement guidelines are developed and followed. The confinement reinforcement was found to enhance flexural ductility over diagonal shear cracking.

Naraine and Sinha (1991) carried out experimental investigation into the behaviour of brick masonry under cyclic biaxial compressive loading. A failure criterion in terms of stress invariants has been presented.

2.2.3 Bare Frames under Static Loads

Verghese and Krishnamoorthy (1989) studied the nonlinear behaviour both experimentally and analytically of closed multistorey frames subjected to a series of static point loads. They proposed a nonlinear frame element to model the behaviour of the frame.

Singh *et al.*(1990) conducted an experimental investigation into the behaviour of building frames with different reinforcement details and subjected to torsion. Overall dimensions of the frame in the test series were kept constant. The effect of amount and type of reinforcement on the torsional resistance and cracking pattern were studied. The development of strains in the columns of the frames was also investigated. The results indicated that the design carrying capacity as stipulated by IS: 456-1978 clause 38.6 was slightly conservative.

2.2.4 Bare Frames under Dynamic Loads

Some of the important contributions through experimental investigations into the behaviour of bare frames under dynamic or cyclic loads are outlined below:

Bertero and McClure (1964), Brown and Jirsa (1971), Gulkan and Suzen (1971), and Otani (1974) made a comprehensive experimental and analytical study of the dynamic inelastic response of reinforced concrete and steel frames.

Chen and Powell (1982) studied both experimentally and analytically the elastic and post-elastic behaviour of 3-D frames. A comparison with generalized plastic hinge concepts for 3-D beam-column elements has been presented.

2.2.5 Infilled Frames under Static Loads

In this section investigations regarding the behaviour of steel and reinforced concrete frames with integral or nonintegral masonry, reinforced or unreinforced concrete infills have been reviewed.

Ockleston (1955), Read (1965) and Fiorato *et al.*(1970) investigated the effect of infill on the lateral stiffness and strength of multistorey reinforced concrete frames. Both the stiffness and strength were found to increase the latter by a factor of the order of seven. However, a reduction in the ductility of the infilled frame as compared to a nominally identical assemblage without infill has been reported.

Benjamin *et al.*(1958) studied the behaviour of reinforced masonry and concrete shear panels enclosed in reinforced concrete bounding frames. Variation in the amount of steel has not been found to influence the rigidity of whole system significantly. The Simple Beam Theory including the effect of shear flexibility was found to predict the behaviour satisfactorily.

Smith (1966) studied the behaviour of square infilled frames loaded diagonally. The frame got separated from the infill over a part of the length on each side depending upon the relative stiffness of the frame and the infill. The part of the infill in contact with the frame behaved as a diagonal strut in compression. The equivalent strut model has been reported to give good results in both single and multistoreyed frames. Two types of failure were observed (a) local crushing of loaded corner of the infill and (b) tension cracking along the compression diagonal of the panel.

Mallick and Severn (1967) studied a series of pairs of rectangular and square steel frame models with plaster infill (aspect ratios varying from 0.5 to 1.0) placed back to back and loaded in lateral shear along the common frame member. It has been reported that the common failure points were either the point of maximum principal tensile stress or the points of contact between the frame and the infill.

Wood (1978) noticed a considerable gain in the shear strength assuming a composite action between the wall and the frame. The composite design was found to be more realistic as compared to the bare wall or the frame designs. Simplified design methods were also proposed.

Liauw and Kwan (1984) investigated the behaviour of steel frame model infilled with non-integral micro-concrete panel and proposed an equivalent strut width.

2.2.6 Infilled Frames under Dynamic Loads

Investigations into the behaviour of steel or reinforced concrete frames with integral or non-integral masonry, plain concrete and unreinforced infills have been reported in the following section:

Muto (1968), and DeLisle and Heiderbricht (1971) studied the response of frames with slitted panel under cyclic load. For a panel reinforcement (0.25 per cent), an increase in energy dissipation per cycle with vertical load has been reported.

Mallick and Garg (1971) obtained values of equivalent viscous damping by a variety of static and dynamic tests on multistorey models. Damping was found to increase with increased amplitudes of vibration. They attributed the development of damping due to internal molecular friction, friction between the frame and the infill, friction between pieces of the infill material itself following onset of cracking, and racking between the frame and the infill.

Mallick (1972) investigated the elastic dynamic response of small scale multistorey steel frame model with cast-in-place infills of plain concrete. The results were compared with the analytical predictions obtained by modelling the infills as equivalent diagonal compression struts. The elastic properties used in the analysis were determined by finite element analysis of a single infilled frame subjected to lateral load.

Yamaguchi and Araki (1973) carried out experimental and analytical investigations into the strength and stiffness characteristics of the infilled frames made of pre-cast reinforced concrete panels connected to the steel bounding frames by flexible connectors. In elastic range the results were found to agree with those obtained using a finite element analysis.

Klingner and Bertero (1976) reported several advantages of infilled frames designed and constructed according to ductility requirements over the comparable bare frames. The increase in strength and energy absorption and dissipation capacities achieved by addition of engineered infills is so large that it far exceeds the detrimental effects of possible increase in inertial forces due to increased stiffness and consequent decrease in the period of vibration.

Tomii *et al.*(1988) found that the ductility of framed shear walls can be improved by increasing the confining reinforcement of end columns or by using steel tubes for end columns. It was also observed that by selecting a particular wall thickness and amount of longitudinal reinforcement bars in the end columns such that the lateral shear capacity, dominated by slip failure is larger than that dominated by tensile yielding of longitudinal bars in end columns and by confining the potential shear failure regions at the top and bottom ends of end columns in steel tube, it is possible to improve drastically the deformation capacity of shear walls reinforced by hoops.

Fukuzawa *et al.*(1988) studied the inelastic behaviour of framed shear wall governed by slip failure of wall panel. To obtain ductile restoring characteristic by slip failure of wall panel, sufficient reinforcement for shear and tensile forces of surrounding frame were needed. They proposed an analytical method wherein the wall panel after cracking, was replaced by tensile and compressive braces.

2.3 Analytical Investigations

Analytical investigations into the linear and nonlinear behaviour of bare walls and frames, and effect of infills on the frames subjected to both static and dynamic loads are presented in the following section.

2.3.1 Bare Walls under Static Loads

Benjamin and Williams (1957) analysed walls with The Simple Beam Theory in elastic range, which was found to be inadequate in case of cracked wall or pierced wall.

Rosenhaupt (1962), and Rosenhaupt and Muller (1963) analysed the pierced wall with truss analogy.

Singh *et al.*(1994) used finite element method to study the response of shear walls with large openings. Variation of stress across the section was found to be parabolic. The maximum stresses were found to be near outer edges. Stress concentrations were observed near openings.

2.3.2 Bare Walls under Dynamic Loads

Heiderbrecht and Raina (1971) evaluated the natural frequency and corresponding mode shapes by assuming the walls to behave as thin plates.

Williams (1971), and Scrivener and Williams (1971) studied the ductility requirements of short period single degree of freedom (SDF) system. The SDF model with flexural mode and stiffness degradation was found to predict the observed behaviour satisfactorily.

2.3.3 Bare Frames under Static Loads

Some of the important studies on the linear and nonlinear static behaviour of plain frames are reported below :

Cohn *et al.*(1983) developed a computer program to predict elastic, inelastic and total responses of reinforced concrete structures. The program assumed the displacements to be small, material to be elastic perfectly plastic, and plasticity being lumped at the ends of element.

Creus *et al.*(1984) presented an approximate method for the analysis of elasto-plastic plane frames. Finite displacements, nonlinear tangent stiffness and generalized plastic hinges at concentrated critical sections were assumed.

Zaupa *et al.*(1984) proposed a pseudo-nonlinear design program, which includes moment-rotation relationships and redistribution of moments at the ends of the beams. The program incorporates generalized limit-states safety check for normal stresses on cross-sections of the specified members.

Rankovic and Coric (1984) used the incremental numerical analysis technique for stability analysis of reinforced concrete members with concrete cracked on tension side. Both the nonlinear bending moment curvature relationship and member curvature were included in the analysis.

Erbatur (1984), and Carol and Murcia (1984) presented techniques and programs for nonlinear analysis of frames and illustrated their applications.

Scholz and Faller (1986) presented a computerised version of an approximate second-order, elasto-plastic analysis of multistorey sway frames. The technique greatly reduced the analysis time and simplified the optimization of members in the design of large multi-storey frames.

Chandra *et al.*(1990) studied both geometrical and material nonlinear models for steel skeletal space frames. In geometric nonlinear analysis, the effects of instability produced by axial forces, the bearing of deformed member and finite deflections were included. Plastic hinges were assumed to be concentrated at the ends of the elements.

Ziemian *et al.*(1992) presented computer applications for the inelastic analysis and design of plane and space steel structures in conjunction with load factors and serviceability criteria.

Singh and Singh (1992) presented a simple technique for second-order analysis of frames. The technique used an iterative process with modification of the stiffness matrix as well as the load vector. Both static and sway effects were included.

King *et al.*(992) presented simplified methods for second-order inelastic analysis of steel frames termed 'Modified Plastic-Hinge and the Beam-Column Strength approaches'. The techniques alleviated the problems associated with over prediction of stiffness and strength by the usual elasto-plastic hinge analysis methods, since the model included the degradation of stiffness as the cross-section's strength approached the critical value at locations along the member length. The beam-column model included equations for the strength of the overall member rather than the expressions for the cross section's strength. The proposed methods were improvements over the conventional elasto-plastic models which consider only sectional stiffness.

Wood (1992) investigated the seismic behaviour of reinforced concrete frames with setbacks. Symmetrical and unsymmetrical arrangements of setbacks were tested. The displacement, acceleration and shear responses of the setback frames during earthquake simulations were compared with those of seven previously tested frames with uniform profiles. The setback frames were observed to be lesser susceptible to damage or lesser susceptible to the higher mode effects than the frames with uniform profiles.

Pankaj and Singh (1992) proposed a method to identify the most unfavourable direction of seismic excitation for various modes in unsymmetrical as well as symmetrical structures having pure torsional modes. The mass perturbation procedure together with critical direction alleviates the need for dual dynamic and static analyses to model the torsional response as all the effects are accounted for within the realm of dynamic analysis only.

Hartley and Abdel-Akher (1993) studied the behaviour of flat-plate structural system, comprising of thin Kirchoff plates (the floor slabs), which are interconnected by one dimensional flexural elements (the columns and column walls) of various shapes and layout. Direct boundary element method (DBEM) was used to generate the stiffness properties of the slab (thin plate) element and to calculate the internal actions. The proposed model was recommended to be used for both research and design practice.

Kuo *et al.*(1993) presented a model for nonlinear analysis of rigid-jointed space frames to include finite rotations, based on Euler's finite rotation formula. Unique analytical expressions valid for updating the geometry of the frame element involving finite rotations were used. The displacement increments observed from the solution of equations of equilibrium at each incremental step were decomposed into rigid body modes and member deformations, there by providing a basis for reckoning and updating the element forces.

2.3.4 Bare Frames under Dynamic Loads

Cheng and Botkin (1972) presented a numerical method for the analysis of tall buildings subjected to various time-dependent excitations. The damping, nonlinear material behaviour and geometric nonlinearity were considered. The displacement response associated with nodal matrix damping was observed to be greater than that associated with the damping proportional to mass and stiffness matrices.

Gillies and Shepherd (1981) developed a computer program to predict the time-history response of elasto-plastic three dimensional frame structures subjected to earthquake ground motions. Bands of plastic hinges have been found to move up and down the structure and the model period got elongated in comparison with the initial elastic period during cycles of yield.

Chen and Powell (1982) and Powell and Chen (1986) developed generalized plastic hinge concepts for the 3-D beam-column elements. Models with both lumped and distributed plasticity have been presented and relative study of various models for static and dynamic load applications have been presented.

Gillies and Shepherd (1984) presented a three dimensional nonlinear model with various types of yield criteria for inelastic dynamic analysis of reinforced concrete frame.

Shi-Ping and Ju-Min (1984) proposed a generalized method taking into account geometrical and material nonlinearities for full range analysis of reinforced concrete frames under cyclic loading. The technique included the effect of slippage of main bars in column-beam joint area.

Anagnostides (1986) used the 'Timoshenko Beam Theory' to compare experimental and analytical results of a steel frame subjected to harmonic excitation. Good agreement between the experimental and analytical results has been reported.

Sedarat and Bertero (1990) studied the effects of torsion on the three dimensional linear and nonlinear seismic response of multistorey building structures. It was found that most of the present building codes are nonconservative.

2.3.5 Infilled Frames under Static Loads

Investigations into the behaviour of steel or reinforced concrete frames with integral or nonintegral infill panels are reviewed in the following section.

Holmes (1961) proposed the concept of infill as an equivalent compression strut of thickness equal to that of the panel and a width equal to one third of the length of the diagonal of the panel. An infilled frame or assemblage of such frames would then be idealized as a pin jointed truss having rectangular panels braced by compression diagonals. Effective elastic modulus for the equivalent struts were computed on the basis of various model tests.

Smith (1962, 66, 68) suggested that the width of the equivalent strut varied with the applied loading and relative stiffness of the frame and the infill. Assuming no bond between the frame and the infill, the empirical relations for contact length have been developed. This concept did not lead to the results consistent with the experimental observations. The discrepancy has been attributed to the incorrect boundary stress distribution.

Mallick and Severn (1967) used finite element analysis with rectangular finite element for the panel and a number of link elements capable of taking compression and shear for interface between the frame and the infill. The result obtained for a two storeyed infilled frames were found to be consistent with those obtained experimentally.

Franklin (1970) studied the behaviour of infilled frames, taking into account the material nonlinearity, cracking of plain concrete infill, separation, and slip between the frame and the panel.

Liauw (1970, 73, 73) analysed the frame with infill assuming the panel to be isotropic and homogeneous and using an eight-term stress function to satisfy the boundary condition of continuous compatibility between the frame and the infill.

Liauw and Kwan (1984) examined the nonlinear behaviour of nonintegral infilled frames using finite element method. The nonlinearities of material, structural interface, effects of initial lack of fit and friction at interface were taken into account. It was shown that the stress redistributions towards collapse were significant and the strength of nonintegral infilled frames was very much dependent on the flexural strength of the frame.

Rao and Seetharamulu (1983) used elements which included inplane rotation to study the behaviour of staggered shear panels in tall buildings. A good agreement between experimental and theoretical results has been reported.

May and Ma (1984) used a nonlinear finite element model for the analysis of infilled frames. An eight noded isoparametric membrane element with two degrees of freedom at each node was used to model the infill. Two noded bar elements with two degrees of freedom per node were used to model the behaviour at the infill frame interface. The theoretical results were reported to be in good agreement with those obtained experimentally.

Papia (1988) used boundary element to model the behaviour at the infill frame interface. A comparison of the results with those of the equivalent strut model was made.

May and Naji (1991) carried out nonlinear analysis of the infilled frames under monotonic and cyclic loading using finite element method. The skeletal frame was modelled with 3-noded frame element and the panel with 8-noded isoparametric element. A 6-noded interface element was used to model the interface between the frame and the infill. The analysis provided good results up to the failure load but predicted lower carrying capacities for given displacements.

Haddad (1991) analysed cracked frames with masonry infill using the finite element method and fracture mechanics. The model takes into account the effects of crack size and its location, relative stiffness of infill and frame, geometry of the frame and frame-infill contact length.

Singh *et al.*(1993) studied the shear wall frame system using finite element analysis. The distribution of lateral forces and their effect on the resisting elements of buildings braced with shear walls was presented. The modification in existing methods of design were also suggested.

2.3.6 Infilled Frames under Dynamic Loads

Studies of the behaviour of steel or reinforced concrete frames with integral or non-integral panels under cyclic and dynamic loads are reported in the following section:

Mallick and Severn (1968) determined the natural frequencies of an infilled frame treating it as an assemblage of linear elastic frame and panel element jointed together along a continuous interface. The natural frequencies were found to be in good agreement with those obtained experimentally as long as vibration amplitudes were kept small.

Kost (1972) developed a computer program considering linear elastic behaviour with the known initial separation between the panels and the frames.

Wilson *et al.*(1975) idealized the building system as a combination of independent frame and shear wall elements interconnected by floor diaphragms which were rigid in their own plane. Based on this model they developed a three dimensional linear structural analysis program for static and dynamic loads.

Klingner and Bertero (1976) investigated the effects of engineered masonry infill panels on the seismic hysteric behaviour of reinforced concrete frames subjected to quasi-static cyclic loads. The equivalent strut idealization for an infilled frame provided an excellent predictions of observed response.

Rao *et al.*(1984) used finite element to study frames with staggered panels and compared the results with those obtained experimentally.

Thiruvengadam (1985) evaluated first few natural frequencies and associated mode shapes of infilled frames by using shear-flexure cantilever, finite element method and multiple strut analogy. In the multiple strut analogy, the infills were modelled by a set of equivalent multiple struts, which were found to predict the best observed behaviour.

May and Naji (1991) carried out nonlinear analysis of infilled frames under monotonic and cyclic loadings using finite element method. The skeletal frame was modelled with 3-noded frame element and the panel was modelled with 8-noded isoparametric element. A 6-noded interface element was used to model the interface between the frame and the infill. The results obtained by the program were in good agreement with those obtained experimentally for first cycle of loads, but in successive cycles the predicted loop was narrower.

Paul *et al.*(1991) made post failure/damage studies of buildings situated in India subjected to severe earthquakes. The improper infill disposition has been reported to result in the damage to the framing system. The 3-D dynamic analysis was proposed for the design of such buildings.

2.4 Nonlinear Analysis

Studies of the static or dynamic elasto-plastic behaviour of the frame and the infill elements are reviewed in this section. The contact problem between frame and infill exhibiting separation, sliding, opening and closing of initial gaps is also discussed.

2.4.1 Frame Nonlinearity

Johnson and Brotten (1966) analysed space frames considering the effects of finite deflections but neglecting the bending and instability effects.

Nigam (1967) proposed a section model with plastic hinge of zero length for inelastic dynamic analysis. The associated flow rule with the element was used to model post yielding response.

Chen and Powell (1982), and Powel and Chen (1986) presented a 3-D beam-column element for the concept of generalized hinge accounting for interaction among axial, torsional and biaxial bending effects. Both distributed and lumped plasticity models taking into account the effect of strain hardening and stiffness degradation were presented. The element behaved excellently for both static and dynamic loads.

Carol and Murcia (1984) developed an iterative method for nonlinear analysis of frames which included time-dependent behaviour of materials and second order effects. Longer elements could be used to discretise the structure without any loss of accuracy, which resulted in to greater economy in the analysis.

Creus *et al.*(1984) proposed an approximate elasto-plastic frame analysis with generalised yield function and finite displacements using a nonlinear tangent stiffness matrix. The interaction between bending moment, shear and axial forces at the critical sections was taken into account.

Gillies and Shepherd (1984) proposed five yield surfaces based on the interaction of:

(i) bending moment about principal axis,
(ii) bending moment and axial force,
(iii) biaxial bending and axial load,
(iv) finite post-elastic characteristics, and
(v) combination of all the above.

These yield surfaces were used successfully for the analysis of post-elastic dynamic response of three dimensional building frames subjected to seismic excitation represented by the concurrent application of orthogonal displacement-time histories.

Chandra *et al.*(1990) presented a second order analysis procedure for steel space frames taking into account both geometrical and material nonlinearities with plastic flow of discrete hinges with instantaneous secant stiffness. The plastic hinges were assumed to be concentrated at the ends of the elements.

2.4.2 Infill Nonlinearity

Little information is available in the literature on three dimensional nonlinear analysis, yield surfaces and nonlinear techniques.

Ilyushin (1956) presented approximate forms of yield surfaces for shell analysis involving the interaction of both in-plane and out of plane forces.

Dinis *et al.*(1980) used a semi-loop shell element to model plates and shells considering the material and large displacement nonlinearity. Both Lagrangian and updated formulations were used for describing large displacement response. An initial stiffness conventional plasticity algorithm and an elasto-viscoplastic models were considered for material nonlinear analysis. The accuracy of the solutions obtained using coarse mesh subdivisions indicate good convergence characteristics of the element.

Naraine and Sinha (1991) reported an experimental investigation into the behaviour of brick masonry wall under cyclic biaxial compressive loading. They proposed a failure criterion expressed in terms of principal stress invariants.

2.4.3 Interface Nonlinearity

The techniques used for modelling the interface between two nonconnected elements are reviewed here:

Goodman *et al.*(1968) formulated a four noded 2-D interface element having two translational degrees of freedom per node. The element models the separation, sliding, opening and closing of initial gaps between two elements. The element was used to model the cracks between rocks and masonry joints.

Liauw and Kwan (1984) and Thiruvengadam (1985) studied the effect of initial lack of fit and friction at the interface between non-integral infills and frames using bar elements between the frame and the infill.

Papia (1988) proposed the use of boundary element to study the interface conditions between frame and nonintegral infill.

May and Naji (1991) modified Goodman's interface element which takes into account rotational degree of freedom on frame side of the interface element to investigate the effects of separation, sliding and initial lack of fit between the frame and the infill.

2.4.4 Nonlinear Solution Techniques

The nonlinear computations are always tedious and time consuming on arrival at convergence at each time step. Some of the important contributions for accelerating nonlinear convergence have been reviewed below:

Wempner (1971) and Riks (1979) have developed a procedure which can be looked upon as an extension of the displacement control method. The procedure is advantageous both near the vicinity of critical points and allows for automatic step size adjustment in the entire range of load. Due to modifications of the original method the constraint equation does not need to be solved simultaneously with equilibrium equations.

Crisfield (1979) presented an accelerated modified Newton-Raphson iteration in which the iterative deflection change is a scalar times the previous iterative change plus a further scalar times the usual un-accelerated change. The method was based on Secant Approach and leads to a significant reduction in the required number of iterations.

Owen and Hinton (1980) discussed in detail various techniques for handling both material and large deformation nonlinearities under static and dynamic loads. The algorithms and computer programs were presented for various nonlinear solution techniques. The elasto-plastic plane stress/strain and plate bending programs were discussed in detail. For elasto-plastic dynamic response, explicit and explicit-implicit algorithms and programs were also presented.

Crisfield (1982) proposed the Secant-Newton methods for nonlinear solution procedures. The Secant methods are closely related with re-started single-cyclic forms of vectorised Quasi-Newton methods. As a consequence, in contrast to similar Conjugate-Newton methods, the Secant-Newton methods do not require accurate line searches. So these methods are efficient as compared to the popular modified Newton-Raphson method.

Bergan (1980) and Bergan (1982) proposed an automated incremental iterative solution scheme which could handle limit points and bifurcations also. With this method, it is possible to choose between load incrementation and displacement incrementation. The method iterates simultaneously for an optimal load and its corresponding equilibrium configuration.

2.5 Concluding Remarks

Based on the above review of literature, the following main points emerge:

- In the dynamic analysis of a complete building system, the inclusion of the effect of the infills is essential for a realistic prediction of the behaviour, even though, it complicates the problem and increases the computational effort.

- Only few investigations on dynamic response of 3-D infilled concrete frames have been reported in the literature.

- Amongst the models for 3-D analysis, the section models with three dimensional point hinges at the ends or Gauss points are prevalent. The point hinges are modelled either on plasticity concepts with different yield surfaces and corresponding flow rules, or are based on force-deformation characteristics of inelastic springs assumed to represent inelastic element characteristics.

- Infilled frames have been mainly analysed by using finite element method. The skeletal frame, the panel and the interface have been modelled by 3-noded frame element, 8-noded isoparmetric element and 6-noded interface element, respectively. The results obtained by this finite element model are reported to be in good agreement with the experimental results.

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CHAPTER : 3

STRUCTURAL MODELLING OF THE SYSTEM

3.1 Introduction

The behaviour of an infilled frame depends upon interaction between the infill and the frame. The composite action depends upon the quality of bonding between the two, and the stress-strain relationships of the materials. Initially the infill may be bonded to the frame by mortar. With the increase of load, the mortar may crack and there may be sliding at interface. The reversal of loading may cause opening and closing of gaps. In addition the infill may get cracked and/or crushed resulting in change in its structural behaviour. Cracking and crushing may render the infill ineffective leaving the bare frame to take all the load which may lead to the failure of the framing system itself. The modelling of all characteristics of infilled frame buildings is quite complex and requires nonlinear analysis using numerical techniques such as the finite element method. In the present investigation finite element models have been used.

3.2 Finite Element Idealisation

Two dimensional infilled frames have been analysed by using the finite element method. The skeleton frame, the panel and the interface have been modelled by 3-noded frame element, 8-noded isoparametric element and 6-noded interface element, respectively [May and Naji (1991), Choubey (1990) and Dhanasekar *et al.* (1985)]. The results obtained by this finite element model are reported to be in good agreement with the experimental results. In the present study, the above model has been used for both 2-D and 3-D analyses of infilled reinforced concrete frames.

Finite element approach based on a displacement field within the element is used. The required relations are [Hinton and Owen (1977) and Zienkiewicz (1977)]:

$$\mathbf{K}^e \delta^e = \mathbf{f}^e + \mathbf{f}_p^e \qquad ...(3.1)$$

where $$\mathbf{f}_p^e = \int_V \mathbf{N}^T \mathbf{p} dv \quad \text{and} \qquad ...(3.2)$$

$$\mathbf{K}^e = \int_V \mathbf{B}^T \mathbf{D} \mathbf{B} dv \qquad ...(3.3)$$

Here $\mathbf{f}^e$ is a vector of nodal forces, δ^e is a vector of nodal displacements, $\mathbf{f}_p^e$ is a vector of equivalent nodal forces due to a set of body forces $\mathbf{p}$, $\mathbf{K}^e$ is the stiffness matrix of the element, $\mathbf{B}$ is the strain matrix, $\mathbf{D}$ is the material property matrix and $\mathbf{N}$ is the shape function.

3.3 Modelling of Infilled Frames

In the present study, the infilled frame is modelled with isoparametric finite elements as shown in Fig. 3.1. Eight-noded panel elements, 6-noded interface elements and 3-noded frame elements have been used to idealise the infill panel, frame-infill interface and surrounding frame, respectively, as have been shown in Fig. 3.2.

3.3.1 Frame Element

The frame element used has six degrees of freedom per node [Hughes (1987)]. The displacement vector is:

$$\delta = \begin{bmatrix} u & v & w & \theta_x & \theta_y & \theta_z \end{bmatrix}^T \qquad ...(3.4)$$

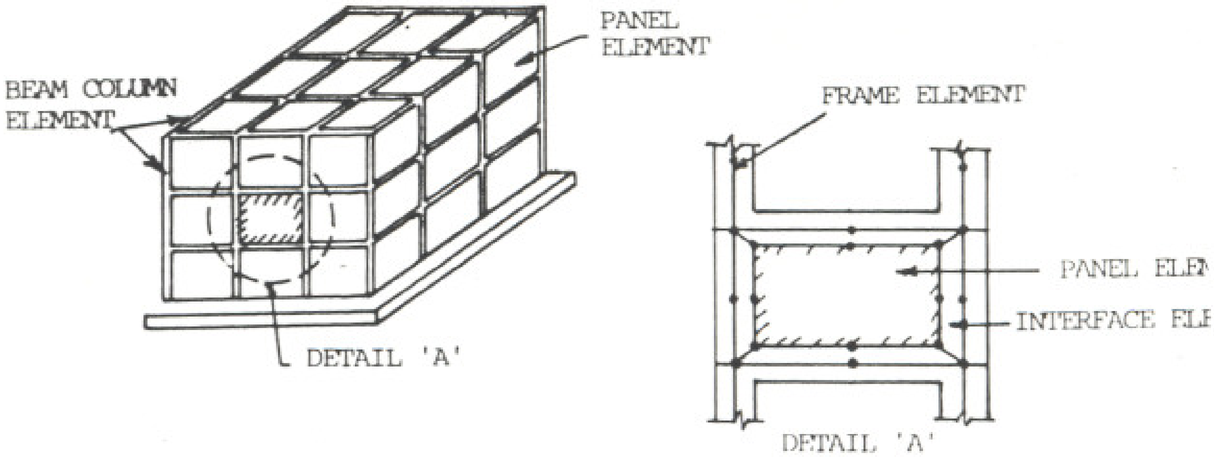


Fig. 3.1 Mathematical Model of a Building Structure.

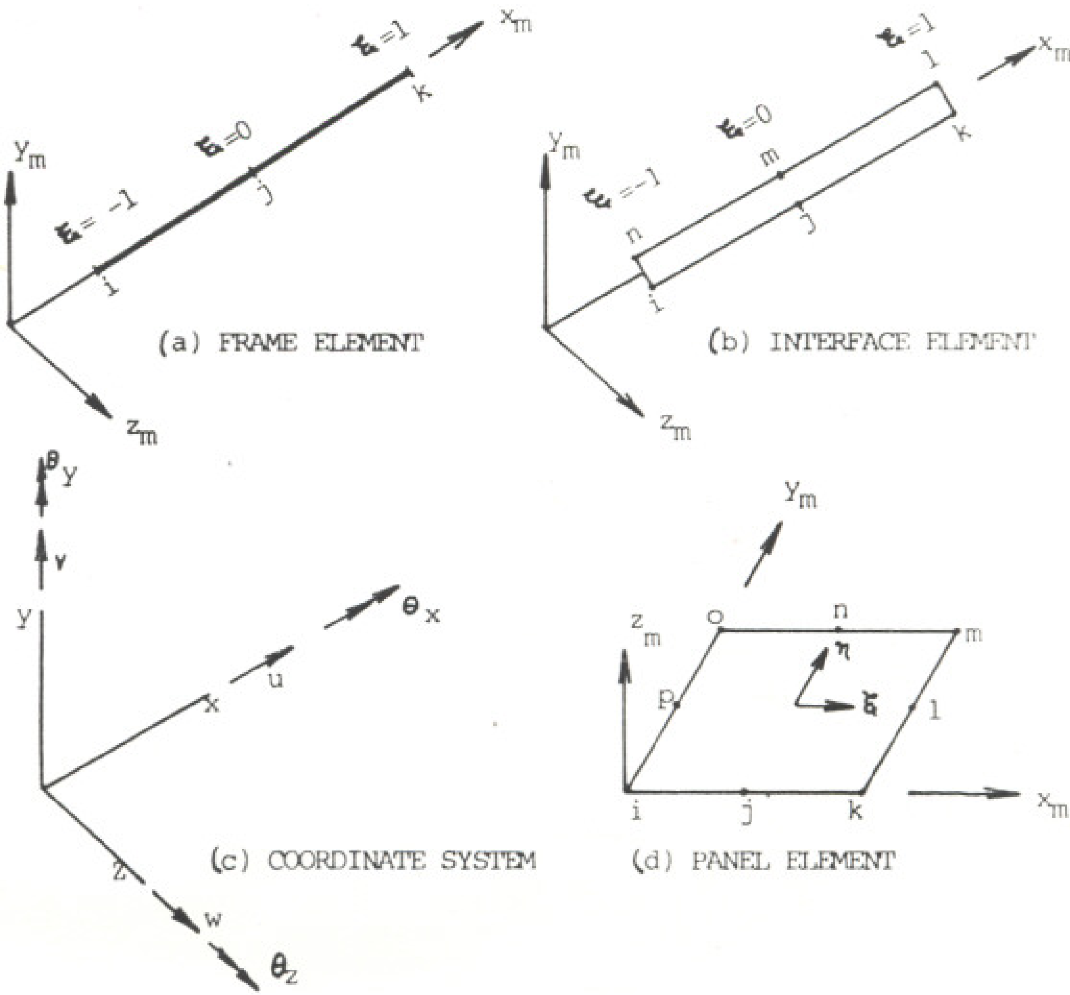


Fig. 3.2 Different Elements used for Modelling the Infilled Frame.

The strain vector is expressed as

$$\varepsilon = \left[\varepsilon_{xx} \quad \phi_{xy} \quad \phi_{xz} \quad \alpha \quad k_{xz} \quad k_{xy}\right]^T \qquad \ldots(3.5)$$

where

$$\varepsilon_{xx} = \frac{du}{dx} \qquad \text{Axial Strain} \qquad \ldots(3.6)$$

$$\left.\begin{aligned} \phi_{xy} &= \frac{dv}{dx} - \theta_z \\ \phi_{xz} &= \frac{dw}{dx} + \theta_y \end{aligned}\right\} \qquad \text{Shear strains} \qquad \ldots(3.7)$$

$$\alpha = \frac{d\theta_x}{dx} \qquad \text{Twist} \qquad \ldots(3.8)$$

$$\left.\begin{aligned} k_{xz} &= \frac{d\theta_y}{dx} \\ k_{xy} &= \frac{d\theta_z}{dx} \end{aligned}\right\} \qquad \text{Curvature or bending strain} \qquad \ldots(3.9)$$

For the 3-noded isoparametric frame element shown in Fig. 3.2(a) the shape functions are expressed as:

$$\begin{aligned} N_1(\xi) &= \frac{1}{2}\,\xi(\xi-1) \\ N_2(\xi) &= 1-\xi^2 \\ N_3(\xi) &= \frac{1}{2}\,\xi(\xi+1) \end{aligned} \qquad \ldots(3.10)$$

The stiffness matrix of a 3-D frame element is expressed as:

$$K^e = K_a^e + K_s^e + K_t^e + K_b^e \qquad \ldots(3.11)$$

where

$$K_a^e = \int B^{a^T} D^a B^a dx \qquad \text{(axial stiffness)} \qquad \ldots(3.12)$$

$$K_s^e = \int B^{s^T} D^s B^s dx \qquad \text{(shear stiffness)} \qquad \ldots(3.13)$$

$$K_t^e = \int B^{t^T} D^t B^t \, dx \qquad \text{(torsional stiffness)} \qquad \ldots(3.14)$$

$$K_b^e = \int B^{b^T} D^b B^b dx \qquad \text{(bending stiffness)} \qquad \ldots(3.15)$$

The equivalent nodal force vector of an element is given by :

$$f_p^e = \left\{ f_p^e \right\} \qquad \ldots(3.16)$$

$$\text{where} \quad f_p^e = \int N_i F_j \, dx \quad \text{for} \quad p = 6i-6+j$$

$$\text{and} \quad f_p^e = \int N_i C_k \, dx \quad \text{for} \quad p = 6i-3+k \qquad \ldots(3.17)$$

where $1 \le i \le n$, n is the number of element nodes, F_j is the applied force per unit length along j^{th} translational degree of freedom ($1 \le j \le 3$) and C_k is the applied couple per unit length along k^{th} rotational degree of freedom ($1 \le k \le 3$).

The axial, shear, torsional and bending strain-displacement matrices are defined as follows :

$$B^a = \left[B_1^a \; B_2^a \ldots\ldots \; B_n^a \right] \qquad \ldots(3.18)$$

$$B^s = \left[B_1^s \; B_2^s \ldots\ldots \; B_n^s \right] \qquad \ldots(3.19)$$

$$B^t = \left[B_1^t \; B_2^t \ldots\ldots \; B_n^t \right] \qquad \ldots(3.20)$$

$$B^b = \left[B_1^b \; B_2^b \ldots\ldots \; B_n^b \right] \qquad \ldots(3.21)$$

where n is the number of element nodes and

$$B_i^a = \left[N_i' \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \right] \qquad \ldots(3.22)$$

$$B_i^s = \begin{bmatrix} 0 & N_i' & 0 & 0 & 0 & -N_i \\ 0 & 0 & N_i' & 0 & N_i & 0 \end{bmatrix} \qquad \ldots(3.23)$$

$$B_i^t = \begin{bmatrix} 0 & 0 & 0 & N_i' & 0 & 0 \end{bmatrix} \qquad \text{...(3.24)}$$

$$B_i^b = \begin{bmatrix} 0 & 0 & 0 & 0 & N_i' & 0 \\ 0 & 0 & 0 & 0 & 0 & N_i' \end{bmatrix} \qquad \text{...(3.25)}$$

where $N_i' = \frac{dN_i}{dx}$ and $1 \leq i \leq n$

The material modulii matrices are defined as:

$$D^a = EA \qquad \text{...(3.26)}$$

$$D^s = \begin{bmatrix} S_{xy} & 0 \\ 0 & S_{xz} \end{bmatrix} \qquad \text{...(3.27)}$$

where $S_{xy} = S_{xz} = GA_s$ and $A_s = A/1.2$ (for rectangular section)

$$D^t = GI_{xx} \qquad \text{...(3.28)}$$

$$D^b = \begin{bmatrix} EI_{yy} & 0 \\ 0 & EI_{zz} \end{bmatrix} \qquad \text{...(3.29)}$$

However the concrete is not a purely elastic material. The plastic flow (creep) has been observed in it. The modulus of elasticity varies with stress rate and magnitude of the stress. The effective reinforced concrete section also varies with the stress level. Both the modulus of elasticity and the effective cross section decrease with the increase in stress level. In the 'elastic' range, either their values should be varied or an average value may be used. The reduction of the elastic rigidity EI by 50 per cent has been suggested by many researchers to define an average value [Anderson and Townsend (1977), Saatcioglu (1984), and Moazzami and Bertero (1987)]. In the present study, 50 per cent reduction in the short term value of static modulus of elasticity of concrete and effective sectional properties both calculated as per

IS: 456-1978 have been assumed for the entire 'elastic' range prior to the development of ultimate yield surface.

The element stiffness matrix has been calculated using selective integration. The non-shear terms are integrated using normal integration with three point Gauss quadrature. The shear terms are evaluated using reduced integration (two point Gauss quadrature) and are extrapolated to match with the integration of other terms.

The stiffness matrix of the beam element in global coordinate system is given by

$$\underline{K}^e = T^T K^e \ T \qquad \ldots(3.30)$$

where **T** is a diagonal transformation matrix of size equal to the size of the element stiffness matrix and is defined as:

$$T = \begin{bmatrix} R & & & & \\ & R & & & \\ & & R & & \\ & & & \ddots & \\ & & & & R \end{bmatrix} \qquad \ldots(3.31)$$

The sub-matrix **R** (3x3) is the member rotation matrix which can be obtained by successive planer rotations from the reference axes (X, Y, Z) to the member axes (x_m, y_m, z_m). Figure 3.3 shows Y-Z-X transformation [Beaufait *et al.*(1970), Krishnamoorthy (1987)]. The reference axes are rotated through angles β, γ and α successively as shown in the Fig. 3.3 to make them coincide with the member axes. The sub-matrix **R** is then obtained as:

$$R = \begin{bmatrix} C_x & C_y & C_z \\ \dfrac{-C_xC_y\cos\alpha - C_z\sin\alpha}{C_{xz}} & C_{xz}\cos\alpha & \dfrac{-C_yC_z\cos\alpha + C_x\sin\alpha}{C_{xz}} \\ \dfrac{C_xC_y\sin\alpha - C_z\cos\alpha}{C_{xz}} & -C_{xz}\sin\alpha & \dfrac{C_yC_z\sin\alpha + C_x\cos\alpha}{C_{xz}} \end{bmatrix} \qquad \ldots(3.32)$$

where $C_x=(X_j - X_i)/L$, $C_y=(Y_j - Y_i)/L$ and $C_z=(Z_j - Z_i)/L$ are the

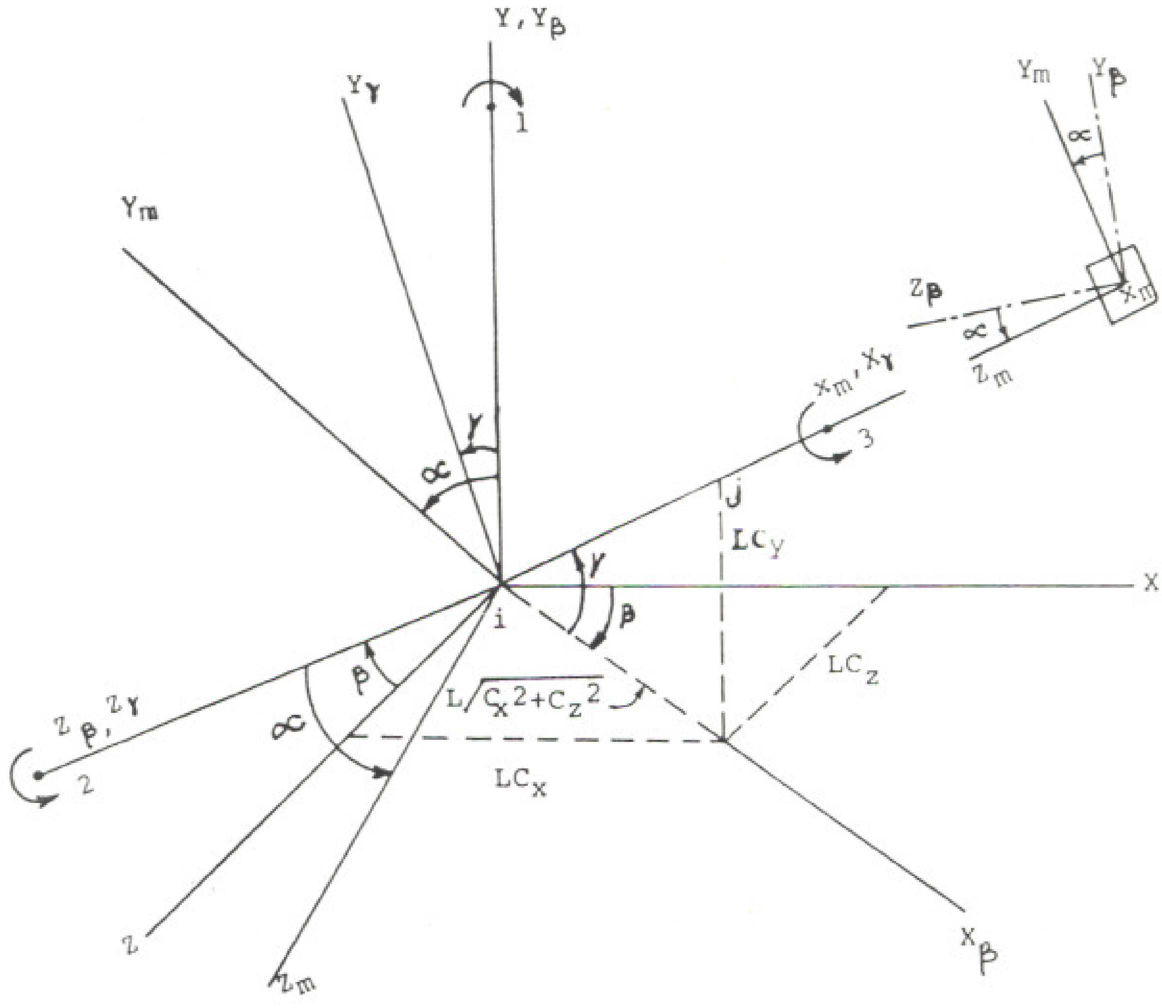


Fig. 3.3 Rotation of Axes in Space: Y-Z-X Transformation.

direction cosines of the member i-j having (X_i, Y_i, Z_i) and (X_j, Y_j, Z_j) as the coordinates of its ends i and j. The term C_{xz} is equal to $(C_x^2 + C_z^2)^{1/2}$. The angle α is the angle between the Y_β and y_m axes or between the Z_β and z_m axes measured in a counter-clockwise direction while viewing the cross-section of the member in a negative x_m direction as shown in Fig. 3.3.

If a member is positioned in the frame of the reference axes such that its longitudinal axis x_m corresponds to the Y-direction of the reference coordinate system, the rotation matrix for Y-Z-X transformation given by Eqn (3.32) can not be defined due to the fact that C_x and C_z become zero. In such cases the rotation matrix is given by,

$$R = \begin{bmatrix} 0 & C_y & 0 \\ -C_y\cos\alpha & 0 & \sin\alpha \\ C_y\sin\alpha & 0 & \cos\alpha \end{bmatrix} \quad \ldots(3.33)$$

It is seen that when describing the orientation of a particular structural member in the frame of the X-Y-Z global axes, the three direction cosines C_x, C_y and C_z define the location of the longitudinal x_m axis and the α or μ angle defines the location of the minor principal axis.

3.3.2 Panel Element

The out-of-plane stiffness of the unreinforced masonry panels is very low as compared to its in-plane stiffness. In the present study only in-plane stiffness has been taken into consideration. For the panel element two in-plane translational degrees of freedom per node have been considered. The displacement vector is

$$\delta = [u \quad v]^T \quad \ldots(3.34)$$

The strain vector is defined as:

$$\varepsilon = \begin{bmatrix} \varepsilon_x & \varepsilon_y & \gamma_{xy} \end{bmatrix}^T \quad \ldots(3.35)$$

For small displacements the strains are given as

$$\varepsilon_x = \frac{\partial u}{\partial x}, \quad \varepsilon_y = \frac{\partial v}{\partial y} \qquad \ldots(3.36)$$

and the shear strain is given as :

$$\gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \qquad \ldots(3.37)$$

$$\sigma = D\varepsilon \qquad \ldots(3.38)$$

where $$\sigma = \begin{bmatrix} \sigma_x & \sigma_y & \tau_{xy} \end{bmatrix}^T \qquad \ldots(3.39)$$

in which σ_x and σ_y are the normal stresses and τ_{xy} is the shear stress. For linear elastic analysis the stress-strain or constitutive matrix is given by:

$$D = [E/(1-\upsilon^2)] \begin{bmatrix} 1 & \upsilon & 0 \\ \upsilon & 1 & 0 \\ 0 & 0 & (1-\upsilon)/2 \end{bmatrix} \qquad \ldots(3.40)$$

in which E and υ are the elastic modulus and Poisson's ratio respectively. The stiffness of panel element is given by

$$K^e = \int B^T D B \, dv \qquad \ldots(3.41)$$

or $$K^e = \int B^T D B \, t \, dxdy \qquad \ldots(3.42)$$

where t is the thickness of the element. The stiffness of the panel element in global coordinates is given by

$$\underline{K}^e = T^T K^e T \qquad \ldots(3.43)$$

where **T** is the transformation matrix defined by Eqn. (3.31). For the 8-noded isoparametric element shown in Fig.3.2(d) used in the present study, the shape functions of the element are given by for corner nodes :

$$N_i^e = \frac{1}{4}(1+\xi\xi_i)(1+\eta\eta_i)(\xi\xi_i+\eta\eta_i-1) \qquad i = 1,3,5,7$$

...(3.44)

for mid side nodes:

$$N_i^e = \frac{\xi_i^2}{2}(1+\xi\xi_i)(1-\eta^2)+\frac{\eta_i^2}{2}(1+\eta\eta_i)(1-\xi^2) \qquad i = 2,4,6,8$$

...(3.45)

Normal integration of an order of 3x3 is performed to calculate the stiffness matrix of the panel element.

3.3.3 Interface Element

This element has been used between the frame element having six local degrees of freedom per node and the panel element having two local translational degrees of freedom per node to model the interface conditions between the two types of elements. For compatibility only two in-plane translational degrees of freedom per node have been considered. The 'thickness' of the element has been taken equal to the distance of the panel edge from the neutral axis of the adjacent frame element to incorporate the moment induced at the neutral axis of the frame due to the friction on the edge of the panel. The displacement vector is

$$\delta = \begin{bmatrix} u & v \end{bmatrix}^T \qquad ...(3.46)$$

The strains are the relative displacements at the top and the bottom of the element. The strain vector is defined as

$$\varepsilon = \begin{bmatrix} \Delta u & \Delta v \end{bmatrix}^T \qquad ...(3.47)$$

where

$$\Delta u = u_{top} - u_{bot} = N_i(u_{top})_i - N_i(u_{bot})_i$$

$$\Delta v = v_{top} - v_{bot} = N_i(v_{top})_i - N_i(v_{bot})_i \qquad ...(3.48)$$

where N_i is shape function.

The relevant stress vector is

$$\sigma = \begin{bmatrix} \sigma_u & \sigma_v \end{bmatrix}^T \qquad ...(3.49)$$

The material modulus matrix is defined as

$$D = \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix} \qquad ...(3.50)$$

where k_s and k_n are the shear and the normal stiffness coefficients, respectively. The strain matrix is defined as

$$B = \begin{bmatrix} -IN_1 & -IN_2 & -IN_3 & IN_3 & IN_2 & IN_1 \end{bmatrix} \qquad ...(3.51)$$

where **I** is identity matrix of order 2x2. The stiffness matrix is calculated as

$$K^e = \int B^T D B dx \qquad ...(3.52)$$

and the stiffness matrix in global coordinate system is calculated as

$$\underline{K}^e = T^T K^e T \qquad ...(3.53)$$

where **T** is transformation matrix defined by Eqn. (3.31). The three point Gauss quadrature rule is used to calculate the stiffness matrix.

3.4 Non-linear Analysis

The response of an infilled frame depends upon the interaction between the infill and the frame. There can be separation, closing of gap and slipping between the frame and the infill. The frame and the infill may exhibit material nonlinearity. To incorporate all these phenomena of interaction a nonlinear analysis has been adopted.

3.4.1 Reinforced Concrete Frame

For reinforced concrete frames section models have been used in preference to the fibre models because of their computational efficiency [Chen and Powell (1982)]. A 3-noded 3-D beam-column element with selective integration has been used. The element stiffness has been calculated using the selective integration. The non-shear terms are integrated using normal integration (three point Gauss quadrature). The shear terms are evaluated using the reduced integration (two point Gauss quadrature) and are extrapolated to match with the integration of other terms. Hinges have been assumed to form at the points of integration which are distributed over the length of the element. One Gauss point is in the centre of the element and other two near the ends. In reinforced concrete framed structures, the frame elements are stiffer near the ends due to joint stiffnesses. So it is appropriate to assume the formation of hinges near the ends of the elements.

Yield Surfaces: Inelastic behaviour of the element is governed by the axial force, two flexural moments and a torsional moment. Chen and Powell (1982), proposed five yield (interaction) surfaces. The surfaces differ, however, mainly in the manner in which the axial force interacts with the three moments. Powell and Chen(1986) have shown that the yield surface given by:

$$f = \left[(M_x/M_{xu})^2 + (M_y/M_{yu})^2 + (M_z/M_{zu})^2\right]^{1/2} + \left[F_x/F_{xu}\right]^n \qquad \ldots(3.54)$$

gives acceptable results in a wide range of practical domain. Here M_x, M_y, M_z are the moments about the X, Y and Z axes, respectively, F_x is the axial force, and M_{xu}, M_{yu}, M_{zu} are the corresponding yield moments; F_{xu} is the axial yield force; and the exponent n is of the order of 2. This yield surface has been used in the investigations reported in the thesis.

Matrix Formulation: The yield criterion determines the stress level at which plastic deformation begins and is written in the general form

$$f(\sigma) = k(\kappa) \qquad \ldots(3.55)$$

where σ is the stress vector, κ is the hardening parameter which governs the expansion of the yield surface.

Rearranging Eqn. (3.55);

$$F(\sigma,\kappa) = f(\sigma)-k(\kappa) = 0 \qquad \ldots(3.56)$$

By differentiating

$$dF = \frac{\partial F}{\partial \sigma} d\sigma + \frac{\partial F}{\partial \kappa} d\kappa = 0 \qquad \ldots(3.57)$$

or $$\mathbf{a}^T d\sigma - A\, d\lambda = 0 \qquad \ldots(3.58)$$

where $\mathbf{a}^T = \frac{\partial F}{\partial \sigma}$ and $A = -\frac{1}{d\lambda}\frac{\partial F}{\partial \kappa} d\kappa$

and $$\mathbf{a}^T = \left[\frac{\partial F}{\partial F_x} \quad \frac{\partial F}{\partial F_y} \quad \frac{\partial F}{\partial F_z} \quad \frac{\partial F}{\partial M_x} \quad \frac{\partial F}{\partial M_y} \quad \frac{\partial F}{\partial M_z}\right]$$

or $$\mathbf{a}^T = \left[\frac{nF_x^{n-1}}{F_{xu}^n} \quad 0 \quad 0 \quad \frac{M_x}{CM_{xu}} \quad \frac{M_y}{CM_{yu}} \quad \frac{M_z}{CM_{zu}}\right]$$

where $$C = \left[(M_x/M_{xu})^2 + (M_y/M_{yu})^2 + (M_z/M_{zu})^2\right]^{1/2}$$

The complete elasto-plastic incremental stress-strain relationship can be written as [Owen and Hinton (1980)]

$$d\sigma = \mathbf{D}_{ep} d\varepsilon \qquad \ldots(3.59)$$

where $$\mathbf{D}_{ep} = \mathbf{D} - \frac{\mathbf{d}_D \cdot \mathbf{d}_D^T}{A + \mathbf{d}_D^T \mathbf{a}} \qquad \ldots(3.60)$$

with $\mathbf{d}_D = \mathbf{D.a}$ and $A = H' = \frac{d\sigma}{d\varepsilon_p}$ = tangent to the effective stress-plastic strain curve and is a function of accumulated effective plastic strain ε_p.

In the present study, the yield moments and axial forces for the reinforced concrete section have been calculated from the appropriate charts given in SP: 16 (1980).

3.4.2 Brick Masonry Infill

Masonry is a complex material consisting of an assemblage of bricks and mortar joints, each with differing properties. Its behaviour is made more complex by the mortar joints acting as planes of weakness due to their low tensile, shear and bond strengths. The material is assumed to be linearly elastic till failure. In compression, on crushing the stress-strain matrix is

$$D_{ep} = \begin{bmatrix} \phi E & 0 & 0 \\ 0 & \phi E & 0 \\ 0 & 0 & \phi G \end{bmatrix} \qquad \ldots(3.61)$$

where all stresses are reduced to zero. In tension, on cracking the stiffness normal to crack is reduced to zero but along the crack partial shear stiffness is maintained. The material property matrix used is:

$$D_{ep} = T^T \begin{bmatrix} \phi E & 0 & 0 \\ 0 & E & 0 \\ 0 & 0 & \beta G \end{bmatrix} T \qquad \ldots(3.62)$$

(in the present investigations the coefficients used are $\phi = 10^{-3}$ and $\beta = 0.25$)

where
$$T = \begin{bmatrix} \cos^2\alpha & \sin^2\alpha & \sin\alpha\cos\alpha \\ \sin^2\alpha & \cos^2\alpha & -\sin\alpha\cos\alpha \\ -2\sin\alpha\cos\alpha & 2\sin\alpha\cos\alpha & \cos^2\alpha - \sin^2\alpha \end{bmatrix}$$

...(3.63)

Here α is the angle between the normal to the crack surface and the X-axis. The stress normal to the crack is reduced to zero, however, a partial shear transfer due to interlocking between the particles is maintained. The stiffness and stresses along the crack are also maintained.

3.4.3 Concrete Mortar Interface

There can be different interface conditions between the infill and the frame. The different interface conditions can be modelled by adjusting the interface stiffness co-efficients. When the normal strain is tensile, a separation is assumed to be taking place and normal stiffness, k_n is taken to be zero. Since a separated interface cannot take up any shearing stress the shear stiffness k_s, is also taken to be zero. If normal strain is compressive, the interface is assumed to be in contact and a very high value is assigned to k_n.

The tangential stress-strain relationship is assumed to be elastic-perfectly plastic [Choubey (1990), and May and Naji (1991)] using a Mohr-Coulomb yield criterion with zero cohesion. When $|\sigma_u| < |\mu\sigma_v|$, firm contact is assumed and k_s is assigned the value obtained from the tangential stress-relative displacement curves resulting from shear box tests [Choubey (1990), and May and Naji (1991)]. Here μ is the coefficient of friction. Slip takes place whenever $|\sigma_u|$ exceeds $|\mu\sigma_v|$. In such a case σ_u is reduced to $\mu\sigma_v$ and k_s is taken as zero. Values of stiffness coefficients k_s and k_n used in the analysis for different interface conditions are listed in Table 3.1

Table 3.1 : Selection of Interface Stiffness Coefficients

Interface conditions	Stiffness coefficients	
	k_s	k_n
Firm contact: $|\sigma_u| < |\mu\sigma_v|$	Experimental	Very high value
Contact with slip: Δv-compressive $|\sigma_u| \geq |\mu\sigma_v|$	Very low value	Very high value
Separation or initial lack of fit: Δv-tensile	Very low value	Very low value

3.4.4 Inelastic Analysis

For the inelastic static analysis, an incremental iterative procedure has been adopted while for dynamic analysis, the procedure presented in Sec. 3.5 is followed. Initially, element forces or stresses are calculated assuming an elastic behaviour for each element. The stress components and/or strains at Gauss points are examined and cracking, yielding, and separation is checked. When any of the above events has occurred, the forces/stresses are reduced to the yield surfaces and the equivalent nodal forces for the element are calculated as

$$f_j^e = \int_V B^T \sigma \, dv \qquad \ldots(3.64)$$

The equivalent nodal forces for the system are given as

$$f_j = \sum f_j^e \qquad \ldots(3.65)$$

The residual force vector at an iteration is calculated as

$$r_{qi} = f - f_j \qquad \ldots(3.66)$$

where f is the load vector at the current iteration and f_j is the equivalent nodal load vector due to the internal stresses reduced within the yield surface.

Convergence Criterion: The convergence criterion employed is based on the ratio of the norms of unbalanced load to the norms of total load i.e.

$$\frac{\left[\sum_{i=1}^{N} r_{qi}^2\right]^{1/2}}{\left[\sum_{i=1}^{N} Q_i^2\right]^{1/2}} \le \varepsilon_{to} \qquad \ldots(3.67)$$

where r_{qi} and Q_i are unbalanced load and total load corresponding to the i^{th} degree of freedom and ε_{to} is the tolerance.

The analysis is terminated when the stiffness matrix becomes non-positive definite indicating a condition of mechanism.

3.5 Time Integration Scheme

The equation of motion for an elasto-plastic system obtained from the consideration of equilibrium of forces is given by :

$$\mathbf{M}\,\ddot{u} + \mathbf{q}(u, \dot{u}) = \mathbf{f} \qquad \ldots(3.68)$$

where $\mathbf{q}$ is the vector of internal resisting forces which depends upon the displacement $\mathbf{u}$ and velocity $\dot{u}$, $\mathbf{M}$ is the mass matrix of the system, $\ddot{u}$ is the acceleration vector and $\mathbf{f}$ is the externally applied load vector. The internal resisting forces are defined by the stiffness matrix $\mathbf{K}$ and damping matrix $\mathbf{C}$. The direct integration of Eqn. (3.68) has been carried out using a numerical step by step procedure. Newmark's predictor-corrector (implicit method) has been selected for dynamic solution.

3.5.1 Newmark's Predictor-Corrector Implicit Scheme

In the Newmark's scheme, the following relations are defined :

$$\mathbf{M}\,\ddot{u}_{t+\Delta t} + \mathbf{q}(u_{t+\Delta t}, \dot{u}_{t+\Delta t}) = \mathbf{f}_{t+\Delta t} \qquad \ldots(3.69)$$

where

$$u_{t+\Delta t} = \bar{u}_{t+\Delta t} + (\Delta t)^2 \beta\, \ddot{u}_{t+\Delta t} \qquad \ldots(3.70)$$

$$\dot{u}_{t+\Delta t} = \bar{\dot{u}}_{t+\Delta t} + \Delta t\, \gamma\, \ddot{u}_{t+\Delta t} \qquad \ldots(3.71)$$

$$\bar{u}_{t+\Delta t} = u_t + \Delta t\, \dot{u}_t + 0.5(\Delta t)^2 (1-2\beta)\, \ddot{u}_t \qquad \ldots(3.72)$$

$$\bar{\dot{u}}_{t+\Delta t} = \dot{u}_t + \Delta t(1 - \gamma)\, \ddot{u}_t \qquad \ldots(3.73)$$

Here β and γ are the parameters which control the accuracy and stability of the method. In this thesis, β and γ are assumed to be equal to 0.25 and 0.5, respectively (as used in average-acceleration method). The quantities $\bar{u}_{t+\Delta t}$, $\bar{\dot{u}}_{t+\Delta t}$ are the historical values and $u_{t+\Delta t}$ and $\dot{u}_{t+\Delta t}$ are the corrector values.

For starting the algorithm, the initial values of accelerations $\ddot{u}_o$, are obtained by solving Eqn. (3.68) at time t = 0 as

$$\ddot{u}_o = M^{-1}[f_o - q(u_o, \dot{u}_o)] \qquad ...(3.74)$$

where $\mathbf{f}_o$ is the applied load vector at time t=0.

The solution for the linear case is obtained by reducing the relations (3.69) to (3.73) to a recurrence relation which involves effective static solutions at intervals of Δt apart. The inelastic solution is obtained in the same way as explained above except that the stiffness matrix and damping matrix are reformulated to take into account the effect of any topological change in the structure due to formation of plastic hinges in the frames and/or post cracking and/or yielding that may occur in the infills.

3.5.2 Newmark's Implicit Predictor-corrector Algorithm [Owen and Hinton (1980)]

The Newmark's algorithm for each time step is applied as follows:

1. Set iteration counter j=0.
2. Predict displacements, velocities and accelerations by using past history at the previous time step as:

$$u^j_{t+\Delta t} = \bar{u}_{t+\Delta t} = u_t + \Delta t\ \dot{u}_t + 0.5(\Delta t^2)(1-2\beta)\ddot{u}_t \qquad ...(3.75)$$

$$\dot{u}^j_{t+\Delta t} = \dot{\bar{u}}_{t+\Delta t} = \dot{u}_t + \Delta t(1-\gamma)\ddot{u}_t \qquad ...(3.76)$$

$$\ddot{u}^j_{t+\Delta t} = (u^j_{t+\Delta t} - \bar{u}_{t+\Delta t})/(\Delta t^2\beta) = 0 \qquad ...(3.77)$$

3. Evaluate residual forces $\mathbf{r}^j$ using the following equations:

$$r^j = f_{t+\Delta t} - M\ \ddot{u}^j_{t+\Delta t} - C^j\ \dot{u}^j_{t+\Delta t} - K^j\ u^j_{t+\Delta t} \qquad ...(3.78)$$

The matrix K is evaluated by considering new events (plastic hinges, cracking, yielding and crushing etc.) in the structure.

4. If required, form the modified effective stiffness matrix using the relation:

$$K^{*} = M/(\Delta t^{2}\beta) + \gamma\, C^{j}/(\Delta t\ \beta) + K^{j} \qquad \ldots(3.79)$$

5. Solve for the incremental displacements

$$K^{*}\ du^{j} = r^{j} \qquad \ldots(3.80)$$

6. Update the displacements, velocities and accelerations as

$$u^{j+1}_{t+\Delta t} = u^{j}_{t+\Delta t} + du^{j} \qquad \ldots(3.81)$$

$$\ddot{u}^{j+1}_{t+\Delta t} = (u^{j+1}_{t+\Delta t} - \bar{u}_{t+\Delta t})/(\Delta t^{2}\beta) \qquad \ldots(3.82)$$

$$\dot{u}^{j+1}_{t+\Delta t} = \dot{u}^{j}_{t+\Delta t} + \Delta t\ \gamma\ \ddot{u}^{j+1}_{t+\Delta t} \qquad \ldots(3.83)$$

7. If du^{j} and/or r^{j} do not satisfy the convergence condition then set $j=j+1$ and go to step 3; otherwise continue.

8. Set $u_{t+\Delta t} = u^{j+1}_{t+\Delta t}$, $\dot{u}_{t+\Delta t} = \dot{u}^{j+1}_{t+\Delta t}$, $\ddot{u}_{t+\Delta t} = \ddot{u}^{j+1}_{t+\Delta t}$ for use in the next time step. Also set $t= t+\Delta t$ to begin the next step.

3.5.3 Element Mass Matrix

If the shape functions used to describe the variation of the acceleration field over the element are the same as those used to describe the displacement variation then the corresponding mass matrix is known as the consistent mass matrix. The consistent mass matrix for an element is given by

$$M^{e} = \int_{v} N^{T} \rho N dv \qquad \ldots(3.84)$$

where **N** is a matrix of shape functions and ρ is mass density.

For lumped mass matrix the diagonal elements of consistent mass matrix are scaled [Hinton et al. (1976)] to preserve total mass corresponding to each degree of freedom.

The mass matrix of an element corresponding to the global coordinates is given by

$$\underline{M}^e = T^T M^e T \qquad \ldots(3.85)$$

where **T** is the transformation matrix for the element defined by Eqn. (3.31).

For the panel elements the consistent mass matrix given by Eqn.(3.84) can be expressed as

$$M^e = \rho t \iint N^T N \, dxdy \qquad \ldots(3.86)$$

where t is the thickness of the element.

For the beam elements the consistent mass matrix can be calculated as

$$M^e = \int N^T C \, N dx \qquad \ldots(3.87)$$

here **C** is a diagonal matrix given by

$$C = \rho \, \mathrm{Diag} \begin{bmatrix} A & A & A & I_x & I_y & I_z \end{bmatrix} \qquad \ldots(3.88)$$

where A is the area of cross section, I_x, I_y and I_z are rotatory inertia about X, Y and Z axes, respectively.

3.5.4 Damping Matrix

Very limited information is available on damping in linear solid mechanics problems and there is even less data available for damping in nonlinear solutions. If the total damping in a structure is assumed to be the sum of the damping of the individual modes present in the system response, a reasonable numerical damping matrix can be derived [Wilson (1972)].

If two representative modes are considered then so called Rayleigh's damping is a linear combination of the mass and stiffness matrices so that

$$C = a_o M + a_1 K \qquad \ldots(3.89)$$

The main advantage of Rayleigh's damping is that it leads to a banded damping matrix, with the same structure as that of stiffness matrix. Thus, consideration of damping does not cause any increase in computational effort while solving the resulting set of equations. However, the use of Rayleigh's damping results in higher damping in the higher modes. This implies that higher modes in the response of the structure will be artificially filtered out. In the investigations reported in the thesis Rayleigh's damping has been assumed.

3.6 Computer Codification: The Program NIFAP

The algorithms described in the previous sections have been codified and a finite element program (**NIFAP** - **N**onlinear **I**nfilled **F**rame **A**nalysis **P**rogram) has been developed for the static and dynamic analyses of three dimensional infilled frame systems of arbitrary configuration. The analysis options provided are:

(i) linear static,
(ii) nonlinear static,
(iii) frequency calculation,
(iv) linear response history analysis using step-by-step direct integration, and
(v) nonlinear response history analysis using step-by-step direct integration.

Two independent horizontal plus a vertical ground motions may be specified. The block diagram of the program has been presented in the

Fig. 3.4.

The program presently contains the following types of elements:

(a) three dimensional three noded frame element,
(b) three dimensional eight noded plane stress element,
(c) three dimensional six noded interface element, and
(d) three dimensional truss element.

These structural elements can be used for linear elastic and nonlinear inelastic static and dynamic analyses. The capacity of the program depends mainly on the total number of nodal points in the system, the number of eigen values required in the dynamic analysis, operating system and type of computer used. The program can efficiently use either consistent or lumped mass matrix for the structural system. Each nodal point in the system can acquire degrees of freedom ranging from zero to six. The element stiffness and mass matrices are assembled in condensed form, therefore the program is equally efficient in the analysis of one, two or three dimensional systems.

The program uses the profile solver [Bathe (1990)]. For eigen value solution, the subspace iteration method [Bathe (1971) and Bathe (1990)] has been used. For nonlinear response history analysis Newmark's Predictor-Corrector Implicit scheme [Paul and Hinton (1980) and Owen and Hinton (1980)] has been used. The Newmark method [Newmark (1959) and Bathe (1990)] has been used for linear response history analysis.

The program indicates formation of hinges in the frame and cracks in the infills at the Gauss points. It calculates plastic strains, if any, at these points and modifies the stiffness matrix accordingly. It calculates displacements and acceleration history at all and the specified nodal points and stress history at the Gauss points at the specified time intervals.

3.7 Validation of Computer Program

To establish the validity of the proposed formulation a number of different types of structures which have been previously studied by

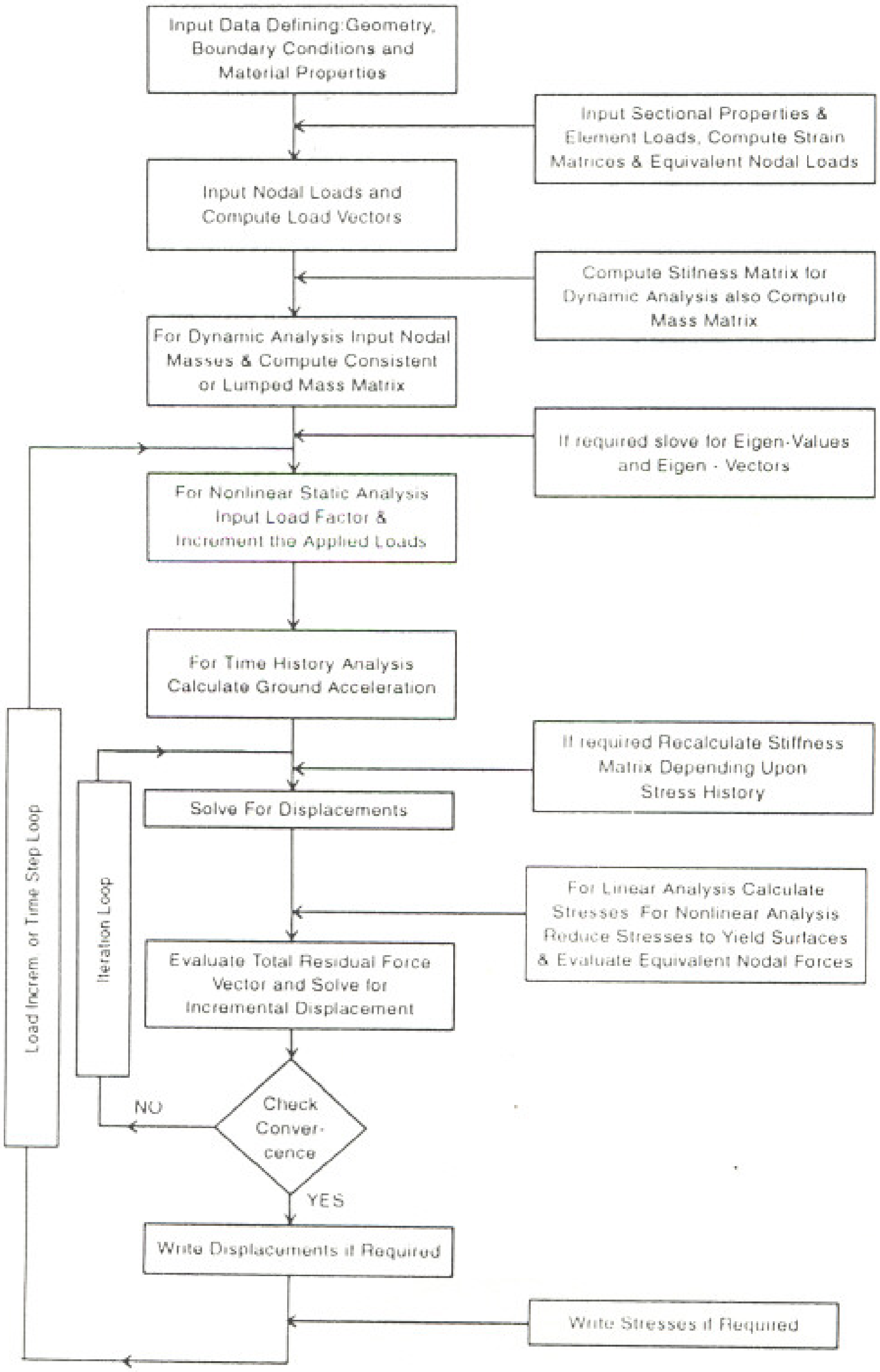


Fig. 3.4 Block Diagram of the Computer Program (NIFAP).

other investigators have been analysed. The analytical results have been compared with experimental results reported in the literature. The structures studied include, bare space and infilled 2-D frames. Dynamic analysis has been performed to check the validity of the time integration algorithm implemented in the program.

3.7.1 Elastic Analysis

□ **Space Frame:** The space frame shown in Fig. 3.5(a) has been analysed to check the validity of the time-integration formulation. This 3-D structure has been previously investigated by Weaver and Johnston (1987). The frame has prismatic members having solid square cross-sections with sides equal to 2.54 mm. The plot in Fig. 3.5(b) represents the forcing function of the ground acceleration in the X-direction which is used in the analysis. The geometric data used is shown in the figure. The previous investigators used normal mode method to calculate the dynamic response. In the present study the time step Δt equal to 0.002 S has been used to perform the time integration analysis. The plots in Fig. 3.5(c) represent the translation response at node 2 in the X, Y and Z directions. A good agreement with the reported results has been observed.

□ **Two-Storey Infilled Frame:** The two-storey infilled frame shown in Fig. 3.6 has been analysed to check the mass matrix formulations and eigenvalue solution. This example has been taken from the experimental and analytical investigations reported by Mallick and Severn (1968) and Thiruvegandam (1985). The frame is made up of members with square steel sections filled with mortar. The first two frequencies have been compared in Fig. 3.6(c). A good agreement with the experimental results is observed.

3.7.2 Inelastic Analysis

To check the validity of the inelastic algorithm and the procedure used in the analysis of the infilled reinforced concrete frames, the following structures have been chosen.

□ **Test Structure 1 - A Portal Frame:** The reinforced concrete portal frame shown in Fig. 3.7(a) tested by Bertero and McGlure (1964) and analysed by Sharma (1983) and Thanoon (1993) has been taken as test structure 1. The frame has been assumed to be fixed at base and idealised as shown in Fig. 3.7(b). The geometry, loads and properties are shown in the figure. The load deflection curve obtained experimentally by Bertero and McGlure (1964) and analytically by Sharma (1983) and Thanoon (1993) are compared with that obtained by using the proposed formulation in Fig. 3.7(d). Sharma used nonlinear moment-curvature relationships and performed numerical integration at gauss points. However, Thanoon used lumped plasticity model with rigid ends and nonlinear stiffness relationships. The load deflection behaviour and the failure load obtained by using NIFAP are in reasonably good agreement with the reported experimental and the analytical results. It is observed that the results obtained by the proposed algorithms using the distributed plasticity are closer to the experimental results than those obtained by the lumped plasticity.

□ **Test Structure 2 - A Space Frame:** The single storey one bay reinforced concrete space frame shown in Fig. 3.8(a) and previously analysed by Thanoon (1993) has been chosen as test structure 2. It has been idealised by eight beam-column elements. The coordinate system, dimensions and other properties are shown in Figs (3.8(a), (b) and (c). A load system which induces all types of stresses i.e. axial, shear, bending and torsion, in the frame is considered for the study. Thanoon analysed the structure with and without slab both by considering and neglecting torsion in its yield criteria. The frame without slab has been analysed for the present study as it is intended to test the inelastic formulation for the frame elements only. The results are compared with those reported by Thanoon in which the slab has not been considered but the torsion in the yield criteria has been considered. Thanoon has used lumped plasticity model with rigid ends and with nonlinear stiffness relations. The comparison of results is presented in Fig. 3.8(d). The results obtained are in good agreement with the reported results.

□ **Test Structure 3 - An Infilled Frame:** The structure shown in Fig. 3.9(a) has been chosen to check the validity of the inelastic

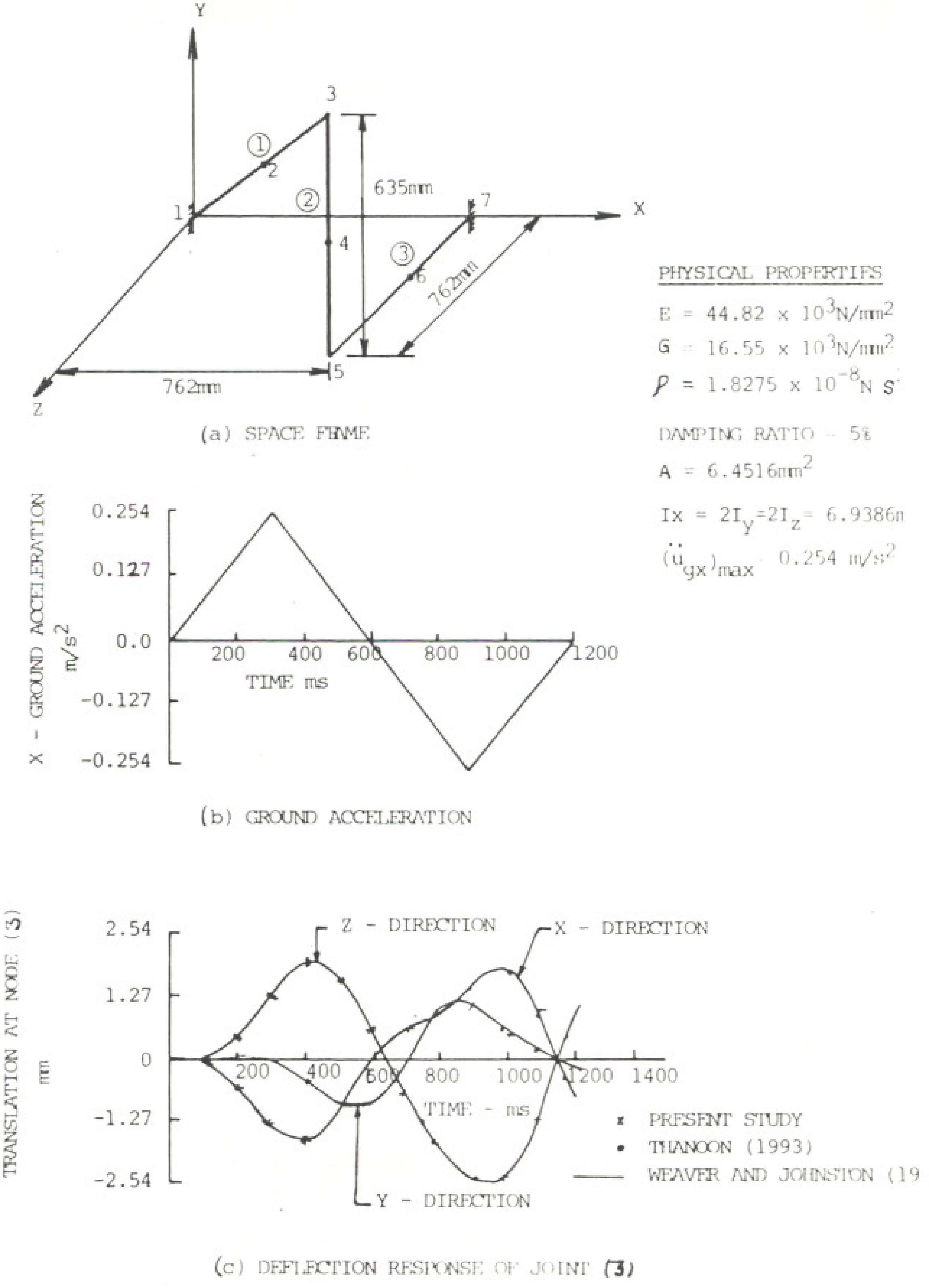
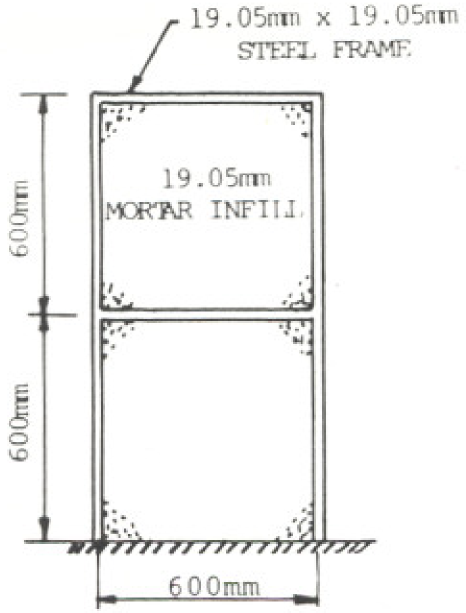
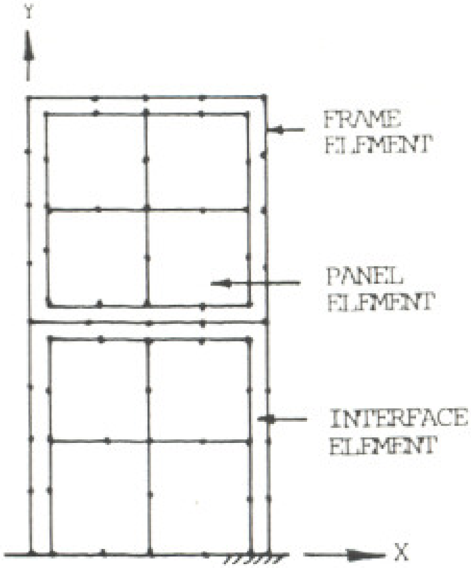


Fig. 3.5 Dynamic Analysis of Space Frame.



(a) INFILLED FRAME



(b) IDEALISED STRUCTURE

PHYSICAL PROPERTIES

PANEL ELEMENT

$E_c = 7.5\ kN/mm^2 \quad \nu = 0.2$

$\rho_c = 0.2 \times 10^{-11} kN.Sec^2/mm^4$

FRAME ELEMENT

$E_s = 210\ kN/mm^2$

$\rho_s = 0.785 \times 10^{-11} kN\text{-}Sec^2/mm^4$

$\mu = 0.44$

INTERFACE ELEMENT

$k_s = 0.768 \times 10^{-3} kN/mm^3$

$k_n = 0.256 \times 10^{4} kN/mm^3$

FREQUENCY	MALLICK & SEVERN (1968)		THIRUVENGADAM (1985)		PRESENT STUDY
	EXPT.	FEM	MULTI STRUT	FEM	
1	142	149	110	143	142.18
2	396	460	246	430	444.19

(c) COMPARISON OF FREQUENCIES

Fig. 3.6 Dynamic Characteristics of the Infilled frame.

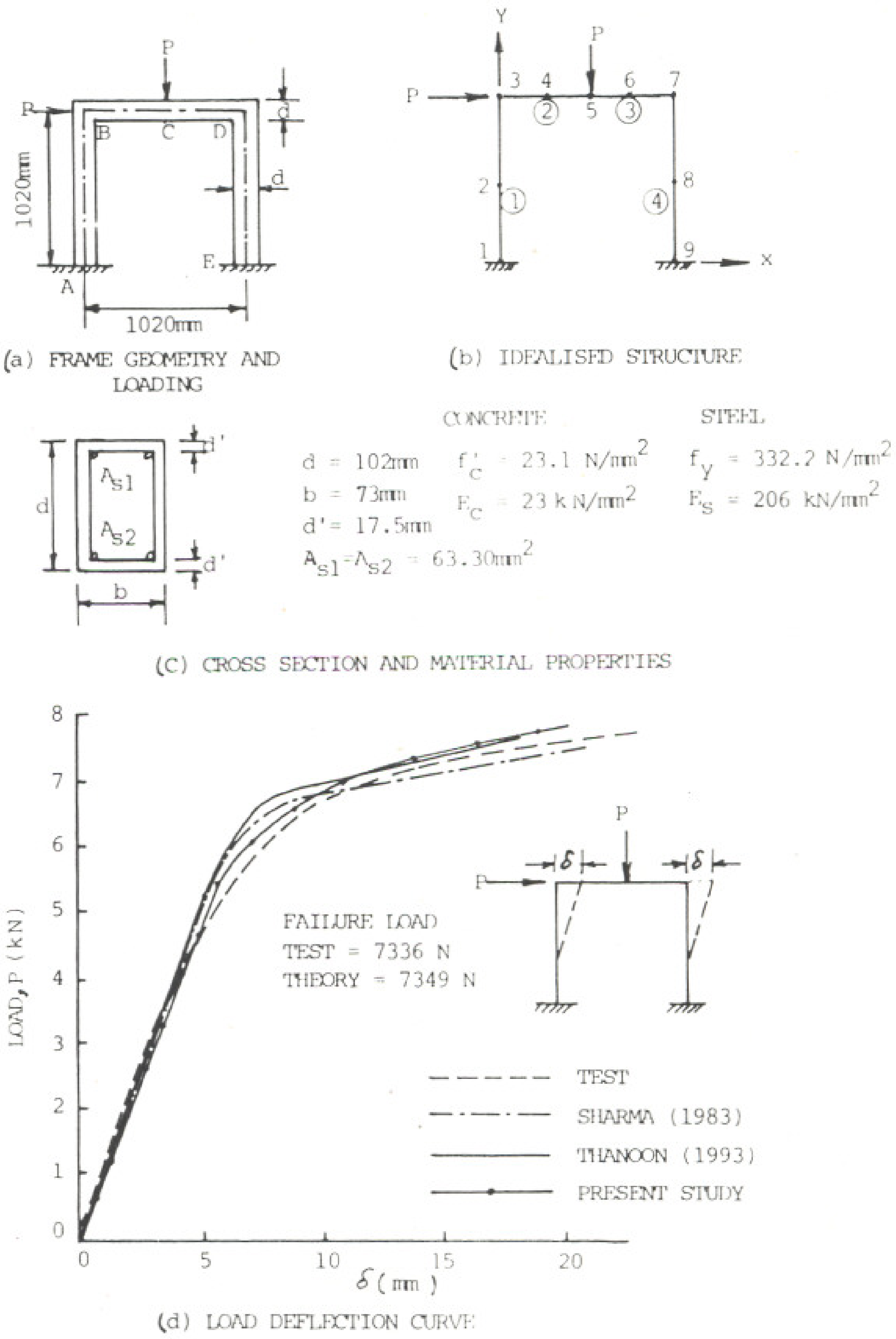


Fig. 3.7 Test Structure 1.

formulations for the panel, frame and interface elements. The structure which consists of a steel frame with Kaffir-D infill panels was investigated by Mallick and Severn (1967) both experimentally and analytically. The results obtained by incremental load are compared with those reported by Mallick and Severn in the Fig. 3.9(c). The load deflection curve has been observed to be nonlinear even when the frame and the infill remained within the elastic range. The nonlinearity may be due to change in the contact length, slippage, and closing and/or opening of gaps between the frame and the infill. The load deflection behaviour is found to agree closely with the experimental behaviour with the maximum difference is of the order of 6.6 per cent.

▫ **Test Structures 4 and 5:** A single bay single storey, and a single bay double storey infilled steel frames previously tested experimentally by Smith (1962) have been analysed using the nonlinear algorithms. The infilled frames and their properties are shown in the Figs. 3.10(a) and 3.11(a) and the results are compared in Figs. 3.10(c) and 3.11(c), respectively. For the single storey structure, the load deflection curve matches with the experimental results at higher loads. At lower loads the computed stiffness is higher as compared to the experimentally observed value. This may be due to improper bond or spread of initial slackness around the panel in the experimental model, whereas in the analytical model bond has been assumed to be perfect at the start of the analysis.

In the double storey structure, the predicted load deflection curve for the roof level matches with the corresponding experimental curve deflection δ_2 . However at the first storey level, the difference in the curves is due to the difference in the computed and the actual stiffnesses of the structure and due to the presence of the initial slackness. The results are also compared with those obtained analytically by Smith (1962) using equivalent strut model. The maximum difference in deflection observed is 26 percent at first storey and 12 percent at roof level.

▫ **Test Structure 6:** This example has been selected to check the validity of the nonlinear formulations up to failure load. This structure consisting of steel frame with masonry infill shown in Fig.

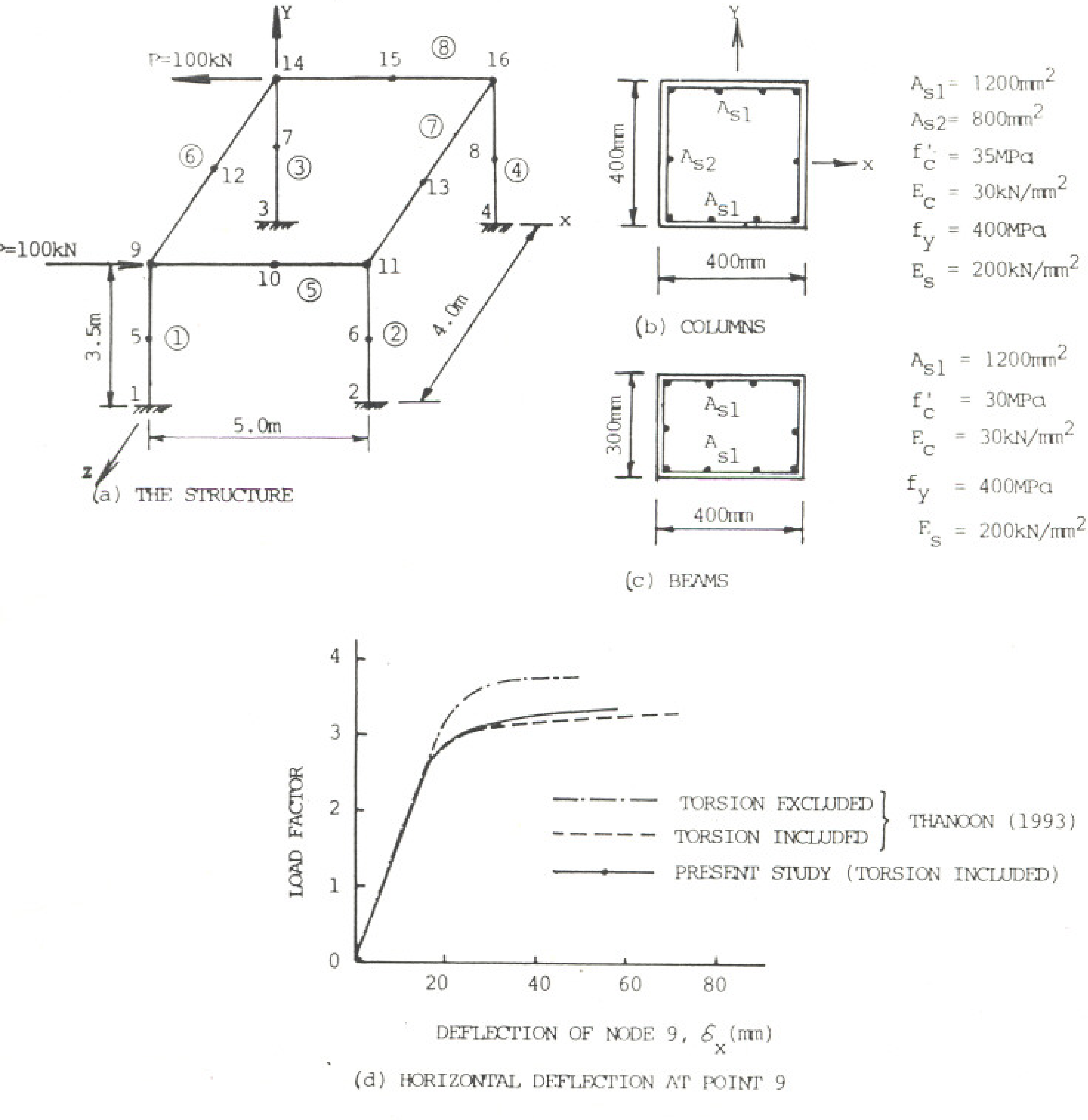


Fig. 3.8 Test Structure 2.

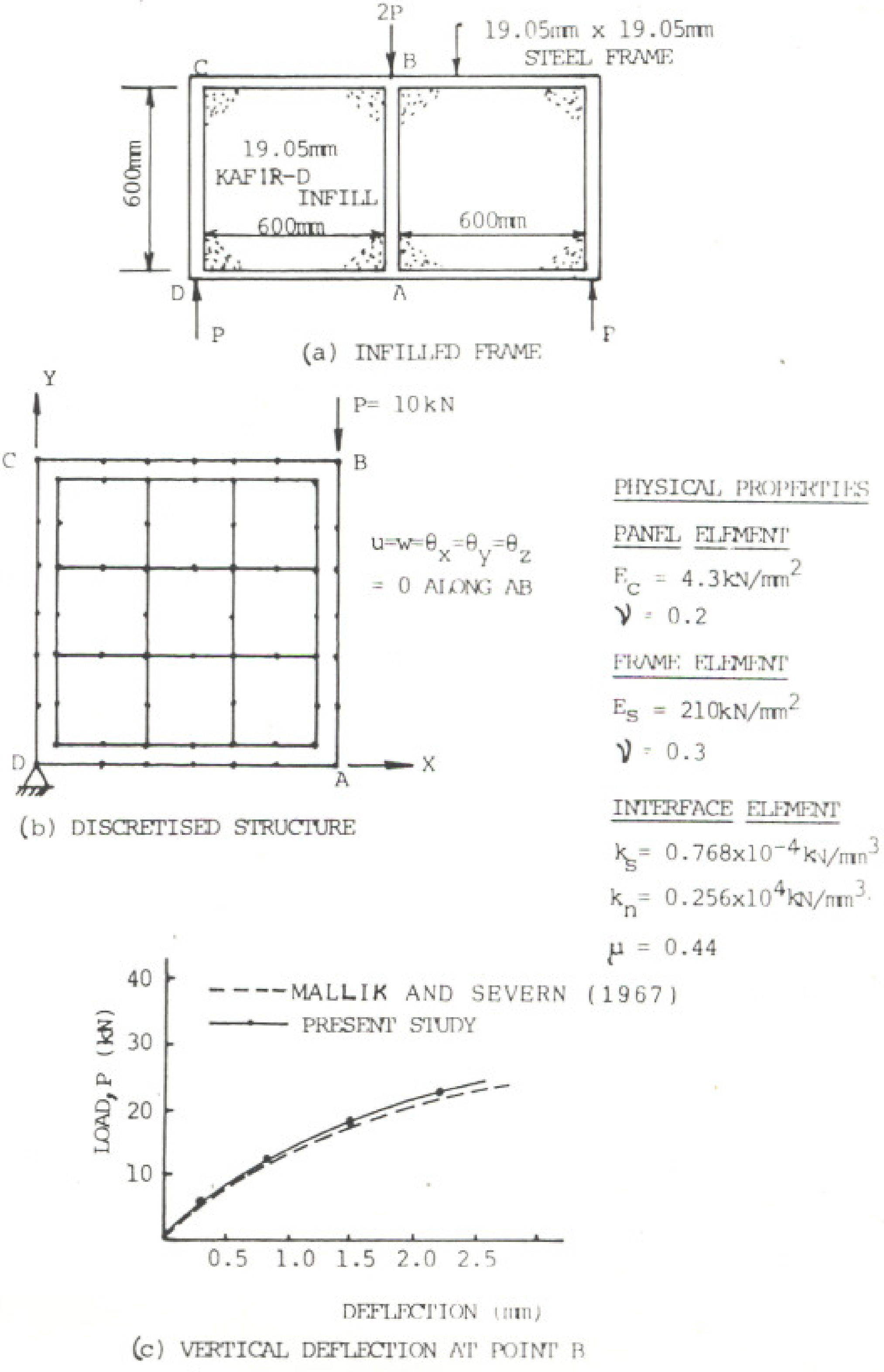
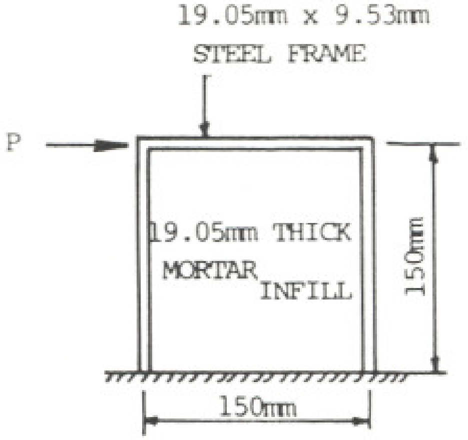
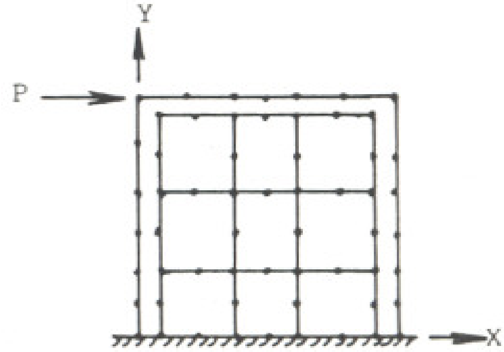


Fig. 3.9 Test Structure 3.



(a) INFILLED FRAME



(b) DISCRETISED STRUCTURE

PHYSICAL PROPERTIES

PANEL ELEMENT

E_c = 5.0 kN/mm^2, ν=0.2

σ_y = 0.01 kN/mm^2

FRAME ELEMENT

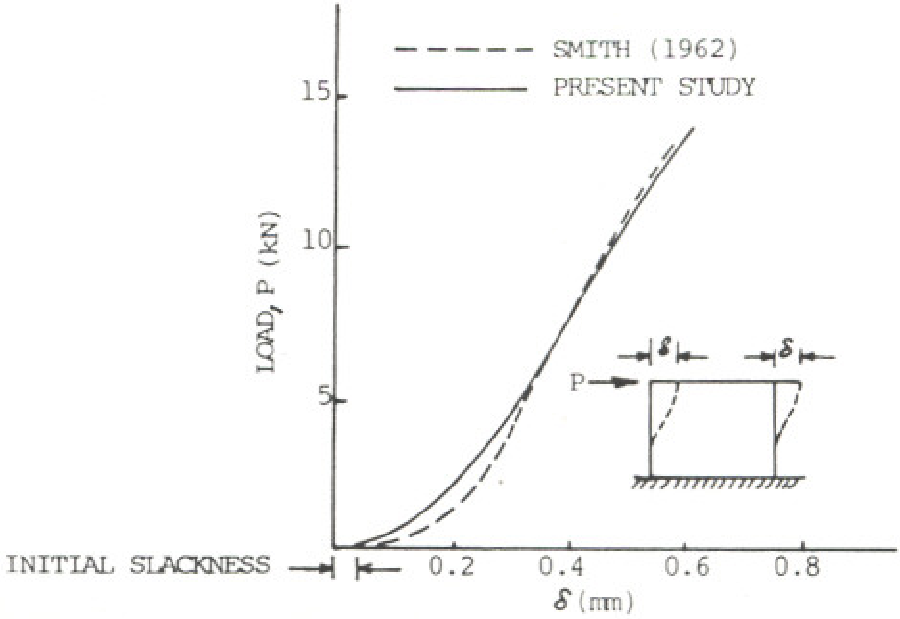
E_s = 210 kN/mm^2 ν=0.3

INTERFACE ELEMENT

k_s = 0.768x10^{-3}kN/mm^3

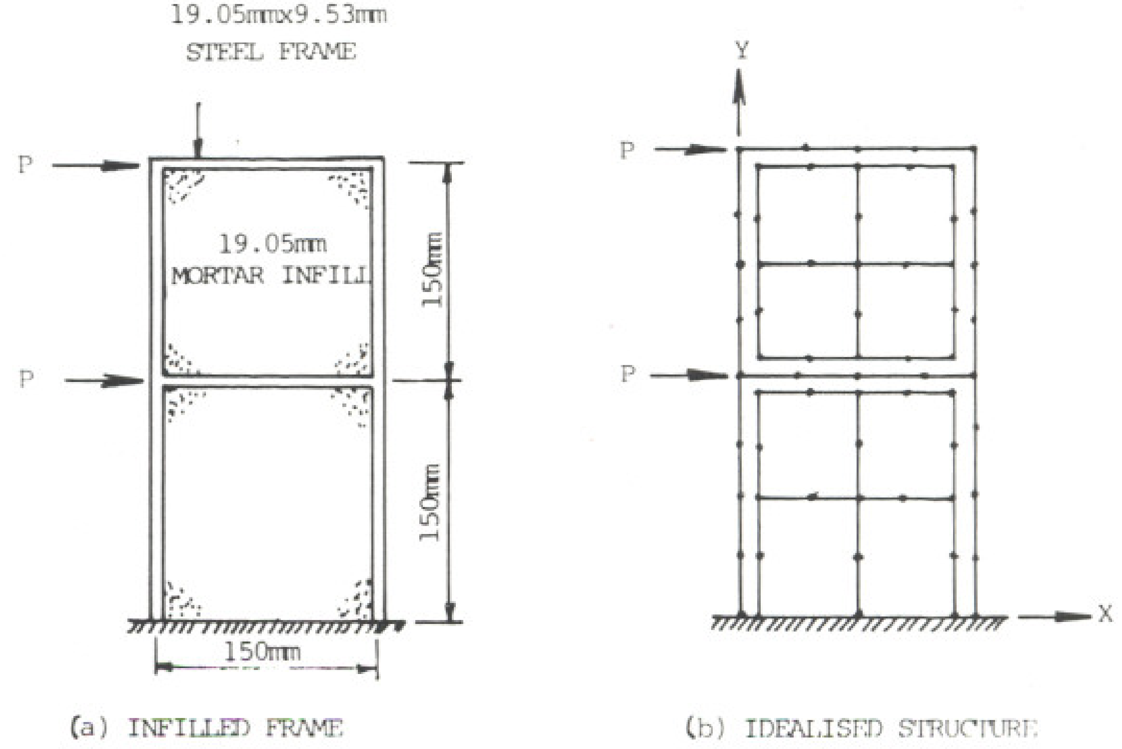
k_n = 0.256x10 ^{4}kN/mm^3

μ = 0.44



(C) LATERAL DEFLECTION

Fig. 3.10 Test Structure 4.



PHYSICAL PROPERTIES

PANEL ELEMENT

$E_c = 7.0$ kN/mm^2 $\nu = 0.2$

$\sigma_y = 0.184 \times 10^{-1}$ kN/mm^2

FRAME ELEMENT

$E_s = 210$ kN/mm^2 $\nu = 0.3$

INTERFACE ELEMENT

$k_s = 0.768 \times 10^{-3}$ kN/mm^3

$k_n = 0.256 \times 10^4$ kN/mm^3

$\mu = 0.4$

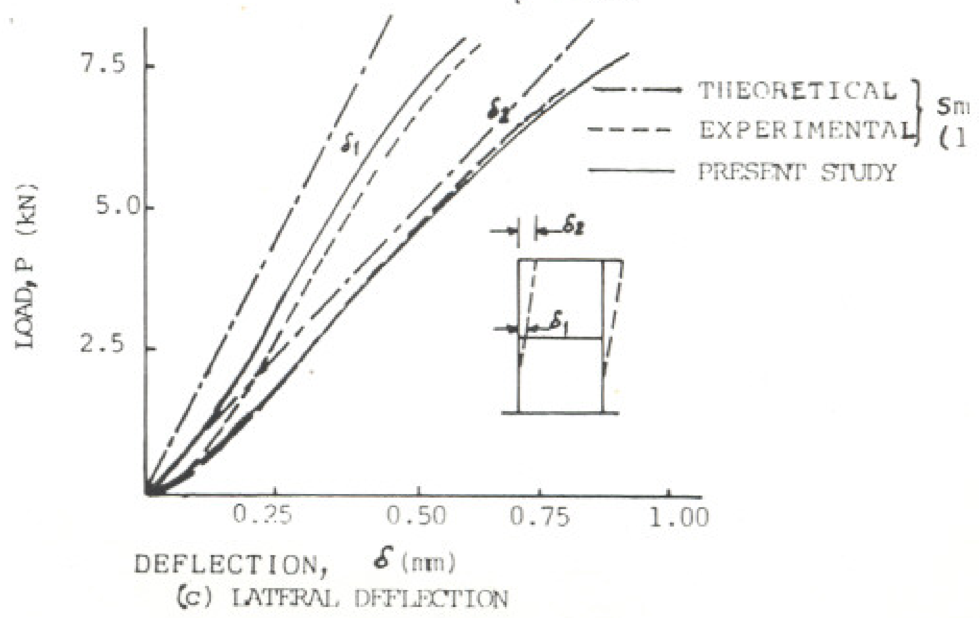
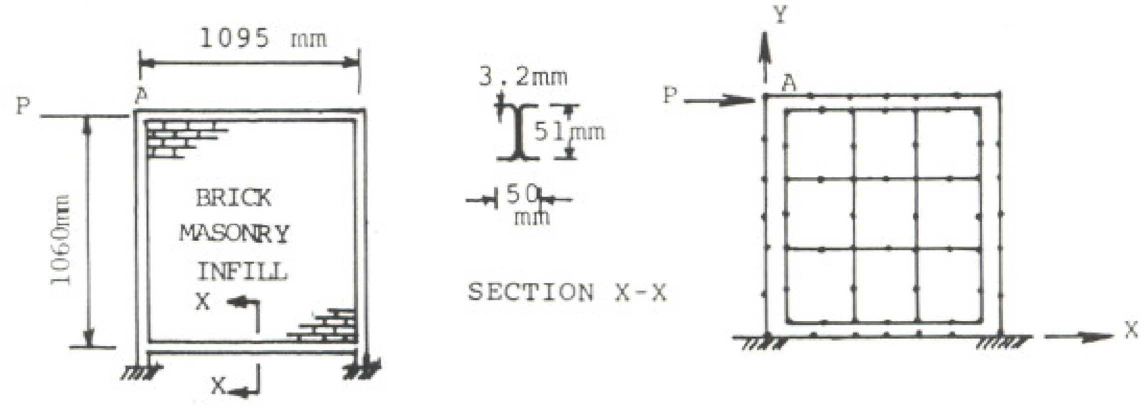


Fig. 3.11 Test Structure 5.



(a) INFILLED FRAME (b) IDEALISED STRUCTURE

PHYSICAL PROPERTIES

PANEL ELEMENT

$E_m = 7.0$ kN/mm^2 $\nu = 0.19$

$\sigma_m = 0.60 \times 10^{-2}$ kN/mm^2

FRAME ELEMENT

$E_s = 210$ kN/mm^2

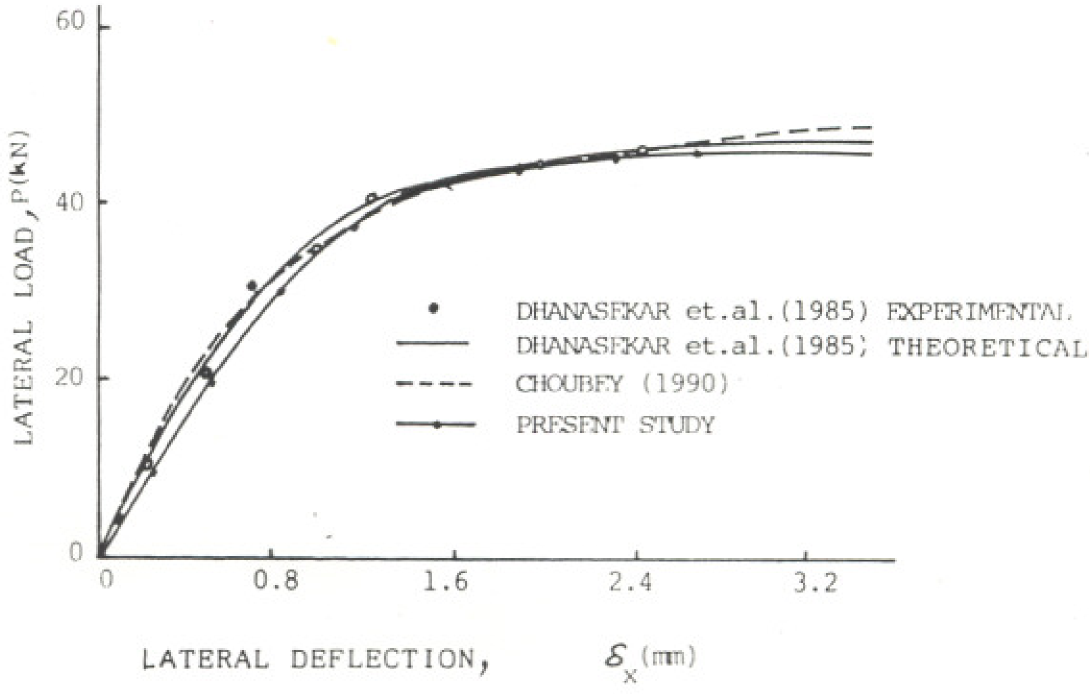
$\nu = 0.3$

$A = 576$ mm^2, $I_z = 0.212 \times 10^5$ mm^4

INTERFACE ELEMENT

$k_s = 6.70 \times 10^{-4}$ kN/mm^3

$k_n = 2.24 \times 10^3$ kN/mm^3



(c) LATERAL DEFLECTION AT A.

Fig. 3.12 Test Structure 6.

3.12 has been investigated both experimentally and analytically earlier by Dhanasekar *et al.*(1985) and by Choubey (1990). The properties and loads are given in the Fig. 3.12.

The load deflection-curve and failure loads predicted by present model are compared with the reported experimental results in the Fig.3.12(c). The proposed formulation of interface element though simpler to that of Choubey (1990) gives comparable results with a step size almost double of that used by Choubey. The number of elements used for infill panel by Choubey was 64 while in the present study only 9 elements have been used. A similar reduction in the requirements of frame and interface elements has been achieved.

3.8. Concluding Remarks

- The formulations for the elements proposed for the investigation of infilled reinforced concrete frame system namely the three noded 3-D beam element, the 3-D panel element and the 3-D interface element have been presented. The usual finite element procedure has been used to formulate the stiffness and mass matrices.

- For the reinforced concrete frame element, the interaction of axial force, biaxial bending and torsion has been considered. For panel element in compression, the material has been assumed to be linearly elastic until failure, and on crushing stiffness has been reduced to zero. On the other hand in tension, on cracking the stress and stiffness normal to crack have been reduced to zero. However, a partial shear transfer due to interlocking between particles has been maintained. The stiffness and stress along the crack have also been maintained.

- At the interface between the frame and the infill, initial gap, closing or opening of gaps or sliding between the frame and the infill have been considered. The tangential stress-strain relationship has been assumed to be elastic perfectly-plastic following the Mohr-Coulomb yield criterion with negligible cohesion. In case the normal strain is tensile, separation has been assumed, otherwise contact has been maintained. If normal strain is compressive but tangential strain exceeds the co-efficient of friction times normal strain slip has been assumed to take place. The stiffness of interface element at each Gauss point has been modified according to the interface conditions at the Gauss point.

□ A general purpose finite element program (NIFAP) for elastic and inelastic static and dynamic response of three dimensional infilled reinforced concrete frames has been developed. The capacity of the program depends mainly on the total number of nodal points in the system, the number of eigen values required in the dynamic analysis, the operating system and the computer used.

The program predicts the formation of hinges in the frame and cracks in the infills at the Gauss points and calculates plastic strains, if any, at these points It calculates displacements and acceleration history at all and the specified nodal points and stress history at Gauss points, at the specified time intervals.

□ The results obtained by the proposed algorithms assuming the plastic hinges at the Gauss points are closer to the experimental results as compared to those obtained by using the lumped plasticity approach although computational time and efforts required are more. In the present study, the plastic hinges have been assumed to form at the Gauss points.

□ The results obtained by using the program NIFAP are reasonably close to those obtained experimentaly. However the results obtained by simplified models like equivalent strut method are too stiff.

□ To establish the validity of the proposed formulations a number of 2-D and 3-D bare and infilled reinforced concrete frames have been analysed for eigen values, inelastic static and dynamic response. The results have been compared with those reported in the literature. It is seen that the eigen values, dynamic response, load-deflection behaviour and the failure loads predicted by NIFAP are reasonably close to the reported results.

3.9 References

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INELASTIC ANALYSIS OF 2-D REINFORCED CONCRETE INFILLED FRAMES

4.1 Introduction

In general, if a framed building is provided with regularly placed infill panels in one or two orthogonal directions, the routine analysis considering the frames in longitudinal and transverse directions independently may be justifiable because the stiffness and strength of the infill panels provided in the longitudinal direction are generally much higher than those provided in the transverse direction. In such a case, a 2-D analysis of infilled framed buildings provides an adequate response for all practical purposes at a much lower cost than a rigorous 3-D analysis. The mathematical models presented in the Chapter three for the inelastic analysis of the infilled reinforced concrete frame, have been used to study its behaviour when subjected to lateral loads.

As described in the previous Chapter the skeleton frame, the panel and the interface have been modelled by 3-noded frame, 8-noded isoparametric membrane element and 6-noded interface element, respectively. For reinforced concrete sections a 50 percent reduction in the short term value of static modulus of elasticity of concrete has been assumed. In addition, effective sectional properties have been used in the entire 'elastic' range prior to the development of ultimate yield surface.

4.2 Nonlinear Analysis

The response of an infilled frame depends upon the interaction between the infill and the frame. There can be separation, closing or opening of gap and slippage between the frame and the infill. The frame and the infill may exhibit material nonlinearity separately. To incorporate all these phenomena of interaction a nonlinear analysis has been adopted.

□ **Reinforced Concrete Frame:** Hinges have been assumed to form at the points of integration (Gauss points) which are distributed over the length of the element. One Gauss point is at the centre of the element and the other two near the ends. In reinforced concrete framed structures, the frame elements are stiffer at the ends due to joint stiffnesses. So it is appropriate to assume the formation of hinges near the ends of the elements. Inelastic behaviour of the element has been assumed to be governed by the interaction of the axial force, two flexural moments and a torsional moment. Chen and Powell's(1982) yield (interaction) surface given by Eq. (3.54) has been used in the investigation.

□ **Brick Masonry Infill:** The brick masonry is assumed to be linearly elastic till its failure. In compression, on crushing the stiffness and the stresses are reduced to zero. In tension, on the other hand, on cracking the stiffness and stress normal to crack are reduced to zero, however, along the crack partial shear stiffness and a partial shear stress transfer due to interlocking between the particles are maintained. The stiffness and the stress along the crack are also maintained.

□ **Concrete Mortar Interface:** Different interface conditions have been modelled by adjusting the interface stiffness co-efficients. When the normal strain is tensile, a separation is assumed to be taking place and normal stiffness, k_n is taken to be zero. Since a separated interface cannot take up any shear stress the shear stiffness, k_s is also taken to be zero. If the normal strain is compressive, the interface is assumed to be in contact and a very high value is assigned to k_n. The tangential stress-strain relationship is assumed to be elastic-perfectly plastic using a Mohr-Coulomb yield criterion with zero cohesion. When $|\sigma_u| < |\mu\sigma_v|$, firm contact is assumed and k_s is assigned the value obtained from the tangential stress-relative displacement curve resulting from shear box tests [Choubey (1990) and May and Naji (1991)]. Here μ is the coefficient of friction. Slip takes place whenever $|\sigma_u|$ exceeds $|\mu\sigma_v|$. In such a case σ_u is reduced to $\mu\sigma_v$ and k_s is taken as zero. Values of the stiffness coefficients k_s and k_n used in the analysis for different interface conditions are listed in the Table 3.1

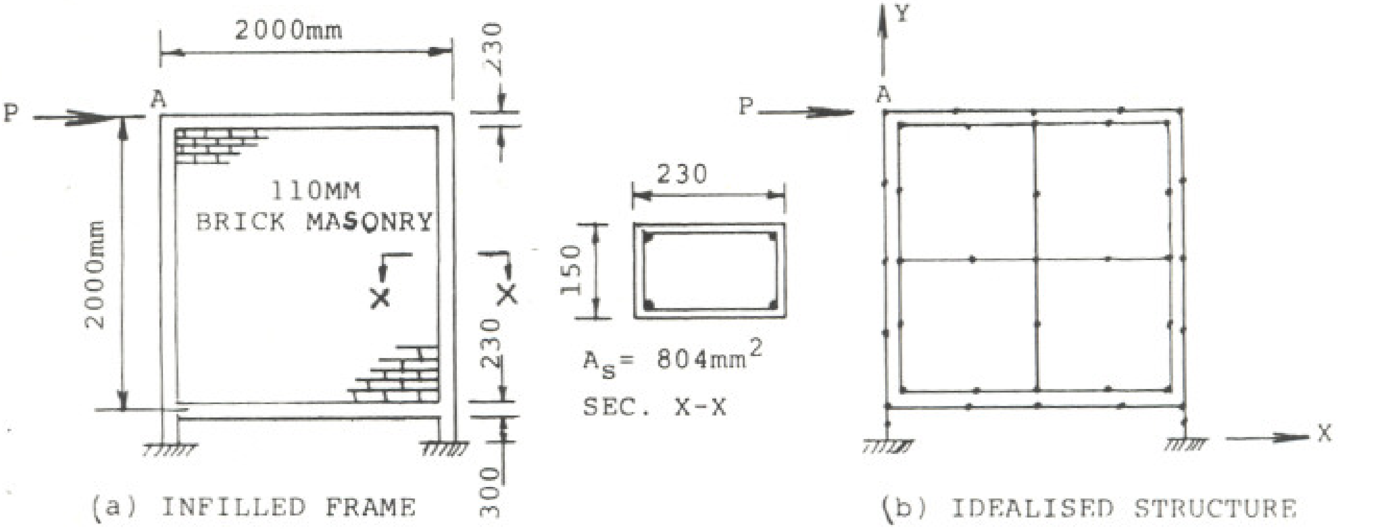
□ **Solution Technique:** In the present investigation a mixed procedure has been adopted to carry out the nonlinear analysis of the infilled frames. For the dynamic solution, direct integration using the Newmark's predictor-corrector (implicit method) technique [Owen and Hinton (1980)] has been used.

4.3 Test Structure 1

To study the behaviour of reinforced concrete infilled frames, a single storey one-bay infilled reinforced concrete frame shown in Fig. 4.1 (a) has been selected. The structure has been previously investigated experimentally and analytically by Choubey (1990). The structure consists of reinforced concrete frame and masonry infill. The physical and material properties and other details of the structure are given in the figure.

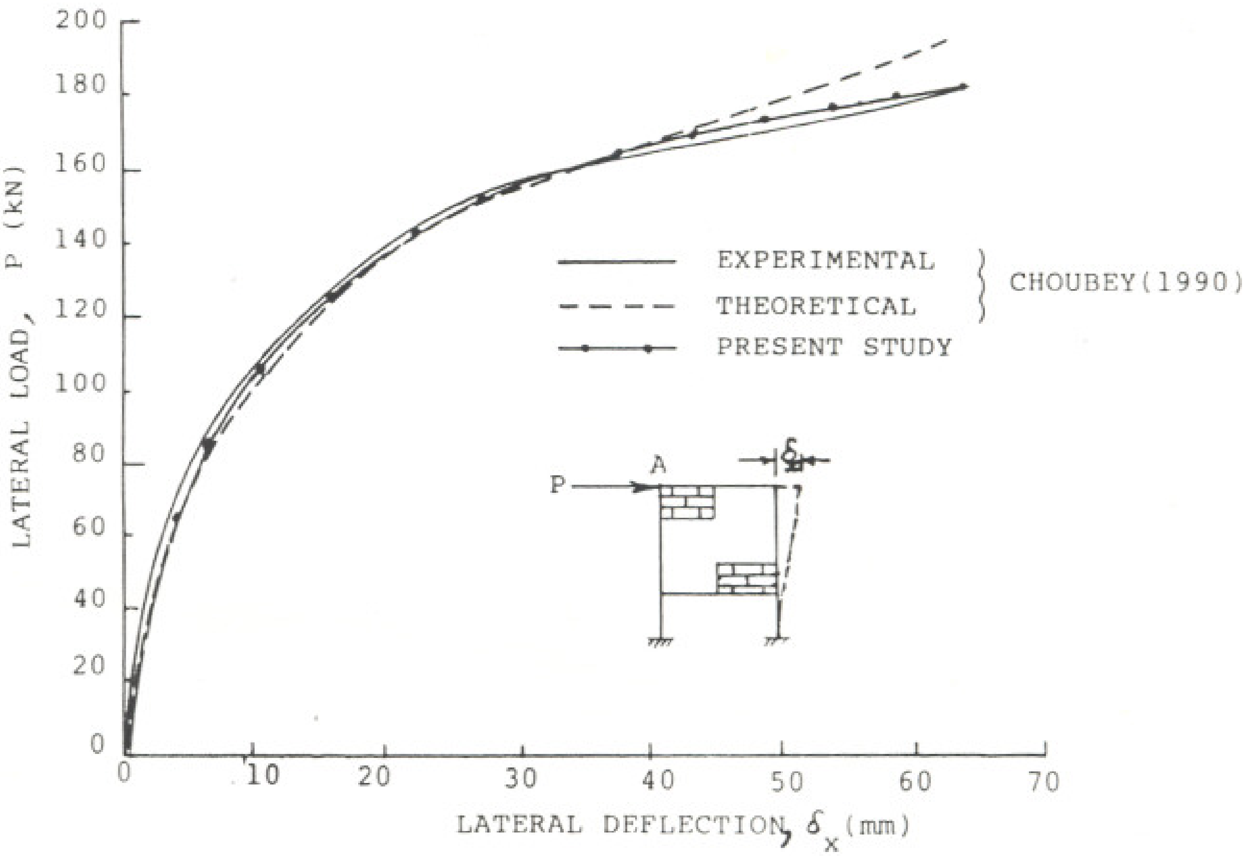
The load deflection curve obtained by using the proposed model has been compared with that reported by Choubey (1990) in the Fig. 4.1(c). A good agreement with the experimental results has been observed. The failure load of 180 kN as predicted by the program NIFAP is close to that obtained experimentally (175.38 kN) by Choubey (1990).

The hinges in the frame and cracks in the infill predicted by the program NIFAP, as well as those obtained experimentally by Choubey (1990) have been presented in the Fig. 4.2. A good agreement between the two has been observed.



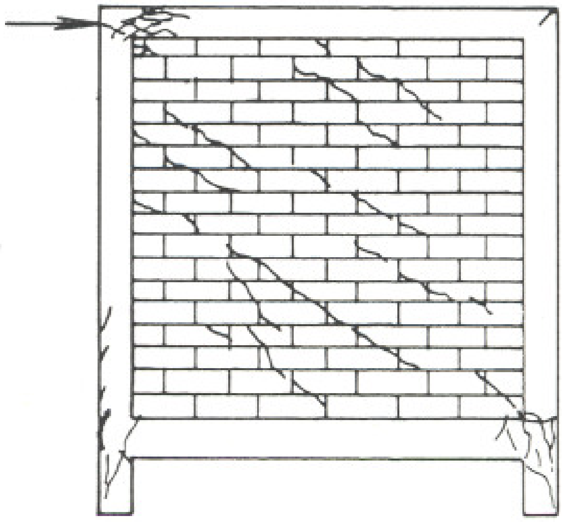
PHYSICAL PROPERTIES

FRAME ELEMENT	PANEL ELEMENT	INTERFACE ELEMENT
E_c = 10.02 kN/mm^2	E_m = 0.7 kN/mm^2	k_s = 0.67x10^{-3}kN/mm^3
ν = 0.2	σ_y = 0.45x10^{-2}kN/mm^2	k_n = 0.224x10^{4}kN/mm^3
f_{ck} = 40.01 N/mm^2	ν = 0.2	

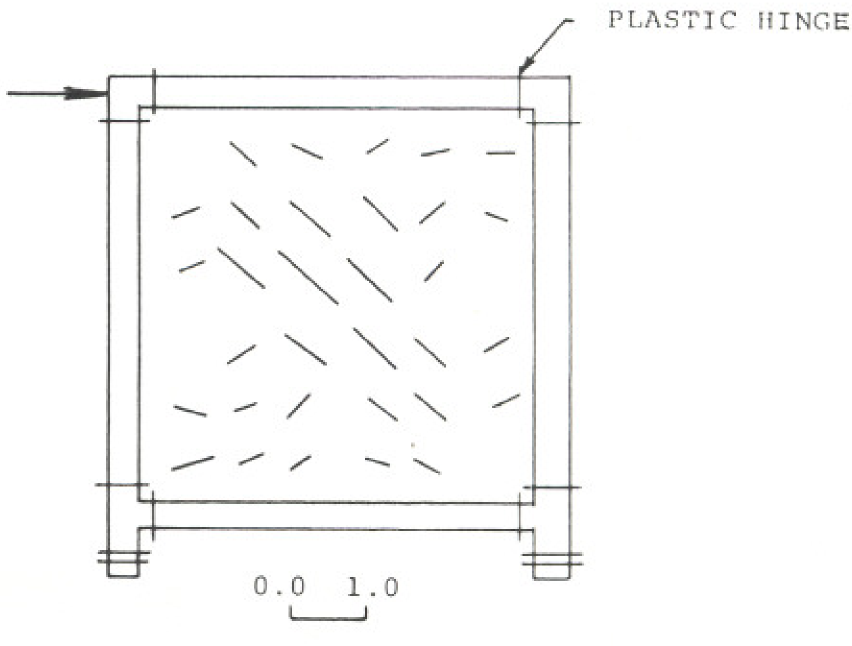


(c) LATERAL DEFLECTION AT A

Fig. 4.1 Load-Deflection Behaviour of the Infilled Reinforced Concrete Frame-1



(a) CRACK PATTERN [CHOUBEY(1990)]



ELASTO-PLASTIC STRAIN

(b) CRACK PATTERN (PRESENT STUDY)

Fig. 4.2 Location of Hinges and Crack Pattern for the Infilled Reinforced Concrete Frame-1

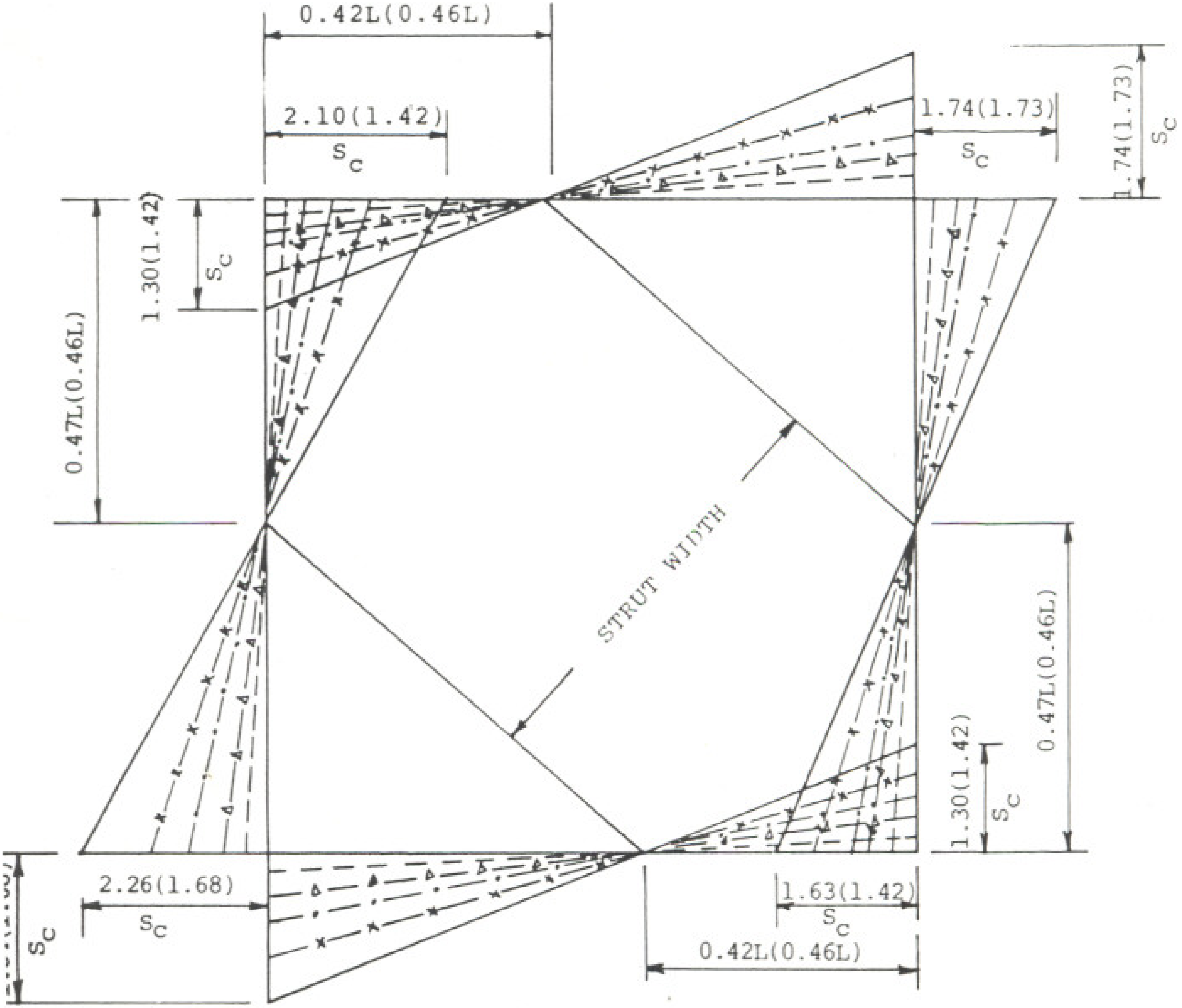
On the application of load, the infilled frame experiences separation at unloaded corners. This separation reduces the lateral stiffness and load carrying capacity of the infilled frames. The transfer of load from frame to infill takes place through the interface, resulting in the development of normal and shear stresses at the interface. The points having tensile normal stresses are assumed to be separated. The separation has been calculated from the relative movement of the pair nodes in the normal direction of the joint element and the slip has been calculated from the relative movement of the pair nodes along the joint element. The separation coefficients, defined as the ratio of separation value to the dimension of the infill, have been plotted in Fig. 4.3. The maximum value of separation coefficient on each side as estimated and those obtained experimentally by Choubey (1990) are shown in the figure. The strut width observed at centre is 0.627 L whereas that proposed by Liauw and Kwan (1984) is 0.707 L.

The closeness between the experimentally observed and estimated load deflection behaviour, failure load, central strut width and mode of failure confirms the reliability of the proposed model in predicting these quantities.

4.4 Test Structure 2

Normally, three types of approaches are used in practice for the analysis of infilled reinforced concrete frames: (a) analysis of infilled frame as a bare frame ignoring the effect of the infill panel, (b) finite element analysis by modelling the infill as a strut element or a panel element monolithic with the frame, and (c) finite element analysis by modelling the infill with a panel element and considering the closing or opening of gaps or sliding between the frame and the panel. To compare these approaches a two-storeyed single bay infilled reinforced concrete frame shown in Fig. 4.4 has been chosen. The dimensions of the frame alongwith member properties are given in the Fig. 4.4.

The discretisation schemes used in the analysis are as shown in Fig. 4.5.



VALUES IN BRACKET ARE FROM CHOUBEY(199

S_C = SEPARATION COEFFICIENTS $\times 10^3$

L = SIDE OF INFILL

LATERAL LOAD IN kN

———————	140
—x—x—x—x—	120
—•—•—•—•—	100
—Δ—Δ—Δ—Δ—	80
– – – – – –	60

CENTRAL STRUT WIDTH	
PRESENT STUDY	LIAUW & KWAN (1984)
0.627 L	0.707L

Fig. 4.3 Separation of Frame with Infill using Program NIFAP.

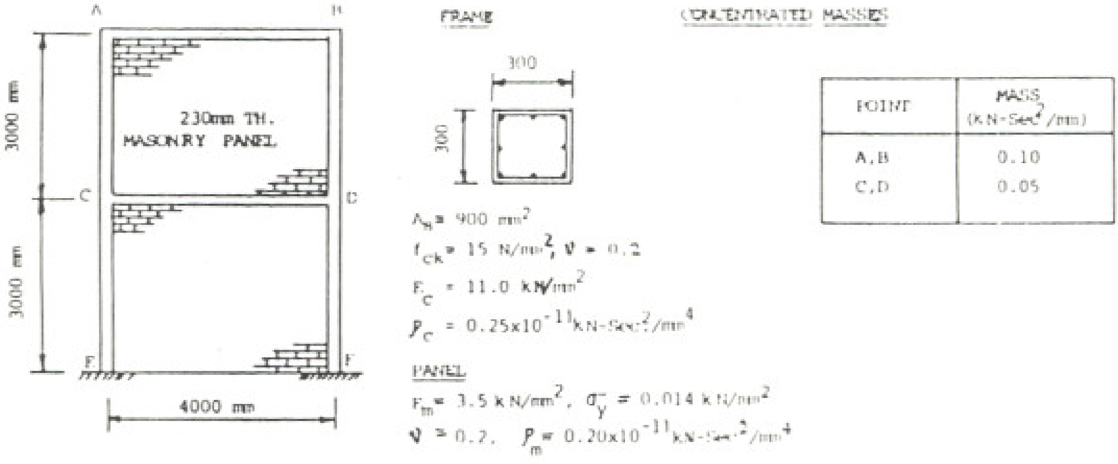


Fig. 4.4 Geometry and X-sectional Details of Reinforced Concrete Infilled Frame-2

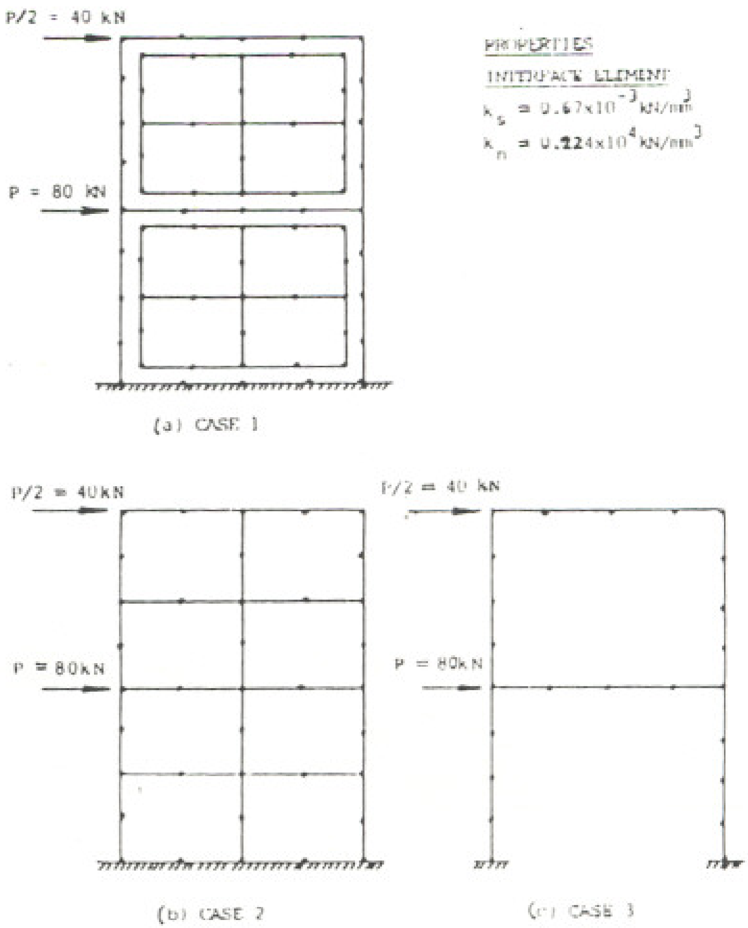


Fig. 4.5 Different Cases of Discretisation of Infilled Frame-2.

Case 1: In the first scheme the frame has been modelled with 12 frame elements and each panel with 4 elements. Fourteen interface elements have been used to model interface conditions between the frame and the infill and also at the damp proof course (DPC) level as shown in Fig. 4.6(a).

Case 2: In the second discretisation scheme the structure has been discretised as in Case 1, but with no interface element i.e. the panel has been assumed to be rigidly connected with the frame as shown in Fig. 4.5(b).

Case 3: The third scheme ignores the effect of infill. The bare frame has been modelled with 12 frame elements as shown in Fig. 4.5(c).

4.4.1 Inelastic Static Analysis

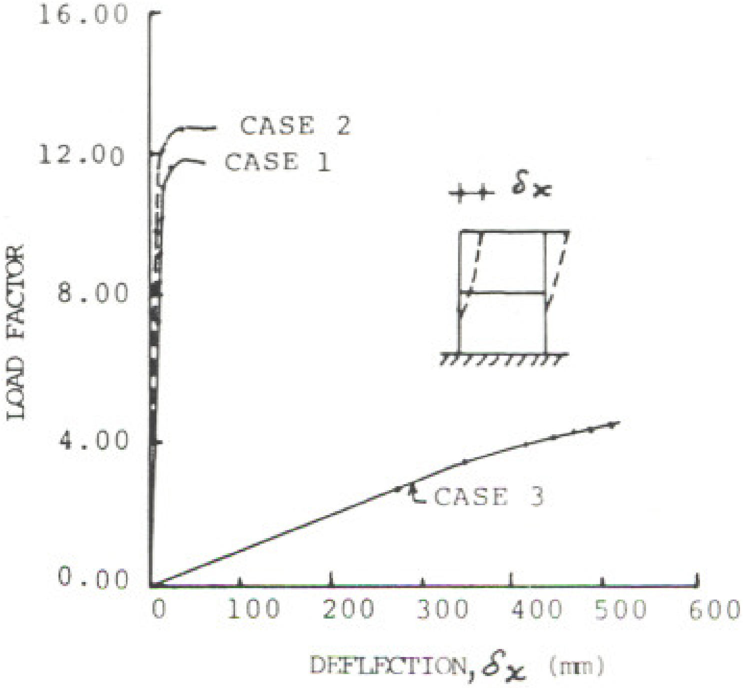
The inelastic static response of the structure shown in the Fig. 4.4 under lateral loads as shown in Fig. 4.5 in terms of deflection δx at roof level using the above discretisation scheme has been shown in Fig. 4.6. It has been observed that the relative stiffnesses for the three schemes are 1.000, 1.640 and 0.012, respectively. The corresponding relative failure loads are 1.000, 1.017 and 0.358, respectively. Thus the stiffness predicted by the Case 3, i.e. using a bare frame idealisation is only 1.2 percent of stiffness predicted by the Case 1. However, the stiffness given by the Case 2 is 64 percent higher than that predicted by the Case 1. In terms of failure load, the load predicted by using the discretisation scheme of the Case 3 is only 35.8 percent of the failure load predicted by discretisation scheme of the Case 1. The failure load predicted by the Case 2 is 1.7 percent higher than that predicted by using the Case 1.

From the above discussion it is evident that the building model ignoring the effect of the infill does not represent the true behaviour. The stiffness and the failure load predicted are unrealistically low.Thus the effect of the infill must be included in the analysis of the infilled frames. However, if only failure load is required to be predicted, the effect of the frame-infill interaction may be ignored. The load-deflection curves at the roof level have been shown in the Fig. 4.6(b) and the sequence of formation of plastic hinges/cracks in the frame and the infill alongwith the corresponding deflections have been listed in the Table 4.1. and are shown in Fig. 4.7. The important features of the response of the problem are discussed below:

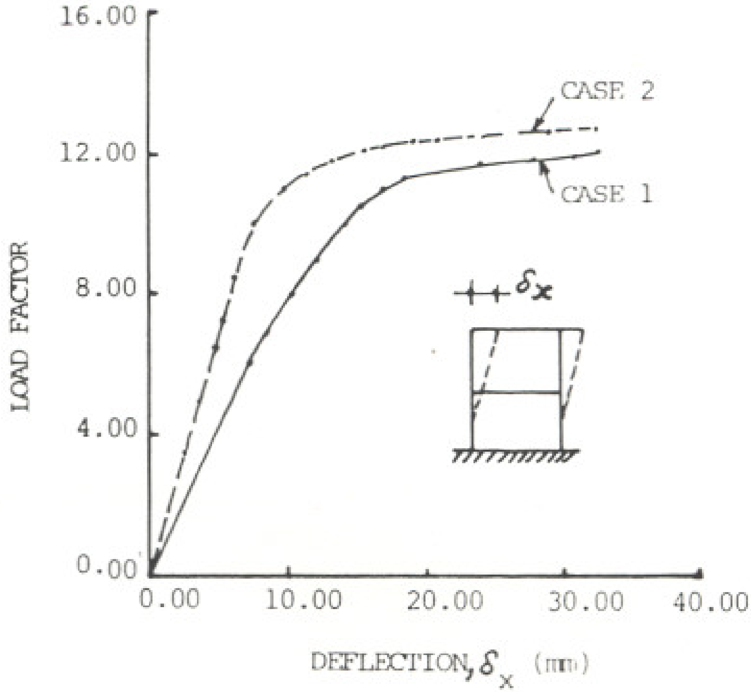
Three distinct types of stress zones namely (a) a compression-compression zone at the ends of the compression diagonal, (b) a tension-tension zone at the ends of the tension diagonal and (c) a compression-tension zone at the centre of the panel have been observed in the infill panels. As the load is increased in decreasing increments, the failure of the infill has been observed to be due to the cracking. At a load factor, defined as the current load divided by the load at the first increment of about 6.0, the cracking in lower panel starts at the ends of the tension diagonal (tension-tension zone) with some cracks at the centre of the infill 4 and 5. With the increase of load, the cracking spreads from the ends of the tension diagonal to the centre, and from the centre to the ends of both the tension and compression diagonals. At a load factor of 10.5, the first hinge forms at the bottom of load ward column and it progresses upwards with the increase of load. At load factor 11.85, six more cracks develop and the structure becomes too flexible to take any additional load. The structure fails at a load factor of about 12.0.

Table 4.1 Sequence of Formation of Plastic Hinges/Cracks in the Infilled frame - 1 (Model 1)

Load Factor	Location & Sequence of appearance of Cracks in the Infill Panels	Location & Sequence of appearance of Hinges in Frames	Deflection at Roof Level (mm)
6.00	1 to 8	—	7.09
7.00	9 & 10	—	8.61
8.00	11 & 12	—	10.28
9.00	13 to 16	—	12.03
10.00	17 & 18	—	14.04
10.50	19	1	15.30
11.00	20 & 21	2	16.92
11.40	22 & 23	—	18.80
11.50	24	—	19.19
11.85	25 to 30	—	30.17



(a) THE THREE CASES



(b) CASE 1 AND CASE 2

Fig. 4.6 Load Deflection Curves for Infilled Frame-2.

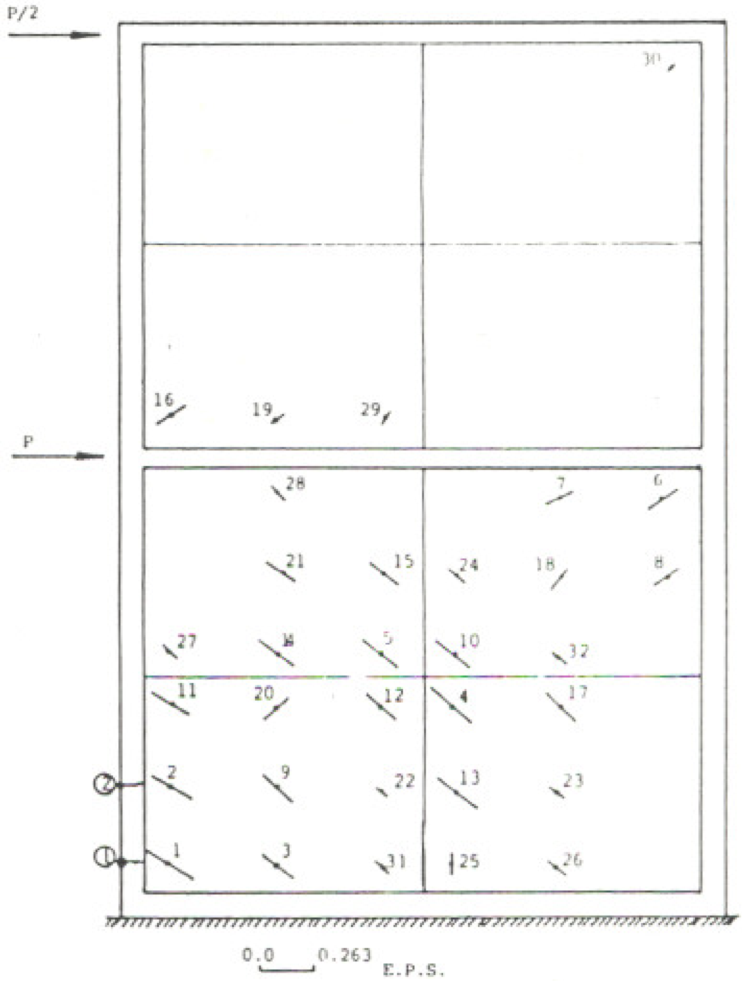


Fig. 4.7 Crack Pattern for Infilled Frame-2

4.4 2 Dynamic Analysis

The infilled frame as shown in the Fig. 4.4 idealised as per scheme of the Case 1 of preceding section has been taken for the dynamic analysis. In addition to the mass of the structure itself four concentrated masses have been considered to be attached at points A, B, C and D. The structure has been subjected to the 20 sec S-0^0-E component of 1940, EL-Centro, Earthquake (Fig. A1) in the X-direction. The earthquake had a peak acceleration of 3417mm/sec^2 at 2.14sec. Both elastic and inelastic responses have been studied during the time interval of 20sec.

The elastic and inelastic responses in terms of deflection at the roof level (point A) have been plotted in the Fig. 4.8. The elastic and inelastic acceleration responses, and the variation of bending moment at the base of the loadward column have been shown in the Figs. 4.9 and 4.10, respectively. The sequence of formation of plastic hinges in the frame and that of cracks in the infill are presented in the Fig. 4.11.

The maximum elastic deflection of 32.0mm occurs after 4.9sec whereas the maximum inelastic deflection of 22.0mm occurs at 9.0sec. The inelastic deflection is 68.8 per cent to that of elastic deflection. The structure remains elastic up to 2.25sec. At 2.5sec, both the bottom storey columns exhibit a number of plastic hinges in the lower half portion and the infill panels of both the storeys exhibit cracking along the tension diagonal as shown in Fig. 4.11(a). The roof deflection at this stage is -10.0mm. At time of 3.0sec, the bottom storey columns are completely plasticised, but most of the cracks in panels 'disappear' since the state of stresses at these points now lie inside the yield surface, however, the plastic strains continue to be present. At 5.0sec some new cracks 'appear' and some old cracks 'disappear', but the bottom storey columns remain plasticised. Subsequently, almost all the cracks in the panels and hinges in the frame 'disappear'.

Figure 4.9 indicates the variation of the elastic and inelastic accelerations at a roof level point A. The maximum acceleration observed for the elastic and the inelastic analyses are 14.7m/sec^2 and

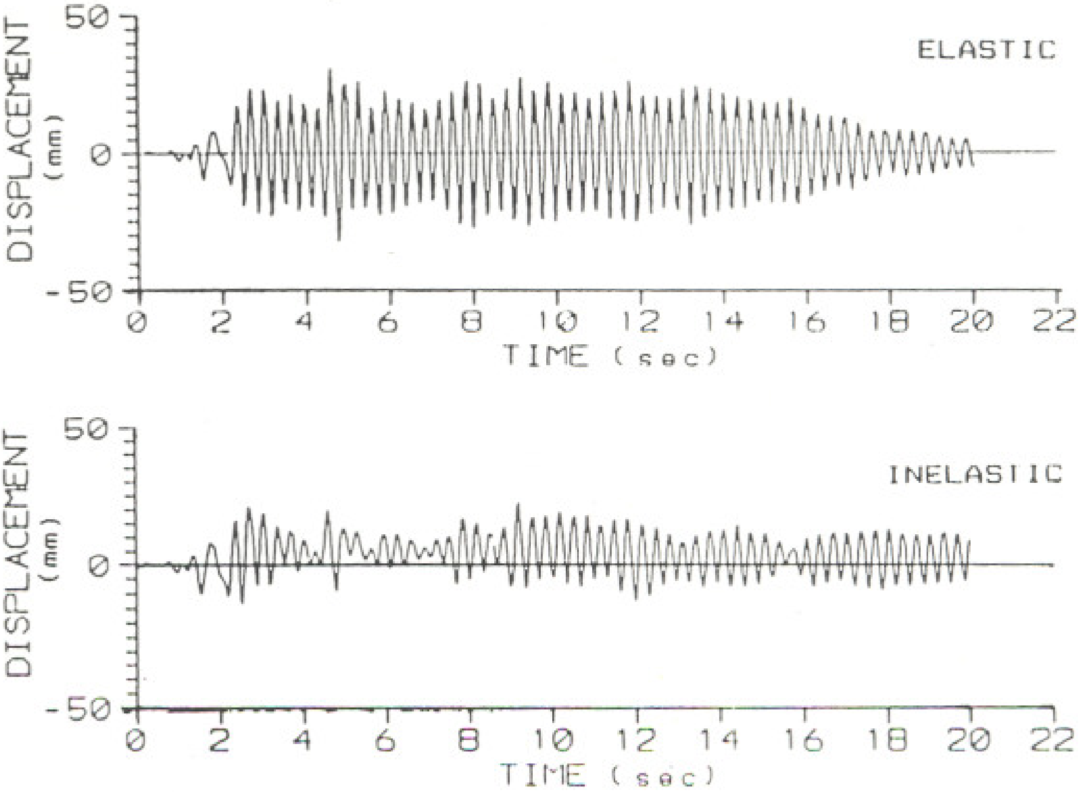


Fig. 4.8 Elastic and Inelastic Displacement Response at Point A: Case 1.

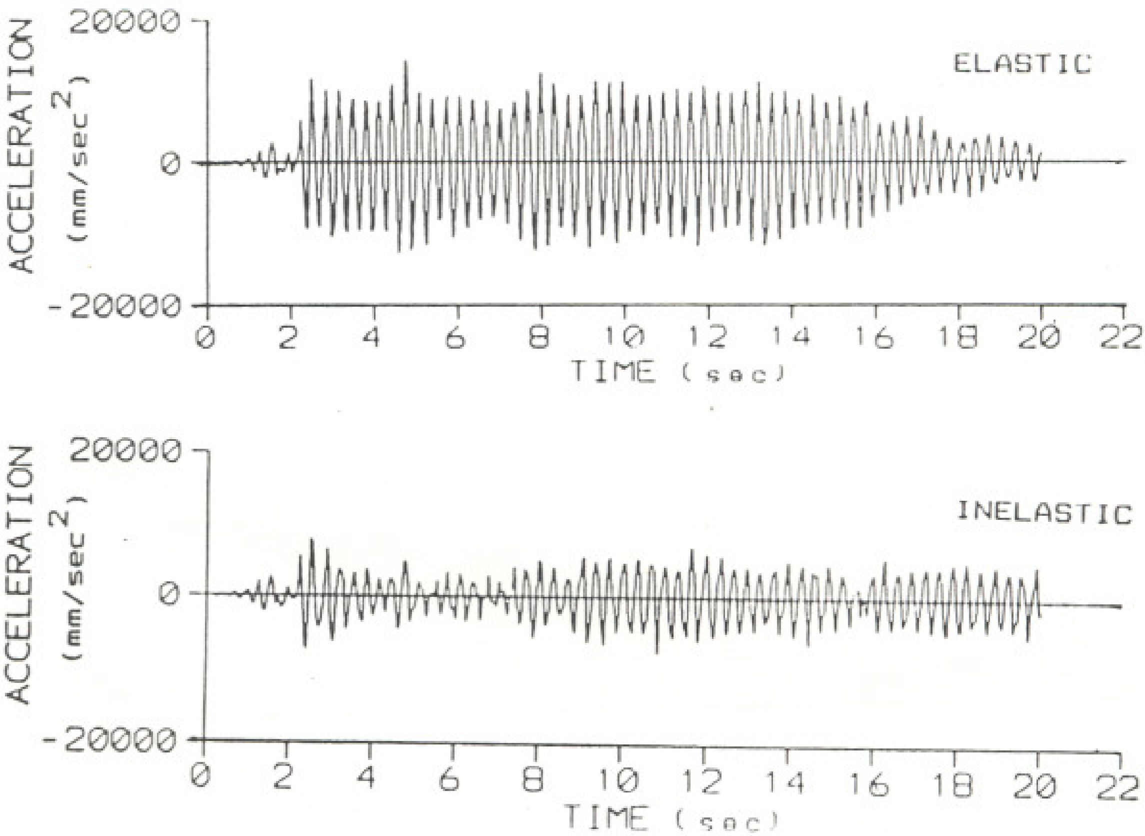


Fig. 4.9 Elastic and Inelastic Acceleration Response at Point A: Case 1.

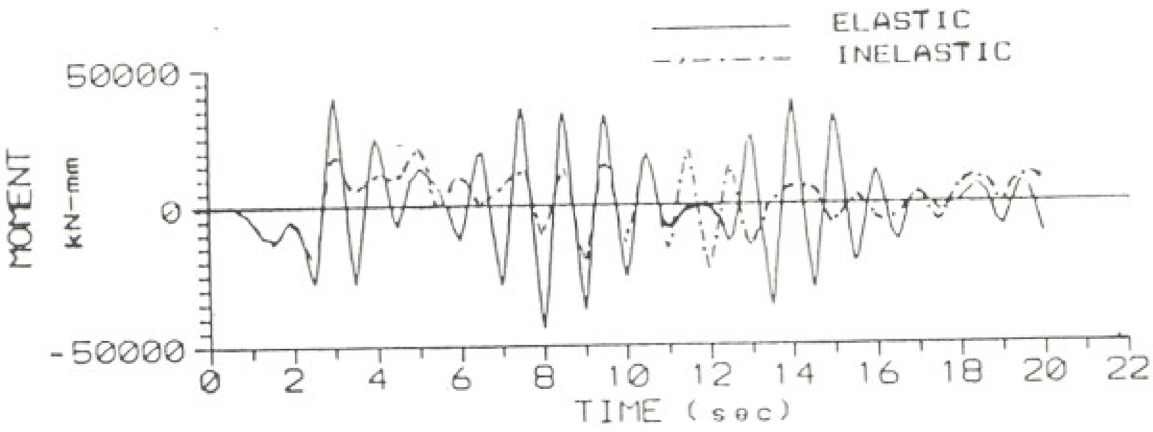


Fig. 4.10 Elastic and Inelastic Variation of Bending Moment at Section F: Case 1.

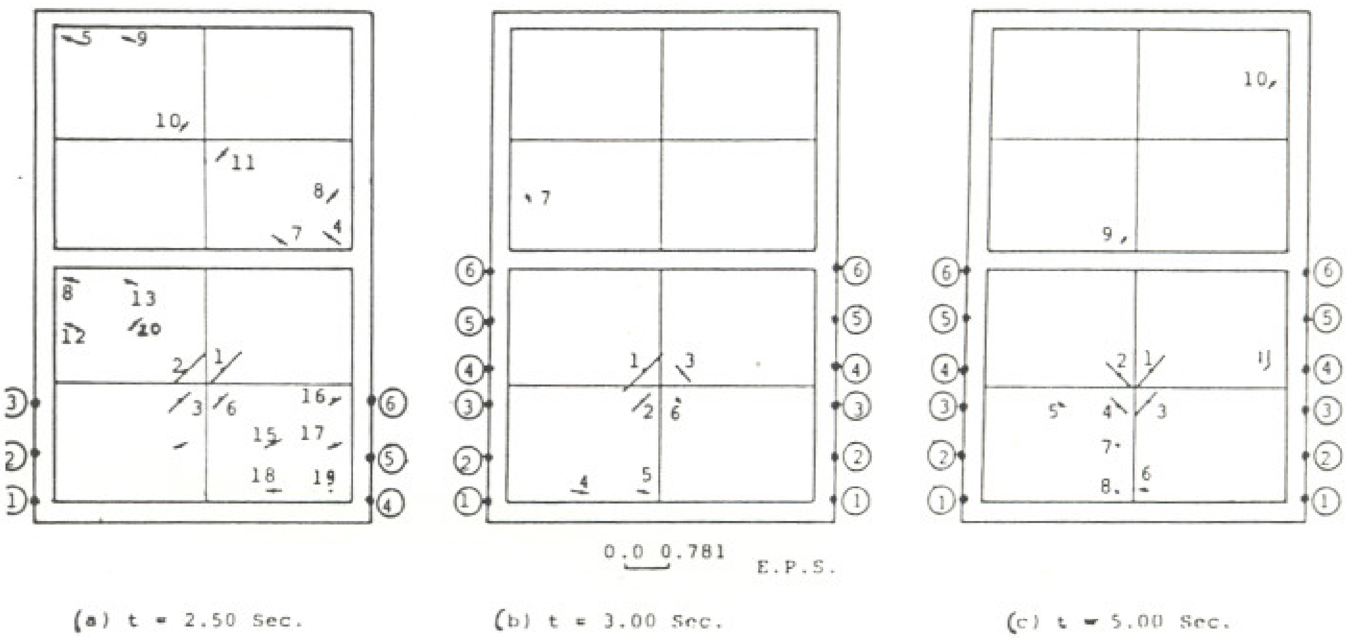


Fig. 4.11 Crack Pattern at Different Times for Infilled Frame-2 (Case 1).

7.4m/sec^2, respectively. The inelastic acceleration being 50.3 per cent of that obtained for the elastic analysis.

Figure 4.10 shows the variation of the moment at a section F near the base. The maximum elastic moment observed is 49.0kNm, while the maximum inelastic moment is 28.0Nm , which is 42.8 per cent less than the elastic moment.

The above analysis demonstrates the necessity of using an inelastic analysis for the realistic prediction of the response of the infilled framed structures.

4.5 Concluding Remarks

Based on the above investigation, the following observations are made:

□ The close agreement between the experimentally observed load deflection behaviour, separation of the infill from the frame, central strut width and failure mode with those predicted by the proposed model establishes the reliability of the model to predict the behaviour, the failure load and the mode of failure.

□ The inelastic algorithms predict the sequence of formation of the plastic hinges in the frame members and the cracks in the infills.

□ A building model which ignores the effect of the infill does not represent the realistic behaviour. For the correct prediction of the response of the infilled reinforced concrete frames the effect of the infill and its interaction with the frame should be included in the analysis. However, the exclusion of the frame infill interaction in the analysis predicts a reasonably correct failure load, but gives a stiffer deflection response. So the interaction may be ignored as it saves a lot of computer time and memory requirements.

□ In the inelastic static analysis, the cracks in the infill are first to develop and subsequently with further increase of the load the infill looses its stiffness which leads to the failure of the frame. With the formation of sufficient number of hinges or cracks, the structure looses most of its stiffness and very large deflections are produced.

□ In inelastic dynamic analysis, the maximum values of the deflection, acceleration and moments at a point are much less as compared to those obtained in the elastic analysis. It is perhaps due to dissipation of energy in elasto-plastic stage.

□ The plastic hinges in the frame and the cracks in the infill appear simultaneously. However, the plastic hinges and the cracks disappear on reversal of loads and on reduction of magnitude of the exciting force.

□ During inelastic dynamic response analysis, most of the plastic hinges and the cracks have been found to develop in the vicinity of the peak acceleration of the earthquake. Permanent plastic deformation has been observed at the end of the analysis due to the formation of the plastic hinges and the cracks.

□ The elastic analysis is not adequate and the inelastic analysis is required to simulate the true behaviour of the infilled frame systems.

4.6 References

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CHAPTER : 5

INELASTIC ANALYSIS OF 3-D REINFORCED CONCRETE INFILLED FRAMES

5.1 Introduction

A 'realistic' model for predicting the earthquake response of a structure should preserve the three dimensional nature of the structure specially in the view of the 3-D effect of an earthquake. Except, probably for the structures with symmetrical distribution of stiffness and mass, the simplified 2-D models may not yield reasonable response to an earthquake excitation and so a complete three dimensional analysis becomes a necessity.

In the following sections static and dynamic analyses of 3-D infilled reinforced concrete frames have been carried out using the computer program NIFAP developed on the basis of the proposed algorithms and procedures. To understand the salient features of the inelastic response, an infilled reinforced concrete space frame has been analysed under static and dynamic loads. Post-earthquake damage analyses of buildings have been conducted to identify the reasons of the observed damage and to assess the reliability of the proposed analytical model. Post-earthquake damage analysis of cycle stand cum canteen reinforced concrete structure at Munghyer, Bihar, India damaged during the Bihar-Nepal Earthquake of Aug. 21, 1988 [Paul *et al.* (1990)] has been carried out. Two four storey reinforced concrete frame buildings, situated in Assam, damaged during the North-East India Earthquake of August 6, 1988 [Paul *et al.* (1990)] have also been studied. First building was extensively damaged at the ground and the first storey. The major damage was concentrated in the failure of many columns just beneath the first floor level. In the second building, the most significant damages were located in the region of articulation joint between the main building and the front staircase.

In the following investigations, the skeleton frame, the panel and the interface have been modelled by 3-noded frame, 8-noded isoparametric and 6-noded interface elements, respectively. For reinforced concrete section, a 50 per cent reduction in the short term static modulus of elasticity and effective sectional properties have been assumed in the entire 'elastic' range prior to the development of ultimate yield surface.

5.2 Nonlinear Analysis

The response of an infilled frame depends upon the interaction between the infill and the frame. There can be separation, closing of gap and slipping between the frame and the infill. The frame and the infill may exhibit material nonlinearity separately. To incorporate all these phenomena of interaction a nonlinear analysis has been adopted.

The various components of the problem are:

□ **Reinforced Concrete Frame:** Hinges have been assumed to form at the points of integration which are distributed over the length of the element. One Gauss point is in the centre of the element and the other two near the ends. In reinforced concrete frame structures, the frame elements are stiffer near the ends due to joint stiffnesses. So it is appropriate to assume the formation of hinges near the ends of the elements. Inelastic behaviour of the element has been assumed to be governed by the interaction of the axial force, two flexural moments and the torsional moment. Chen and Powell's (1982) yield (interaction) surface given by Eq. (3.54) has been used in the analysis.

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□ **Concrete Mortar Interface:** The different interface conditions have been modelled by adjusting the interface stiffness co-efficients. When the normal strain is tensile, a separation is assumed to take place and normal stiffness k_n is taken to be zero. Since a separated interface cannot take up any shearing stress, the shear stiffness k_s is also taken to be zero. If normal strain is compressive, the interface is assumed to be in contact and a very high value is assigned to k_n. The tangential stress-strain relationship is assumed to be elastic-perfectly plastic using a Mohr-Coulomb yield criterion with zero cohesion. The values of stiffness coefficients k_s and k_n used in the analysis for different interface conditions are listed in Table 3.1

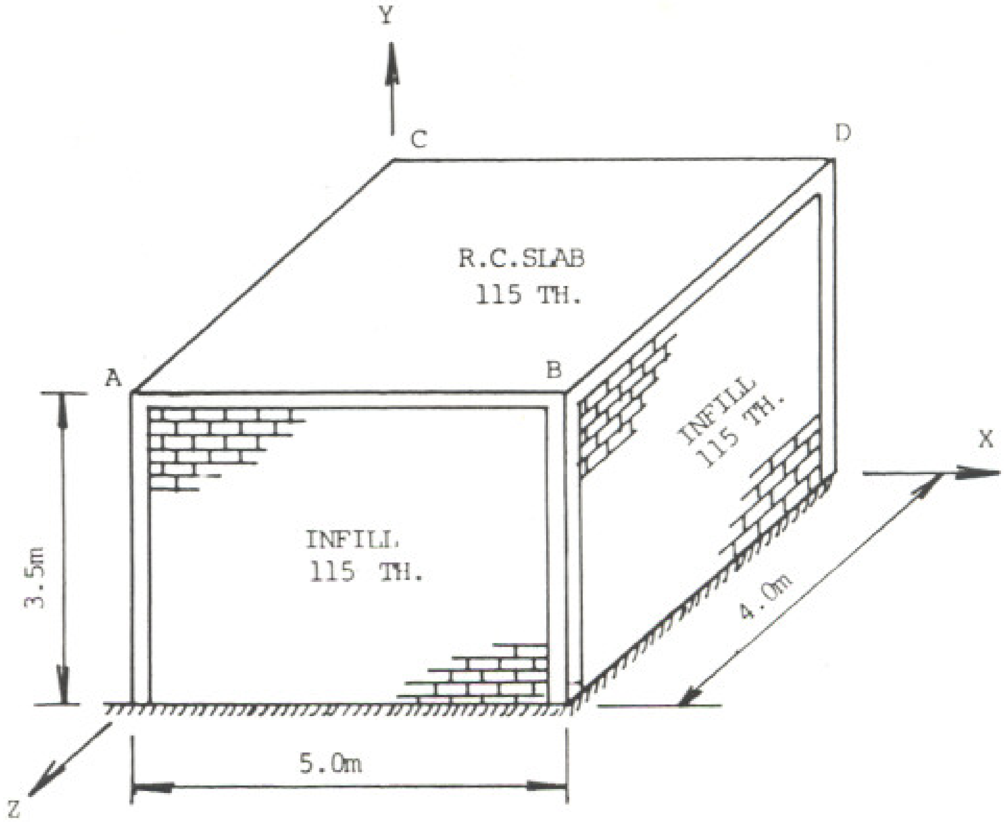
□ **Solution Technique:** In the present study, a mixed procedure has been chosen to carry out nonlinear analysis of the infilled frame. For dynamic solution, direct integration using Newmark's predictor-corrector (implicit method) technique [Owen and Hinton (1980)] has been used.

5.3 Test Structure

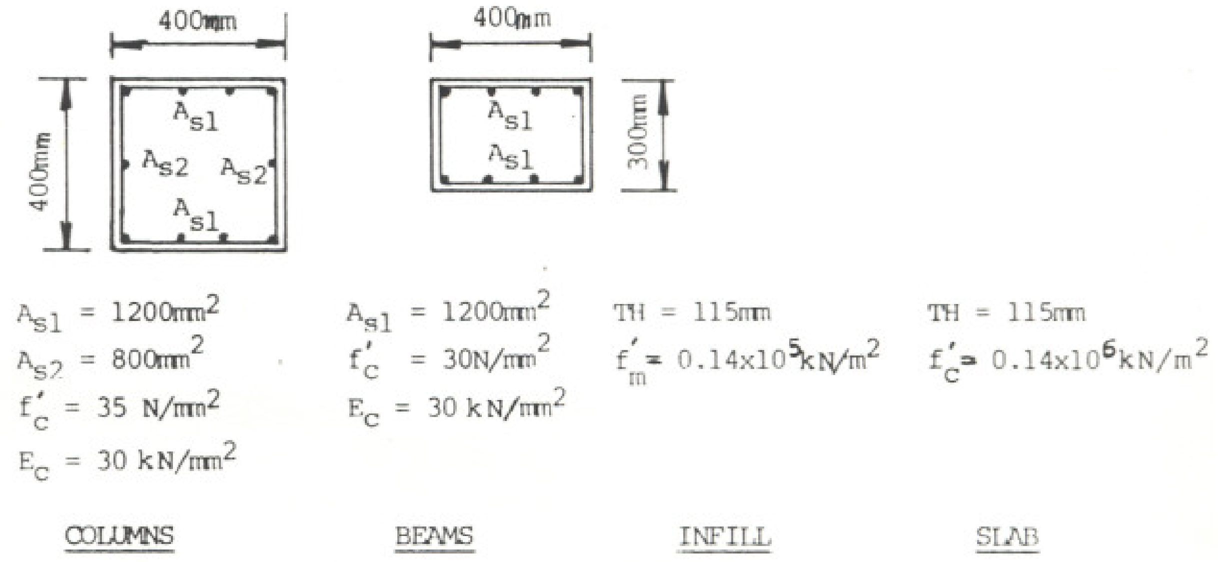
A single storey one bay infilled reinforced concrete space frame shown in Fig. 5.1(a) has been studied for the inelastic behaviour under static and dynamic loads. The frame has a roof of reinforced concrete slab and all the other four sides have masonry infill. The properties and the cross-sectional details are shown in Fig. 5.1(b). For analysis, the roof slab is assumed to have only the in-plane stiffness, since the out of plane flexural stiffness of slab being negligible as compared to the inplane stiffness of the slab and the stiffness of the beams.

5.3.1 Static Analysis

The infilled space frame of Fig. 5.1 is discretised as shown in Fig.



(a) THE INFILLED R.C. FRAME



(b) PROPERTIES

Fig. 5.1 The Geometry and the X-sectional Details for the Infilled Space Frame-1.

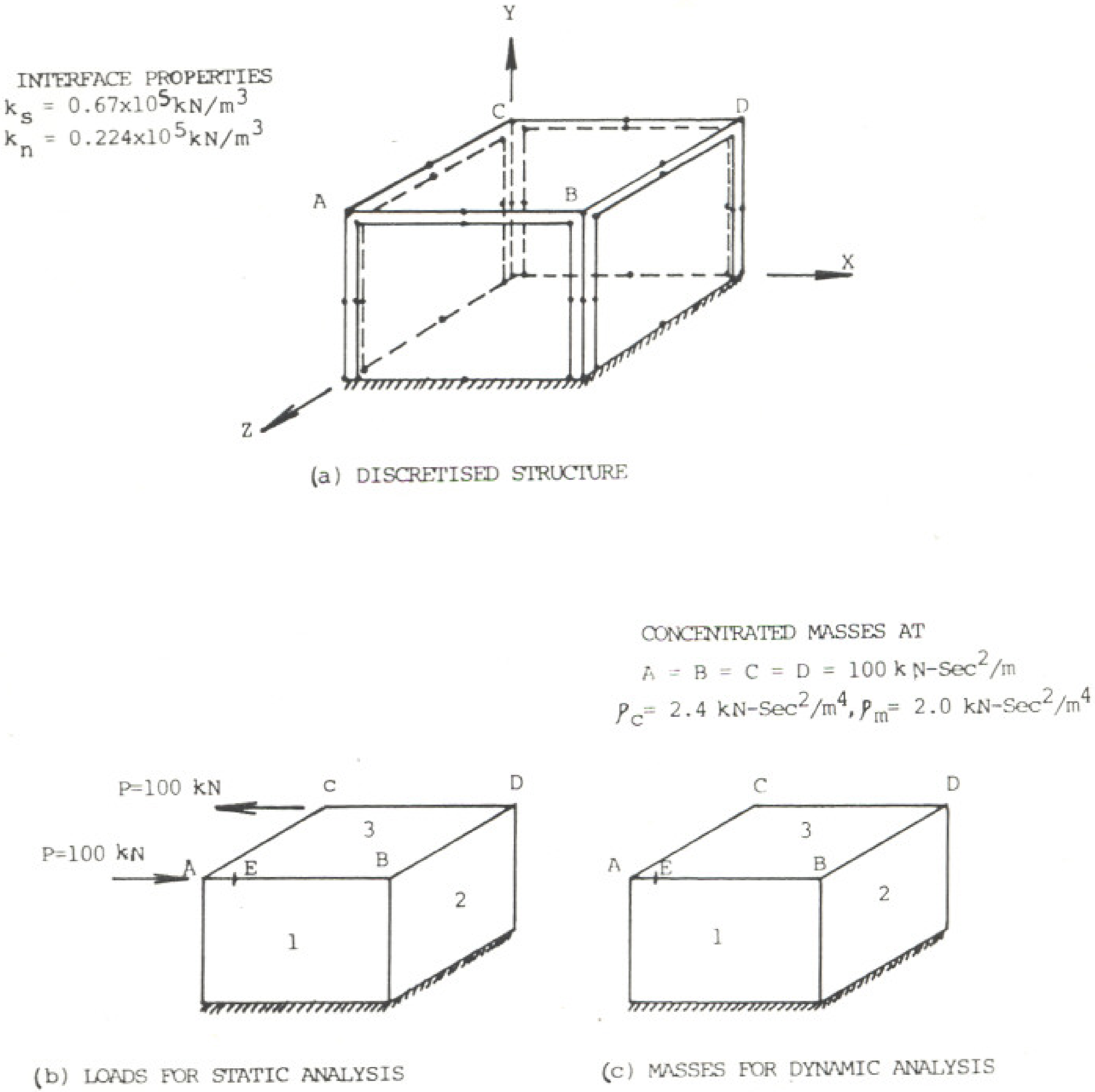


Fig. 5.2 The Discretisation, Loads and Masses for the Infilled Space Frame-1.

5.2(a). Two equal and opposite lateral loads are applied along the X-axis at points A and C to simulate the torsional loading as shown in Fig. 5.2(b). The inelastic response in terms of load-deflection curve in the X-direction at the point A is shown in Fig. 5.3. The sequence of formation of the plastic hinges in the frame and the cracks in the infill panel 1 are shown in Fig. 5.4 and listed in the Table 5.1. At the load factor (defined as the current load divided by the load at the first increment) of 16.0, cracks 1 and 2 appear in the panel 1. When the load factor reaches 22.0, two more cracks 3 and 4 appear in the panel 1. At a load factor of 28.0, eight plastic hinges form in the frame. The infilled frame loses most of its stiffness as the load factor reaches a value of 67.0. Beyond the load factor of 78.0, the solution is not possible even with a very small load increment.

Table 5.1 The Sequence of Formation of the Plastic Hinges/Cracks in the Infilled Space Frame - 1

Load Factor	Sequence of Appearance of Cracks in Panel 1	Sequence of Appearance of Hinges in Frame	Deflection at A δ_x (mm)
16.0	1 & 2	—	5.85
22.0	3 & 4	—	8.84
28.0	—	1 to 8	18.08
34.0	5 & 6	9 to 14	25.25
39.0	—	15 to 19	23.72
44.0	7 & 8	20	33.48
57.0	9	—	67.45
61.0	—	21	88.41
67.0	—	22	107.20

5.3.2 Dynamic Analysis

For dynamic anlysis, the frame is idealised as per the scheme shown in Fig. 5.2(a). The geometry, sectional details and the properties are shown in Fig. 5.1. In addition to the mass of the structure, four concentrated masses are attached at the points A, B, C and D as shown in Fig. 5.2(c). The S-0^0-E component of EL-Centro, Earthquake (Fig. A.1.) of 1940 8sec duration has been applied in the X-direction of the frame. The earthquake had a peak acceleration of 3417mm/sec^2 in S-0^0-E

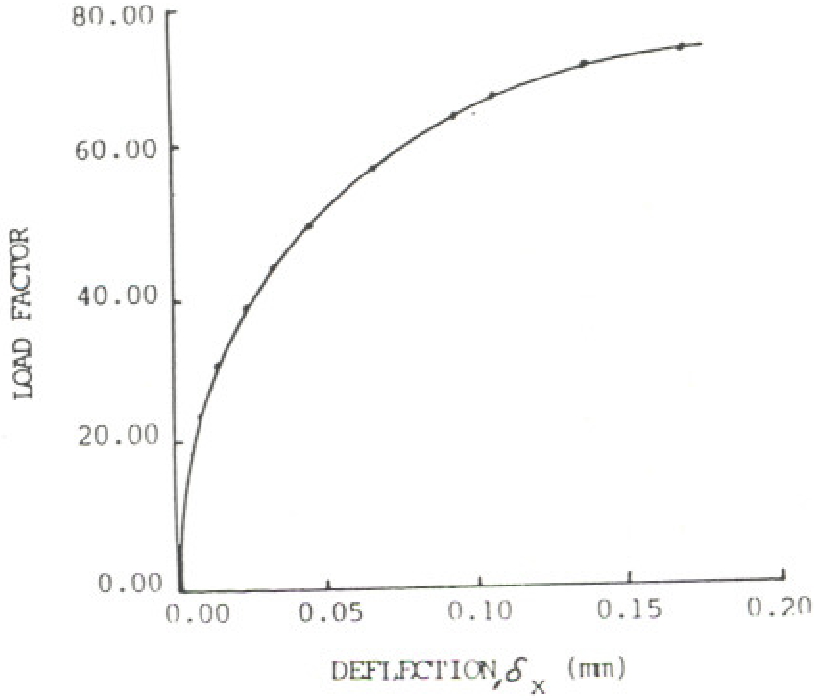
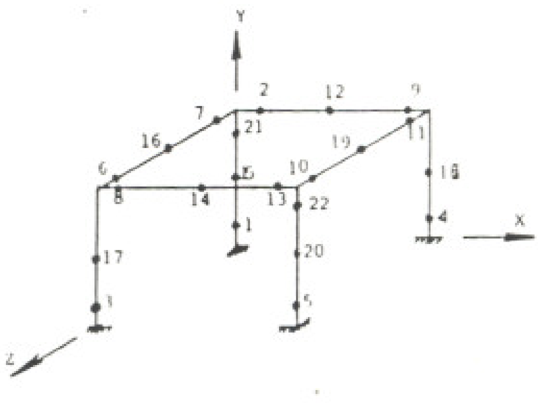


Fig. 5.3 The Inelastic Response: Deflection at the Node A.



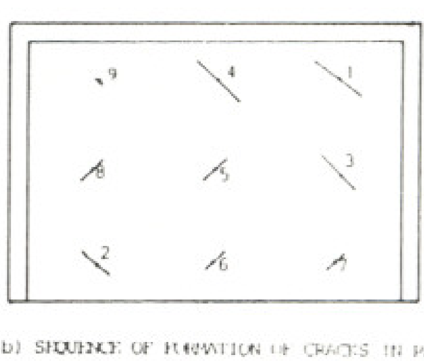


Fig. 5.4 The Sequence of Formation of the Cracks and the Hinges in the Infilled Space Frame-1.

direction at 2.14sec. A damping ratio of 0.05 has been assumed for the analysis.

The elastic and inelastic responses in terms of roof deflection at the point A have been plotted in Fig. 5.5. The elastic and inelastic acceleration responses at the roof level and the variation of the bending moment in the beam AB at the Gauss point near end A is shown in Fig. 5.6. The variation of bending moment at the section E has been presented in Fig. 5.7. The infill panel 1 and the one opposite to it have shown cracks during the application of the earthquake load. Only the beams along AB and CD have shown plastic hinges at the Gauss points near the ends. The behaviour has been found to be symmetrical along the edges AB and CD. So the panel 1 and the beam AB have been studied. The sequence of formation of the plastic hinges in the frame and that of cracks in the infill panel 1 are shown in Fig. 5.8.

The maximum inelastic deflection has been found to be 19.0mm at 4.27sec whereas the maximum elastic deflection is 14.0mm at 2.6sec which is 26.3 per cent less than inelastic deflection. It has been noticed that after 2.6sec, the deflection occurs only on one side of axis indicatig a permanent deformation set. The maximum inelastic and elastic accelerations have been found to be 3.0m/sec^2 at 2.5sec, and 5.9m/sec^2 at 3.5sec, respectively. The former is 96.6 per cent higher than the latter. Maximum moments in frame at the section E have been observed to be 69.0kNm at 2.0sec and 27.0kNm at 2.6sec for the inelastic and elastic responses, respectively.

At time 2.5sec, four plastic hinges are formed in the frame and three cracks appear in the panel 1. Little later at time of 3.0sec, the cracks disappear (since the stresses are now within the yield surface limit, however the plastic strains continue to be present) and one new crack appears. At the time of 6.5sec, one crack reappears in the panel 1, but the others vanish. After this stage all the cracks 'disappear'.

There may be degradation of the stiffness of the panels and the frame due to appearing, disappearing and reappearing of plastic hinges and cracks. The stiffness degradation has not been taken into consideration in the present study as literature review has revealed

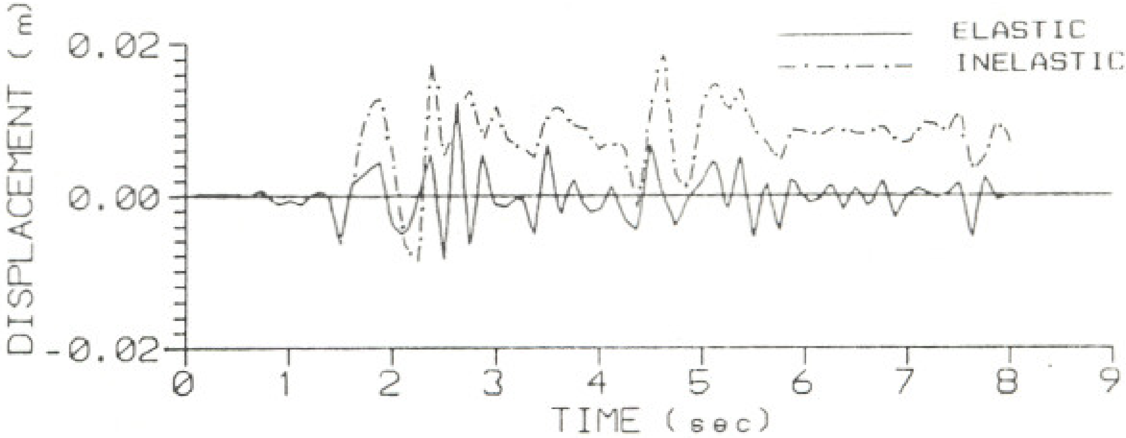


Fig. 5.5 The Elastic and Inelastic Responses: Deflection at a Point A of the Infilled Space Frame-1.

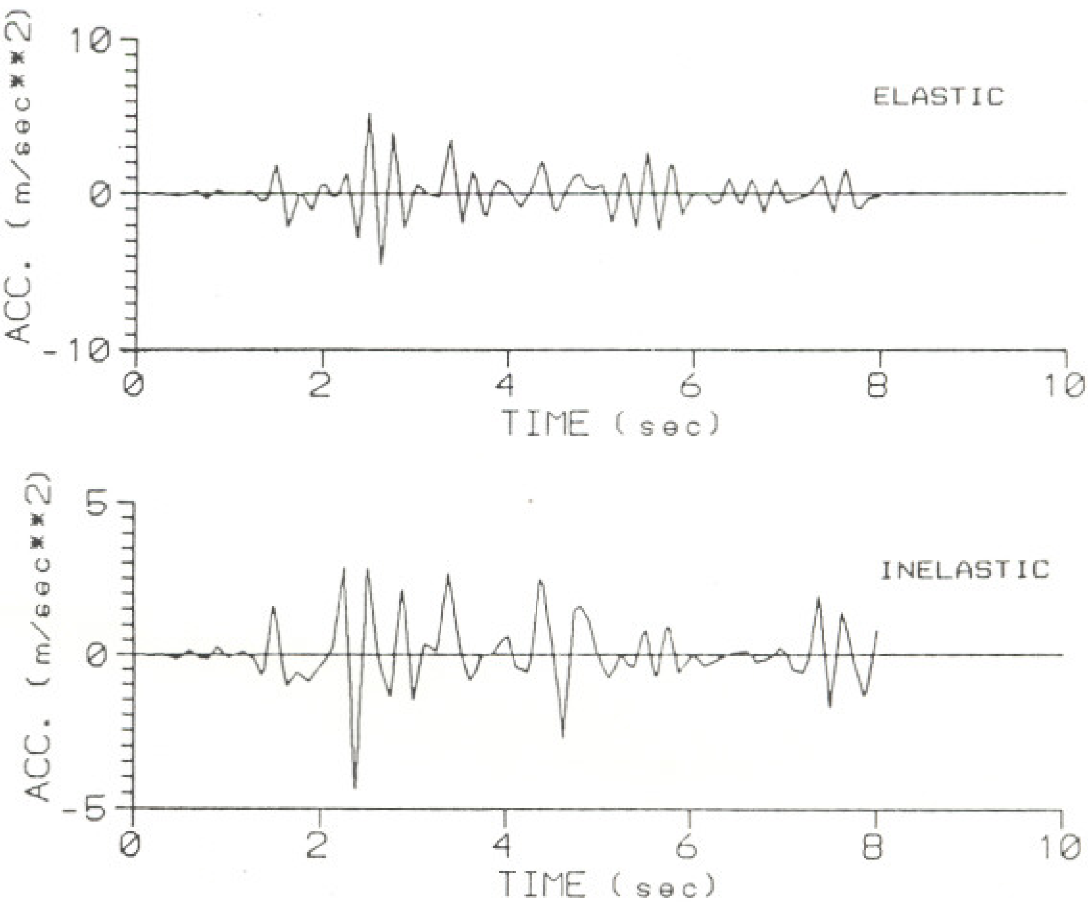


Fig. 5.6 The Elastic and Inelastic Responses: Acceleration of the Point A of the Infilled Space Frame-1.

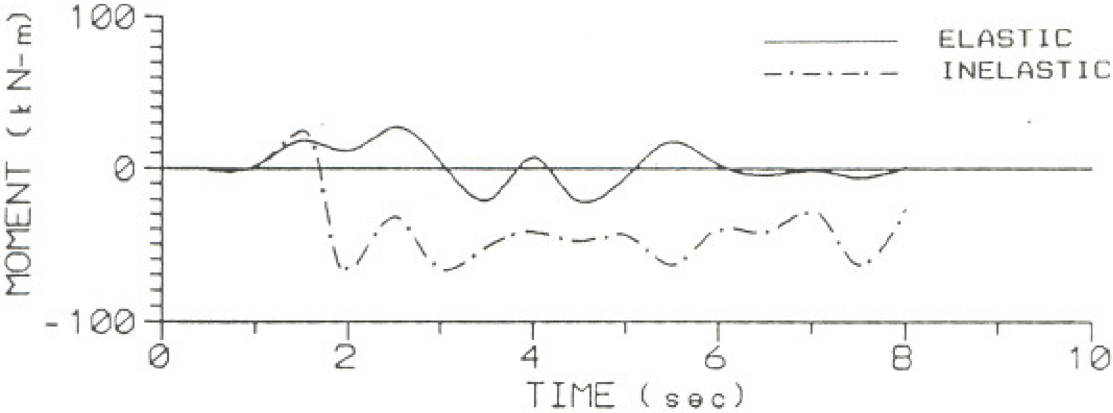


Fig. 5.7 The Elastic and Inelastic Responses: Bending Moment of Section E of the Infilled Space Frame-1.

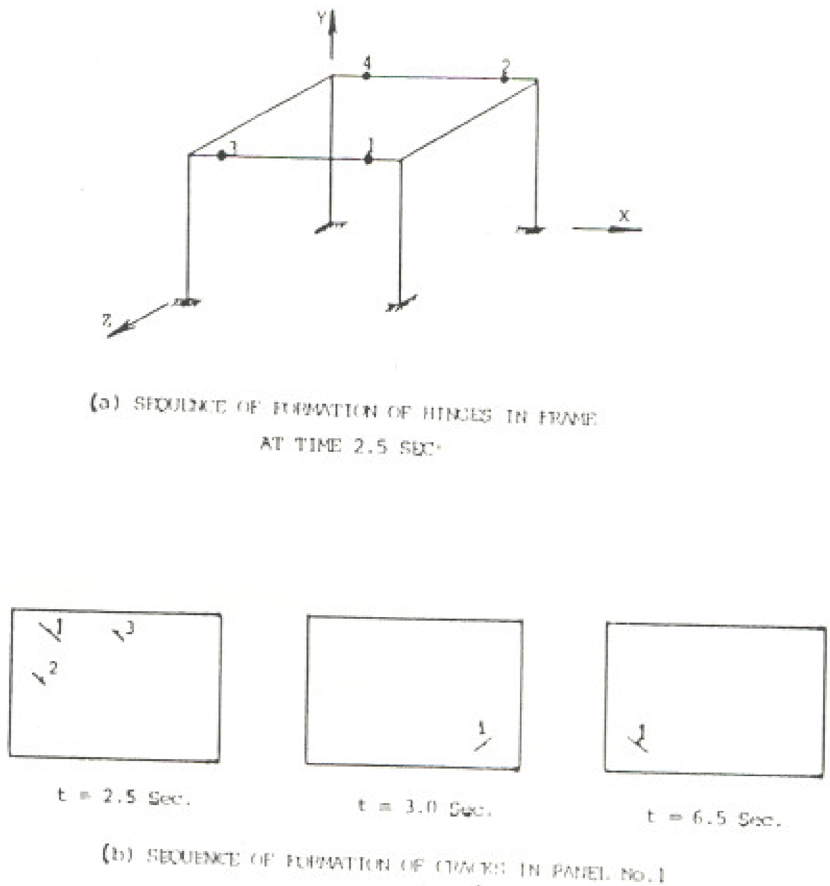


Fig. 5.8 The Sequence of Formation of the Cracks and the Hinges in the Infilled Space Frame-1 at Different Times.

that this type of degradation has negligible effect on the results.

5.4 Simulation of Post-Earthquake Damage/Failure

The post-earthquake damage/failure analysis of the buildings situated in India has been carried out with the recorded time histories of the earthquakes.

5.4.1 Cycle-Stand-cum Canteen Structure

Cycle-stand-cum canteen structures were constructed for I.T.Is. at Darbhanga and Munghyer, Bihar, India in 1968. The buildings were basically framed structures with two rows of five circular columns each. There were no filler walls at the ground storey level (flexible storey). The upper storey had ten reinforced concrete rectangular columns exactly above the circular columns of ground storey and connected with beams and filler walls in between. The unsymmetry arised due to a staircase connecting the ground floor to the first floor as shown in Fig. 5.9.

The structures at both the places were located on the alluvial soil and were subjected to Bihar-Nepal Eartquake of Aug. 21, 1988. The structure at Darbhanga was nearer to the epicenter than that of Munghyer.

□ **Earthquake Damage/Failure:** During August 21, 1988, Bihar-Nepal Earthquake, the structure at Darbhanga had totally collapsed. But the structure at Munghyer developed cracks at the ends of most of the ground storey columns and is still standing in the damaged condition [Thakkar *et al.* (1990), Kaushik (1990) and Paul *et al.* (1990)].

□ **Dynamic Analysis:** The idealised cycle-stand-cum canteen structure has been shown in Fig. 5.10. For dynamic analysis of the structure each frame member has been modelled with one frame element and each infill wall with one panel element. The interface between the frame and the infill has been modelled with interface elements. The reinforcement details in various members of the building frame are not available. The reinforcement in the columns of such buildings is

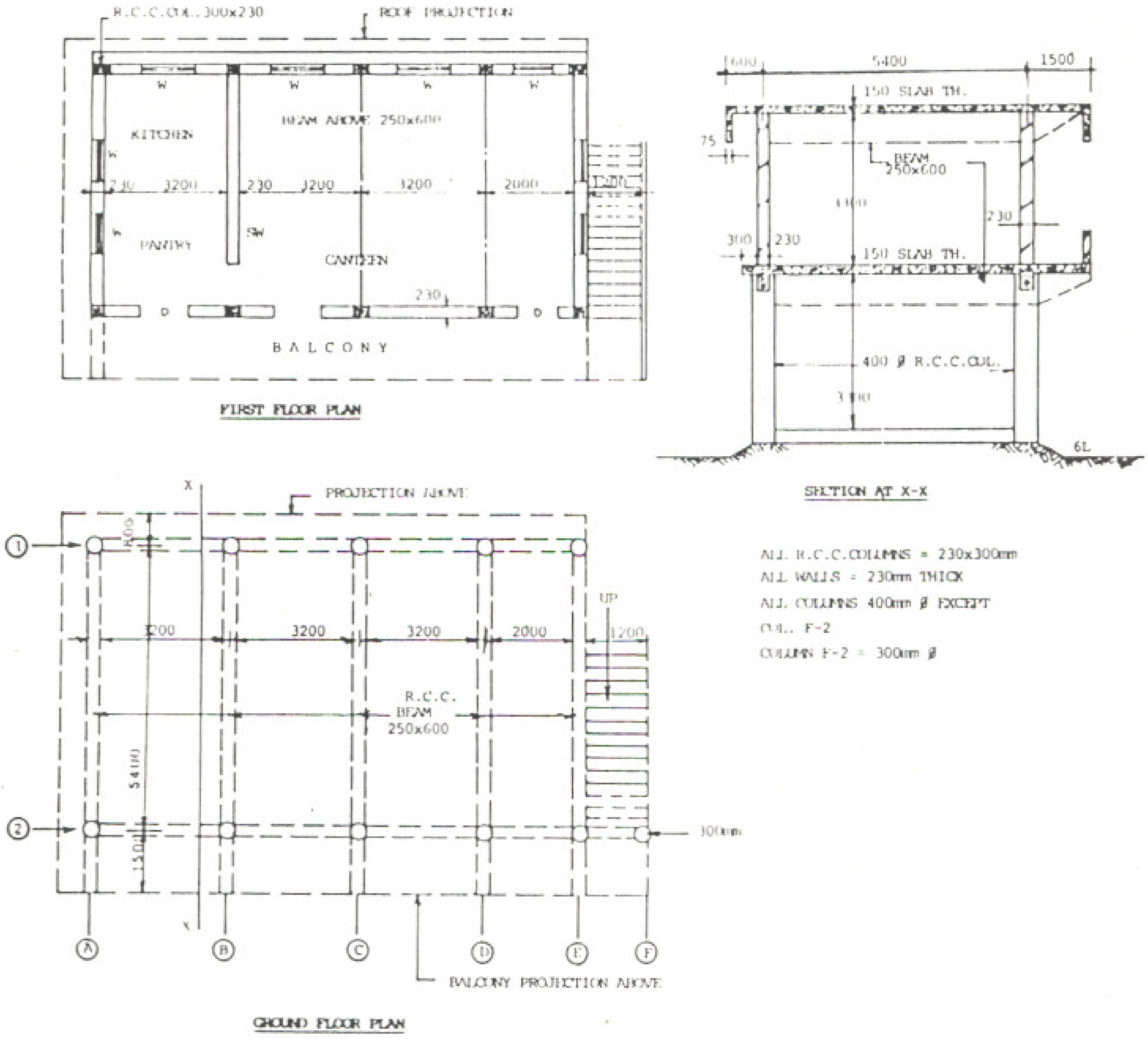


Fig. 5.9 Cycle Stand cum Canteen Structure.

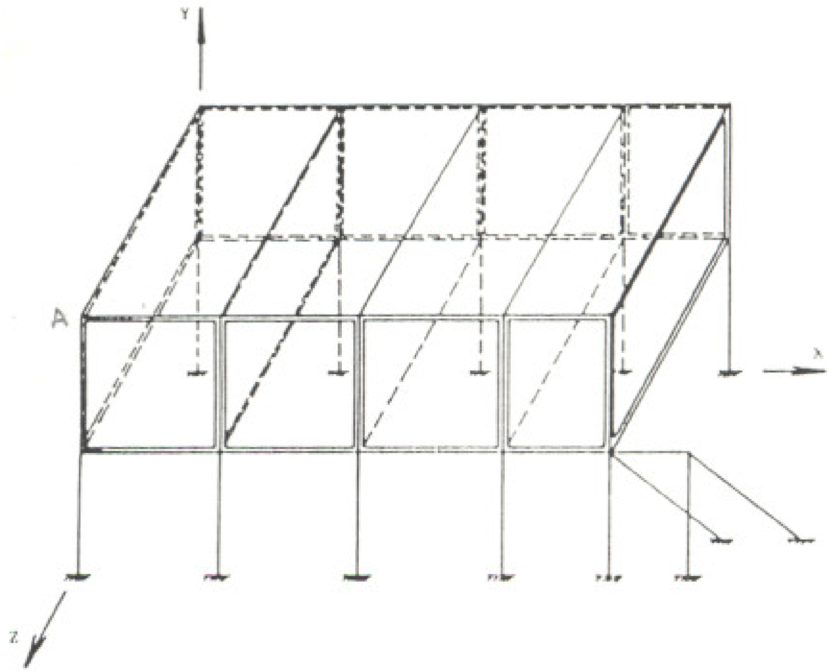


Fig. 5.10 Cycle Stand cum Canteen Structure: Skeletal Frame with Infills.

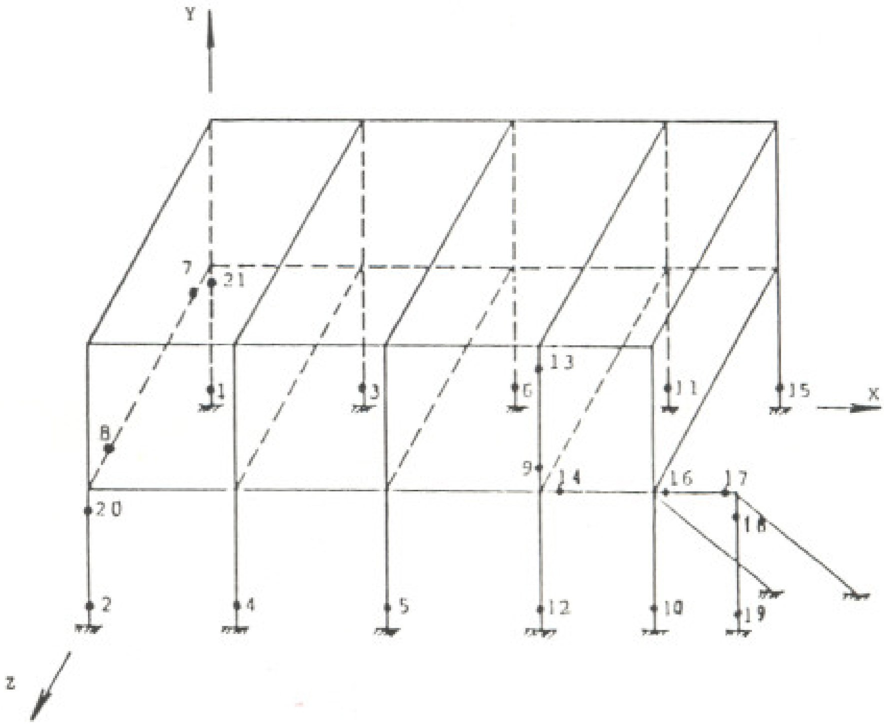


Fig. 5.11 Formation of Plastic Hinges at Various Times.

generally of the order of one per cent. To investigate the reason for the collapse of the structure an analysis has been carried out assuming one per cent steel in the column sections.

The time history record of earthquake i.e accelerogram is available only for Munghyer, no such data is available for Darbhanga as the instrument went out of scale during shaking. So the investigation has been carried out using the data at Munghyer only. The structure has been analysed with all the three components of the earthquake namely, the transverse, the vertical and the longitudinal components (Fig. A.2) acting in the X, Y and Z directions of the structure, respectively, shown in the Fig. 5.10. The damping has been assumed to be 5 per cent of critical damping. Apart from self weight of the structure, the mass due to 25 per cent of live load at first floor has been considered.

□ **Dynamic Response :** The inelastic response of the structure with one per cent of steel in the columns has been obtained. The sequence of formation of plastic hinges in the frame is shown in Fig. 5.11 and listed in Table 5.2. The response in terms of deflection and acceleration at the point A at the roof level have been plotted in Figs. 5.12. and 5.13, respectively. The response has been observed to be maximum in the Z-direction. The maximum inelastic deflection of 20mm occurs at 9.5sec and the maximum acceleration of 5.00m/sec^2 in the Z-direction.

Initially, the structure behaves elastically. At the time of 1.0sec, the plastic hinges appear at the bottom ends of almost all the columns and in the beam on extreme side opposite to that of stair case as shown in Fig. 5.11. At the time of 2.0sec, the hinges appear at the bottom of the remaining columns, stair case and beam at the first storey where there has been no infills. At the time of 3.0sec, hinge appeared on the top of left hand front column and on the top of column at its back. No hinge in the frame or crack in the infills appeared in the structure after this time. The inclined legs of the stair case have not shown any plasticisation and have possibly prevented the structure at Munghyer from collapse.

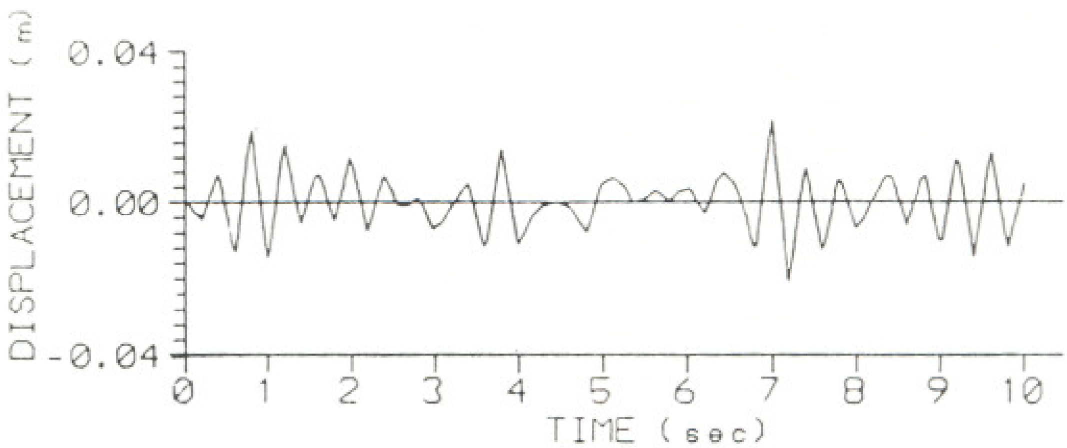


Fig. 5.12 The Earthquake Response: Deflection of the Point A.

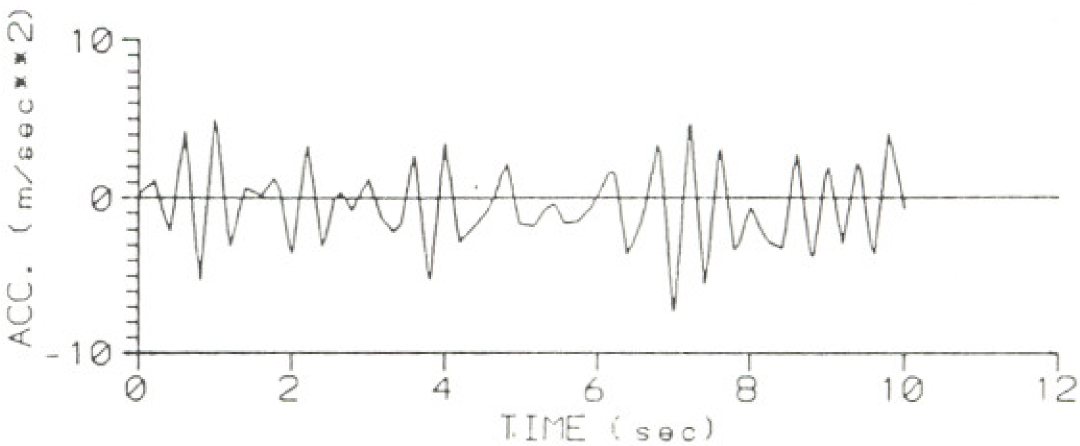


Fig. 5.13 The Earthquake Response: Acceleration of the Point A.

Table 5.2 Sequence of Formation of Plastic Hinges in Cycle stand cum Canteen Structure

Time sec	Sequence of Plastic Hinges in the Skeletal Frame	Deflection δx (mm)
1.0	1 to 10	-14.00
2.0	11 to 19	8.00
3.0	20	7.00
7.0	21	16.00

The analysis has exhibited the formation of hinges in most of the columns and in some of the beams of the ground storey without converting the structure into a mechanism. The post-earthquake observations have also shown the development of wide cracks at the ends of most of the columns of the ground storey without resulting in the collapse of the structure.

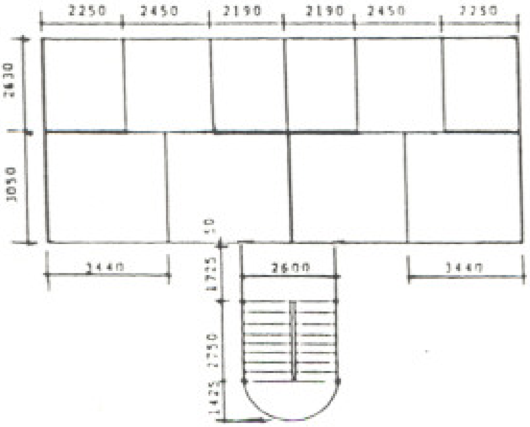
5.4.2 Staff Quarter, Reserve Bank of India, Guwahati, Assam

One of the residential buildings of the staff quarter complex of Reserve Bank of India, situated at Hatigaon, Guwahati consisting of a four storey reinforced concrete frame with masonry infills has been studied for earthquake damage simulations. The building is symmetrical in plan with about 80 square meter of floor area. There is an independent front staircase having an expansion gap of 50mm as shown in Fig. 5.14. The important features of the building are:

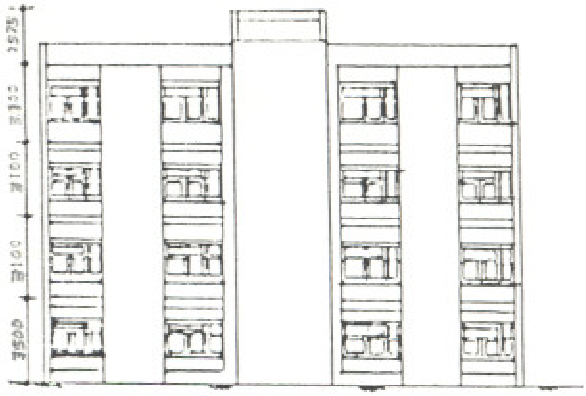
(i) The ground storey is 3.55m high with the foundation level being 3.05m below the made up ground level. Other three storeys are 3.10m high each.

(ii) The internal and external masonry walls are of 125mm and 225mm thick, respectively.

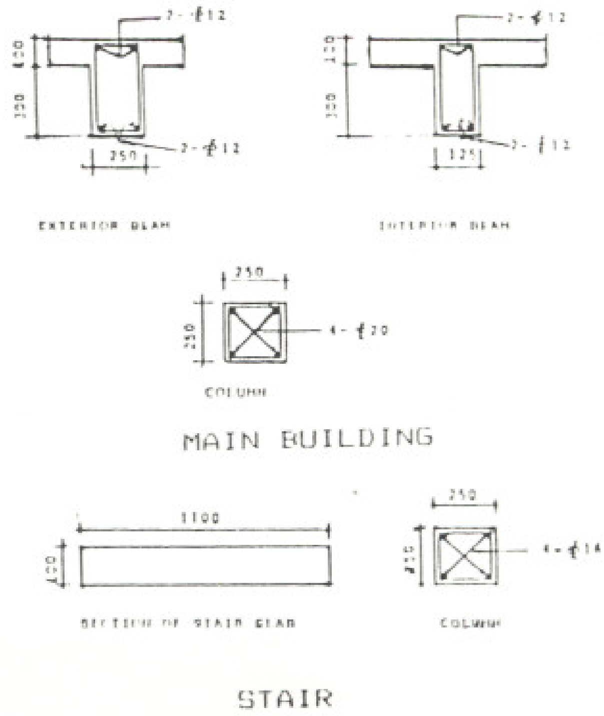
(iii) The foundation consists mainly of isolated footings below the columns. However, there are two numbers of combined footings connecting four columns.



(a) PLAN



(b) FRONT ELEVATION



(c) SECTIONS OF THE BUILDING

Fig. 5.14 Staff Quarter R.B.I., Guwahati.

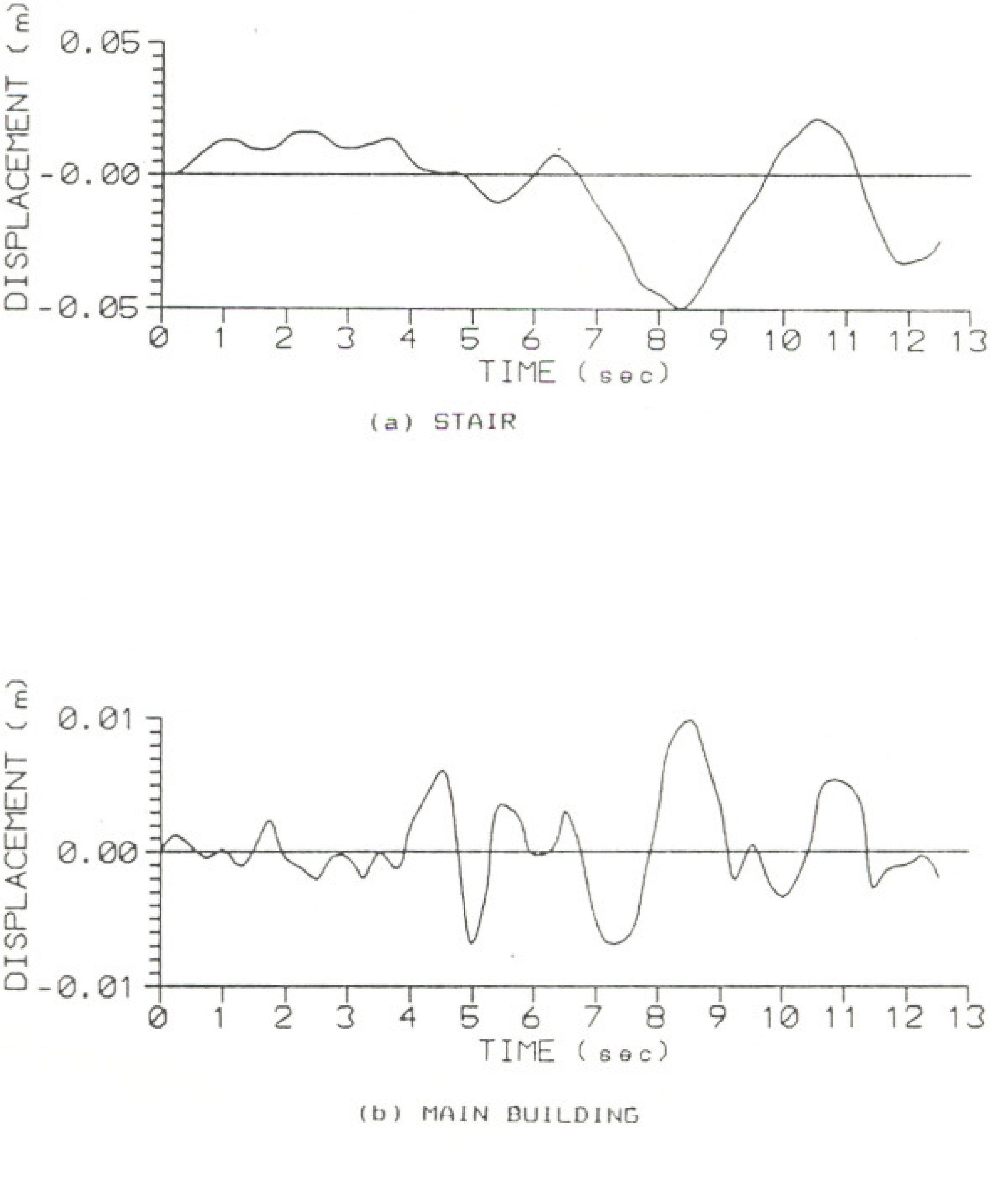


Fig. 5.15 The Displacement Response at Roof Level for Stair and Main Building, RBI, Guwahati.

□ **Earthquake Damage:** During the North-East India Earthquake of August 6, 1988, the building was damaged. The major damage was observed in the region of expansion joints connecting floor slab of the main building and the landing slab of the front staircase. The extent of the damage was relatively severe at the terrace level as compared to the bottom floors due to the large relative displacement at the top floors leading to hammering and heavy pounding. The other forms of damage [Gupta(1990) and Paul *et al.* (1990)] were:

(i) cracking/crushing of concrete at the floor levels;

(ii) spalling of concrete, falling of plaster from the ceilings, floor and adjoining walls;

(iii) collapse of masonry walls of the staircase at the expansion gap region;

(iv) exposure of reinforcements due to hammering and pounding; and

(v) horizontal cracks in the masonry wall at the junction of the brick wall and the floor beam; and

(vi) damage to the door frames.

□ **Ground Motion Record:** During the North-East India Earthquake of August 6, 1988, no strong motion recorder was in operation close to the building under investigation [Chandrasekaran et al. (1989)]. In order to arrive at reasonable time history records, the records at stations near the building are taken into consideration. Considering their distances and local site conditions the time history records of station at Loharghat (Fig. A3) has been chosen for the analysis.

□ **Dynamic Analysis:** The main building and the staircase structures have been considered as two separate structures. Each frame member has been modelled with one frame element and each infill wall with one panel element. The interface between the frame and the infill has been modelled with the interface elements. The effect of the infills in the staircase has not been considered as the infills had openings. The structure has been excited by the three components of earthquake for 10sec. The damping has been assumed 5 per cent of critical damping. Apart from the self weight of the structure, the mass due to 25 per cent of live load at various floors has been considered.

Both the structures have been observed to remain within elastic limit. The displacement response of the main building and of the staircase at the roof level have been shown in the Fig. 5.15. The maximum out-of-phase displacement of 59.50mm has been observed at roof level. If the soil-structure interaction is considered then this out of phase displacement will be even more. Therefore, the gap of 50mm provided between the two separate structures was inadequate to accommodate their total out-of-phase displacements at the upper floor levels and explains the heavy pounding and hammering action at the upper storeys causing damage. Response analysis indicates that the stresses in the building and the staircase members, were within the safe limit and hence members are not damaged. This fact is correlated by the post-earthquake observations.

5.4.3 Reinforced Concrete Frame Office Building, Diphu

This office building is comprised of four storey reinforced concrete frames with masonry infills. The building is L-shaped in plan as shown in Fig. 5.16. The other important details are:

(i) The ground storey is 4.24 m high, except for the two rows of the column on the right side of the building where the height is 2.40m. This arrangement was adopted because the ground was not level.

(ii) The first four left hand side exterior panels of the ground storey are having ventilator openings just beneath the first floor beam level. The infill walls are present only below the ventilator openings. The infill walls were not provided in the last two panels on the right hand side of the building. All the exterior infills of the first, second and third storeys are having window openings at the two sides of each panel and therefore not treated as the infill panels in the analysis.

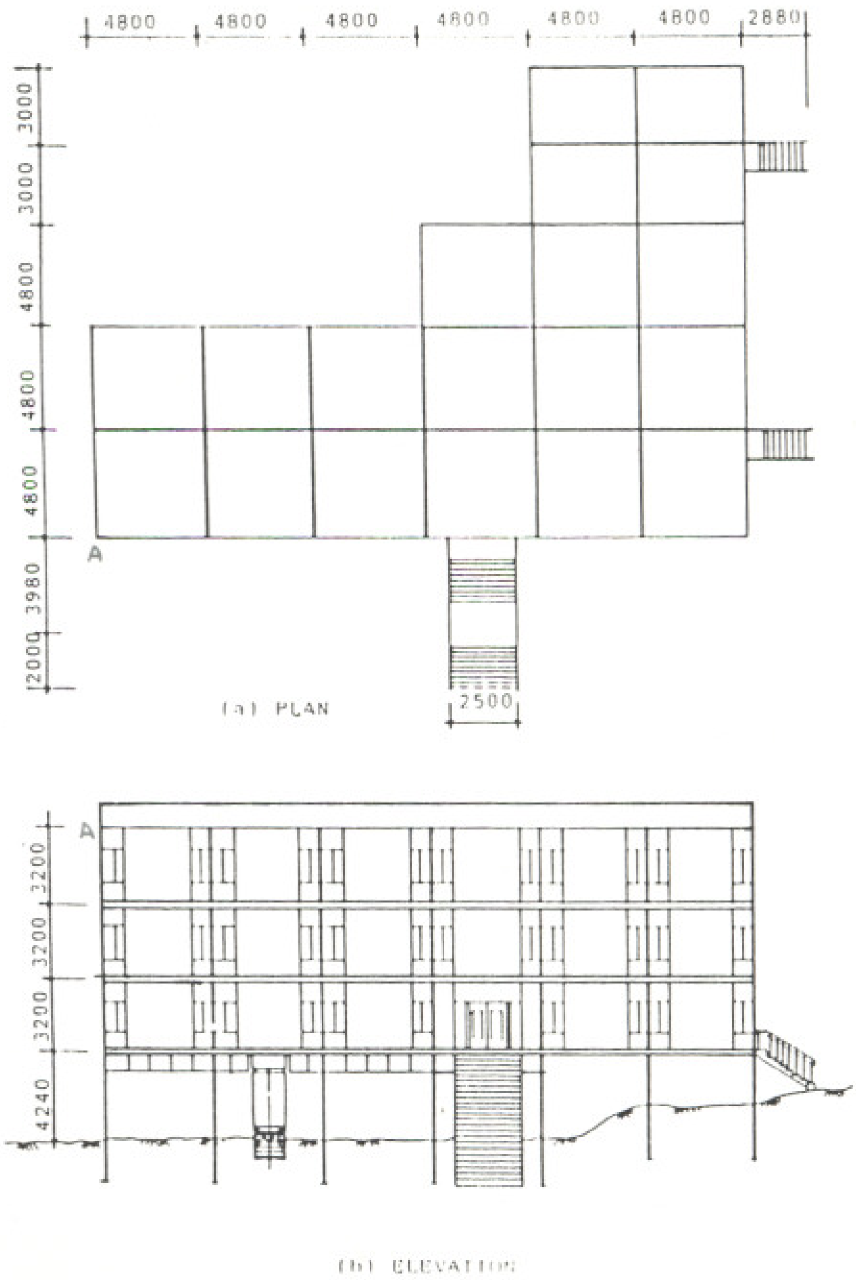


Fig. 5.16 Office Building, Diphu.

□ **Earthquake Damage:** During the North-East India Earthquake of August 6, 1988, the structure was damaged. Most of the damage was concentrated at the ground and the first storeys, the upper storeys being lightly damaged. Some of the important post-eartquake observations [Gupta(1990) and Paul *et al.*(1990)] were:

(i) Cracks have appeared in most of the ground floor columns. The columns were severely damaged in the region of ventilator/window openings or where there were no infill walls. This may be due to the sudden change in stiffness caused by the openings at the floor levels. Cracks in some of the first floor columns were also noticed.

(ii) horizontal cracks along the line joining floor beams and masonry walls;

(iii) diagonal cracks at the masonry walls;

(iv) failure at the beam-column joint at the first floor level;

(v) vertical cracks along the line joining ground and first floor columns and the masonry walls, and

(vi) falling of plaster from the walls causing exposure of bricks

□ **Ground Motion Record:** During the North-East India Earthquake of August 6, 1988, a Strong Motion Instrument was in operation near the building site [Chandrasekaran *et al.* (1989)]. The earthquake record has been shown in the Fig. A4. It is assumed that similar acceleration values would have been experienced by the damaged building at the time of the earthquake.

□ **Dynamic Analysis:** Each frame member has been modelled with one frame element and each infill wall with one panel element. The interface between the frame and the infill has been modelled with interface elements. The structure is subjected to the longitudinal, transverse and vertical components of the earthquake acting in the X, Y and Z directions, respectively. The columns have been assumed to be fixed at ground level. Only interior infill panels have been considered

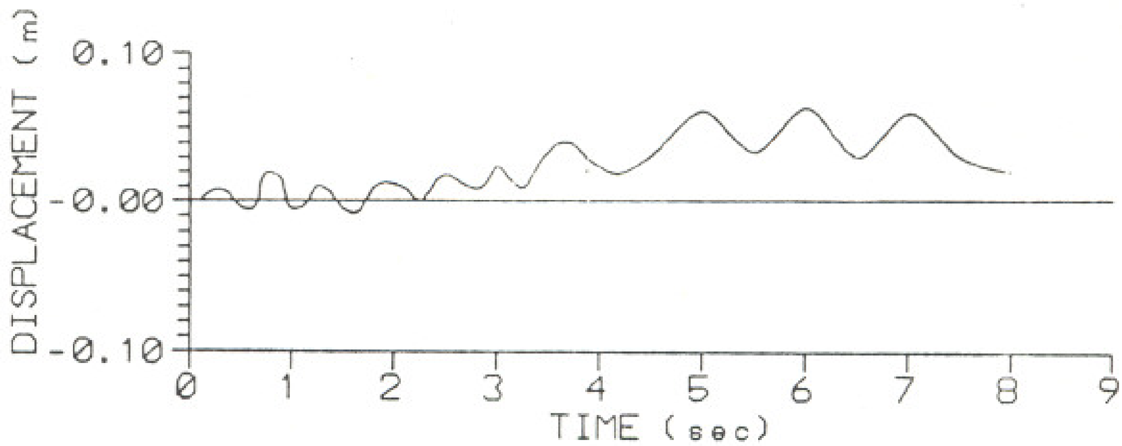


Fig. 5.17 The Earthquake Response: Deflection of the Point A.

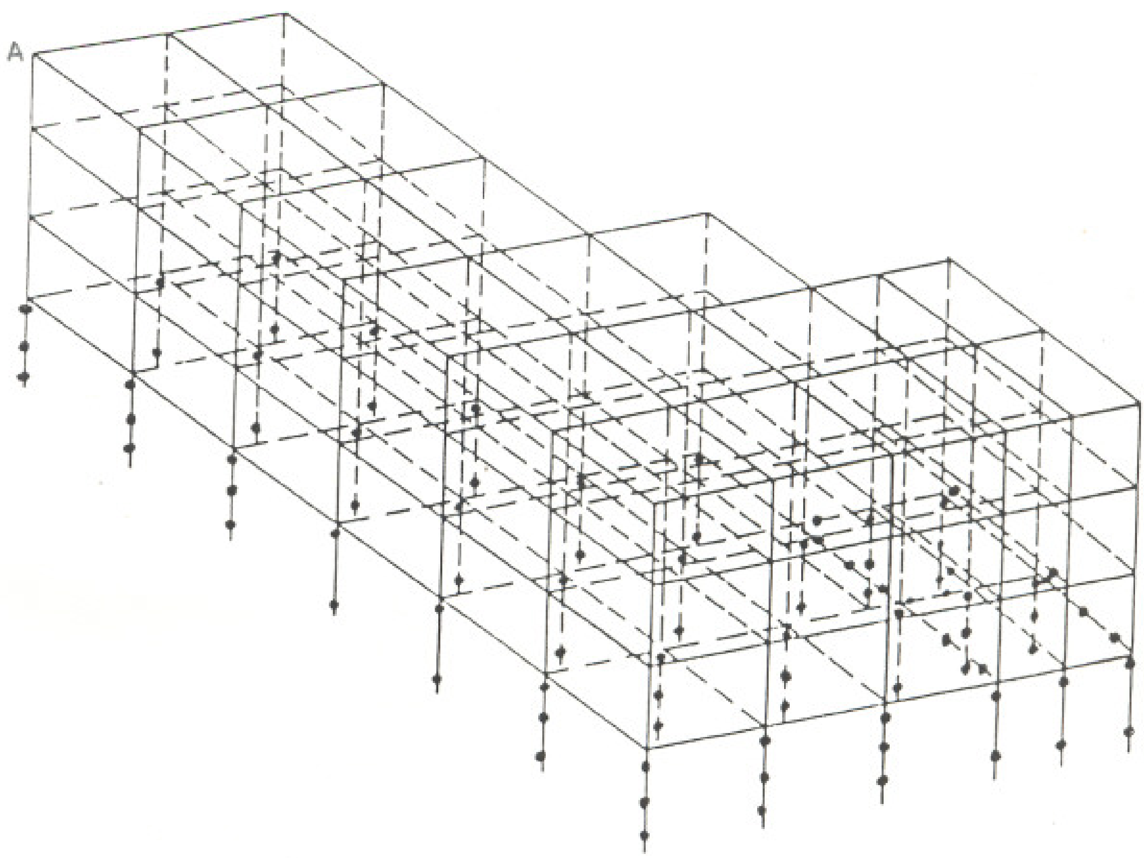


Fig. 5.18 The Formation of Plastic Hinges in Frame for Office Building.

as the outer panels had openings on two sides of the panels. The damping has been assumed 5 per cent of critical damping. The masses are lumped at node points. Apart from the mass of the structure itself, the mass due to 25 per cent of live load at various floors has been considered.

□ **Dynamic Response:** The inelastic response in terms of deflection at the point A at the roof level has been plotted in Fig. 5.17. The maximum inelastic deflection having a value of 40.0mm occurs at 3.8sec in the X-direction at the point A.

The location of formation of plastic hinges in the frame has been shown in Fig. 5.18. No cracks have been observed in the infills. The plastic hinges appeared at the top and bottom of all the ground storey columns. Some ground storey columns plasticised near the centre also. The beams at the first floor level at the rear of the left hand part of the building also indicated plasticisation. It may be due to the unsymmetry of the building. This explains the cause of observed damage to the columns of the ground storey at the ends and cracks in the beams at first floor level.

5.5 Concluding Remarks

Based on the predicted inelastic response of 3-D infilled reinforced concrete frames and post-earthquake failure/damage studies following conclusions are drawn:

□ The computational model and the procedures developed are able to analyse and represent the inelastic behaviour of three dimensional infilled reinforced concrete frame systems adequately. The model can predict the entire time history response of infilled frame systems under earthquake excitation.

□ The proposed inelastic algorithms for inelastic analysis predict the sequence of formation of plastic hinges in the frame members and of the cracks in the infills.

- During inelastic dynamic response analysis, most of the plastic hinges and cracks have been found to form in the vicinity of peak acceleration of earthquake excitation. Permanent plastic deformation has been observed at the end of the analysis due to the formation of plastic hinges and cracks.

- In the inelastic dynamic analysis, the plastic hinges in the frame and the cracks in the infill appear simultaneously. However, the plastic hinges and the cracks 'disappear' on reversal of loads and also on reduction of magnitude of exciting force.

- The post-earthquake damage/failure simulation studies of infilled reinforced concrete frame buildings using the proposed algorithms clearly identify the critical locations where the damage can occur. The analysis represents realistically the dynamic response over the given time interval and indicates the structural deficiencies with regard to their earthquake resistance and the reasons for the actual post-earthquake damages observed. The stiffness of the infill in the building plays an important role and should be considered in the analysis. The close agreement between the observed and the predicted behaviour establishes the reliability of the proposed model.

- The elastic analysis is not adequate and an inelastic analysis is required to simulate the real behaviour of infilled frame buildings.

5.6 References

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SIMPLIFIED MODEL FOR REINFORCED CONCRETE INFILLED FRAMES

6.1 Introduction

Although the analysis of infilled reinforced concrete frames by the proposed algorithms is accurate but it is computationally costly and time consuming for the design of such building systems. For the analysis of real sized buildings using the proposed algorithms, the analyst has to use the panel element and the interface element with the frame element. It increases the computer memory and time requirements tremendously. So a simplified method for the analysis of such systems is needed. In this Chapter a simplified model for predicting the behaviour of the reinforced concrete infilled frames with a reasonable accuracy has been proposed and the results are compared with those obtained from the rigorous analysis using the computer program NIFAP.

6.2 Simplified Approach using Diagonal Member

The study of the infilled reinforced concrete frames in the previous Chapters indicate that in an infilled frame the panel behaves predominantly like a strut along the compression diagonal and a tie along the tension diagonal. However, a masonry panel under tension takes very small load. Under dynamic loads, the compression and tension diagonals keep on interchanging, the compression diagonal taking the load while the tension diagonal taking negligible load. To model this type of behaviour the infilled panel has been replaced by a single diagonal member acting both as a strut and a tie as shown in Fig. 6.1.

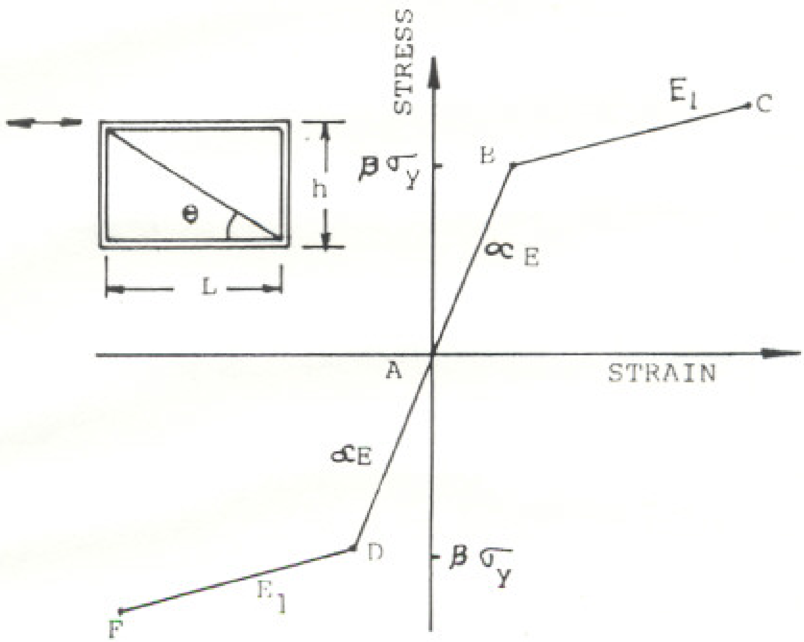


Fig. 6.1 Stress-Strain Behaviour of the Equivalent Diagonal Member.

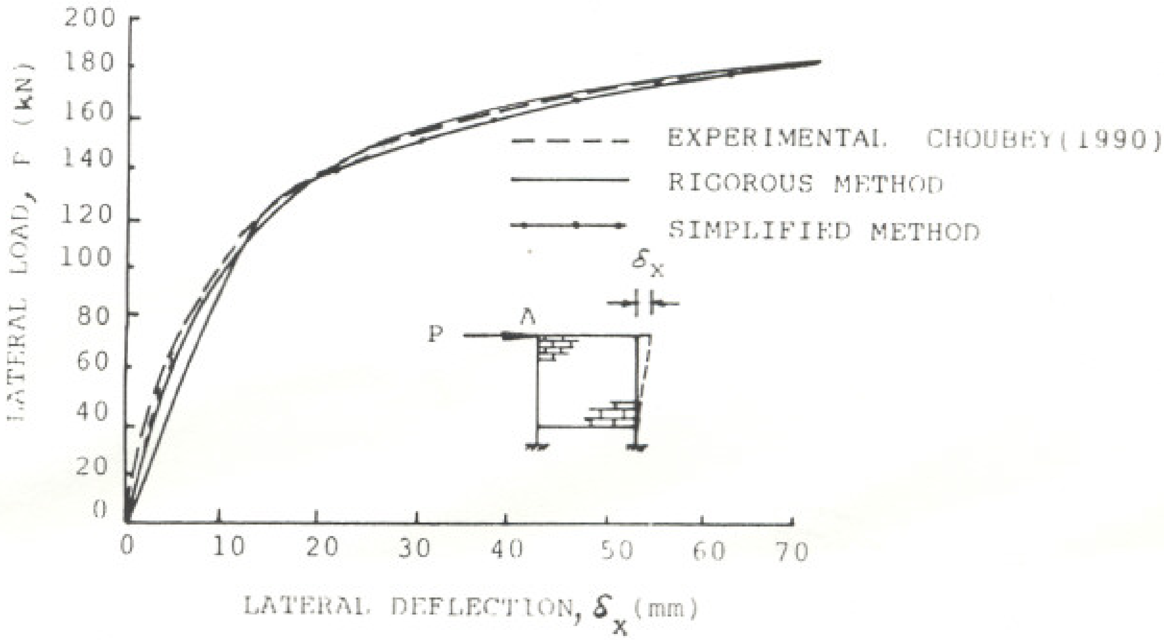


Fig. 6.2 Load-Deflection Behaviour of the Infilled Reinforced Concrete Frame-1

In the inelastic phase, cracking and crushing of the masonry infill results in the stiffness degradation and deformed shape results in the reduction in the contact surface of the panel with the frame. For modelling the nonlinear behaviour due to stiffness degradation of the panel a bilinear stress strain behaviour as shown in Fig. 6.1 has been assumed. Here σ_y and E are the yield stress and Young's modulus of elasticity of masonry under uniaxial compression, respectively. A factor α has been used to incorporate the effect of reduction in the stiffness due to separation and slippage of the panel from the frame. With the increase of the load, due to the deformed shape there is reduction in the contact area between the frame and the panel resulting in the reduction of the equivalent width of the diagonal member and hence the load carrying capacity of the diagonal member near the yield load. To incorporate this effect the yield stress of masonry has been reduced by a factor β. From the studies the values of α and β have been found to be 0.85 and 0.67, respectively. The thickness of the diagonal member has been taken to be equal to the thickness of the panel.

Smith [1962] has proposed a method in which the width of the equivalent strut of the panel depends upon its aspect ratio. However, Liauw and Kwan [1984] have suggested that the equivalent strut width is independent of the aspect ratio and is given by

$$w = 0.45\ h \cos \theta \qquad (6.1)$$

where h is the height of the panel as shown in Fig 6.1. This strut width has been used in the present study.

6.3 Validation of the Simplified Method

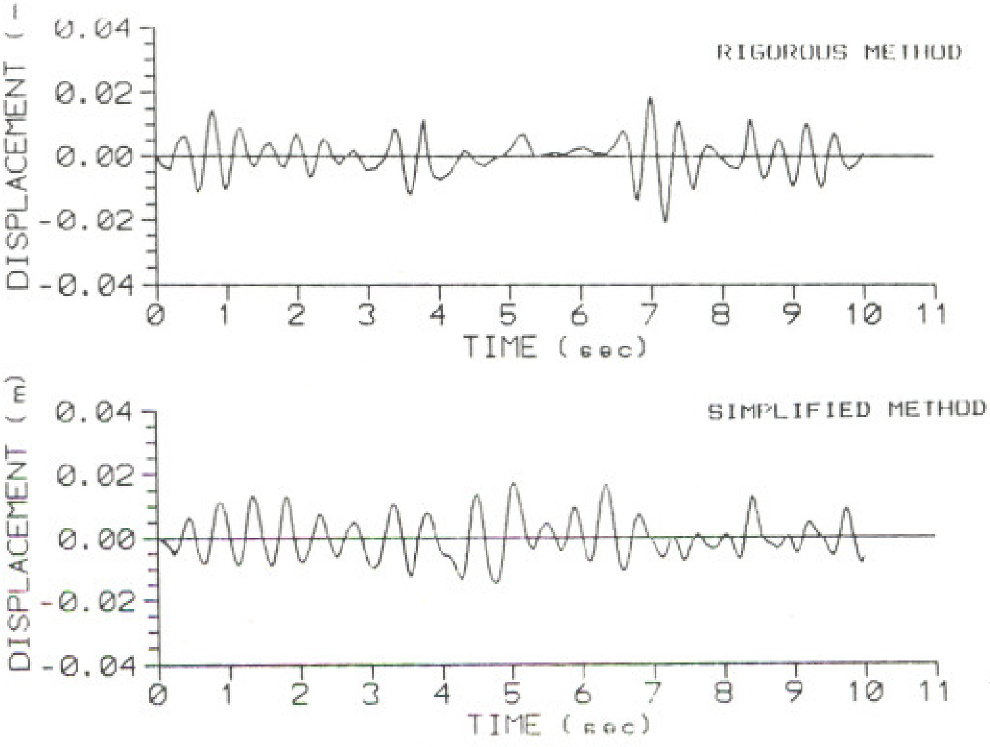
To establish the validity and effectiveness of the simplified diagonal member, the method proposed for the analysis of the reinforced concrete infilled frames for the structure studied earlier in the Sec. 4.3 has been reanalysed. The frame has been discretised using the frame element shown in Fig. 4.1(c) but the panel has been replaced by an equivalent diagonal member as explained above. The load deflection curve has been compared with that obtained by the rigorous method and the experimental results [Choubey (1990)] in Fig. 6.2. The locations of the plastic hinges developed in the frame have been found to be exactly the same as predicted by the rigorous method as indicated in Fig. 4.2(b). The CPU time required for the simplified analysis has been found to be about one third of that required for the rigorous analysis.

The second structure analysed is that of cycle stand cum canteen discussed earlier in the Sec. 5.4.1. The panels have been discretised by the equivalent diagonal members with each frame member being modelled with the three noded frame element. The structure has been analysed for all the three components of the Bihar-Nepal earthquake recorded at Munghyer as discussed in the Sec. 5.4 and shown in the Fig. A.2. Both the linear and nonlinear displacement response in the Z-direction at the point A have been obtained and compared with those obtained by the rigorous method in the Fig 6.3. Although the amplitude of the response waves differ but the time periods of the waves remain the same. The maximum deflection obtained both by the rigorous and the simplified methods are 20mm.

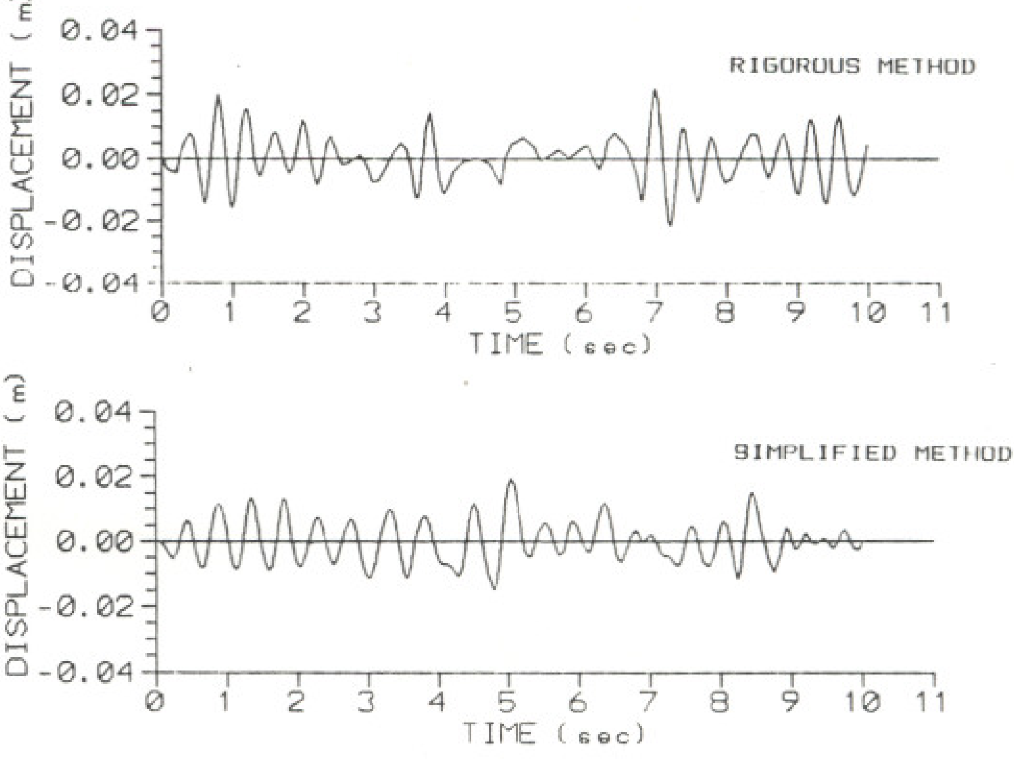
The plastic hinges predicted by the simplified method are shown in Fig. 6.4 and those predicted by NIFAP program in Fig. 5.11. The sequence of their formation is given in Table 5.2. The locations of most of the plastic hinges predicted by the two methods are the same. However, the sequence of their formation differs in the case of a few hinges.

Table 6.1 Sequence of Plastic Hinge Formation in Cycle Stand cum Canteen Structure.

Time sec	Sequence of Plastic Hinges formed as given by	
	Simplified Method Fig. 6.4	NIFAP Fig. 5.11
1.0	—	1 to 10
2.0	—	11 to 19
2.4	1 to 4	—
3.0	—	20
4.8	5	—
5.6	7	—
6.4	8 to 22	—
7.0	—	21



(a) LINEAR



(b) NONLINEAR

Fig. 6.3 Earthquake Response: Deflection at the Point A.

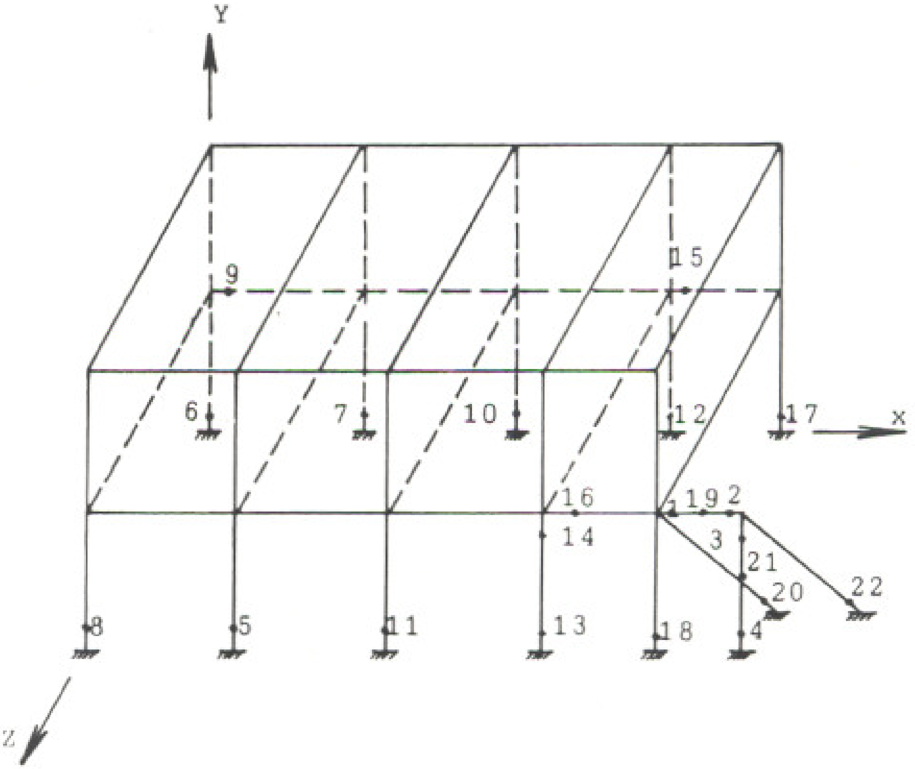


Fig. 6.4 Formation of Plastic Hinges in the Frame for Cycle Stand cum Canteen Structure.

The CPU time required for the simplified analysis has been found to be about one third of that required for the rigorous analysis. The in-core memory requirement for the simplified method is about 50 per cent of that required by the rigorous method.

6.4 Concluding Remarks

□ For all practical purposes the proposed simplified method using the equivalent diagonal member gives reasonably comparable results with those obtained by the rigorous method. The time period and maximum displacements which are required for the design of the infilled reinforced concrete systems are comparable.

□ The locations and the sequence of formation of plastic hinges in the frame as predicted by the simplified method for inelastic static analysis are exactly the same as predicted by the rigorous method. However, the location and sequence differ for some of the plastic hinges in the dynamic analysis.

□ The computational time and in-core memory requirements for the simplified method are about 30 and 50 per cent, respectively, of that required by the rigorous method.

□ Since the proposed simplified model gives the acceptable results required for the design of the reinforced concrete frame systems at much lesser computational effort, it may be used in the design offices.

6.5 References

1. Liauw, T.C. and Kwan, K.H.(1984), Non-Linear Behaviour of Non-Integral Infilled Frames, Computers & Structures, 18(3), 551-560.

2. Smith, S.B.(1962), Lateral Stiffness of Infilled Frames, J. Struct. Div., ASCE, 88(ST6), 183-199.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 Introduction

In this thesis, investigations have been conducted to study the response of infilled reinforced concrete frames under earthquake excitation and to effectively simulate the post earthquake damage/failure of the structures. Various aspects of the problem viz structural modelling, material modelling, development of an algorithm and its computer implementation/codification for the inelastic analysis, prediction of overall earthquake response and post earthquake failure/damage simulation have been examined. Throughout the investigations, an attempt has been made to explore the areas of current state of knowledge which are less clearly understood and to highlight some problems where emphasis in analysis and design has been lacking. The significant conclusions on the basis of the present study are summarised in the following sections.

7.2 Review of Literature

Based on the critical review of the literature on the linear and nonlinear, static and dynamic behaviour of the infilled frames and other related structural systems, the following observations are made.

- In the dynamic analysis of a complete building system, the inclusion of the effect of infills is essential for a realistic prediction of the behaviour, even though, it complicates the problem and increases the computational effort.

□ There is very limited literature available on dynamic response of 3-D infilled reinforced concrete frames.

□ The finite element method has been extensively used in the analysis of infilled frames. The results obtained by 3-noded frame element for skeleton frame, 8-noded isoparmetric element for the panel and 6-noded interface element for the interface joint have been reported to be in good agreement with the experimental results.

□ Amongst the models for inelastic 3-D analysis of reinforced concrete frames, the section models with three dimensional point hinges at the ends or at the Gauss points have been found to be satisfactory.

7.3 Structural Modelling

A 'realistic' mathematical modelling for an infilled reinforced concrete frame system should be able to simulate the actual observed response of such systems. To account for the probable unsymmetry in plan or/and in elevation of an infilled reinforced concrete frame system a 3-D mathematical model has been used to simulate the response. The model uses 3-D beam-column element to idealise the space frame. The frame infill mortar joint has been modelled by 3-D interface element and the infill has been discretised by a membrane type of panel element. The model is capable of predicting the sequence of formation of the plastic hinges in the frame and the cracks in the infill.

The analysis of a number of structures has demonstrated that the proposed model is capable of simulating the behaviour of such systems realistically. The eigen values, time history response, load deflection behaviour and failure loads predicted by the proposed model have been observed to be reasonably close to the experimental values. The location and sequence of formation of the plastic hinges in the frame and cracks in the infill have been found to follow the pattern given by the experimental investigations. Thus, the simplifications made in modelling have been found to be reasonable. However the results obtained by the simplified models like an equivalent strut method are too stiff.

7.4 Material Modelling

The frame element used for the discretisation of the reinforced concrete frame member considers the interaction of axial force, biaxial bending and torsion for the development of the surface. For the panel element in compression, the material has been assumed to be linearly elastic until failure and on crushing the stiffness has been assumed to reduce to nearly zero. In tension, on cracking the stresses and the stiffness normal to the crack have been reduced to zero, however, a partial shear transfer due to interlocking between the particles has been maintained. The stiffness and stresses along the crack have also been maintained.

At the interface between the frame and the infill, initial gap, closing or opening of gaps or sliding between the frame and the infill have been considered. The tangential stress-strain relationship has been assumed to be elastic perfectly plastic. It is based on Mohr-Coulomb yield criterion with negligible cohesion. In case the normal strain is tensile, a separation has been assumed, otherwise contact has been maintained. If the normal strain is compressive and tangential strain exceeds the co-efficient of friction times the normal strain, a slip has been assumed to take place. The stiffness of the interface element at each Gauss point has been modified according to the interface conditions at the Gauss point.

7.5 Development of Computer Program

A finite element program (NIFAP) based on the proposed algorithms has been developed for the elastic and inelastic, static and dynamic response of three dimensional infilled reinforced concrete frames. It is a general purpose program with the capacity depending mainly on the total number of nodal points in the desired system, the number of eigen values required in the dynamic analysis, the operating system and the computer system used.

The program predicts the formation of hinges in the frame and cracks in the infills at the Gauss points. It calculates plastic strains, if any, at these points and gives displacements and acceleration history at the desired points and stress history at the desired Gauss points, at the specified time intervals.

7.6 Inelastic Analysis

The proposed algorithms for the inelastic analysis are simple and suitable for 2-D and 3-D infilled frame structural systems. In the frame the plastic hinges have been assumed to form at the Gauss points. The algorithms predict the sequence of formation of the plastic hinges in the frame elements up to 'failure' or development of mechanism. The interaction of axial force, biaxial bending and torsional moment has been considered in the yield criterion for the formation of a plastic hinge in the frame element. The results obtained by the proposed algorithms considering the plastic hinges at the Gauss points are closer to the experimental results as compared to the lumped plasticity approach although computational time and efforts required for convergence are more. In the present investigations of the reinforced concrete infilled frames the plastic hinges at the Gauss points are considered.

Inelastic algorithm considers the gradual deterioration of the stiffness of the structure due to the onset of plasticity in the frame members and cracking in the infill panels. The algorithm adequately incorporates the initial gap, closing or opening of gaps or sliding between the frame and the infill. It has been observed, that it is economical to modify the stiffness matrix after the formation of each new plastic hinge(s) as it leads to the faster convergence.

The validity of the proposed procedure and algorithm for the inelastic analysis of 2-D bare and the infilled frames, and 3-D bare frame have been extensively checked. The close agreement between the experimentally observed load deflection behaviour, separation of the infill from the frame, central strut width and failure mode with those predicted by the proposed model establishes the reliability of the model to predict the behaviour, the failure load and the mode of failure. The inelastic algorithms predict the sequence of formation of the plastic hinges in the frame members and the cracks in the infills.

7.7 Overall Earthquake Response

The objective is to establish the feasibility of both the elastic and inelastic analyses for earthquake response highlighting the importance of certain aspects affecting the infilled reinforced concrete frame behaviour. A number of 2-D and 3-D structures have been analysed using the proposed algorithms.

A building model which ignores the effect of the infill does not represent the realistic behaviour. For the correct prediction of the response of the infilled reinforced concrete frames the effect of the infill and its interaction with the frame should be included in the analysis. However, the exclusion of the frame infill interaction in the analysis predicts a reasonably correct failure load, but gives a stiffer deflection response. So the interaction may be ignored as it saves a lot of computer time and memory requirements.

In the infilled frame systems, formation of hinges in the frame members, cracking of infill panels, closing or opening and sliding between the frame and the infill all have been found to be responsible for their nonlinear behaviour. In the inelastic static analysis, the cracks in the infill are first to develop and subsequently with further increase of the load the infill looses its stiffness which leads to the failure of the frame. With the formation of sufficient number of hinges or cracks, the structure looses most of its stiffness and very large deflections are produced.

In the inelastic dynamic response, the study predicts permanent plastic deformations due to the formation of plastic hinges in the frame and cracks in the infill. In general, it has been observed that the plastic hinges form during the cycle of peak deflection and 'disappear' (the plastic strains continue to be present but stresses are now within the yield surface) when the direction of vibration reverses. Some plastic hinges and cracks disappear from certain locations and new plastic hinges and cracks develop at the other locations.

The study concludes that the elastic analysis is not adequate and the inelastic analysis is required to simulate the true behaviour of the infilled frame systems.

7.8 Post Earthquake Damage/Failure Simulation

The post earthquake damage/failure analyses of infilled reinforced concrete framed buildings using the proposed algorithms have clearly identified the locations of damages, the structural deficiencies with regard to their earthquake resistance and the reasons of the actual damages observed. The infills in the buildings have been observed to play an important role. The realistic behaviour of the buildings with the infills cannot be predicted without considering the effect of the infills in the analysis. The good agreement between the observed and the predicted behaviour establishes the reliability of the proposed model.

From the analyses carried out in the present study, it can be concluded that the computational model, procedure developed and the computer program are able to analyse and predict the inelastic behaviour of three dimensional infilled reinforced concrete frame systems adequately. The program can predict the entire time history response to an earthquake excitement.

7.9 Simplified Equivalent Diagonal Member Approach

A diagonal member replacing the infill panel has been proposed to predict the nonlinear behaviour of the infilled reinforced concrete frames. For all practical purposes the proposed simplified representation of the infill by the diagonal member gives reasonably comparable results with those obtained by the rigorous method and the reported experimental results. The time period and maximum displacements which are required for the design of the infilled reinforced concrete systems are predicted with acceptable accuracy. The locations of the plastic hinges and the sequence for their formation as predicted by the simplified method for the inelastic static analysis are exactly the same as obtained by the rigorous method. However their locations differ for some of the plastic hinges for the dynamic analysis. For the problems solved the computational time and incore memory requirements for the simplified method are about 30 and 50 per cent, respectively, of those required for the rigorous method. The proposed simplified equivalent diagonal member method may be used in the design offices.

7.10 Suggestions for Further Research

It has been recognised that the analysis of the infilled frame systems under earthquake excitation is essential. Detailed studies are required to propose codal provisions for the design of the infilled framed buildings, particularly, in the regions of high seismicity. Some relevant suggestions for future research directions are made below:

- Several improvements in the structural and material modelling are possible to predict inelastic response. The effect of considering the spread of plasticity in the frame element needs to be studied. The appropriate yield criteria to include the effects of shear is required to be established. Stability considerations may be introduced in the analysis. Large deformations alter geometry and could at least be considered in the form of simplified p-Δ effect.

- The experimental and analytical studies are needed to establish the parameters of the proposed simplified model for the infilled reinforced concrete frames.

- The properties of the interface between the reinforced concrete frame and masonry infill need to be established experimentally.

- The soil structure interaction studies provide yet another important area for investigation especially for the structures on the soft soils.

- The effect of degradation of reinforced concrete section due to the development of the plastic hinges and their disappearance need to be established for a 'realistic' inelastic analysis.

APPENDIX A

GROUND ACCELERATION RECORDS OF EARTHQUAKES

A-1 Ground Acceleration Records

The three translational components of ground acceleration records for the various earthquakes used in the investigation have been presented.

A-1.1 El Centro Earthquake

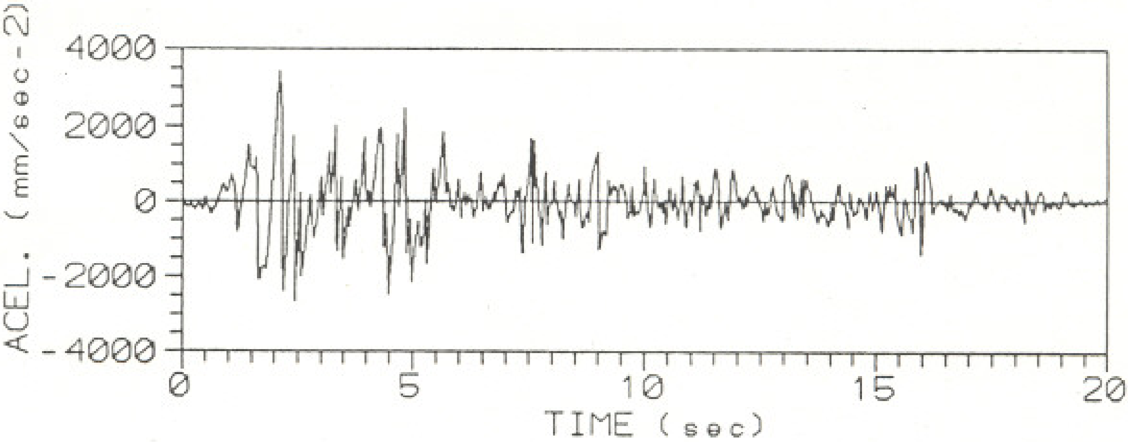
The ground acceleration records for El Centro, California Earthquake of May 18, 1940 have been presented in Fig. A1. The peak acceleration was about 3136mm/sec^2.

A-1.2 Bihar-Nepal Earthquake

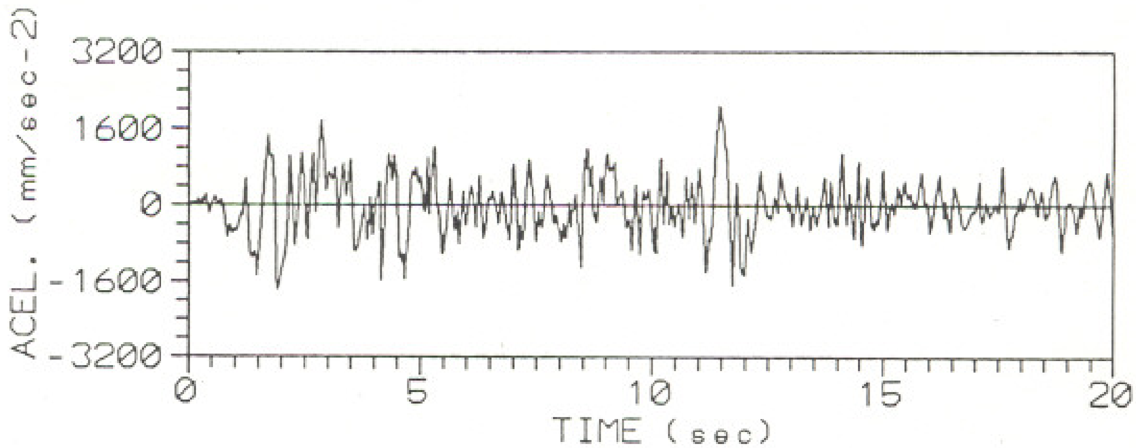
The ground acceleration records for Bihar-Nepal Earthquake of August 21, 1988 recorded at Munghyer have been presented in Fig. A2. The peak acceleration was about 1900mm/sec^2.

A-1.3 North-East India Earthquake

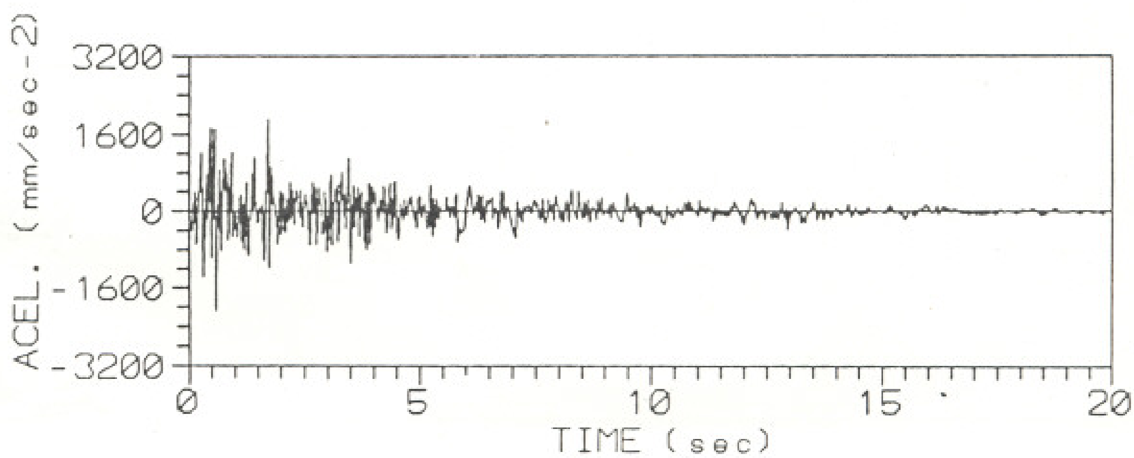
The ground acceleration records for North-East India Earthquake of August 6, 1988 recorded at Loharghat have been presented in Fig. A3. The peak acceleration was about 550mm/sec^2. Figure A4 shows the ground acceleration records of the earthquake recorded at Diphu. The peak acceleration observed was about 3400mm/sec^2.



(a) S 0° E COMPONENT

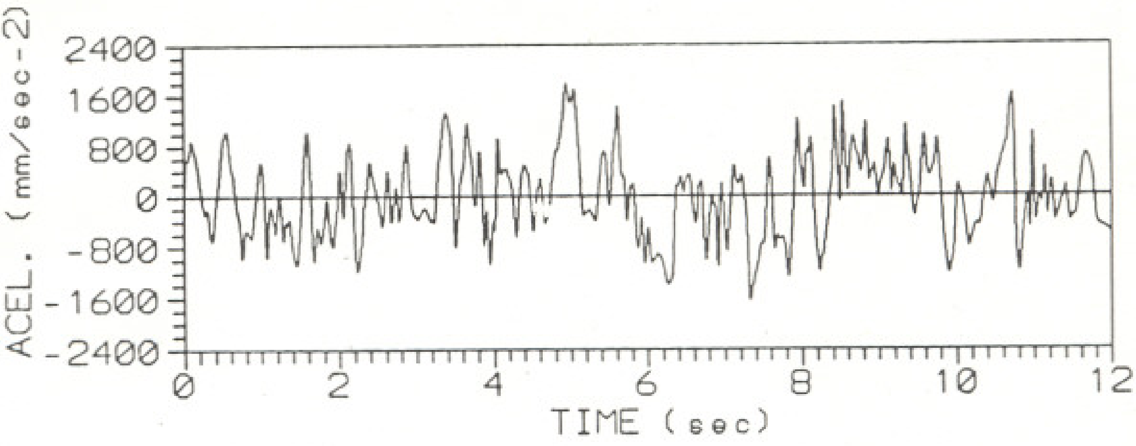


(b) S 90° W COMPONENT

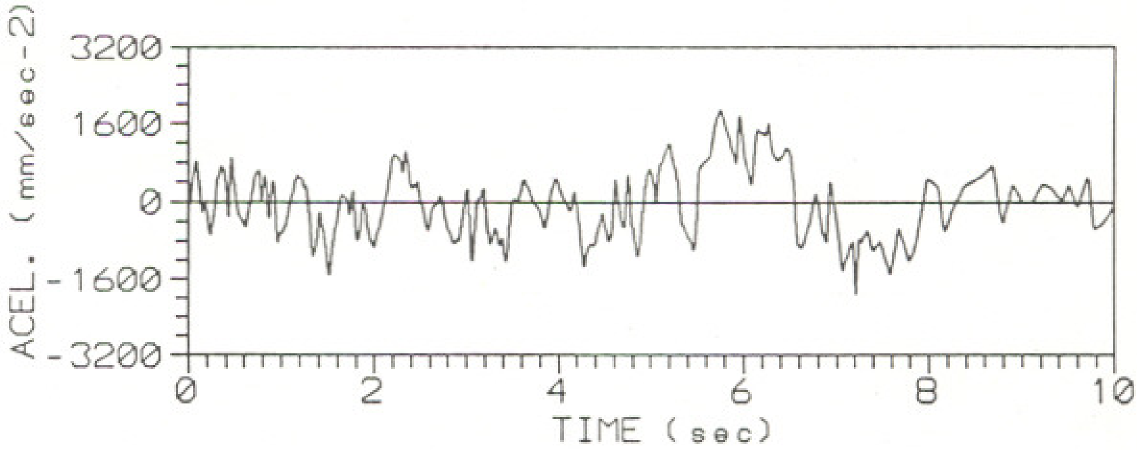


(c) VERTICAL COMPONENT

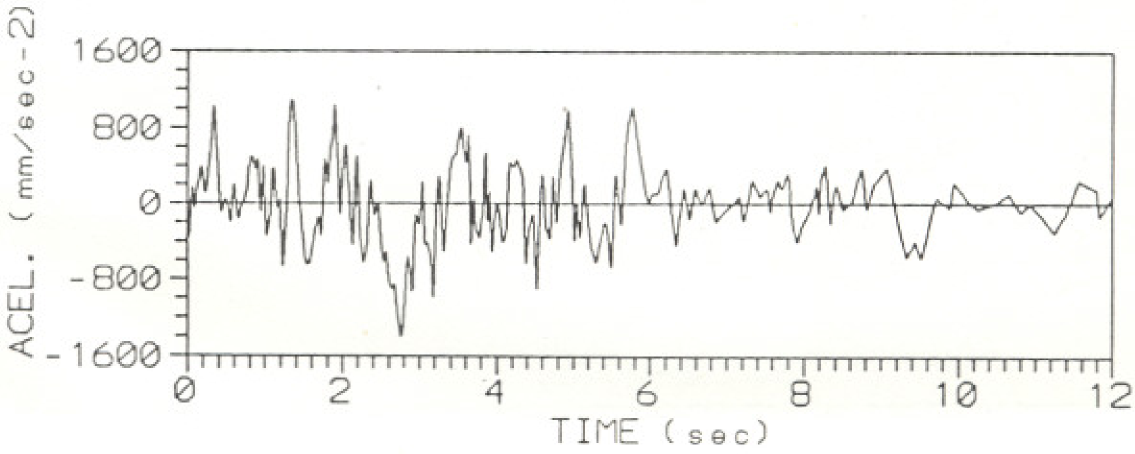
Fig. A.1 **Ground Acceleration Record of El. Centro, California, Earthquake, May 18, 1940.**



(a) LONGITUDINAL COMPONENT

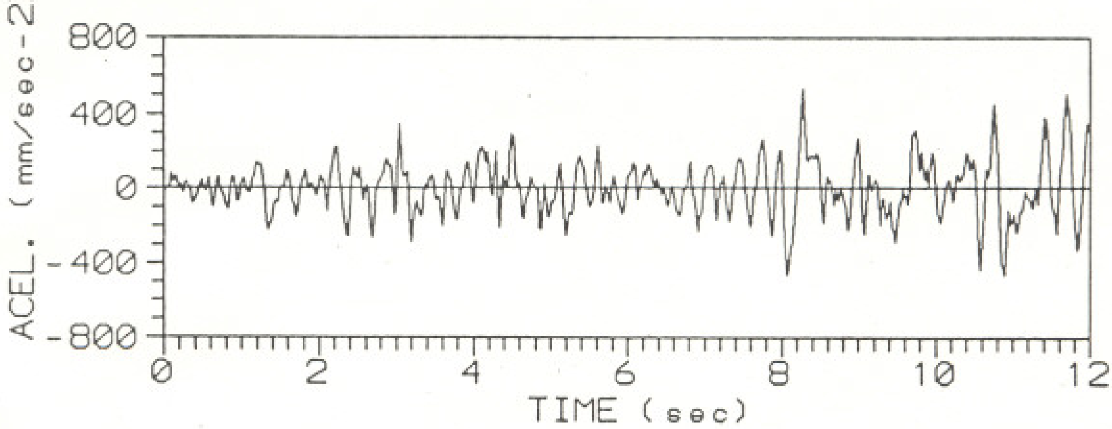


(b) TRANSVESE COMPONENT

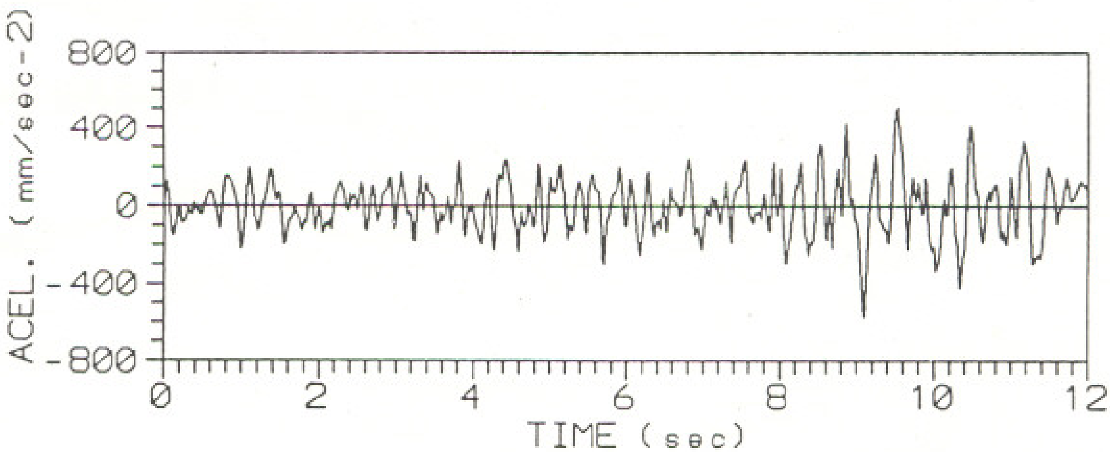


(c) VERTICAL COMPONENT

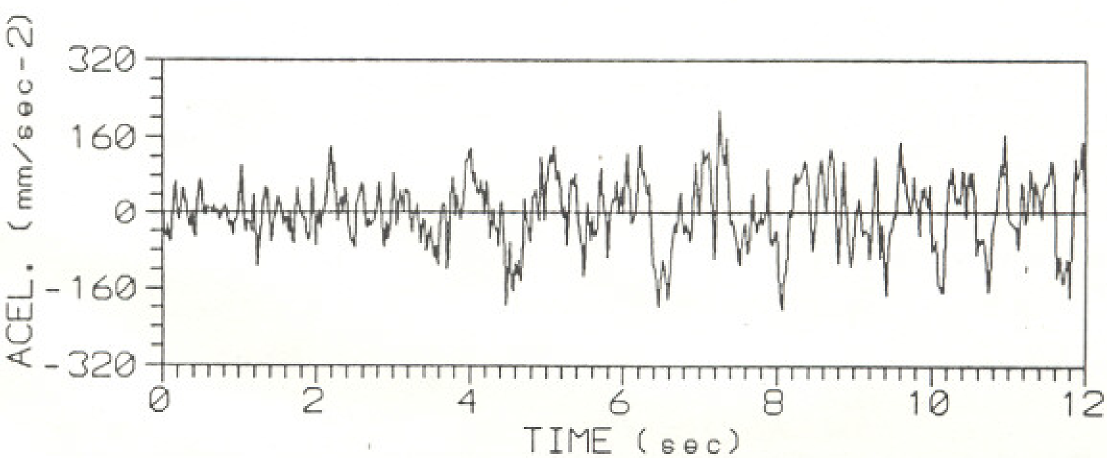
Fig. A.2 **Ground Acceleration Record of Munghyer (Bihar-Nepal) Earthquake, August 21,1988.**



(a) S 36° E COMPONENT

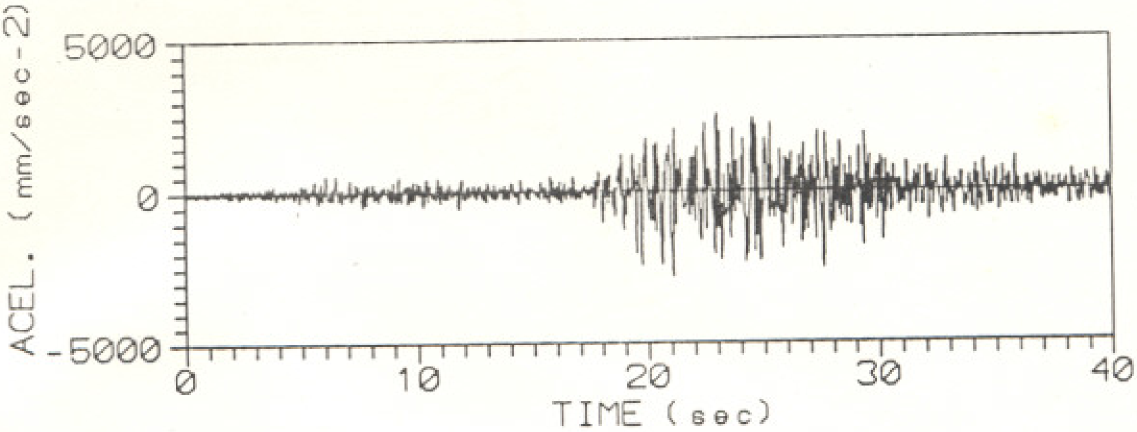


(b) N 54° E COMPONENT

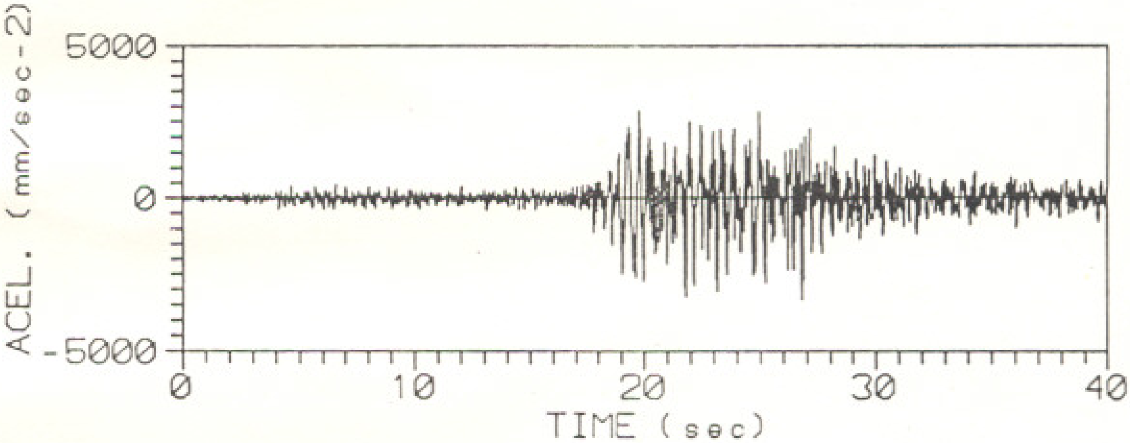


(c) VERTICAL COMPONENT

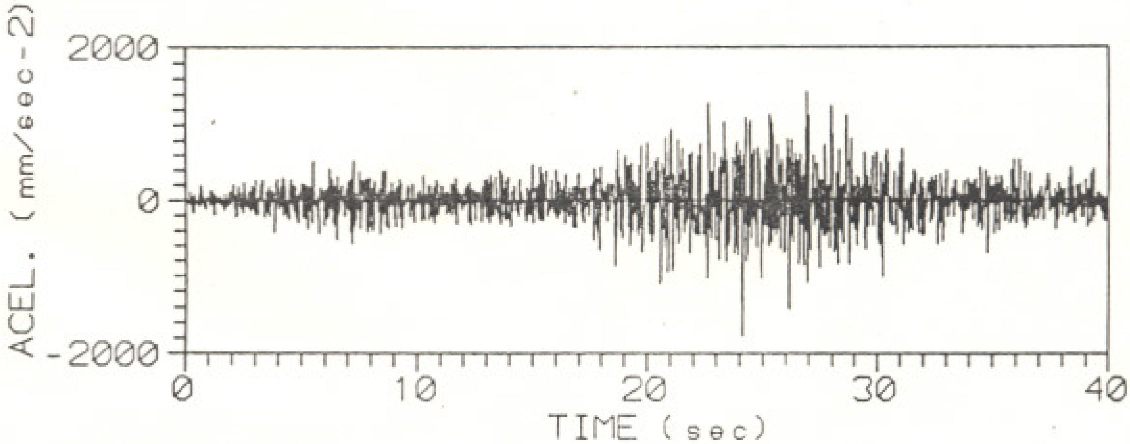
Fig. A.3 Ground Acceleration Record of Loharghat (N-E India) Earthquake, August 6, 1988.



(a) LONGITUDINAL COMPONENT



(b) TRANSVESE COMPONENT



(c) VERTICAL COMPONENT

Fig. A.4 Ground Acceleration Record of Diphu (N-E India) Earthquake, August 6, 1988.