SEISMIC ANALYSIS OF STEPBACK AND SETBACK BUILDINGS

A THESIS

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By

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Gratis

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled **'Seismic Analysis of Stepback and Setback Buildings'** in fulfilment of the requirement for the award of the Degree of **Doctor of Philosophy** and submitted in the **Department of Earthquake Engineering** of the University is an authentic record of my own work carried out during a period from 29.7.1993 to 10.7.1996 under the supervision of **Dr. D.K. Paul.**

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other University.

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Plain land in hills is scare and therefore sloping land is being increasingly used for buildings. Economic development of hilly areas, have a marked effect on the buildings in terms of style, material and method of construction. Stone, wooden load bearing buildings are common in hilly areas. Traditionally, the hill buildings are constructed in stone masonry with mud mortar. Loss of lives and property are mainly due to damage of these buildings during earthquakes. The existing r.c.c. buildings have performed well. Therefore r.c. framed buildings are getting popular in hilly areas. In hilly areas, many multistoreyed r.c. framed buildings rests on hill slope. The various floors of the building stepback towards the hill slope and at the same time the building may have setback also. The stepping back of the building towards hill slope result into unequal column heights at the same floor level.

Building constructed on hill slope poses special structural and constructional problems. The various floors of the building on hill slope may be supported on two types of columns (i) columns resting on the floors below and (ii) columns resting on the sloping ground. These buildings are highly irregular and asymmetric. The centres of mass of various floors of the building on hill slope lies on different vertical axes and so is its centre of stiffness unlike symmetrical buildings. Most of the hill areas falls in active seismic belts. These buildings are subjected to severe torsion in addition to lateral shears under the action of earthquake loads. The non uniform soil profile on the hill slope result into different soil properties at different levels. It may result into unequal settlement of foundations and local failure of slope. Landslides and unstable slope creates problem to buildings on hill slope causing total collapse. Not much studies have been made on the various problems facing hill buildings. Climatic conditions and heavy rains is a big problem for buildings in hill areas. This thesis looks into the solution of some of the special problems related to buildings on hills.

To capture the real behaviour of buildings on hill slope 3D modelling of the building is required. In the present study two different 3D modelling of the structure have been taken for seismic analysis.

In the rigorous method of dynamic analysis, the floor slab of the building is taken as flexible and the building has been modelled as having 6 d.o.f. per node. The mathematical model consists of 3D frame elements, r.c.c. panels, brick masonry infills, r.c.c. slabs, interface elements. Special attention has been given to the nonlinear modelling of the various components of buildings.

The r.c. beam/column section has been analysed using nonlinear stress-strain relation for concrete and an elasto-plastic model for steel. The regression analysis is used to fit a third degree polynomial to the points obtained from the actual analysis of the r.c. cross section. The irregular buildings such as on hill slopes are subjected to severe torsional moment and lateral shears under the action of earthquake loads in addition to bending moments and axial forces. The yielding of the frame members takes place under the combined action of the bending moments, axial force, torsional moment and shears. The presently available yield criteria take interaction of some of the components of forces, all components are not considered in the available yield criteria. To study the inelastic behaviour of the buildings on hill slope subjected to severe torsion and shears in addition to bending moments, axial forces requires the yield criteria which consider the interaction of all the six components of forces. Therefore in the present study effort has been made to develop a yield criterion considering the interaction of all the six components of forces.

In case of r.c.c. panel elements, concrete is modelled as an isotropic material under biaxial stress condition and the material modelling for different phenomenon such as cracking, yielding and crushing of concrete and yielding of steel are modelled using available models. The brick masonry elements has been modelled considering crushing and cracking condition.

The interface elements have been modelled considering separation and slippage. The tension and compression at the interface determines the separation and contact. While in contact, the normal and shear stress at the interface determines the slippage at the interface.

To analyse the structure in the inelastic range the frame elements have been modelled by lumped plasticity theory. The algorithm predicts the formation and disappearance of plastic hinges. Ductility is an important parameter in earthquake resistant design of buildings. To study the ductility requirement of r.c. members, an inelastic analysis is necessary. Ductility of a member cannot be realistically determined unless appropriate inelastic degrading stiffness model is used. Therefore degrading modified Takeda's model has been implemented. On unloading the plastic hinge, stiffness degradation has been considered for all the six components as the yield criteria used in the present study takes interaction of all the six components of forces. Ductility requirement of all the yielded members have been evaluated. There is a gradual deterioration of stiffness of the structure due to plastic hinges formed and cracking, yielding and crushing of r.c.c panels, cracking and crushing of brick masonry infills.

The results of inelastic analysis obtained from present study compares well with the available experimental results in the literature. It is observed in the present study that the buildings which are subjected to severe torsion and lateral shears in addition to bending moments and axial loads, the yielding of r.c. members takes place at a lower load factor as compared to buildings which are not subjected to severe torsion and lateral shears.

In the simplified method of dynamic analysis the floor system is considered as rigid under lateral loads, then the building modelling is much simplified and can be modelled as 3 d.o.f. per floor at the centre of gravity of the floor i.e. two translation and one rotation about vertical axis passing through centre of gravity. The model consists of frame elements and infill panels. Building on hill slope is characterised by the location of centre of mass of different floors lying on different vertical axes, and so is the case with the centre of stiffness. The existing methods of dynamic analysis of such irregular buildings are too complicated to be used in the design offices. Therefore a simplified method for seismic analysis of these buildings based on transformation of mass and stiffnesses of various floors about a arbitrarily choosen common vertical reference axis is developed. The mass of different floors lying on different vertical axes gets transferred to common vertical reference axis and so is the stiffness of various floors. In this modelling the overall size of the problem gets reduced tremendously requiring much less time for data preparation and computational effort. In this modelling accidental eccentricity can be taken into account by simply shifting the centre of mass of the floor equal to accidental eccentricity. The results obtained from this method compares well with 6 d.o.f./node analysis with rigid floor diaphragm.

A few real building problems having stepback configuration on hill slope have been studied for its seismic response using the two methods of analyses. It has been found that the results of free vibration time periods, mode shapes, inter storey column shears, ground column shears and infill shears, lateral floor displacements obtained by simplified method are comparable to the results obtained by rigorous method.

Code of Practices(UBC,NBCC,NZS etc.) recommends 3D dynamic analysis for irregular buildings such as on hill slopes. Although many computer codes are available for seismic analysis of irregular buildings, still there is a need of simplified method for seismic analysis of stepback and setback buildings such as that on hill slope to be used in design offices, which gives an insight into the real behaviour of hill buildings under seismic conditions. It is suggested to adopt the simplified method for 3D dynamic analysis of irregular buildings in the Code of Practices. It has also been observed that the base shear concept is not applicable in these types of buildings.

A procedure for stability analysis of the slope with building loads has been developed based on limit plastic equilibrium using simplified Bishop's method. The building loads in the form of vertical loads, horizontal loads and bending moments transferred at the foundation level to the hill slope has been considered in addition to the self weight of the sliding mass of the soil. The dry/wet condition of the material of the hill slope can be considered in the analysis. The minimum factor of safety against sliding failure of slope is evaluated by taking various trial slip circles automatically in the computer program. The different layers of the soil in the slope can be taken into account considering different properties of the soil mass. Earthquake effects can be considered in the analysis. It is found from the study that the stability of slope depends on the type of loads, location of loads, configuration of the building transferring the load, drainage condition of the area. It is found that the stepback type configuration of building gives better stabilising effect as compared to combination of stepback and setback. The stability of slope decreases with earthquake loads. Buildings on flat ground adjacent to hill slope should be so configured that heavier part of the load should be transferred at uphill side of the slope otherwise there is a chance of local failure. Taking foundation deeper on upstream side of slope increases the stability of slope. The reduction in pressure due to building loads enhances the stability of slopes and can be achieved by providing strip foundation across the slope for all the columns in one row. It has been observed that factor of safety against sliding of slope increases with increase in distance of location of footing from free edge of slope. The distance between the two column loads also affects the factor of safety. Proper drainage arrangement should be provided around the building complexes so as to avoid soil erosion and landslides.

The results of inelastic analysis of real buildings on hill slope having stepback configuration shows that the plastic hinges forms in the members located on periphery of the building and mostly are in columns. It shows that the stepback buildings are torsionally unbalanced. The ductility requirement has been evaluated for the yielded members and it is found that the ductility demand is higher for members located at the outer periphery. The too short and too long columns at the same floor level in these buildings are the worst affected and are to be avoided.

Soil structure interaction study has been carried out for few cases of hill buildings. It is observed that for loose and medium soil with shear wave velocity up to 300m/sec, the free vibration time periods increases from 1 to 5% as compared to fixed base condition. For dense soil with shear wave velocity 600m/sec and above, the results of free vibration time periods are almost the same as that of fixed base condition. It is also observed that ductility demand of the yielded members increases, where the buildings are supported on the loose and medium soil base.

A few different configuration of buildings on hill slope (i.e. regular frame building on flat ground, setback building on flat ground, stepback building on sloping ground, stepback and setback building on sloping ground) have been studied from structural and stability considerations under the action of dead, live, and earthquake loads. It has been found that there is not much difference in the structural behaviour and member forces for these configurations under the action of dead and live loads. But under the action of earthquake load the behaviour of different configurations of the buildings is quite different. It advocates the use of different configurations in different situations so as to get the better structural performance and economical design of buildings on hill slope. A combination of stepback and setback type configuration of building gives better response as compared to stepback configuration only because of neutralizing effect of torsion. It has been observed that ductility requirement of r.c. members in stepback configuration is more as compared to combination of stepback and setback configuration building. Incidentally the outer profile of a combination of stepback and setback configuration building follows the natural profile of hill slope, which is architecturally more acceptable. Therefore a combination of stepback and setback configuration of buildings are recommended for construction in hill areas.

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(SATISH KUMAR)

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NOTATIONS AND SYMBOLS

A_c	Area of concrete in compression
A_s, A_{si}	Area of steel in general and in ith layer
a_i, b_i	i = 1, 2, 3, 4 Coefficients of third degree polynomials which defines the interaction diagram between axial loads and ultimate moments about major and minor axes
A	Area of cross-section of the member, base area of the footing
a_x, b_x	Dimensions of the flat shell element
a, b	The lengths of rigid ends at the member ends
B	Strain displacemnet matrix
B_s	width of slice of column load
c	Length of central segment of the member
C_{an}	Setback seismic coefficient
C_{tn}	Torsion bending coefficient
C_x, C_y, C_z	Direction cosines in x, y, z directions
C_1, C_2	Material parameter
C_m	Centre of mass
C_s	Seismic coefficient
C	Damping matrix of the system
c	Cohesion of soil
D_n	Plan dimension of the building in the direction of computed eccentricity
$\mathbf{D}$, $\mathbf{D_c}$, $\mathbf{D_s}$, $\mathbf{D}^r_{ep}$	Material property matrix in general, for concrete and steel, Elasto-plastic material property matrix for r.c.c. panel
$\mathbf{D_{ce}}$, $\mathbf{D_{be}}$	Material property matrix for r.c.c. panel in cracking condition, for brick masonry
d_1, d_2	Coefficients to determine Rayleigh's damping
du_{max}, dy	Maximum deformation and yield deformation
d	Total depth of r.c. section
e_1, e_s	Effective eccentricity, static eccentricity
e_c	Strain in concrete
E, E_c, E_b, E_s	Young's modulus of elasticity in general, for concrete, brick masonry, steel
E_{st}	Elasto-strain hardening Young's modulus of steel
e_u	Ultimate strain of concrete in compression
EI	Flexural rigidity
$\mathbf{F_m}$	Member force vector
$f_c^{'}$	Compressive strength of concrete
$f_t^{'}$	Tensile strength of brick masonry
f_b	Yield strength of brick masonry
f_b^t	Tensile strength of brick masonry
f	Force vector
F	Factor of safety
f_y	Yield stress of steel
G, G_b, G_1, G_2	Shear modulus of rigidity in general, brick masonry, cracked shear modulus

g_k	Gradient vector of the yield surface
H	Horizontal load
h_w	Pore pressure head
I_1, I_1'	First stress invariant, Strain invariant
I_x, I_y, I_z	Moment of inertia of member in x, y, z directions
J_2, J_2'	Second deviatoric stress invariant, strain invariant
K_g	Initial stiffness based on gross cross-section
K_1', K_2'	Unloading stiffness for cracking and yielding subhinge
K_u	Unloading stiffness
K_c	Stiffness matrix of column member at member axis
K_c^r	Stiffness matrix of column member at reference axis
$\mathbf{K_d}, \mathbf{K_d^r}$	Stiffness matrix of diagonal member at local axis, at reference axis
$\mathbf{K_m}$	Stiffness matrix of the member in local axis
Kn, Ks	Normal and shear stiffness coefficients for interface element
$\mathbf{K_e}$	Stiffness matrix of the member in global axis
$\mathbf{K_{ep}}$	Elasto-plastic stiffness matrix
$\mathbf{Kp}$	Plastic reduction matrix
K_x', K_y', K_z' $K_{mx}', K_{my}', K_{mz}'$	Elastic stiffness of the plastic hinge after unloading in x, y, z directions
$\mathbf{K_h}$	Stiffness matrix of the plastic hinge
K_l	Reloading stiffness
K_o	Initial elastic stiffness
K_{pe}	Stiffness of plastic hinge
$\mathbf{K_m}$, $\mathbf{K_b}$	Stiffness matrix of shell element membrane, bending
$\mathbf{K_e^j}$	Stiffness matrix of joint element at local axis
$\mathbf{K^j}$	Stiffness matrix of joint element at global axis
$\mathbf{K^*}$	Effective stiffness matrix
$K_x, K_y, K_z,$ $K_{x\theta}, K_{y\theta}, K_{z\theta}$	Spring constants for representing the soil stiffness
L	Total height of the frame
L_s	Setback level ratio
L'	Height of the base portion of the frame
L	Length of the member
L_i	Length of slice i
L	Length of footing
m_x, m_y, m_z M_x, M_y, M_z	moments in x, y, z directions
$\mathbf{M_f^r}$	Mass matrix of the floor at reference axis
m_{zo}, m_{yo}	Uniaxial moment capacity of the section about z and y axis
m	Mass of the floor in x, y, z directions
mI_x, mI_y, mI_z	Mass moment of inertia of the floor in x, y, z directions
M_{ux}, M_{uy}, M_{uz}	Moments in x, y, z directions
M_{zo}, M_{yo}	Ultimate moment capacity of the section
M	Moment load transferred at slope

N_i, N_j	Shape functions for shell element
N_1, N_2	Shape functions for line interface element
n	No. of slices of soil
$\overline{N}_i$	Effective normal force
n_c	No. of slices of column loads
N_j	Normal component of force of slice j due to column load
$\mathbf{dp_k}$	Incremental nodal forces corresponding to plastic deformation
$\mathbf{dp}$	Vector of incremental forces
P_u	Axial force in member
$p_{i1}', p_{i2}', p_{i3}'$	Forces at storey axis in x, y, z directions
p_{i1}, p_{i2}, p_{i3}	Forces at vertical reference axis in x, y, z directions
$\mathbf{P_i}, \mathbf{P_j}$	Member end actions at reference axis
$\mathbf{P_i'}, \mathbf{P_j'}$	Member end actions at storey axis
P_o	Axial force capacity of the section
$\mathbf{P_{mb}}$	Vector of end actions for the shell element
R_e	Elastic deformation
R_p	Plastic hinge deformation
$\mathbf{R_s}$	Matrix of direction cosines for shell element
$\mathbf{R}$	Transformation matrix
r_u	Pore pressure ratio
R	Radius of slip circle
r_o	Radius of footing
S_x, S_y	Shape factors of the cross-section in x and y directions
S	Shear strength of soil
$\mathbf{T}, \mathbf{T_s}, \mathbf{T_j^J}$	Transformation matrix for beam and flat shell element, joint element
T_{xo}	Torsional moment capacity of the section
t	Time
Δt	Time increment
u', v', θ'	Displacements at storey axis
u, v, θ	Displacements at vertical reference axis
$u, \dot{u}, \ddot{u}$	Displacement, velocity and acceleration vectors
$\ddot{u}_{gx}, \ddot{u}_{gy}$	Base input acceleration in x, y, z directions
u_i, u_j	i = 1, 6, j = 7, 12 Member end displacements
$\mathbf{d_{ue}}, \mathbf{d_{up}}$	Vector of incremental displacement(elastic and plastic)
$\mathbf{u_{mb}}$	Vector of displacement for flat shell element
u, v	Displacement in x, y, z directions for interface element
V_{Bo}	Base shear
V_{uy}, V_{uz}	Shear forces in y and z direction
W_1	Effective weight
w_i	Weight of slice i
w_j	Weight of slice j due to column loads
x, y	Distance between reference axis and centre of gravity of floor in x and y directions
X_c, Y_c	Co-ordinates of the column location at foundation level
$\mathbf{z_m}$	Member displacement vector
z	Section modulus of the footing

α	Angle between Y_β and Y_m axis
$\alpha,\beta,\alpha_1,$ α_2,β_1,β_2	Coefficients for stiffness degradation model
α_m	Tension stiffening coefficient
β,θ,μ	Rotation angles used for Z-Y-X transformation of space member
γ	Density of the material
γ_w	Density of the water
ζ_i	Damping ratio for the ith mode
v	Poisson's ratio
θ_x,θ_y	Angle of inclination of diagonal member with x and y axis
θ_j	Angle of inclination of normal to vertical of slice j due to column load
ϕ_x	Shear deformation parameter
ϕ	Angle of internal friction of soil
χ	Curvature of the section
Ω_θ	Frequency ratio
λ_k	Flow constant
ρ	Material mass density
ρ_{si},ρ_t	Steel ratio in the ith layer and total steel ratio in r.c. cross-section
μ_d	Displacement ductility
μ_c	Curvature ductility
σ_o	Uniaxial compressive strength
σ_x,σ_y	Compressive stress in x, y directions
σ_u,σ_v	Normal and shear stress for interface
σ_c	Compressive strength of Concrete
σ_y	Yield strength of steel
σ_1,σ_2	Stresses at extreme edges of the footing
τ	Shear stress in the soil
τ_{xy}	Shear stress in flat shell element
ε_y	Yield strain in the material
$\varepsilon_1,\varepsilon_2$	Tensile strain in crack direction
ε_m	Limiting value of tensile strain
ε_u	Ultimate strain in concrete
$d\varepsilon_e,d\varepsilon_{sp}$	Elastic and plastic component of incremental strain of steel
ε_t	Tolerance limit

Chapter 1

INTRODUCTION

1.1 Prelude

Plain land in hills is scare and therefore sloping land is being increasingly used for buildings. Economic development of hill areas have a marked effect on the buildings in terms of style, material and method of construction. Buildings constructed in hill areas poses special structural and constructional problems. Due to the scarcity of plain land in hill areas, the buildings are required to be constructed on the hill slope. Stone, wooden load bearing buildings are common in hilly areas. Traditionally the hill buildings are constructed in stone masonry with mud mortar and results in heavy loss of lives and property due to the damage of these buildings during earthquakes. The existing r.c.c frame buildings have performed well. Due to harsh weather conditions and durability of concrete over bamboo and timber, reinforced concrete construction although very expensive is becoming more and more popular in the hilly regions.

Buildings on hill slope differs in a way from other buildings. The various floors of such buildings steps back towards the hill slope and at the same time building may have setbacks also. A setback is a sudden change in plan dimension or a sudden change in stiffness along the height of a building. The stepping back of building towards hill slope may result into unequal column heights at the same floor. The floors may be supported on two types of columns (i) columns resting on floors below and (ii) columns resting on sloping ground. Some columns of the building may be resting in the cutting(i.e. on firm ground) and some columns in filling(i.e. on soft ground). Due to the varied configurations of buildings in hill areas, these buildings become highly irregular and asymmetric. When subjected to lateral loads these buildings result in significant torsional response in addition to translational response.

As most of the hill areas falls in active seismic belts, the buildings constructed in hill areas are much more vulnerable to seismic environment. Landslides and unstable slopes creates great problem to buildings on hill slopes causing total collapse. Sufficient information is available for construction of earthquake resistant wooden, stone and brick masonry buildings and some of these information is also available in I.S. Codes 13827, 13828 and 4326(1993).

Very little information is available in literature about the r.c. frame buildings on hill slope under the action of seismic excitation. The safety of buildings on hill slope in active seismic belts is of great concern due to the loss of lives in the recent past Bihar-Nepal 1988 and Uttarkashi 1991 earthquakes. Need is felt to study the seismic behaviour of reinforced concrete frame buildings on hill slopes under earthquake excitations. The present study is an effort in this direction.

1.2 Damage to Buildings on Hill Slopes during Past Earthquakes

During the Tokachi-oki earthquake in north eastern region of Japan in 1968 many buildings located near the edge of a stretch of hills in the city of Hachinohe suffered serious damages. The ground floor columns of a 4 storeyed reinforced concrete hotel building of modern design built on a hill slope were severely damaged during the Oita earthquake of Nov. 23, 1980. Towns located atop hill sides were severely damaged and some of them were completely razed to the ground.

The Bihar Nepal earthquake of Aug. 21, 1988 led to great loss of lives and severe damage to buildings[Thakkar *et al.*(1988)]. The major loss of life was in villages due to collapse of mud houses and brick houses laid in mud mortar. Many brick buildings in lime mortar were severely damaged due to failure of arches, cracks in exterior and interior walls. Masonry arch and the roofs of civil surgeon house, Darbhanga was badly damaged. Brick buildings with cement mortar were also severely damaged. The reinforced concrete framed buildings performed well with a little damage.

Many buildings in hill areas of Uttar Pradesh were severely damaged during the Uttarkashi earthquake of Oct. 20, 1991. Old stone masonry buildings in Uttarkashi town were badly damaged. Buildings constructed with cement mortar solid blocks load bearing walls in cement mortar did not collapsed, although some of them had serious damage to walls in the first storey. Buildings constructed as per the provisions of I.S. 4326 with seismic bands did not suffered any damage except minor cracking at some locations.

1.3 Problems Associated with Hill Buildings

There are additional problems of buildings on hill slope as compared to buildings in plain areas as listed below.

(i) Buildings in hill areas are irregular and asymmetric and therefore are subjected to severe torsion in addition to lateral forces under the action of earthquakes;

(ii) many buildings on hill slope are supported by columns of different lengths. The shorter columns attracts more forces as the stiffness of the short columns is more and undergo damage when subjected to earthquakes;

(iii) buildings in hill areas are subjected to lateral earth pressure at various levels in addition to other normal loads as specified on buildings in plain areas;

(iv) building loads transmitted at the foundation level to the slope creates problem of slope instability and may result into total collapse of the building;

(v) the soil profile is non uniform on the hill slope and result into different properties of soils at different levels. The bearing capacity, cohesion, angle of internal friction etc. may be different at different levels. It may result into unequal settlement of foundations and local failure of slope;

(vi) climatic conditions and heavy rains is a big problem for buildings in hill areas requiring special attention for drainage, temperature control and lighting arrangements in the buildings.

1.4 Statement of the Problem

Buildings constructed on hill slopes having stepback and setback configurations are highly torsionally coupled due to the fact that the centre of mass of various floors lies on different vertical axis and so is the case with centre of stiffness. Prediction of seismic response of such structures is highly indeterminate and cumbersome. Pachau(1992) has analysed three buildings on hill slope using 1 D modelling technique and torsional shears are accounted for separately. The one dimensional modelling does not give the true behaviour of the torsionally coupled buildings. Paul(1993) described the problems of hill buildings and suggested simplified method for seismic analysis of hill buildings. Torsionally coupled buildings have been studied by various researchers(i.e. Gupchup(1978) and Penzien(1969)) in the past. However most of the work on torsionally coupled buildings is confined to buildings on flat grounds. To analyse and study the true behaviour of such irregular asymmetric buildings it is required that 3D model is to be adopted to get the true coupled translational and torsional response of the structure. Buildings on hill slope are highly irregular due to their configurations and asymmetry due to variation in mass and stiffness distributions. The various types of building configurations in hilly areas are shown in Fig. 1.1. The available literature shows that most of the work is carried out for regular asymmetric buildings.

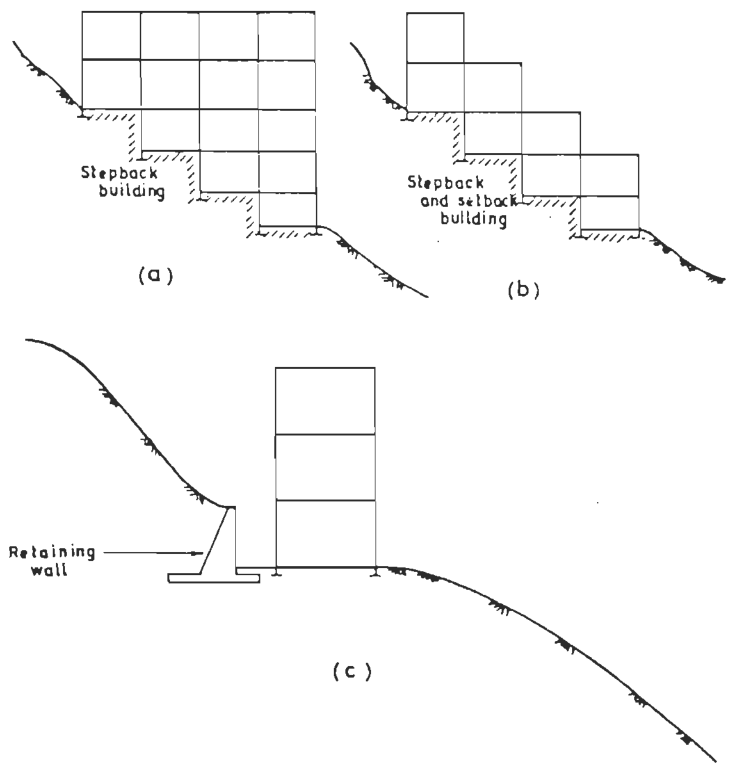

Fig. 1.1 - Buildings in Hill Areas

In order to get the realistic behaviour of the stepback and setback buildings under the action of earthquake loads, 3D modelling of the building structure is required. Generally the r.c.c. framed buildings consists of beam/columns, r.c.c. slabs, r.c.c. panels, brick masonry infills. To analyse the building realistically under the action of gravity and earthquake loads, the stiffness and mass contribution from all the elements is required to be considered. This further requires the proper modelling of all the components of the building. To carry out the dynamic analysis of the multistoreyed stepback and setback building considering all the components, the problem gets complicated too much. To carry out preliminary analysis and design of stepback and setback buildings, a 3D simplified method of dynamic analysis is required to be developed to be used in the design offices and to decide upon seismically better configuration of the buildings.

To study the inelastic behaviour of stepback and setback buildings, rigorous method of dynamic analysis is required. The beam/column elements of such irregular buildings will be subjected to severe torsional moments and lateral shears under the action of earthquake loads. The yielding of beam/column elements takes place under the combined action of all the six components of forces generated under the action of gravity and earthquake loads.

Therefore a suitable yield criteria is to be developed considering the interaction of all the six components of actions in the 3D beam/column elements. To study the realistic ductility requirements of the yielded members, stiffness degradation needs to be considered in the inelastic analysis.

Nonlinear modelling of r.c.c. panels and brick masonry infills considering cracking, yielding and crushing needs to be considered in the analysis procedure. Interface elements are also required to be considered in the inelastic analysis considering tension and compression conditions at the interface.

Study of stability of slope is equally important as unstable slope may cause total collapse of the buildings. To study the stability of slope with building loads transferred at foundation level, loads in the form of vertical, horizontal and moments needs to be considered in addition to the soil mass for finding the factor of safety against sliding failure of the slope. The dry/wet/saturated condition of the soil mass needs to be considered in the analysis.

In the present study, effort has been made to develop simplified and rigorous method of analysis considering various complexities of the stepback and setback buildings and nonlinear modelling of the various components of the building by making suitable assumptions and simplifications, where ever necessary. Efforts has also been made to develop an analysis procedure for finding the minimum factor of safety against sliding failure of slope.

1.5 Objectives of the Study

The objectives of the present study are

(i) to review the literature related to the irregular, asymmetric buildings exhibiting torsionally coupled behaviour, modelling of 3D r.c. beam/column, panel elements (r.c.c., brick masonry), stability of hill slope with building loads;

(ii) to look into the Codal provisions of different countries for dynamic analysis procedure on such buildings and suggest simplified dynamic analysis procedure based on the study;

(iii) to develop a simplified dynamic analysis procedure for stepback and setback buildings under the action of seismic loads;

(iv) to develop a rigorous static/dynamic analysis procedure for modelling 3D buildings with 3D beam/column elements, r.c.c. panel elements, brick masonry elements, interface elements, r.c.c. slab elements taking into account the linear and nonlinear behaviour of members in the structure under the action of combined gravity, live and seismic loads;

(v) to develop a analysis procedure to find the factor of safety against sliding failure of the hill slope under the action of building loads considering dry/wet conditions of the soil including earthquake effects;

(vi) to develop a computer program of the above analyses procedures;

(vii) to compare the seismic response of real buildings on hill slope using simplified and rigorous method of analysis, and to study the inelastic behaviour of the hill buildings under the action of combined gravity, live and seismic loads;

(viii) to carry out the parameteric study of the stability of slope under the action of various kinds of building loads;

(ix) to study the seismic behaviour of the differently configured buildings on hill slope and to finally recommend some better configuration of buildings on hill slope from seismic and stability considerations.

1.6 Layout of the Thesis

The thesis has been put in total of 8 chapters and the brief description about the chapters is described below:

In Chapter 1 the significance and importance of the problem has been highlighted along with objectives of the present study.

Chapter 2 covers the review work related to torsionally coupled buildings, modelling of 3D elements, panel elements.

Chapter 3 covers the 3D simplified dynamic analysis procedure for seismic analysis of the stepback and setback buildings under the action of seismic loads. Validation of the simplified model developed by comparing the results of present study with the existing results.

Chapter 4 covers the rigorous analysis procedure of the 3D buildings including the nonlinear modelling of beam/column elements, panel elements, interface elements. Validation of the model choosen by comparing the results with existing experimental/analytical results.

Chapter 5 describes the analysis procedure for finding the factor of safety against sliding failure of the slope with building loads acting on it and validation of the computer program by comparing results with the available results.

Chapter 6 compares the seismic behaviour of hill buildings obtained from simplified and rigorous methods of analysis. Stability analysis of slope and inelastic behaviour of the hill buildings has also been presented in this Chapter.

Chapter 7 shows the seismic behaviour of differently configured buildings on hill slope.

Chapter 8 covers the summary of this investigation and the conclusions drawn from the present study and suggestions for further research work.

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Chapter 2

LITERATURE REVIEW

2.1 Introduction

Multistoreyed r.c framed buildings are getting popular in hilly areas and many of them are constructed on hill slope. Setback multistoreyed buildings are not uncommon over level grounds whereas stepback buildings are common on hill slope. A combination of stepback and setback buildings are also common on hill slopes. Setback and stepback buildings are shown in Fig.2.1. The buildings may have setbacks in one or both the principal directions located symmetrically or unsymmetrically about the vertical axis. At the location of setback stress concentration is expected when the building is subjected to earthquake excitation. These are generally not symmetrical due to setback and/or stepback and result into severe torsion under a earthquake excitation. Current building codes suggest detailed dynamic analysis for these types of buildings. For symmetrical multistoreyed setback buildings, the building may be de coupled where as for unsymmetrical buildings, a coupled analysis is required. Literature on the seismic behaviour/analysis of stepback and setback type of buildings is scanty. Literature on dynamic behaviour of these types of buildings and its analysis procedures which take into account the asymmetry in the buildings i.e. torsional coupling effects has been reviewed in this Chapter. Various different analytical models for r.c. beam/column elements and panel elements for inelastic behaviour including stiffness degrading models have been reviewed in this Chapter.

2.2 Setback and Stepback Buildings

Berg(1962) studied the earthquake stresses in buildings with setbacks. In his study the building with setback has been represented by a rectangular cantilever shear beam with a setback at one location along its height. In the model the x-section is assumed uniform above and below the step. The rigidity per unit area and density are taken uniform throughout the height of the beam but the rigidity is taken different in both the directions. The modal equations of motion are derived and computed modes are examined to show the effects of symmetric and unsymmetric setbacks upon the vibrational characteristics and upon the dynamic stresses induced by earthquakes.

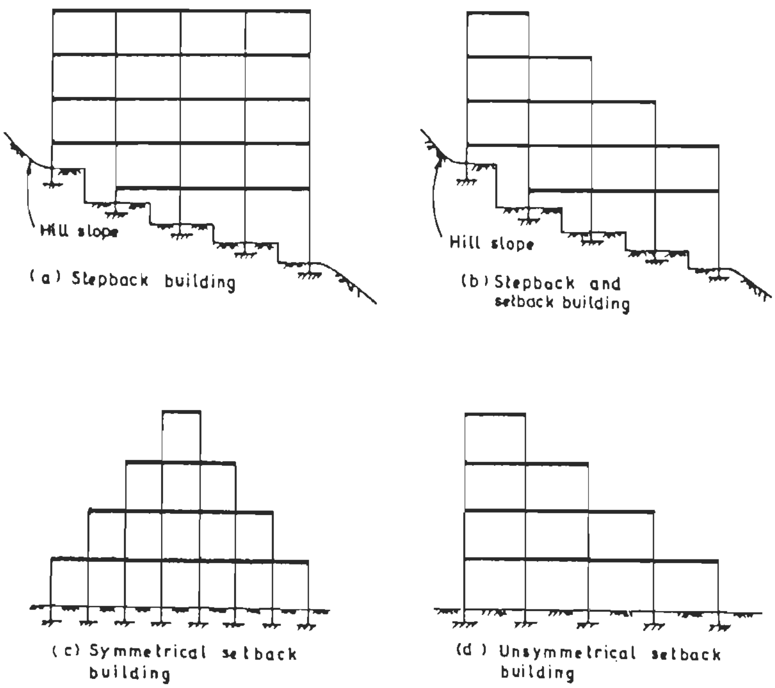


Fig. 2.1 - Stepback and Setback Building

Blume and Jhaveri(1969) investigated a highrise building with one or more setbacks involving special problems. The codes have dealt this problem by specifying the alternative to treat the tower portion of the building as a separate structure. The separate tower concept however does not take into account the fact that the ground motion is modified greatly by the base portion of the building before it affects the tower portion. The tower is subjected to an essentially harmonic forced vibration instead of the nearly random motion of the ground. The torsional and translational vibrations of a building with unsymmetrical setbacks is coupled in general. They carried out the analysis of a multistoreyed building with a setback.

The actual values of the code shear coefficients are found to be much smaller than the corresponding computed response values. The effect of setback on the base shear coefficient is strongly dependent on the characteristics of the ground motion and in turn these characteristics are significantly influenced by geologic conditions. It is essential for setback structures to reconcile analysis and design procedures with the real earthquake problem and its probabilistic aspects.

Penzien(1969) Presented an approximate method of determining the peak seismic response of irregularly shaped buildings when subjected to base acceleration. Irregularly shaped buildings may have two contributing mode shapes with frequencies of nearly the same magnitudes. Two degrees of freedom system method has been applied to the lateral motion of buildings having large setbacks and to the coupled lateral motion of eccentric buildings. A comparative study of results has been carried out for the methods i.e. two degrees of freedom system and single degree of freedom system. It is concluded that two degree of freedom method accurately predicts the setback seismic coefficients C_{an} and torsion bending coefficients C_{tn} even when the period of the setback structure coincides with the fundamental period of the building and period of fundamental torsional mode of vibration equals the period of fundamental lateral mode of vibration. The single degree of freedom method is considerably in error when the period of the setback structure is near one of the lower periods of the building which supports it and when the period of the fundamental torsional mode is near the fundamental lateral vibration mode. Setback structures should be so designed that its fundamental period of vibration differs considerably from the first lateral vibration mode of the building and also does not coincide with the periods of other lower lateral vibration modes. The seismic forces developed in a setback structure and the seismic torsion forces developed in an eccentric building assuming elastic systems are much larger than standard code values. Therefore for determining seismic forces in a setback structures and seismic torsion forces in eccentric buildings the desirable effects of inelastic deformations must be considered.

Pekau and Green(1974) investigated earthquake response of yielding frame structures with setbacks. Keeping in view the serious stress concentrations at the level of setback the effect of inelastic action is established. The base portion of the building consist of three equal bays for a fraction of overall height given by the level of setback. The tower portion is a single central bay where the storey sums of girder and column properties equal corresponding sums in the uniform three bay structures multiplied by degree of setback. The storey drift response is not sensitive to level of setback for relatively small towers. The tower and base shear coefficients both increase for decreasing size of setback. It is interesting to note for high level of setback the base shear coefficient for setback and uniform structures tend to be the same. When the degree of setback is greater than 0.67 the presence of the setback has small effect.

Humar and Wright(1977) carried out analytical study of the dynamic behaviour of selected series of multistorey steel frames with symmetric setbacks. The models of the structure were having finite number of degrees of freedom with masses lumped at the floor levels. Conclusions derived are that the relative contributions of the higher modes to the base shear in general increase with decreasing tower base plan area for setback type buildings. The maximum interstorey drifts are substantially greater than the comparable responses for comparable uniform buildings in the inelastic range. The maximum shear coefficients are substantially greater than the comparable responses for comparable uniform buildings. Codes underestimates the distribution of base shear throughout the building height for setback buildings. In the setback buildings, the shear coefficients shows a sudden and marked increase in the transition region between the base and the tower. For a setback building with the mass and stiffness substantially proportional to the plan area, the seismic response depends upon the ratio of the tower plan area to the base plan area, rather than upon the ratio of the plan dimensions of the tower and base in the direction of vibration as specified in 1973 SEAOC Code. There appears to be a strong correlation between general nature and distribution of the elastic and inelastic seismic responses of setback buildings. Thus a less expensive elastic analysis in most practical design applications can be employed.

Cheung and Tso(1987) presented a simple method for lateral load analysis of buildings with setback for preliminary design purposes. The concept of compatibility has been employed in this method. The lateral load acting on the structure is divided into two sub components. The sub component consists of applied load acting on the tower portion of the structure together with a set of compatible loads acting at and below the setback level. This part of load is resisted by tower portion only. The compatible loads are to offset the effect of loadings above the setback ensuring compatibility between the tower and the base portion of the structure. The second component will then consists of applied loads at and below the setback level less the compatible loads. This second set of loads will be resisted by base portion only. The final response of the structure will be the sum of responses under each of the two loading sub component as discussed above. For eccentric setback structures, additional computation is necessary to take into account the torsional effect. The lateral loading is first subdivided into translational and torsional loadings, Then effect of translational loading is worked out as in the symmetric setback structures. Then effect of torsional loading is to be worked out. For this location of centres of rigidity are to be worked out. Then distribution of torsional shear can be carried out by using the compatible concept again. Then total response will be the sum of translational response and torsional response. This method can be used as design tool as well as it provides an insight into the load transfer mechanism involved in such type of structures especially in the region where setback occurs.

Sobaih, Hindi and Al-Noury(1988) studied the effect of different parameters on the nonlinear dynamic analysis of setback reinforced concrete frames. The parameters are setback level ratio'L_S'; variation of beam properties; earthquake intensity and the type of nonlinear model. The tower portion exhibits larger displacements as 'L_S' decreases compared with uniform frame. The response of such frames is affected by setback ratio, beam properties, earthquake intensity and nonlinear model used in the analysis.

Satake and Shibata(1988) carried out dynamic inelastic analysis for torsional behaviour of a setback type building using three dimensional frame model. First, the original designed building model is analyzed to see the torsional behaviour of a setback type building subjected to strong earthquake. Secondly, the modified model whose strength is modified as per the torsional response properties is analyzed and found that if the torsional response is not considered in the seismic design, it affects the response and damage distribution. The strength of each frame must be determined according to its torsional response properties to control the damage level. The distribution of elastic response shear distribution can be used to determine adequate strength distribution. By taking into account the torsional response properties, the requirement of total strength can be reduced about 20% as compared with current design value.

Shahrooz and Moehle(1990) carried out the experimental and analytical studies of seismic response of setback structures. Only two dimensional response parallel to the setback was considered. The influence of setback on dynamic response, the adequacy of current static and dynamic design requirements for setback buildings, design methods to improve the response of setback buildings were the main points under consideration in the study. Only responses parallel to setback buildings are analyzed so that torsional effects do not arise. The results of the test structure were similar to those for a structure with regular configuration except torsion. The resulted behaviour of the structure using modal spectral analysis & static analysis design method did not differ notably. Both the methods were found inadequate to prevent concentration of damage in the members near the setback. For the setback structures, it was concluded that the design should be such which will impose increased strength on the tower relative to the base. A static analysis is proposed by the author which amplifies the forces. The ductility demand according to the proposed method is reduced considerably.

Wood(1992) investigated influence of setbacks on nonlinear response of R/C framed buildings. The displacement and shear responses of the setback frames were governed by effective first mode. Maximum top storey displacement and maximum interstorey drift for all frames increased with increasing ground motion intensity. However, the magnitude of displacement response and interstorey drift did not depend upon the frame profile as observed. Maximum storey displacement and interstorey drift were well represented by linear first mode shapes, The linear mode shapes for setback structures exhibit kinks, that were not present in case of uniform frames. But kinks did not influence the dynamic behaviour of setback frames. Maximum storey inertial forces and maximum storey shear were similar to the equivalent lateral force distribution used for design. Differences between nonlinear behaviour of regular and setback frames does not warrant the different design procedures to be adopted in the current building codes. There was no indication of concentration of forces or displacements in different stories with different mass or stiffness.

Jain and Mandal(1992) studied multistoried buildings with V-shaped plan by modelling each wing as a vertically oriented anisotropic plate for the motion in the transverse direction. All modes exhibit floor flexibility in case of unequal stiffness in transeverse and longitudinal directions, Both rigid floor as well as flexible floor modes existed in case of equal stiffness in transverse and longitudinal directions. In this study torsional stiffness of floors and frames is neglected. The modes involving floor deformations are not excited by a spatially uniform ground motion. Problem of stress concentration can be taken care of by designing the structure in such a way that longitudinal & transverse stiffness of the structure are equal. Various parameters like relative values of stiffness in the longitudinal & transverse direction of each wing, angle, aspect ratio, height to length ratio have very significant effects on the relative significance of floor flexibility. Floor flexibility affects the shear distribution among transverse frames thus leading to unsafe design for some frames. If the total transverse stiffness is more than the longitudinal stiffness the first floor mode involves more deflection at the junction than that at the free end and vice-versa. As the angle between the wings increases(decreases), floor flexibility effects decreases (increases), These effects increase significantly with an increase in aspect ratio and with decrease in building height. Depending upon the configuration of the structure, floor flexibility may overload some of the transverse frames.

Paul(1993) suggested a simplified method for analysis of stepback and setback buildings by taking one d.o.f. per floor(i.e. translational either in x or y directions) and studied the hill buildings with this method. Results obtained from this method has been compared with 6 d.o.f. per node analysis.

Kumar and Paul(1994) developed a simplified method of dynamic analysis for irregular buildings such as on slope having stepback and setback configurations characterised by centre of mass of various floors of the building lying on different vertical axes and so is its stiffness, with 3 d.o.f. per floor assuming floor diaphragm as rigid . This simplified method is based on the concept of transformation of mass and stiffness about a common vertical reference axis located anywhere in the space. The overall size of the problem is reduced tremendously. The result obtained from present formulation are almost the same as obtained from 6 d.o.f. per node analysis with rigid floor diaphragm.

2.3 Torsional Coupling and Dynamic Behaviour

Kan & Chopra(1977) has studied the torsionally coupled buildings in which centre of mass of all the floors lies on one vertical axis with 3 d.o.f. per floor with rigid floor diaphragm assumption and found that any lower mode of vibration of torsionally coupled building can be approximated as a linear combination of three vibration modes of the corresponding uncoupled system i. e. the jth mode in translational vibration in x-direction, the jth mode in torsional vibration and jth mode in translational vibration in y-direction. This has facilitated the procedure to be simpler as compared to the standard procedure for analysing the response of torsionally coupled multistorey buildings to earthquake ground motion. Numerical examples has been solved and it is found that the approximate procedure is sufficiently accurate for purposes of design of most multistorey buildings. Idealized system has been shown in Fig.2.2. The effect of torsional coupling depends strongly on the ratio of natural frequencies for uncoupled torsional and lateral motions of the corresponding uncoupled systems.

Humar & Wright(1977) studied dynamic behaviour of multistorey steel rigid frame building with setbacks. The steel frame was modelled as dynamic systems having finite number of degrees of freedom with the masses lumped at the floor levels. In the setback type buildings maximum utilized girder ductility ratios are substantially greater than the comparable uniform building. It is evident that setback buildings with slender towers designed according to such codes may undergo serious distress in the tower portion when subjected to severe earthquakes.

Tso and Biswas(1977) presented a procedure to compute the response of asymmetrical buildings subjected to two orthogonal components of ground motion. it is an extension of response spectrum technique for structures under uni directional excitation.

The accuracy of the procedure for the realistically proportioned asymmetrical building is checked with results obtained using time history dynamic analysis.

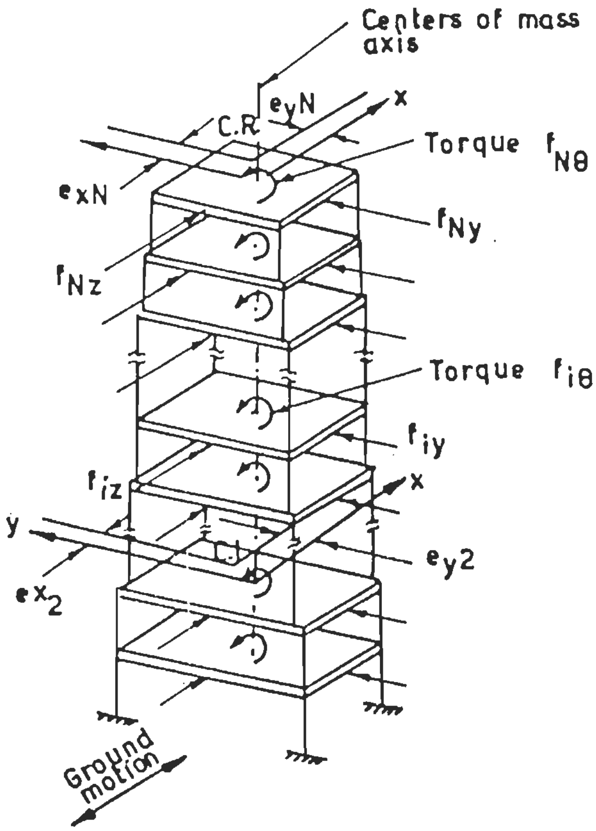


Fig. 2.2 - Idealized System for Setback Building

Rutenberg *et al.*(1978) proposed a scheme to calculate the effect of torsion on each lateral load resisting element of asymmetrical buildings in the context of the response spectrum techniques. The scheme consists of (i) obtain the modal shear and torque on the building by the response spectrum technique (ii) compute the total modal shear forces on each frame by resolving the modal shear and torque on the building according to principles of structural mechanics. Then obtain the total shear force on each frame by combining the total modal shears on that frame by combining the total shears on that frame in a root sum square manner.

Irvine & Kountouris(1980) investigated the inelastic seismic response of a simple torsionally unbalanced building. The building is modelled as two degree of freedom in which two identical frames support a diaphragm, the centre of mass of which may be offset from the centre of stiffness. It was found from the comprehensive parametric study that the eccentricity is not a significant parameter for peak ductility demand

Tso and Dempsey(1980) studied the dynamic torsional response of a single mass partially symmetric system to ground excitation. The torsional response and dynamic eccentricity are shown as functions of the eccentricity of the system and its uncoupled frequency ratio. It is shown that the dynamic eccentricity can best be expressed as a bilinear function of eccetricity for the critical ratio is unity. A comparison with the torsional provisions of five seismic Codes(Canada, Mexico, Newzealand, ATC3 and Germany) shows that the torsional moment and edge displacement of the system is underestimated by the first four Codes when the eccentricity is small and the uncoupled torsional and lateral frequencies are close.

kan & Chopra(1981) studied the coupling of lateral and torsional motion under earthquake excitations for buildings where the centre of mass do not coincide with centre of resistance. It is found that effect of torsional coupling depend significantly on ratio of uncoupled torsional to lateral frequency. For relatively large value of this ratio these effects are simple and can easily be generalized. For systems with ratio equal or greater than 2 these lateral deformations are unaffected. The responses primarily in translation and most buildings are strong in torsion, yielding of the system is controlled primarily by the yield shear, after initial yielding, the system has a tendency to yield further primarily in translation and behave more and more like an inelastic single degree of freedom system, responding primarily in translation. The torsional coupling generally affects maximum deformation in inelastic system to a lesser degree compared to corresponding linearly elastic systems.

Volcano(1982) studied the influence of the structural properties and earthquake features on differently defined ductility requirements and it is observed that damage level was similar for the structures with differently defined ductility factors. Weak or stiff structures requires greater ductility requirements. Duration of earthquake ground motions causes an increasing effect on the ductility factors which account for hysteretic energy. Hardening gives generally a more uniform distribution of ductility requirements but in some systems, total damage can increase inspite of an increase of hardening ratio. Softening produces a detrimental effect on the structures. The viscous damping produces a reduction of the mean and maximum ductility requirements.

Aranda(1984) studied ductility demands for r.c. frames irregular in height taking into account the inelastic behaviour. The importance of inelastic effects on the seismic analysis of r.c. frames irregular in elevation was shown with the exact step by step analysis. Irregularity in elevation increases the ductility factors by a factor approximately 2. This effect was significant where there was sudden change in the stiffness distribution of the building. This type of analysis is sensitive to the characteristics of the history of the record, It was important to define a procedure to use records of reduced duration to represent the overall inelastic behaviour of the structure. A procedure to scale maximum acceleration based on existing statistical information is presented.

Tso and Sadek(1985) studied the ductility demand and the edge displacement of a simple eccentric model in the inelastic range. It is found that unlike elastic response the coincidence of uncoupled torsional and lateral frequencies does not lead to exceptionally high inelastic response. It was also found that the system eccentricity has a large effect on ductility demand than earlier studies indicated. Eccentricity has the effect of increasing the edge displacement of the structure by a factor up to three when compared with that of a symmetrical systems. Increase in torsional stiffness of the structure tends to reduce this factor.

Bozergnia and Tso(1986) studied the inelastic earthquake response of a one way torsionally coupled systems subjected to two types of ground motion excitations. The effects of eccentricity, yield strength, uncoupled torsional to lateral frequency ratio and uncoupled lateral period on the response of the system were examined. The ductility demand on the critical element in the eccentric model can be up to about three times that for the corresponding symmetrical systems. Asymmetry affects the right edge displacement more than it affects the element ductility demand, especially for torsionally flexible structures. The ductility demand does not depend much on uncoupled torsional to lateral frequency but edge displacement is more sensitive to this ratio especially for stiff structures with low yield levels. It is shown that the stiff eccentric structures are vulnerable to such high ductility demand. Therefore a design strength of stiff eccentric buildings should not be reduced from the elastic strength demand.

Costa, Oliviera and Duarte(1988) studied the buildings exhibiting the vertical irregularities. The building was idealized as a set of plane moment resisting frames connected to shear walls by rigid diaphragms. Nonlinear behaviour for both the frames and walls were considered. It is found that ductility demand distribution are irregular in shear walls but fairly regular in the frames except for storeys immediately above a discontinuity, where there is a significant increase in the frame ductility demand. The ductility demands in the frame and shear wall are almost the same for regular buildings, For irregular building the ductility demand can be nearly twice the ductility demand for regular buildings. In general if irregularity occurs in frame than the ductility demand is increased in shear wall and if irregularity occurs in shear wall than the increase in ductility demand is observed in frame.

Sobaih, Hindi and Al-Noury(1988) studied the effect of different parameters on the nonlinear dynamic analysis of setback reinforced concrete frames. The parameters are setback level ratio'L_S' defined as L/L' where L is total height of the frame and L' is the height of the base portion of the frame; variation of beam properties; earthquake intensity and the type of nonlinear model. Maximum interstorey drifts occurs at the intermediate floors for L_S =0.375. At upper floors ductility demands for beams increases as L decreases. Also ductility demand for external columns may exhibit larger values as Ls decreases as shown in Fig. 2.3. The response of such frames is affected by setback ratio, beam properties, earthquake intensity and nonlinear model used in the analysis.

Hejal and Chopra(1989) studied the effects of lateral torsional coupling on the earthquake response of multistorey buildings. The effects of lateral torsional coupling on the responses of multistorey building and its associated one storey system are similar. It causes a decrease in the base shear, base overturning moment and top floor lateral displacement at the centre of rigidity, but an increase in the base torque, These effects are directly dependent on e/r ratio. Torsional coupling effects in the response of multistorey buildings depend on ρ (i.e. beam to column stiffness ratio). The effects of lateral torsional coupling on the height wise variations of forces is not very significant. It is more pronounced for storey shears and storey torques than storey overturning moments. Lateral torsional coupling affects the response spectra also for torsionally stiff systems the effect is very small but for torsionally flexible system the effect is significant. For torsionally stiff systems with closely spaced uncoupled frequencies and larger e/r values, the base torque at the centre of rigidity is approximated by the quantity $V_{BO}e_1w_1$. The product of base shear V_{BO} in the corresponding torsionally uncoupled multistorey system, e_1 is the effective eccentricity and $w_1$is the effective wt. in the fundamental vibration mode of the associated one storey system normalized by its total wt.

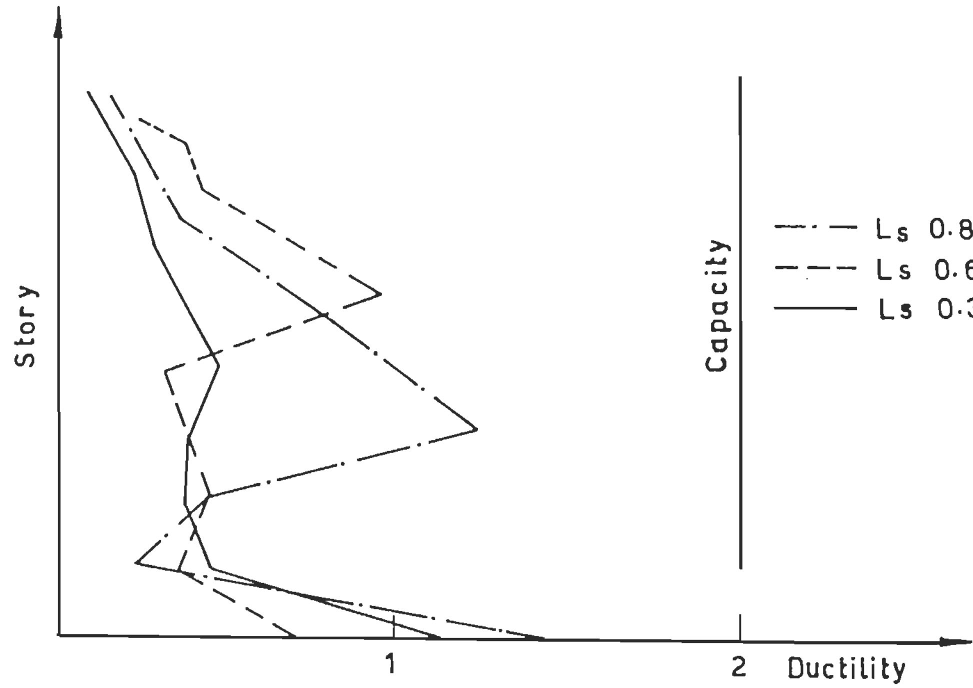


Fig. 2.3 - Column Ductility Demand for Unsymmetrical Frame

Ruiz, Rosenbleuth, Eeri and Diederich(1989) Studied seismic response of asymmetrically yielding structures having single degree of freedom hysteretic bilinear systems subjected to simulated accelerograms. It is concluded that the ductility demands of asymmetrically yielding hysteretic structures subjected to narrow band earthquakes tend to be much higher than that of symmetric systems. The duration of motion has a decisive influence on ductility demand. The longer the duration of the earthquake, the effect of the asymmetry is more on the ductility demand.

Prasad and Jagdish(1989) presented the inelastic response of single storey structure, square in plan supported on four columns subjected to earthquake assuming that the model is having three degrees of freedom per floor. The responses of the structure to simultaneous action of the two orthogonal horizontal components of the ground motion and to one of the two components have been compared.The torsion does not seem to significantly influence the result for small eccentricities, but for the large eccentricities like e/a=.0 3, the maximum ductility demand of some columns reduced due to torsion, the largest of maximum ductilities is increased. This increase ranges from 5% to 50% when compared with zero eccentricity case. Disparity between largest of the maximum ductilities to the smallest of the maximum ductilities increases with eccentricity for the two components while it decreases with eccentricity for one component input. For zero eccentricity case torsional response was noticed for two components input and not for one component case. The response spectra has showed that the columns in short period structures experience larger ductilities.

Hamzeh *et al.*(1990) investigated inelastic response of torsionally coupled system to an ensemble of real earthquake records in terms of system parameters such as lateral frequency, uncoupled torsional to lateral frequency ratio and eccentricity of the system. Angle of incidence of earthquake has significant effect on both ductility ratio and torque, especially in the low period range. For the smaller period range the system ductility ratio, the system torque and ductility ratios for the weakest column are significantly influenced by ratio of torsional and lateral frequency for the two component earthquake.

Goel and Chopra(1991) presented the influence of system parameters, uncoupled lateral vibration period, uncoupled torsional to lateral frequency ratio, stiffness eccentricity, relative values of strength and stiffness eccentricity, yield factor on the inelastic response of one storey asymmetric plan systems to two excitations. It is found that the torsional deformation of elastic as well as inelastic systems tends to increase with increasing stiffness eccentricity e_s/r and decreasing frequency ratio Ω_θ over a wide range of structural vibration periods. For very long period, displacement sensitive elastic systems, the torsional deformations tends to zero regradless of e_s/r and Ω_θ values. The lateral deformation of elastic as well as inelastic systems generally decreases with increasing e_s/r and decreasing Ω_θ. The element deformation of elastic systems is affected more by e_s/r and Ω_θ compared to the inelastic systems. It is also concluded that the torsional deformation of the system decreases if it is excited well into the inelastic range. Inelastic action influences the largest of peak deformations among all resisting elements of systems in a manner similar to the way it influences the lateral deformation. The ratio of the lateral deformations at the C_s of inelastic and elastic asymmetric plan systems is significantly different than symmetric plan systems, then the effects of plan asymmetry are significant. The ratio of element deformations for inelastic and elastic systems is affected by plan asymmetry to a greater degree compared to the ratio for deformation at C_s and is smaller for asymmetric plan systems.

Maheri, Chandler and Bassett(1991) tested models designed with variable ratios of torsional to lateral stiffness and with both symmetric and asymmetric mass distributions under earthquake base loading and it was concluded that the analysis of dynamic structural properties leads to very accurate predictions of frequencies and mode shapes. The analysis showed that earthquake response in the asymmetric cases is dominated by first mode but the experimental results showed that the second mode is much more significant than the theory predicts. In torsionally coupled structures, the theory overestimates the contribution of the first mode. The difference between theoretical and experimental responses identified are particularly significant for structures with low uncoupled frequency ratio R_f. The experimental results and conclusions drawn in comparison with theoretical predictions are considered to be widely applicable to seismic analysis and design of asymmetric multi-storey frame structures.

Chandler and Duan(1991) evaluated the factors which affects the inelastic seismic performance of torsionally asymmetric buildings. It is shown that Mexico 76 Code torsional provisions are inadequate and on other hand Mexico 87 Code torsional provisions are over conservative. It was found that the element at the stiff edge is the critical element which suffers severe damage than the corresponding symmetric structures. The peak ductility demand of the element at the flexible edge is always lower than that of corresponding symmetric ones. It has been recommended that the design eccentricities expressions of Mexico 87 Code be changed to $1.5e_s + 0.1b$ and $0.5e_s - 0.1b$. It will lead to minimum strength design in resisting elements.

Zeriss, Tassios and Zhang(1992) evaluated strength reduction factor q of plane reinforced concrete frames designed by Euro Code using computer algorithm. Conventional drift limits and local curvature ductility checks has been adopted as criteria for estimating q. Three reinforced concrete frames having similar geometry and different first storey heights has been considered. The controlling criteria for defining q are local demanded ductilities rather than drift. The estimated response reduction factors are higher than those assumed for design but for the frame with a relatively tall first storey, tighter local detailing restrictions are required to achieve the design reduction factor assumed in the design.

Rutenberg, Benbenishti and Pekau(1992) presented a parametric study of earthquake response of single storey asymmetric structures designed by the static provisions of various codes. It is shown that SEAOC/UBC and NBCC designs lead to lower ductility demand than the ATC/NEHRP and CEB designs. The presence of elements normal to the direction of excitation usually moderates peak ductility demand displacement and rotation but the effect is not appreciable. In the asymmetric systems design results in larger ductility demand than in symmetric systems. Ductility demand response is affected by the type of model chosen. Increase in torsional to lateral frequency ratio tends to lower peak ductility demand. The maximum displacement of asymmetric systems is larger than that for the similar symmetric system and the factor of 2 is possible.

Nassar, Osterass and Krawinkler(1992) presented seismic design based on strength and ductility demand. It highlights the significance of ductility in seismic performance. Serviceability and collapse limit states has been expressed explicitely. Ductility capacity is the basic seismic design parameters for collapse limit state. Statistically inelastic response of SDOF and MDOF systems provides a means of developing strength design criteria based on ductility capacity. This approach is more complicated than the current code approach.

Goel and Chopra(1992) evaluated the effects of plan asymmetry on earthquake response of code designed one storey systems to know that how these effects are well represented by various building codes. It was concluded that the stiff side element with design force smaller than its symmetric plan value experienced increased ductility demand because of plan asymmetry. The ductility demand on the flexible side element is significantly smaller than in the symmetric plan system. Asymmetric plan systems with reduction factor R=1 may experience structural damage due to yielding and non structural damage resulting from increased deformations. Present building codes does not ensure that deformation and ductility demands for symmetric and asymmetric plan systems are similar. It is also concluded that additional deformations due to plan asymmetry cannot be reduced by modifying the design eccentricity in the codes.

Zhou and Minoru(1992) presented pulse response analysis to evaluate the maximum responses of asymmetric structures and is applied to an idealized monosymmetric system. Results were compared with those given by time history analysis and it was concluded that proposed procedure gives reasonable estimate of responses.

Boroschek and Mahin(1992) presented dynamic torsional behaviour of an existing building that responded severely during service level earthquake. Parametric studies are carried out on linear and nonlinear models of the building. The torsional behaviour in regular structures increases stress and ductility demands in element located away from the centre of rotation and translational displacement are also affected. These effects are influenced by the characteristics of the input ground motion and these effects are more severe for elastic structures than inelastic structures and are highly dependent on the characteristics of the input ground motion.

Fukada, Kobayashi, Adachi, Nagata and Hayashi(1992) presented the results of vibration tests performed on the building after its completion and result of simulation analysis. It was found that the plane frames with different heights in a setback building are to be given the same lateral rigidities to their own weights. Then the natural vibration periods of such structures are shorter than those of design analysis due to the fact that additional rigidities due to non structural elements give its own contribution. When the vibration amplitudes are larger, then these additional rigidities disappear.

Ayala, Garcia and Escobar(1992) evaluated the seismic performance of nonlinear asymmetric building structures with resisting elements in one and two orthogonal directions and the suitability of different design recommendations. Seismic performance is measured by the ratio of maximum ductility demand for asymmetric structures to the maximum ductility demand for the corresponding symmetric ones. The torsional response of the building structures is significantly affected by the in-plan distribution of the strength. The coefficients involved in the design eccentricities recommended in the current code for Mexico city which follows the distribution of mass, it may lead to the values of performance indexes in excess of those considered adequate. To keep the values within the acceptable limits, the design coefficients are to be modified in such a way that the torsional overstrength is kept constant, the interstorey resisting force is moved toward a position between C_m and C_s.

Corderoy and Thambiratnam(1993) presented a simple method for earthquake analysis of torsionally coupled setback buildings on flat grounds. The analysis uses the shear beam model in which floors are assumed to be rigid, each with three degrees of freedom. The system can be analysed elastically or elasto-plastically. The whole procedure has been programmed in such a manner that any degree of asymmetry can be taken care of. The sequence of columns yielding demonstrated the effect of asymmetry. The columns closest to the floor centroids yield first. The time step has been selected 1/10th of the structure's fundamental period for elastic analysis, For elasto-plastic range the time step must be smaller than this. In the design situations the column stiffness and strength are to be chosen such that behaviour of the building is consistent with the function to be performed by the building under consideration.

Cruvellier and Smith(1993) presented a method for static and dynamic analysis of 3 dimensional asymmetric buildings composed of intersecting bents of any structural type by modelling it in two dimensions. The two dimensional model is simple which neither requires pre nor requires post analysis transformations. In this method, the engineer is forced to understand structural action in order to translate it from three to two dimensions. It is more relevant to educational considerations rather than to the practical purposes.

Colajani *et al.*(1993) studied the response of structures taking into account large displacement effects. In the elastic field these effects can be qualitatively evaluated by a one storey equivalent system for a particular class of structures. The one storey inelastic system has also been analysed by assuming a yielding surface. It is confirmed that the torsional deformable structures in the absence of symmetrical distribution of the translational stiffnesses are affected by a dynamic response which is unforeseeable by small displacement analysis. In the elastic field a high risk condition can occur particularly for small values of damping ratios, when the dissipative capacity of the system is involved.

Correnza *et al.* (1994) analysed series of models subjected to both uni and bi directional ground motion input and found that for the flexible edge element, accurate estimates of additional ductility demand arising from torsional effects may be obtained from uni-directional models only for medium range to long period systems. These estimates may be over conservative for short period systems, which constitute a large proportion of system for which Code static torsional provisions are utilized. It is further concluded that models incorporating the transverse elements but analysed under uni-directional lateral loading may under estimate by up to 100% the torsional effects in such systems. But are reasonably accurate for medium and long period structures.

Llera *et al.*(1994) studied the accidental torsion effects in buildings due to stiffness uncertainty. Symmetric plan buildings can be asymmetric due to the discrepancies between the computed and actual values of the structural element stiffness and under go torsional vibrations under the effects of purely translational ground motion. Such accidental torsion leads to increase in structural element deformations which is shown essentially insensitive to the uncoupled lateral vibration period of the system but is strongly affected by the ratio of uncoupled lateral and torsional vibration periods. It has been found that the structural deformations due to stiffness uncertainty is shown to be much smaller than implied by the accidental torsional provisions in the building code and most other building codes.

Wong and Tso(1994) studied the inelastic seismic response of the torsionally coupled unbalanced structural systems with strength distributed using elastic response spectrum analysis. It has been shown that inelastic responses depend strongly on the torsional stiffness of the system. For torsionally stiff system, the torsional response leads to decrease in the stiff edge displacement but for torsionally flexible systems, the torsional response leads to increase in the stiff edge displacement by taking accidental torsion effects into account, the response spectrum analysis gives strength distribution such that there is no excessive ductility demands on the lateral load resisting elements.

Llera *et al.***(1994)** studied the differences between the increase in building response due to accidental eccentricity predicted by Code specified static and dynamic analysis for symmetric and unsymmetric single and multistorey buildings. Upper and lower bounds for differences in response computed from static and dynamic analysis are obtained for general multistorey systems. These differences in response primarily depends upon the ratio of the fundamental torsional and lateral frequency of the building. These are larger for small values of the frequency ratio and decrease to zero as the frequency ratio becomes large. The discrepancies between the increase in response due to accidental eccentricity predicted by dynamic and static analyses is in many cases is of the same order of magnitude as the response increase itself. This suggests that the Code specified static and dynamic analysis to account for accidental torsion should be modified to be mutually consistent.

Llera *et al.***(1995)** Suggested the simplified model for analysis and design of multistorey buildings based on single super element per building storey by matching the stiffness matrices and ultimate yield surface of the storey with that of the element. The errors in peak responses are expected to be less than 20% for most practical structures. The model uses an accurate representation of storey shear and torque surfaces, which capture the fundamental features controlling the inelastic behaviour of the building.

2.4. Codal Provisions of Various Countries

Australian Standard 2121-1979 recommends that where a regular building or framing system has one setback in which the plan dimension of the tower in each direction is at least 0.75 times the corresponding plan dimension of the lower part, such a building may be considered as being without a setback for the purposes of determining and distributing earthquake forces. Buildings with other conditions of setback in either zone A or 1, the tower shall be designed as a separate building using the larger of values of the seismic response factor C at the base of the tower determined by considering the tower as a separate building for its own height or as part of the overall structure. The resulting shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height. For buildings with other conditions of setback shall be analysed by considering the dynamic characteristics of such buildings. Horizontal torsion can be accounted for by taking the design eccentricity e_d as $1.7e_s-e_s^2/b+0.1b$ or $e_s-0.1b$ where b is the maximum lateral dimension of the building perpendicular to the horizontal loading direction under consideration and e_s is the static eccentricity.

National Building Code of Canada(1990) specifies that where the centroids of mass and the centres of stiffness of the different floors do not lie approximately on vertical lines, a dynamic analysis shall be carried out to determine the torsional effects. A setback is a sudden change in plan dimension or a sudden change in stiffness along the height of a building. The effects of major changes in stiffness and geometry are best investigated by dynamic methods. The design eccetricity for regular asymmetric structures has been specified as $1.5e+0.1D_n$ or $0.5e-0.1D_n$. where D_n is the plan dimension of the building in thr direction of computed eccentricity, e is the distance between the location of the resultant of all the forces at and above the level being considered and the centre of rigidity at the level being considered.

National Standard of People's Republic of China Aseismic Building Design Code GBJ 11-89 says the effect of structural torsion can be taken into account by assigning 3 d.o.f. per floor; i.e. mutully orthogonal two components of translation and one component of rotation. Seismic loads and actions are correspondingly evaluated by means of the spectral and modal methods.

Japan Earthquake Resistant Design Method for Buildings specifies that a coupled system consisting of appendage and the main structure must be analyzed according to modal analysis procedure which include evaluation of natural periods and associated oscillating modes for a structural model.

New Zealand Standard NZS 4203:1992 says that a three dimensional modal analysis or a three dimensional numerical integration time history analysis shall be used for structures having horizontal and vertical irregularity.

Swiss Standard SIA 160 (1989), , says that if the plan layouts of the structure and the distribution of mass are not approximately symmetric and with significant discontinuities through out the height of the structure, then a more detailed method of calculation shall be used, such as the response spectrum method.

Regulations for Earthquake Resistant Design of Buildings in Egypt - 1988 Code of Practice for Loading, Ethiopia ESCPI-1983, Indonesian Earthquake Code 1983, National Structural Code for Buildings Philippines recommends that for buildings with setback where the plan dimension of the tower portion in each direction is at least 75 percent of the corresponding plan dimension of the lower part, the effect of the setback may be neglected for the purposes of determining seismic forces by the equivalent static force method. For other conditions of setback in buildings the detailed dynamic analysis is to be carried out.

I.S.1893-1984 recommends that buildings having irregular shape and or irregular distribution of mass and stiffness in horizontal and/or vertical plane shall be analysed by modal analysis and torsional shears are to be accounted for separately by taking eccentricity equal to 1.5 times the static eccentricity between the centre of mass and centre of stiffness. Negative torsional shears are to be neglected.

Earthquake Resistant Standards National Regulations of Construction Peru recommends that if the reduced dimension in plan is not less than 3/4 parts of the dimension of the immediate lower story in the direction in which the earthquake is considered, the force H shall be calculated and shall be distributed in height according to the usual practice. Similarly if the base of the building with reduction has the height less or equal to 30% of the total height of the building, it shall be considered that the reduction will not modify the distribution of H force. If the reduced dimension in plan is less than the 3/4 parts of the dimension of the immediate lower story in the direction considered, the reduced part shall be determined at its base according to the following criteria:

(a) In the case when the reduction is between 50% and 75%, the reduced part will be treated like one independent tower and the base shear force will be determined according to its base multiplied by an amplification factor of 1.25.

(b) In the case when the reduction is more than the 50%, the reduced part will be treated like one independent tower and the base shear force will be determined according to its base multiplied by an amplification factor of 1.5.

(c) To the base of a building considering as a whole with the reduction referred above shall be applied the shear force calculated according to the indication of the same paragraph adding the forces that will be determined for this lower portion, as indicated above.

Uniform Building Code of U.S.A.(1991) recommends that structure having irregularity of the type as (1) a story in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above (2) where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story, a roof which is lighter than the floor below need not be considered.(3) Where the horizontal dimension of the lateral resisting system in any story is more than 130 percent of that in an adjacent story, one story penthouses need not be considered, are to analysed with dynamic analysis. A three dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity and having a rigid or semi rigid diaphragms. Accidental eccentricity is defined as 5% of the plan dimension in the direction of stiffness eccentricity.

2.5 Analytical Models

2.5.1 3D R.C. Frame Members

Buildings with irregular shapes are highly torsionally coupled under the action of lateral loads such as earthquake loads. Therefore analysis of a structure is to be carried out in the 3D space. This requires the modelling of the members such as(i.e. beams/columns) should be three dimensional in nature. Therefore r.c. frame members should be modelled as three dimensional frame members with 6 d.o.f. per node of the structure. Different models of 3D r.c. members are presented by various authors for non linear analysis of building frames under earthquake loading.

Nigam(1967) presented nonlinear analysis of frame structure under dynamic loading. The yield condition of the member is governed by the interaction of different stress resultants. The plastic deformation after yielding of the section are assumed to be concentrated on single section that is a plastic hinge of zero length is assumed to exist. The plastic flow is assumed to be along the gradient of the yield function evaluated at the point representing the current state of the stress resultants.

Wen and Farhoomand(1970) modified Nigam's model to allow for plastic hinge to extend over a finite length.

Tseng and Penzien(1975) presented a model consisting of two parts in series, linear and nonlinear. The nonlinear part is assumed to be concentrated at the beam ends in the form of 3D plastic hinges. The elastic-plastic tangent stiffness matrix has been derived using the generalized yield function $F(P_u, M_{yu}, M_{zu})$ and the associated flow rules for elasto-plastic solid section.

Takizawa and Aoyama(1976) introduced a model which takes the interaction of biaxial bending moment and stiffness degrading effects. Prager-Ziegler kinematic hardening theory was used to shift from the cracking stage to the yield stage.

Gillies and Shepherd(1981) presented a three dimensional elasto-plastic model. The inelastic actions were confined to the ends of the element. Rigid end blocks were specified to simulate the joint core zones. Two rotational springs were provided at each end of the beam elements to model the flexural behaviour along the local y and z axes. This also includes axial spring which has the capability to represent the axial yield under combined bending and axial tension or compression.

Lai *et al.*(1984) presented a model consisting of two inelastic elements at the two ends of a reinforced concrete member sandwiching a linear elastic line element as shown in Fig. 2.4. For each inelastic element there are four inelastic springs at each of the four corner regions with a fifth spring at centre of the section. Each of the four exterior springs represents the stiffness of the effective reinforcing steel and effective compression concrete. The fifth spring has only one component which is from the effective concrete in centre region. Inelastic behaviour is fully concentrated within the inelastic elements located at two ends of a member.

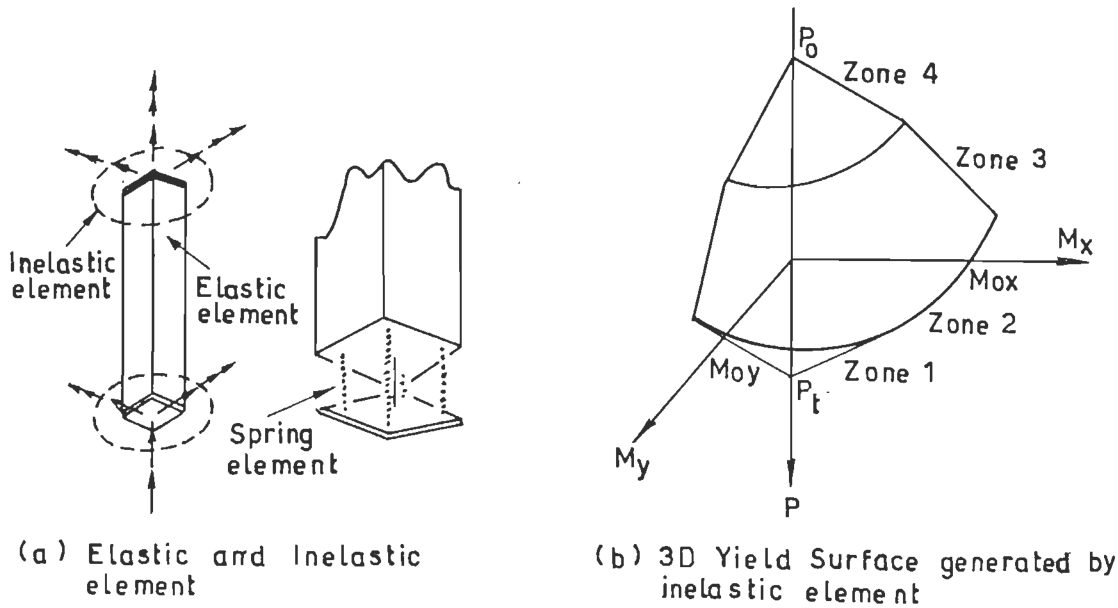
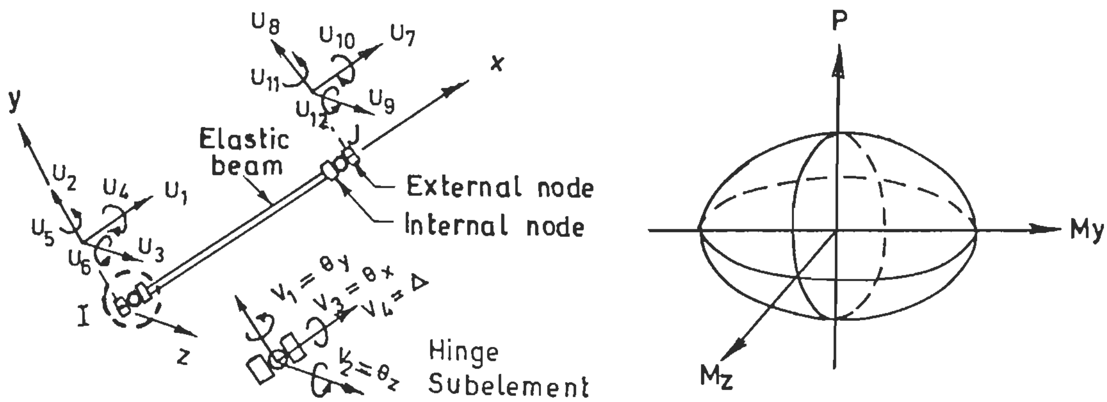


Fig. 2.4 - Inelastic Model for 3D R/C Member: Lai etal(1984)

Powell and Chen(1986) presented a 3D beam column element with a generalized plastic hinge as shown in Fig.2.5. This model takes into account the interaction of axial forces, biaxial bending moments and torsion. This model also takes into account the stiffness degradation and the rate dependent material properties.

Sfakianakis and Fardis(1991) presented a new model for r.c. columns subjected to a cyclic biaxial bending with axial force. Its basic constituent is the tangent flexibility matrix of a section which relates the set of increments of the 3 normal stress resultants P, M_y, M_z to that of the corresponding section deformation. This incremental flexibility relation is based on the bounding surface concept which is constituted as the locus of points (P, M_y, M_z) at ultimate strength of the r.c. cross section.



(a) Elastic and inelastic (hinge) subelement

(b) Force moment yield surface

Fig. 2.5 - Inelastic Model for 3D R/C Member: Powell and Chen

Thanoon(1993) proposed yield criteria for 3 D r.c. frame members taking interaction of axial forces, bending moments M_y and M_z and torsional moment T. Shear forces effects have been neglected. The yield function is described as

$$(m_y/m_{yp})^2 + (m_z/m_{zp})^2 + (t/t_{uo})^2 = 1.0$$

$$\text{where } m_{zp}/m_{zo} = a_1+a_2(p_u/p_o)+a_3(p_u/p_o)^2+a_4(p_u/p_o)^3$$

$$\text{and } m_{yp}/m_{yo} = b_1+b_2(p_u/p_o)+b_3(p_u/p_o)^2+b_4(p_u/p_o)^3$$

where m_{zo} and m_{yo} are the dimensionless values of the ultimate bending moment capacities of a section about z and y axes respectively, when the axial force is equal to zero. m_{zp} and m_{yp} are the dimension less values of the ultimate flexural strengths about the z and y axes respectively, for a fixed value of P_u. The constants a_1 to a_4 and b_1 to b_4 are the polynomial constants.

2.5.2 Stiffness Degrading Models

The inelastic analysis of reinforced concrete buildings subjected to strong earthquake motions requires a realistic model which takes into account the continually varying stiffness and energy absorbing characteristics of the structure due to the inelastic behaviour of the structural members under strong motions. Various models available in the literature are described briefly here.

Takeda *et al.*(1970) presented a model by defining a primary curve for initial loading and a set of rules are described for reversal loading. The primary curve is characterised by three linear segments, such as cracking point, yielding point and point beyond yielding. depending upon the loading state the set of rules are prescribed for reversal loading to take its paths. Seven condition hysteretic model has been formulated.

Imbeaut and Nielson(1973) suggested the use of degrading bilinear model. This model uses a deteriorating elastic stiffness that represents the average value of the unloading and reversed loading stiffness. The reduced elastic stiffness is a function of the maximum displacement ductility and is given as $K=K_g(1/\mu_d)^{\alpha}$ in which μ_d = displacement ductility = D_{max}/D_{yield}, α=constant $0.5<\alpha<0.6$,K_g=initial stiffness based on gross section.

Anderson and Townsend(1977) presented two degrading stiffness models. The first referred to as the degrading trilinear model (DTL) has the following properties. The initial loading branch and all unloading branches have stiffness K_g based on the gross section properties. The strain hardening branch has a stiffness = 0.03 K_g and reversed loading branches have stiffness = $K_g(1/\mu_c)^{1.5}$ in which μ_c is the maximum curvature ductility of the member. The second model is referred to as a degrading trilinear connection model. In this the initial loading branch stiffness and subsequent unloading branch stiffnesses of the hinge element=$K_g/4$. This takes into account the loss of stiffness produced by concrete cracking in the beam and joint rotation generated by bond failure around the longitudinal beam steel anchored in the joint core.

Chen and Powell(1982) took stiffness degradation into account when reversed loading is applied. It is assumed that the stiffness degrades independently for each force component of each subhinge in inverse proportion to the largest previous hinge deformations. The unloading stiffness K_1' for cracking hinge and K_2' for yielding subhinge depend on the previous maximum positive and negative hinge deformation and are controlled by input coefficients α_1 and α_2 for cracking subhinge and β_1 and β_2 for yielding subhinge. These coefficients control the reloading stiffnesses K_1'' and K_2'' also as shown in Fig. 2.6.

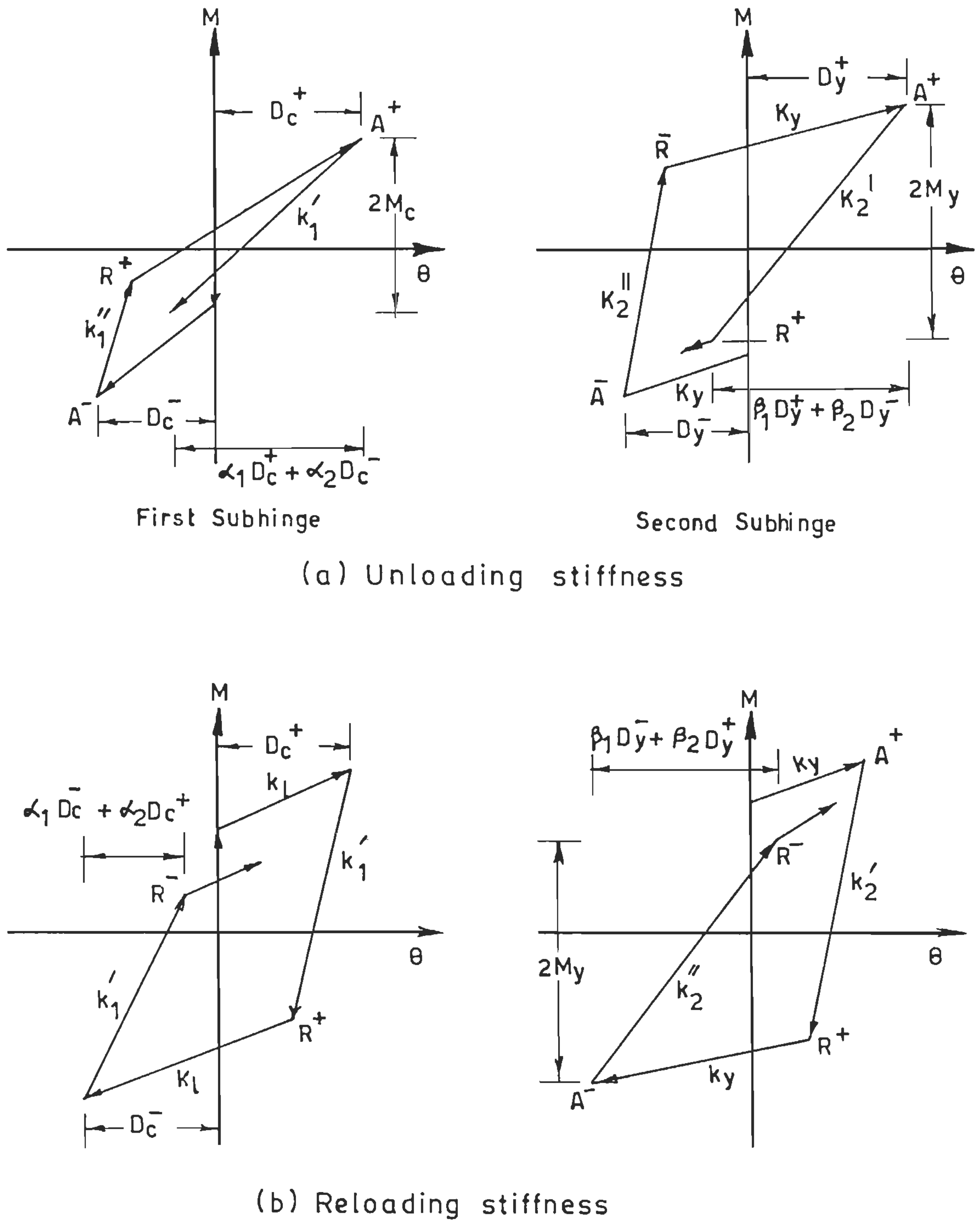


Fig. 2.6 - Stiffness Degrading Model

Allahabadi and Powell(1988) introduced stiffness degrading model for r.c. beams under cyclic loads. Strain hardening and degrading flexural stiffness are approximated by assuming that the element consists of a linear elastic beam element with nonlinear rotational springs at each end. All plastic deformation effects including the effects of degrading stiffness are introduced by means of the moment rotation relationships for the hinge rotations. The moment rotation relationship for each hinge is an extended version of Takeda's model which has the behaviour as shown in Fig.2.7. The extension to Takeda's model are (i) a reduction of unloading stiffness by an amount which depends on the largest previous hinge rotation. (ii) Incorporation of variable reloading stiffness which is larger than that of the Takeda's model and also depends upon the past rotation history. The unloading stiffness K_u depends on the maximum hinge rotation and is controlled by input parameter α as shown in Fig. 2.7(a) and its value varies from 0.0 to 0.4. The reloading stiffness as controlled by input parameter β as shown in Fig.2.7(b) and its value varies from 0.0 to 0.6.

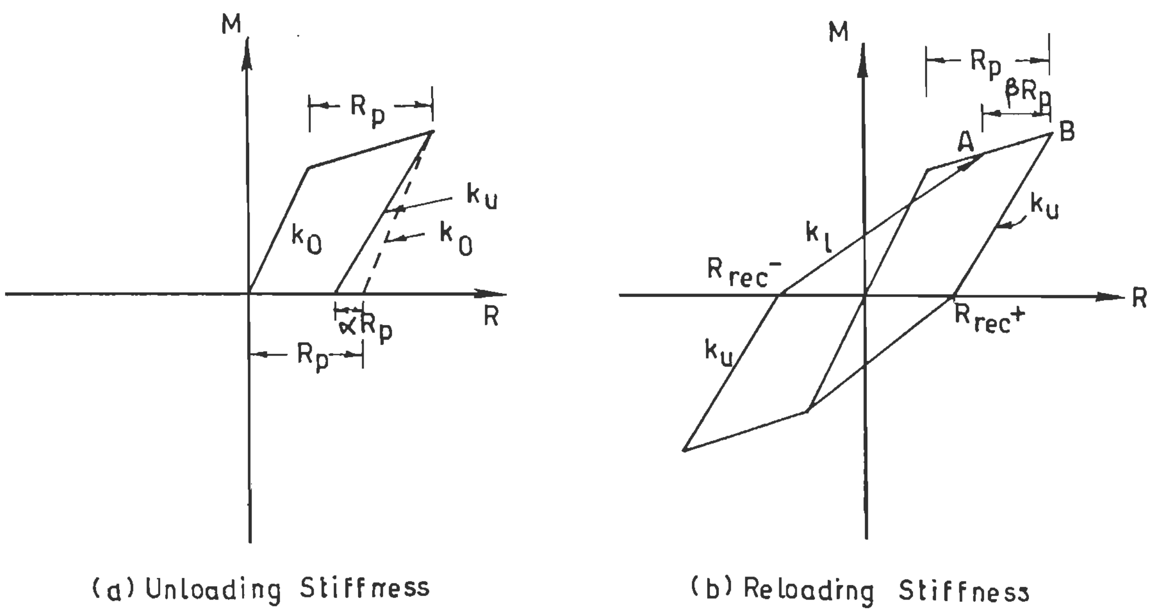


Fig. 2.7 - Extended Takeda's Model

Kunneth *et al.*(1990) presented the hysteretic model which uses three parameters α, β, γ in conjuction with nonsymmetric trilinear curve. Stiffness degradation represented by α is introduced by setting a common point on the extrapolated initial stiffness line and assumes that unloading lines target the point until they reach the x-axis, after which they aim the previous maximum or minimum points. Pinching behaviour is introduced by lowering the target maximum or minimum point to a straight level γP along the previous unloading line. Reloading lines now aim this new point untill they reach the crack closing point after which they target the previous maximum or minimum point. Strength degradation is introduced by parameter β.

2.5.3 Panel Elements

Reinforced cement concrete framed building structures consists of r.c. space frame with brick work or concrete block masonry infills and stiffened by floor slabs acting as rigid diaphragms. Slabs and infill panel increases the load carrying capacity of the r.c. framed structures. The analysis of the above system may be carried out either on approximate basis or on finite element basis. To analyse the structure into elastic range the approximate methods are sufficient and to extend the analysis into inelastic range the finite element method is to be employed. The various modelling techniques available in the literature for panel elements is briefly described here.

Holmes(1961) proposed the concept of panel element as an equivalent compression strut of thickness equal to that of the panel and a width equal to one third of the length of diagonal of the panel. Effective elastic modulus for the equivalent strut were computed on the basis of various tests. **Smith(1962,66,68)** proposed that the width of equivalent strut depends upon the loading applied, relative stiffness of the frame and the infill.

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 element between the frame and the infill. The results obtained for two storeyed infilled frame were found to match with experimental results. **King and Pandey(1978)** described procedure based on finite element method for analysing infilled framed structures. It is shown that the fairly coarse finite element meshes can be used for finding the lateral stiffness of the single frame. The infill is idealised as four noded rectangular elements with 2 degrees of freedom at each node. **Liauw and Kwan(1984)** examined the nonlinear behaviour of non integral infilled frames using finite element method. It was shown that the stress redistribution towards collapse were significant and the strength of non integral infilled frames was very much dependent on the flexural strength of the frame. **May and Naji(1991)** carried out nonlinear analysis of infilled frames. The frame members has been modelled with 3-noded frame elements, and panel elements has been modelled as 8-noded element. A 6-noded interface element has been used to model the interface between the frame and the panels. The analysis provided good results up to the failure load.

Macleod(1969) proposed a new model for shear walls which has rotational degree of freedom at each node.

Franklin(1970) studied the behaviour of infilled frames taking into account material non linearity, cracking, separation and slip between the frame and the panel.

Liauw(1970,73) analysed the frame with infill by using the eight term stress function to satisfy the boundary condition of continuous compatibility between frame and the infill.

Kost *et al.*(1974) described a method for dynamic analysis of frames with shear walls with pre existing gaps at the interface of walls and frames. All parts of the structure are assumed to be linear but the response of the structure is nonlinear due to opening and closing of gaps. Each panel is modelled as four noded elements with 16 generalised displacements.

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 analytical results has been reported.

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

2.6 Concluding Remarks

Based on the literature review the following points emerge

- Setback type and regular asymmetric buildings on flat grounds have been analysed extensively with approximate and rigorous methods of analyses. The buildings on sloping ground have not been studied much as yet.

- It has been observed that torsional deformations of systems tends to increase with increasing eccentricity.

- Building Codes suggests, a detailed dynamic analysis is to be carried out for irregular asymmetric buildings. But for regular asymmetric buildings static analysis procedure is recommended by taking the design eccentricity as suggested in various Codes.

- For nonlinear dynamic analysis of buildings r.c. member modelling based on plastic hinge concept has been used. These model takes into account the interaction of axial forces and moments in three directions. The effect of shear forces has not been considered.

- Stiffness degrading models based on Takeda's model for 2D r.c. members has been used extensively.
- Various models based on finite element approach for panel elements has been used. It is very important to account for stiffness due to panel elements while analysing frames under seismic loading.

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BUILDING MODELLING WITH RIGID FLOOR IDEALISATION

3.1 Introduction

Many multistoreyed reinforced concrete framed buildings rests on hill slope having stepback and setback configurations as shown in Fig. 3.1. Buildings on hill slope are good examples of irregular buildings which are highly unsymmetrical in plan and elevation. The centres of mass of different floors of the buildings on hill slope lie on different vertical axes. The centres of stiffness of the different storeys also lie on the different vertical axes. These buildings undergo severe torsion under earthquake excitation and it is the main cause of damage to such buildings during earthquakes. For such irregular buildings all Codes recommend to carry out full 3D dynamic analysis. Such irregular buildings requires full 3D analysis needing lot of data preparation, handling and interpretation of voluminous results. In preliminary analysis and design of irregular buildings such as stepback and setback, lot of trials are required to be carried out to decide about the configurations and layout of the building at the initial planning stage. Simple method which can give good results will be very much helpful/suitable. Simple method of dynamic analysis is needed for stepback and setback buildings characterised by centre of mass of various floors lying on different vertical axes and so is its stiffness. It may be supported at different levels by two types of columns (i) columns resting on sloping ground and (ii) columns resting on floor below.

The mathematical model of a building on hill slope should represent the spatial distribution of mass and stiffness of the structure to be able to represent significant features of its dynamic response. A simplified 3D model for dynamic analysis of stepback and setback buildings needs to be developed which represent 3D behaviour of the building. The Codal provisions of various countries for dynamic analysis, development of the simplified method, analytical procedure and validation of the model using numerical examples have been presented here.

3.2 Codal Provisions of Different Countries

Code of Practices such as National Building Code of Canada(1990), National Standard of people's Republic of China GBJ(11-89), Japan Earthquake Resistant Design

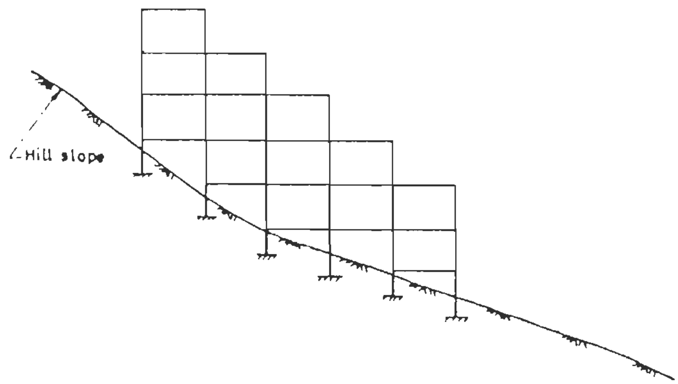


Fig. 3.1 - Stepback and Setback Building

Method of Buildings, New Zealand Standard NZS(4203-1992), Swiss Standard SIA 160(1989), Uniform Building Code of U.S.A.(1991) recommends for structures having major changes in stiffness and geometry are to be analysed by dynamic analysis using three dimensional model. It is also recommended that accidental eccentricity for asymmetric buildings should be taken into account in addition to structural eccentricity in order to account for the uncertainties in the evaluation of stiffnesses, location of masses, disturbances in these quantities during construction etc.

I.S. Code 1893(1984) recommends modal analysis for buildings having plan irregularity and says that torsional shears are to be evaluated separately by taking design eccentricity as 1.5 times the distance between centre of mass and centre of stiffness at various floors. Negative torsional shears are to be neglected.

The Code of Practices recommends dynamic analysis for these highly irregular, asymmetric buildings. . At the same time base shear concept of Codes for regular buildings is not applicable to buildings on sloping ground as these are supported at different levels along the hill slope. Buildings on sloping ground having irregularity in plan and elevation can not be analysed by pseudo-static analysis procedure suggested by[Cheung and Tso(1987), Smith and Cruvellier(1990), Goel and Chopra(1993)]. Planar modelling techniques suggested by[Cruvellier and Smith(1993)] is too complicated to be used in the design offices.

3.3 Representation of Stiffness and Masses about a Common Reference Axis

In a building with rigid floor diaphragm following three situations may arise with respect to the centre of mass and centre of rigidity.

(i). Coincident centre of mass and centre of rigidity of each floor lying on the same vertical axis. In this case the building does not undergo torsional motion under lateral excitation and the standard stiffness approach for each storey is applicable. This is applicable to regular and symmetrical buildings.

(ii) Centre of mass of each floor lying on the same vertical axis where as the centre of rigidity of each floor does not lie on the same vertical axis. In this case the stiffness of each storey is formulated about a common vertical axis. Kan and Chopra(1976, 1977, 1981) transferred the storey stiffness from the centre of rigidity to the common vertical axis passing through the centre of mass of each floor. Figure 3.2 shows that centre of mass of all the floors lies on the same vertical axis passing through c.g. of the floors.

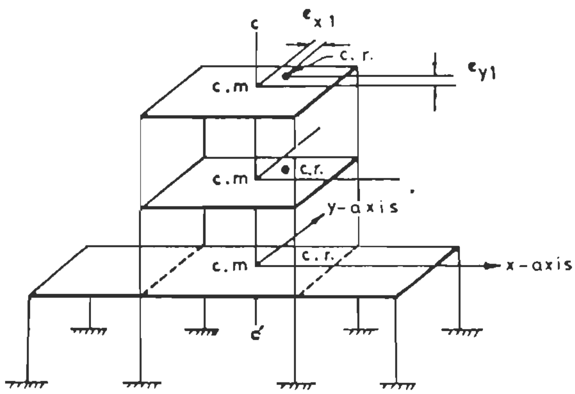
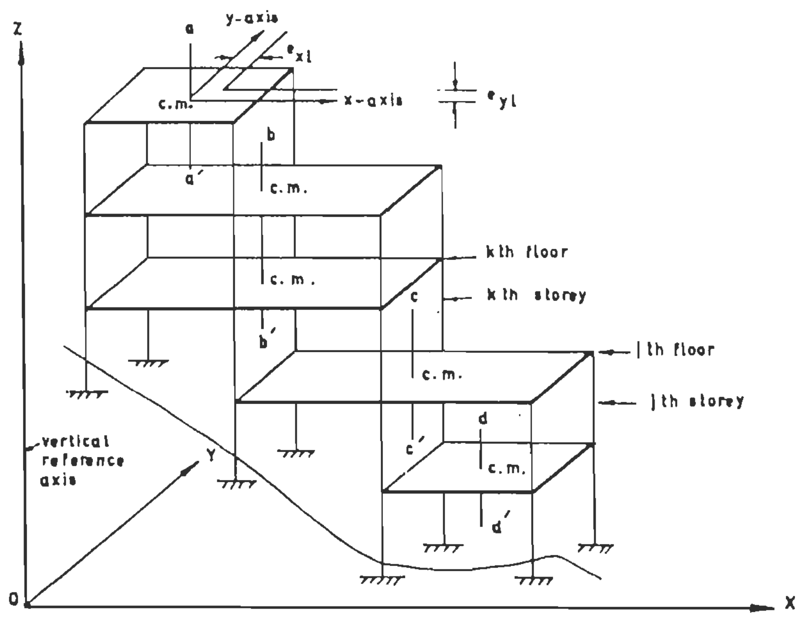


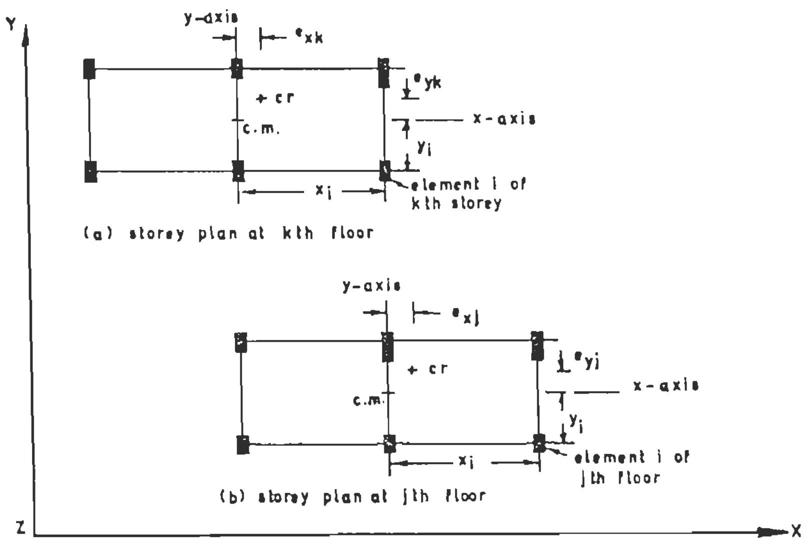
Fig. 3.2 - Idealized Symmetrical Setback building

(iii)Centre of mass of each floor not lying on the same vertical axis as well as centre of stiffness of each floor also not lying on the same vertical axis. The approach used in the first two cases cannot tackle such problems. Therefore a simplified model needs to be developed for dynamic analysis of such irregular buildings. Figure 3.3(a) shows that centre of mass and centre of stiffness of all floors lies on different vertical axis.

The third case is the generalised case and the first two cases can be derived from this case. This is applicable to buildings with a setback and stepback configurations. Here, formulation for the transfer of storey stiffness and floor mass to a common arbitrary vertical reference axis is presented. An irregular building where the centres of mass of different floors lies on different vertical axes and the centres of stiffness of the floors also lying on different vertical axes is shown in Fig. 3.3. In this case the stiffness due to all the columns, walls and diagonal bracings is transferred from its position to the common vertical reference axis OZ as shown in Fig. 3.3. This axis can be chosen any where in the space. Similarly the floor masses of the structure are to be transferred to this vertical reference axis. By this simplification, the over all size of the problem gets reduced tremendously. All the members between any two adjacent floors are represented by one member itself located on the vertical reference axis.



a - Idealized multistorey stepback and setback building



b - Storey plan at kth and jth floor

Fig. 3.3 - Irregular Building Frame

3.4 Transformation of Forces and Displacements

3.4.1 Transformation of Forces

A multistoreyed framed building is shown in Fig. 3.4. Let $p_{i1}^{'}, p_{i2}^{'}, p_{i3}^{'}$ are the three forces acting at a point in a storey i at c.g. of the floor in x, y and z directions respectively which are orthogonal to each other. The effects of these forces on the common vertical reference axis OZ be p_{i1}, p_{i2}, p_{i3} in x,y and z directions respectively and is given by (3.1). This reference axis can be suitably chosen any where as shown in Fig. 3.4.

$$\begin{bmatrix} p_{i1} \\ p_{i2} \\ p_{i3} \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ -y & x & 1 \end{bmatrix} \begin{bmatrix} p_{i1}^{'} \\ p_{i2}^{'} \\ p_{i3}^{'} \end{bmatrix}, \tag{3.1}$$

$$\begin{bmatrix} \mathrm{p}_i \\ \mathrm{p}_j \end{bmatrix} = \begin{bmatrix} \mathrm{R} & 0 \\ 0 & \mathrm{R} \end{bmatrix} \begin{bmatrix} \mathrm{p}_i^{'} \\ \mathrm{p}_j^{'} \end{bmatrix} \qquad \text{or } \mathrm{p} = \mathrm{R_T p}^{'} \tag{3.2}$$

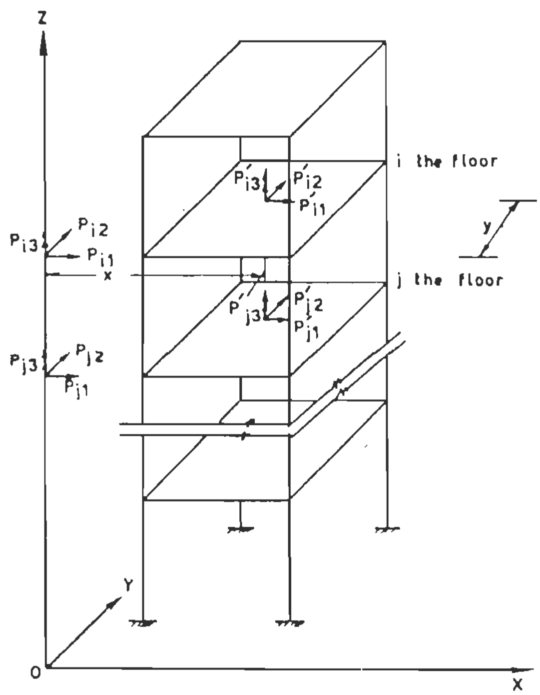


Fig. 3.4 - Reference axis OZ and Member i j between Floors i and j

$$\text{where } \mathbf{R} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ -y & x & 1 \end{bmatrix}$$

where $\mathbf{p}_i = [p_{i1}, p_{i2}, p_{i3}]$, $\mathbf{p}_j = [p_{j1}, p_{j2}, p_{j3}]$ and $\mathbf{p}_i' = [p_{i1}', p_{i2}', p_{i3}']$, $\mathbf{p}_j' = [p_{j1}', p_{j2}', p_{j3}']$ are the member end actions at vertical reference axis and storey axis respectively.

3.4.2 Transformation of Displacement

In the same manner, if u, v, θ are the three displacements of a storey i at reference axis, then u', v', θ' are the three corresponding displacements of that storey at storey axis and it is expressed as

$$\begin{bmatrix} u' \\ v' \\ \theta' \end{bmatrix} = \begin{bmatrix} 1 & 0 & -y \\ 0 & 1 & x \\ 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} u \\ v \\ \theta \end{bmatrix} \tag{3.3}$$

$$\begin{bmatrix} \mathbf{u}_i' \\ \mathbf{u}_j' \end{bmatrix} = \begin{bmatrix} \mathbf{R}^T & 0 \\ 0 & \mathbf{R}^T \end{bmatrix} \begin{bmatrix} \mathbf{u}_i \\ \mathbf{u}_j \end{bmatrix} \qquad \text{or } \mathbf{u}' = \mathbf{R}_T^T \mathbf{u} \tag{3.4}$$

where $\mathbf{u}_i = [u_{i1}, u_{i2}, u_{i3}]$, $\mathbf{u}_j = [u_{j1}, u_{j2}, u_{j3}]$ and $\mathbf{u}_i' = [u_{i1}', u_{i2}', u_{i3}']$, $\mathbf{u}_j' = [u_{j1}', u_{j2}', u_{j3}']$ are the member end displacement at reference and storey axes respectively.

3.5 Mathematical Model

3.5.1 Roof / Floor Slabs

The inplane deformation of a floor slab can be neglected as compared to the lateral deformation of the storey under lateral loads and therefore assumption of rigid floor diaphragm for an idealized multistorey building structure supported on massless axially inextensible columns, walls and bracing members can safely be made.

3.5.2 Column Members

A column contributing to storey stiffness is shown in Fig. 3.5. The stiffness matrix of a member is derived by giving unit displacement in x, y directions and unit rotation in z direction.

(i) Unit displacement in X-direction: - Let unit displacement be given in X direction to the c.g. of the floor of a storey. Keeping the base fixed then the column will be displaced as shown in Fig. 3.5(a). The force induced in the X direction and torsional moment at the vertical reference axis Z will be $12EI_y/(L^3(1+\phi_x))$ and $-12EI_y y/(L^3(1+\phi_x))$ respectively.

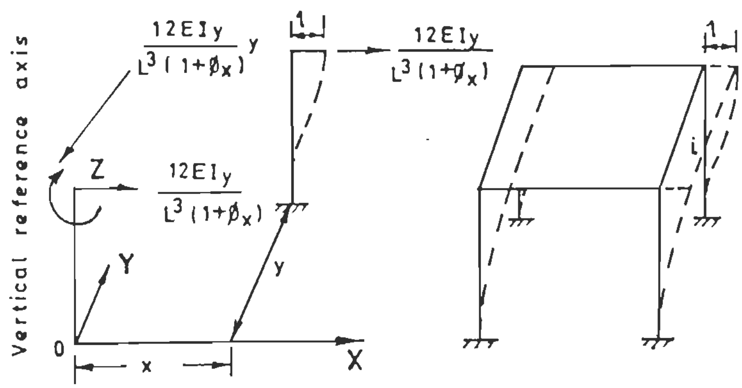
(ii) Unit displacement in Y- direction:- Let unit displacement be given in Y direction to the c.g. of the floor of a storey. Keeping the base fixed then the column will be displaced as shown in Fig. 3.5(b). The force induced in the Y direction and torsional moment at vertical reference axis Z will be $12EI_x/(L^3(1+\phi_y))$ and $12EI_x x/(L^3(1+\phi_y))$ respectively.

(iii) Unit rotation in Z- direction: - Let unit rotation be given in Z direction to the c.g. of the floor of storey. Keeping the base fixed then the column will be displaced as shown in Fig. 3.5(c). The force induced in X direction, Y direction and torsional moment in Z direction at vertical reference axis will be $-12EI_y y/(L^3(1+\phi_x))$, $12EI_x x/(L^3(1+\phi_y))$ and $GI_z/L+12EI_y y^2/(L^3(1+\phi_x)+12EI_x x^2/(L^3(1+\phi_y))$ respectively.

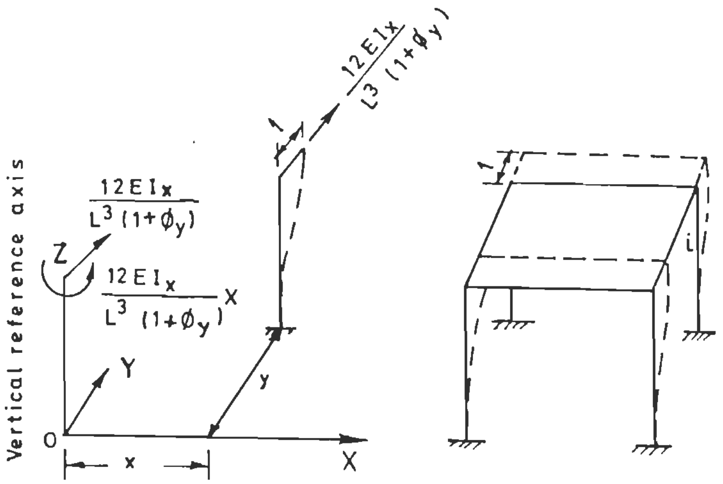
Therefore the stiffness matrix $\mathbf{K}_c^r$ due to a column member of the storey at the common vertical reference axis can be expressed as

$$\mathbf{K}_c^r = \mathbf{R}_T \mathbf{K}_c \mathbf{R}_T^T \tag{3.5}$$

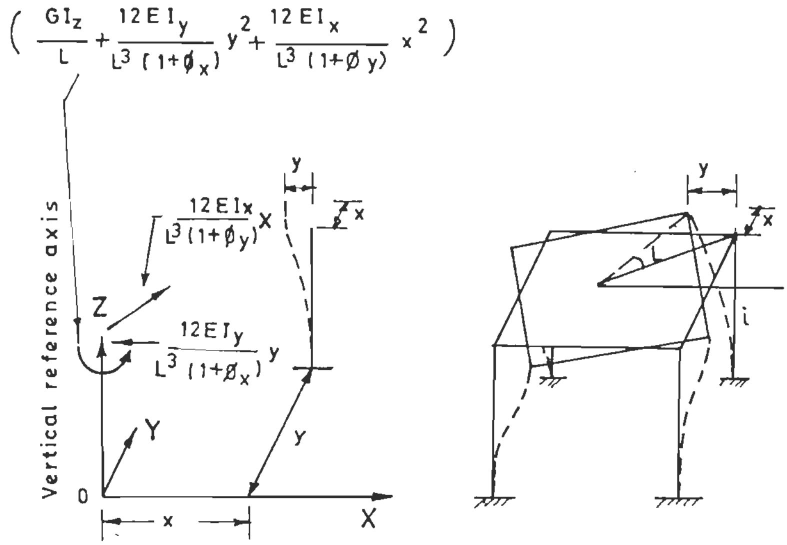
and $\mathbf{K}_c$ is stiffness matrix for column member at member axis and is taken from[Gere and Weaver(1986)] and $\mathbf{K}_c^r$ is expressed as



(a) Unit displacement in X-direction



(b) Unit displacement in Y-direction



(c) Unit rotation about Z-axis

Fig. 3.5 - Forces at Vertical Reference Axis

$$
\begin{bmatrix}
\frac{12EI_y}{L^3(1+\phi_x)} & 0 & -\frac{12EI_y}{L^3(1+\phi_x)}Y & -\frac{12EI_y}{L^3(1+\phi_x)} & 0 & \frac{12EI_y}{L^3(1+\phi_x)}Y \\
0 & \frac{12EI_x}{L^3(1+\phi_y)} & \frac{12EI_x}{L^3(1+\phi_y)}X & 0 & -\frac{12EI_x}{L^3(1+\phi_y)} & -\frac{12EI_x}{L^3(1+\phi_y)}X \\
-\frac{12EI_y}{L^3(1+\phi_x)}Y & \frac{12EI_x}{L^3(1+\phi_y)}X & (\frac{GI_z}{L}+\frac{12EI_y}{L^3(1+\phi_x)}Y^2+\frac{12EI_x}{L^3(1+\phi_y)}X^2) & \frac{12EI_y}{L^3(1+\phi_x)}Y & -\frac{12EI_x}{L^3(1+\phi_y)}X & -(\frac{GI_z}{L}+\frac{12EI_y}{L^3(1+\phi_x)}Y^2+\frac{12EI_x}{L^3(1+\phi_y)}X^2) \\
-\frac{12EI_y}{L^3(1+\phi_x)} & 0 & \frac{12EI_y}{L^3(1+\phi_x)}Y & \frac{12EI_y}{L^3(1+\phi_x)} & 0 & -\frac{12EI_y}{L^3(1+\phi_x)}Y \\
0 & -\frac{12EI_x}{L^3(1+\phi_y)} & -\frac{12EI_x}{L^3(1+\phi_y)}X & 0 & \frac{12EI_x}{L^3(1+\phi_y)} & \frac{12EI_x}{L^3(1+\phi_y)}X \\
\frac{12EI_y}{L^3(1+\phi_x)}Y & -\frac{12EI_x}{L^3(1+\phi_y)}X & -(\frac{GI_z}{L}+\frac{12EI_y}{L^3(1+\phi_x)}Y^2+\frac{12EI_x}{L^3(1+\phi_y)}X^2) & -\frac{12EI_y}{L^3(1+\phi_x)}Y & \frac{12EI_x}{L^3(1+\phi_y)}X & (\frac{GI_z}{L}+\frac{12EI_y}{L^3(1+\phi_x)}Y^2+\frac{12EI_x}{L^3(1+\phi_y)}X^2)
\end{bmatrix}
\tag{3.6}
$$

where E is the Young's modulus of elasticity, G is the shear modulus, I_x, I_y, I_z are the moment of inertia about X, Y and Z axes respectively, L is the length of the member, A is the x-sectional area of the member, ϕ_x and ϕ_y are the shear deformation parameters expressed as

$$
\phi_x = \left[\frac{24(1+\nu)I_y S_y}{AL^2}\right] \text{ and } \phi_y = \left[\frac{24(1+\nu)I_x S_x}{AL^2}\right] \tag{3.7}
$$

where ν is Poisson's ratio and S_x and S_y are the shape factors of the x-section of the member with respect to X and Y directions respectively.

3.5.3 Panel Elements and Diagonal Bracings

Inter storey walls and diagonal bracing members provide significant stiffness to whole structure. A wall member can be modelled as equivalent diagonal member taking its effective width. Diagonal members are idealized as strut elements. Stiffness matrix $\mathbf{K}_d^r$ due to a diagonal bracing at common vertical reference axis with 3 d.o.f. per floor can be derived in the same manner as has been derived for column members by giving unit displacements in X, Y directions and is expressed as

$$
\mathbf{K}_d^r = \mathbf{R}_T \mathbf{K}_d \mathbf{R}_T^T \tag{3.8}
$$

and K_d is the stiffness matrix due to strut element at member axis and is taken from[Gere and Weaver(1986)] and K'_d is expressed as per (3.9). Where $C_x = \cos\theta_x$, $C_y = \cos\theta_y$. The θ_x and θ_y are the angle of inclination of diagonal bracing with storey axes X and Y respectively.

The stiffness matrix of each floor due to vertical members and diagonal bracings can be worked out at the reference axis directly as explained above and these can be superimposed to get the overall stiffness matrix of the structure at the common vertical reference axis.

Formulation with 3 d.o.f. per floor will give two lateral forces and one torsional moment at each floor level and correspondingly in the members. The same model having rigid floor diaphragm has been extended as having 6 d.o.f. per floor. In the same manner stiffness matrices have been derived due to vertical members and diagonal bracings at a vertical reference axis.

$$\begin{bmatrix}
\frac{EAC_x^2}{L} & 0 & -\frac{EAC_x^2}{L}Y & -\frac{EAC_x^2}{L} & 0 & \frac{EAC_x^2}{L}Y \\
0 & \frac{EAC_y^2}{L} & \frac{EAC_y^2}{L}X & 0 & -\frac{EAC_y^2}{L} & -\frac{EAC_y^2}{L}X \\
-\frac{EAC_x^2}{L}Y & \frac{EAC_y^2}{L}X & (\frac{EAC_x^2}{L}Y^2+\frac{EAC_y^2}{L}X^2) & \frac{EAC_x^2}{L}Y & -\frac{EAC_y^2}{L}X & -(\frac{EAC_x^2}{L}Y^2+\frac{EAC_y^2}{L}X^2) \\
-\frac{EAC_x^2}{L} & 0 & \frac{EAC_x^2}{L}Y & \frac{EAC_x^2}{L} & 0 & -\frac{EAC_x^2}{L}Y \\
0 & -\frac{EAC_y^2}{L} & -\frac{EAC_y^2}{L}X & 0 & \frac{EAC_y^2}{L} & \frac{EAC_y^2}{L}X \\
\frac{EAC_x^2}{L}Y & -\frac{EAC_y^2}{L}X & -(\frac{EAC_x^2}{L}Y^2+\frac{EAC_y^2}{L}X^2) & -\frac{EAC_x^2}{L}Y & \frac{EAC_y^2}{L}X & (\frac{EAC_x^2}{L}Y^2+\frac{EAC_y^2}{L}X^2)
\end{bmatrix}$$

(3.9)

3.5.4 Stiffness Matrix with 6 d.o.f. per Floor

(i) Column and Diagonal Members

Stiffness matrices due to column and diagonal member of a storey have been derived at a vertical reference axis by considering 6 d.o.f. per floor and by giving unit displacements in X, Y, Z directions and unit rotations about X, Y, Z directions and is expressed as

$$\mathbf{K}_c^r = \mathbf{R}_{T6}\mathbf{K}_c\mathbf{R}_{T6}^T \qquad (3.10)$$

where $\mathbf{K}_c^r$ is the stiffness matrix at common vertical reference axis and $\mathbf{K}_c$ is the stiffness matrix at member level and $\mathbf{R}_{T6}$ is expressed as

$$\begin{bmatrix} \mathbf{R}_6 & 0 \\ 0 & \mathbf{R}_6 \end{bmatrix} \qquad (3.11)$$

and $\mathbf{R}_6$ is expressed as

$$\mathbf{R}_6 = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & y & 1 & 0 & 0 \\ 0 & 0 & -x & 0 & 1 & 0 \\ -y & x & 0 & 0 & 0 & 1 \end{bmatrix} \qquad (3.12)$$

3.6 Mass Matrix

The mass of each floor is lumped at the centre of mass of each floor. The mass in either X, Y and Z directions is equal to m and mI_X, mI_y, mI_Z are the mass moment of inertia of the floor about X, Y and Z axes respectively. Where the centre of mass of each floor does not lie on the same vertical axis of the structure, the masses and mass moment of inertia have to be transferred to the same vertical reference axis.

3.6.1 Transformation of Mass Matrix About Reference Axis

Transformation of mass matrix from the storey axis to a reference axis can be carried out with the transformation matrix R. Let $\mathbf{M}_f^r$ represents the mass matrix of a floor at a reference axis and $\mathbf{M}_f$ represents the mass matrix of the floor at the centre of mass. Then

$$\mathbf{M}_f^r = \mathbf{R}\mathbf{M}_f\mathbf{R}^T \tag{3.13}$$

The $\mathbf{M}_f^r$ works out to

$$\mathbf{M}_f^r = \begin{bmatrix} m & 0 & -my \\ 0 & m & mx \\ -my & mx & (mx^2 + my^2 + mI_z) \end{bmatrix} \tag{3.14}$$

where x and y are the distances between the vertical reference axis and the centre of mass of the floor in X and Y directions respectively. The mass matrix of the overall structure can be worked out about the same vertical reference axis using the above transformation. To take into account the accidental eccentricity the mass is to be displaced from centre of mass axis by a distance equal to accidental eccentricity as recommended in Code of Practices. The x, y distances will be the distances between the reference axis and the displaced centre of mass.

3.6.2 Mass Matrix with 6 d.o.f. per Floor

The mass matrix of the rigid floor with 6 d.o.f. per floor is worked out as

$$\mathbf{M}_f^r = \begin{bmatrix} m & 0 & 0 & 0 & 0 & -my \\ 0 & m & 0 & 0 & 0 & mx \\ 0 & 0 & m & my & -mx & 0 \\ 0 & 0 & m & (my^2 + mI_x) & -mxy & 0 \\ 0 & 0 & my & -mxy & (mx^2 + mI_y) & 0 \\ -my & mx & 0 & 0 & 0 & (my^2 + mx^2 + mI_z) \end{bmatrix} \tag{3.15}$$

The formulation with 3 d.o.f. per floor yields result of two lateral forces/displacements and one torsional moment/rotation, where as results with 6 d.o.f. per floor gives all the three forces/displacements and three moments/rotations of the rigid floor.

3.7 Mode Superposition Method

The equation of motion for the free vibration of building[Clough and Penzien(1975)] is written as

$$\mathbf{M\ddot{u}} + \mathbf{C\dot{u}} + \mathbf{Ku} = 0 \tag{3.16}$$

The equation of motion under earthquake excitation is given as

$$\mathbf{M\ddot{u}} + \mathbf{C\dot{u}} + \mathbf{Ku} = -\mathbf{Mi_x}\ddot{u}_{gx} - \mathbf{Mi_y}\ddot{u}_{gy} \tag{3.17}$$

where $\ddot{\mathbf{u}} = [\ddot{\mathbf{u}}_{1,}\ddot{\mathbf{u}}_{2,} ------------ \ddot{\mathbf{u}}_n]$

and $\ddot{\mathbf{u}}_1 = [\ddot{u}_{x,}\ddot{u}_{y,}\ddot{\theta}_z]$ for 3 d.o.f. per floor

and $\ddot{\mathbf{u}}_1 = [\ddot{u}_{x,}\ddot{u}_{y,}\ddot{u}_{z,}\ddot{\theta}_{x,}\ddot{\theta}_{y,}\ddot{\theta}_z]$ for 6 d.o.f. per floor

M, **C** and **K** are the overall mass, damping and stiffness matrices about a common vertical reference axis, and

$\mathbf{i_x} = [1,0,0,1,0,0\ldots\ldots\ldots\ldots]^T$, $\mathbf{i_y} = [0,1,0,0,1,0\ldots\ldots\ldots\ldots]^T$,

$\ddot{u}_{gx}$ and $\ddot{u}_{gy}$ are the ground acceleration in X and Y directions respectively.

The eigen value problem is solved to get the frequencies of vibration and mode shapes at the reference axis. The mode shape coefficients are to be transferred to the centre of mass axis with the RT matrix described earlier and then the mode participation factors are to be evaluated in the usual manner at the centre of mass axis.

3.7.1 Storey Forces and Displacements

The dynamic response of a structure under earthquake excitation is dependent upon the natural time periods, mode shapes, damping characteristics and wave form of the accelerogram. The maximum displacement response and the forces are worked out using the response spectrum method. For combining the response of various modes CQC [Wilson, Kiureghian and Bayo(1981)] method has been adopted. Newmark's implicit predictor corrector scheme[Hinton and Owen(1980)] is used for solving the forced vibration equation of motion.

3.7.2 Member Forces and Displacements

The displacements at all the column member ends can be determined from the storey displacements using the transformation matrix as described earlier. The seismic forces in a member m are obtained as

$$F_m = K_m \cdot z_m \tag{3.18}$$

where $\mathbf{F_m}$ is the member forces, $\mathbf{K_m}$ is the member stiffness matrix at local axis, $\mathbf{z_m}$ are the member displacements.

3.8 Validation of the Computer Program Developed

A computer program 'SAIRFB' (Seismic Analysis of Irregular idealised Rigid Floor Buildings) has been developed based on this algorithm with 3 d.o.f. per floor and 6 d.o.f. per floor to find out the natural time periods, mode shapes, storey displacements, storey forces, member forces, member displacements and time history response of irregular buildings supported on vertical columns and diagonal bracings.

A Few numerical examples have been solved with the developed program to check the validity of the program.

3.8.1 Numerical Example 1.

A multistoreyed frame structure located on hill slope[Paul(1993)] is analysed. See Fig. 3.6. Following four different locations of the vertical reference axis are taken to show that the results of analysis are independent of its location.

(i) Along the centre of mass of roof.

(ii) Along the centre of mass of second floor.

(iii) Along the centre of mass of first floor.

(iv) Along the centre of mass of the whole structure which is located at 900 mm from the centre of mass of roof along x-axis.

Given E=25491 N/mm2, γ= 2.4t/m2, L=3500 mm, Mass of 3.5m x 4.0m slab = 5.14 N-sec2/mm, Size of columns 350mm x 250mm

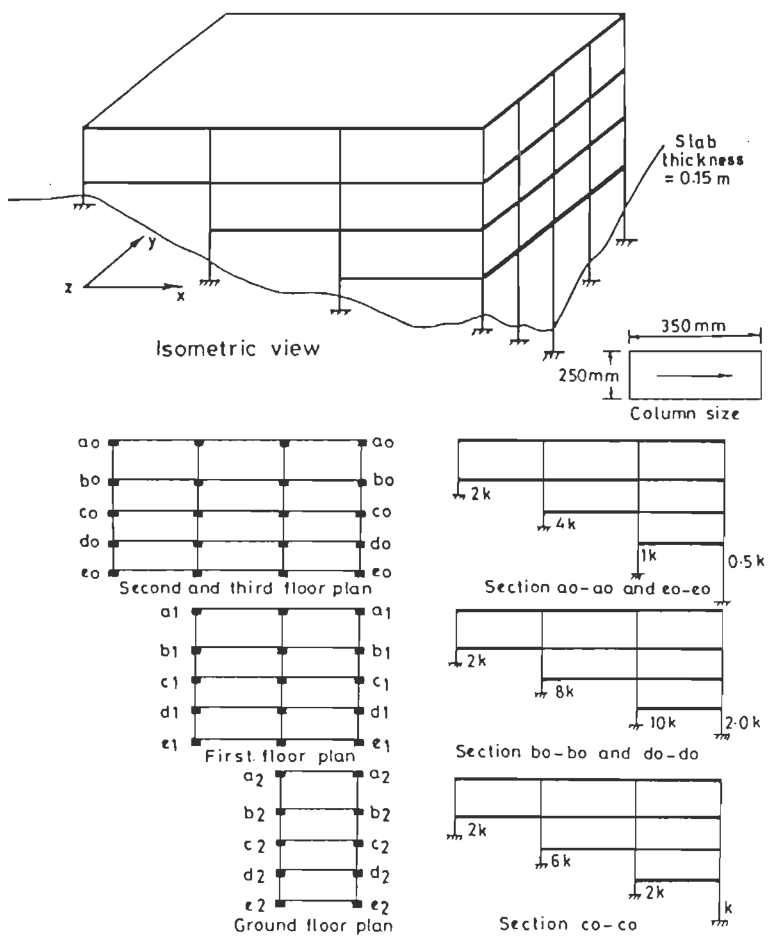


Fig. 3.6 - Multistorey R.C. Building on Hill Slope

The problem has been analysed for time periods of vibration. The results are shown in Table 3.1. It shows that these are exactly same implying that the vertical reference axis can be suitably chosen anywhere.

(i) Along the centre of mass of roof,(ii) Along the centre of mass of second floor, (iii) Along the centre of mass of first floor, (iv) Along the centre of mass of the whole structure.

Table 3.1. - Natural Time Periods of Vibration

MODE	Time periods with reference axis at			
	i	ii	iii	iv
1	0.33722	0.33722	0.33722	0.33722
2	0.22838	0.22838	0.22838	0.22838
3	0.20466	0.20466	0.20466	0.20466
4	0.13019	0.13019	0.13019	0.13019
5	0.10328	0.10328	0.10328	0.10328
6	0.09204	0.09204	0.09204	0.09204
7	0.07893	0.07893	0.07893	0.07893
8	0.07828	0.07828	0.07828	0.07828
9	0.07092	0.07092	0.07092	0.07092
10	0.06284	0.06284	0.06284	0.06284
11	0.05643	0.05643	0.05643	0.05643
12	0.05091	0.05091	0.05091	0.05091

3.8.2 Numerical Example 2

A space frame shown in Fig.3.7 has been chosen to check the validity of the time integration and the present formulation. This example is taken from [Corderoy and Thambiratnam (1993)]. All the columns are of symmetric x-sections having second moments of area $I_x = I_y = 42 \times 10^6$ mm^4 about their centroidal axes parallel to X and Y axes respectively. The first and second floor slabs are assumed to have a lumped weight of 5 kPa, while the roof slab is assumed to have a lumped weight of 2.5kPa. Young's modulus of elasticity and Poisson's ratio for the steel buildings are taken to be 200 GPa and .30 respectively. The ground acceleration is taken in the form $\ddot{u}_g = A \sin 2\pi t/T$ where $A = 0.3g$ and $T = 2$.

Figure. 3.8 shows the displacement time history at levels 1 and 3 as per the present study which overlaps with the displacement time history as reported by [Corderoy and Thambiratnam(1993)]. Thus validates the program for its formulation and time history analysis.

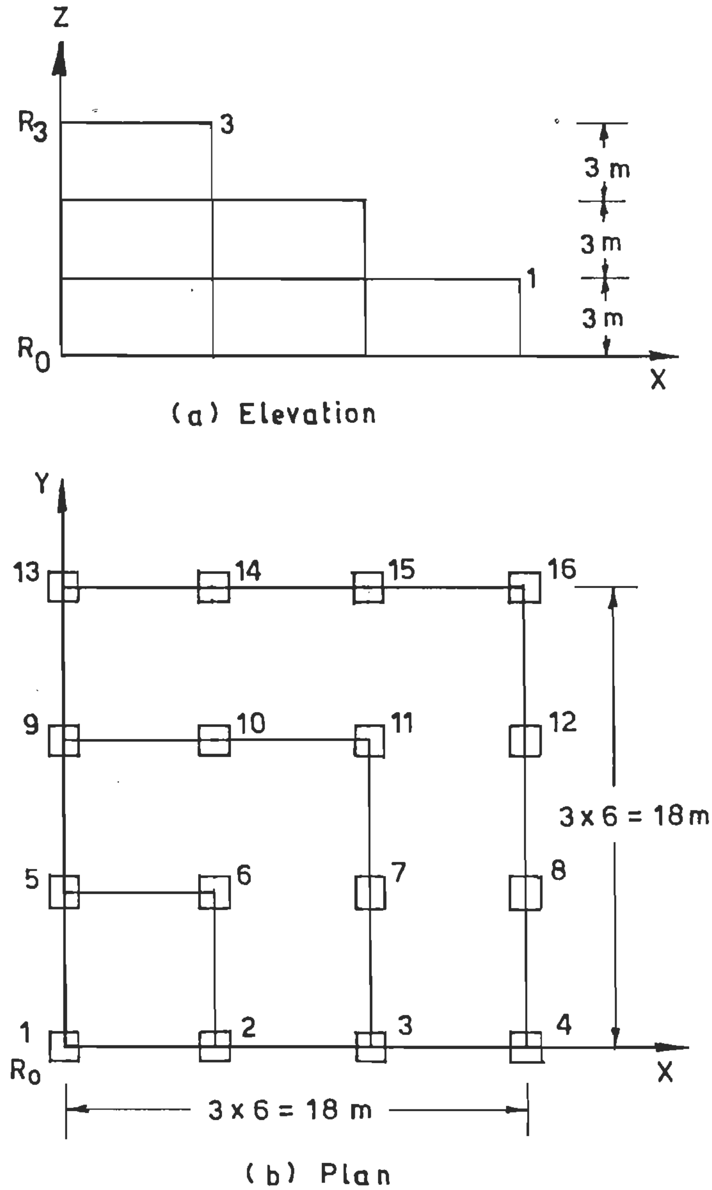


Fig. 3.7 - A Space Frame

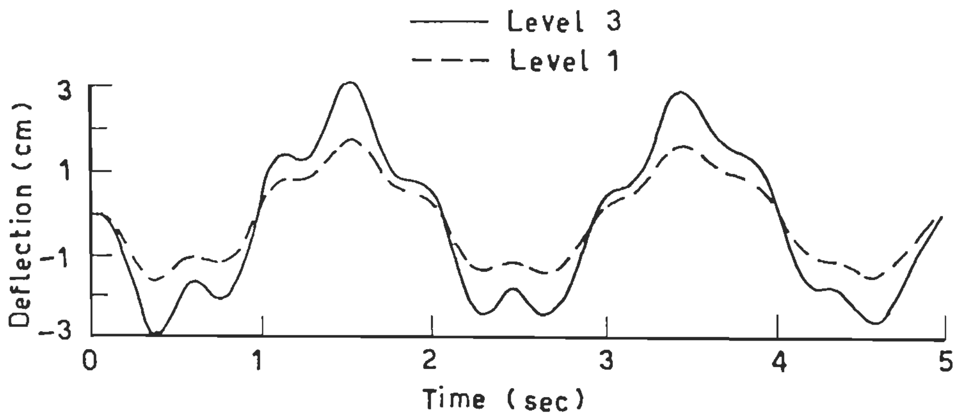


Fig. 3.8 - Displacement Time History at level 1 and 3(Present study and Corderoy and Thambiratnam(1993)

3.8.3 Numerical Example 3

Figure 3.9 shows the two storey r.c. symmetrical and unsymmetrical setback frame. Size of column is 500mmx500mm. Given E = 25491 N/mm^2, ν =0.15.

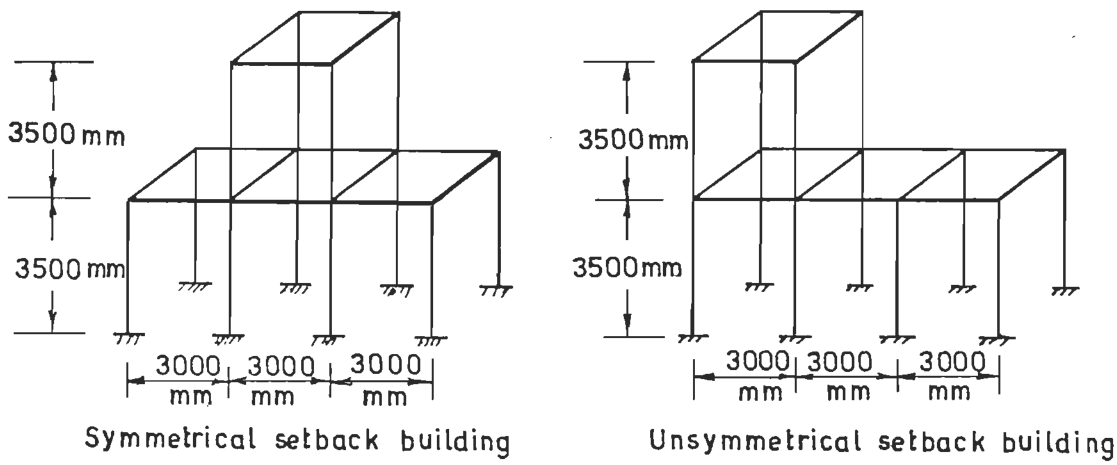


Fig. 3.9 Two Storey r.c. Setback Frame Building

The time periods of vibration and mode shapes for the above cases are shown in Figs. 3.10 and 3.11. It shows that the symmetrical setback frame is subjected to pure translation or torsional motion whereas the unsymmetrical setback frame is subjected to coupled translational and torsional motions.

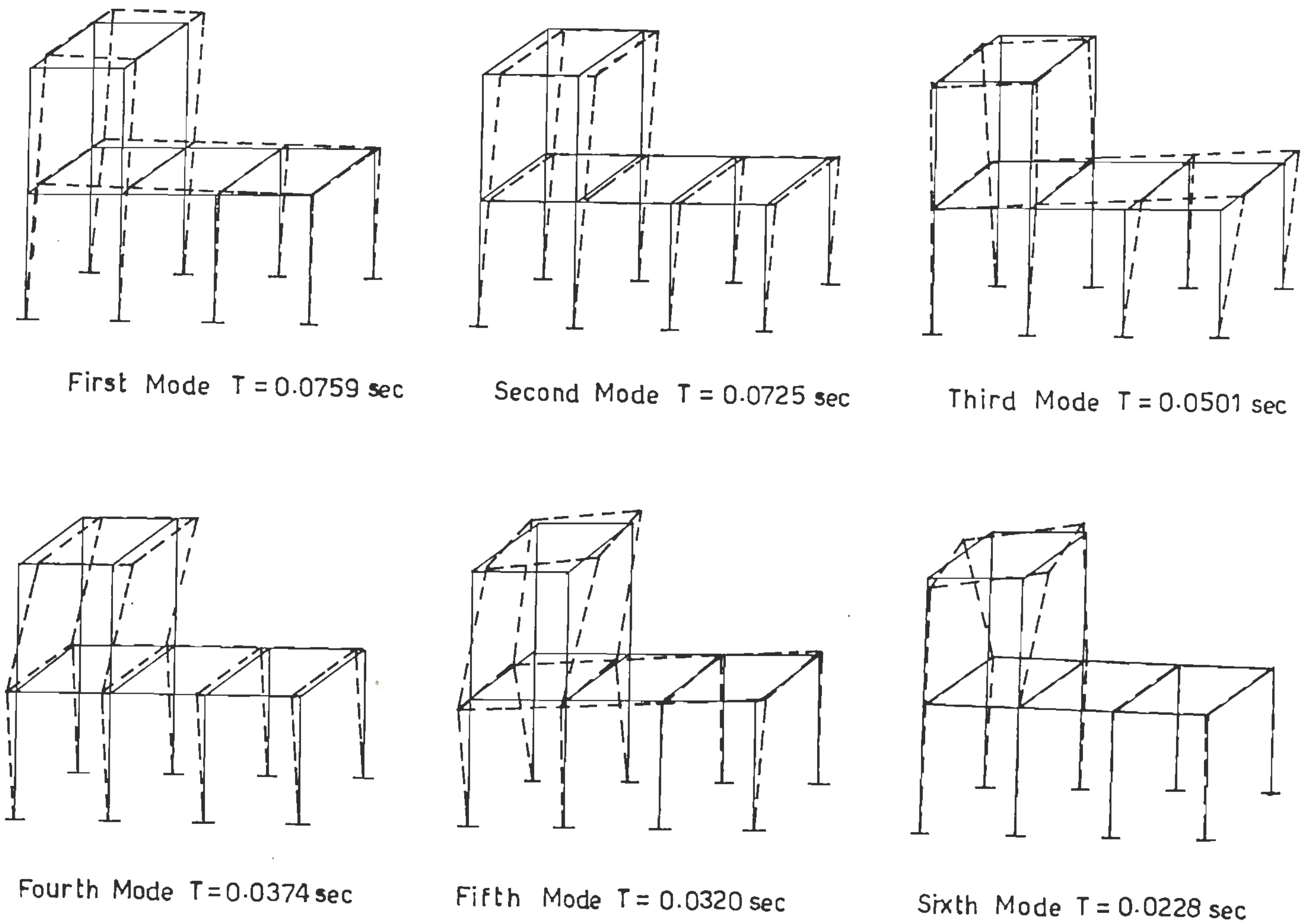


Fig. 3.10 - Mode Shapes of two Storey r.c. Unsymmetrical Setback Building

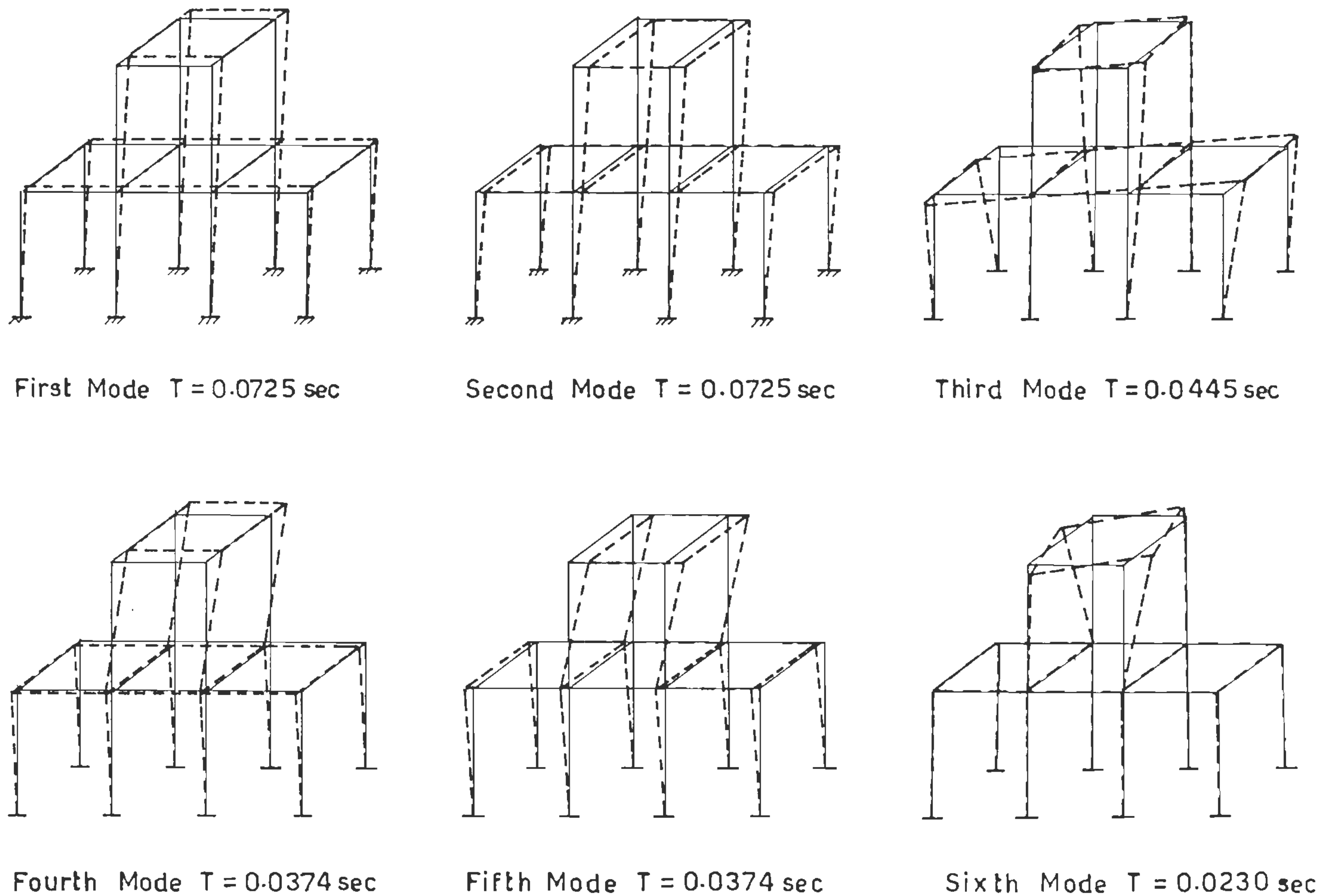


Fig. 3.11 - Mode Shapes of two Storey r.c. Symmetrical Setback Building

3.9 Dynamic Analysis of Stepback and Setback Buildings

Two problems of hill building frames are analysed for the seismic response using 6 d.o.f per floor, 3 d.o.f per floor, I.S. Code method and with 3 d.o.f. per floor considering accidental eccentricity as per UBC and 6 d.o.f. per node analyses procedures with rigid floor diaphragm. I.S. Code 1893(1984) recommends modal analysis with one d.o.f. per floor and says that torsional shears are to be evaluated separately by taking design eccentricity as 1.5 times the distance between centre of mass and centre of stiffness at various floors. Negative torsional shears are to be neglected.

Modulus of elasticity has been taken as 25491N/mm^2 and 5% damping has been assumed. I.S. Code(1893-1984) spectra has been used to find out the displacements and forces. Results of time periods, storey shears, displacements and maximum column shears obtained from all the methods are compared, discussed and therefrom conclusions drawn.

3.9.1 Numerical Example 1

Figure 3.12 shows a stepback and setback building. All column sizes are 350mmX350mm. The plan dimensions are indicated in the figure itself. The mass has been worked out for each floor with load intensity of 7 kN/m^2. The natural time periods of vibrations of the building obtained by various methods are presented in Table 3.2. Storey shears, floor displacements and maximum column shears obtained using different methods are presented in Fig 3.13.

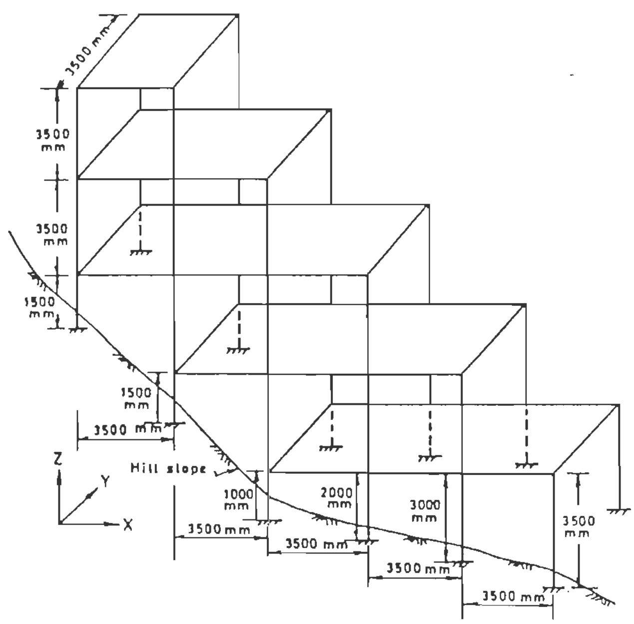


Fig. 3.12 - Stepback and Setback Building Frame

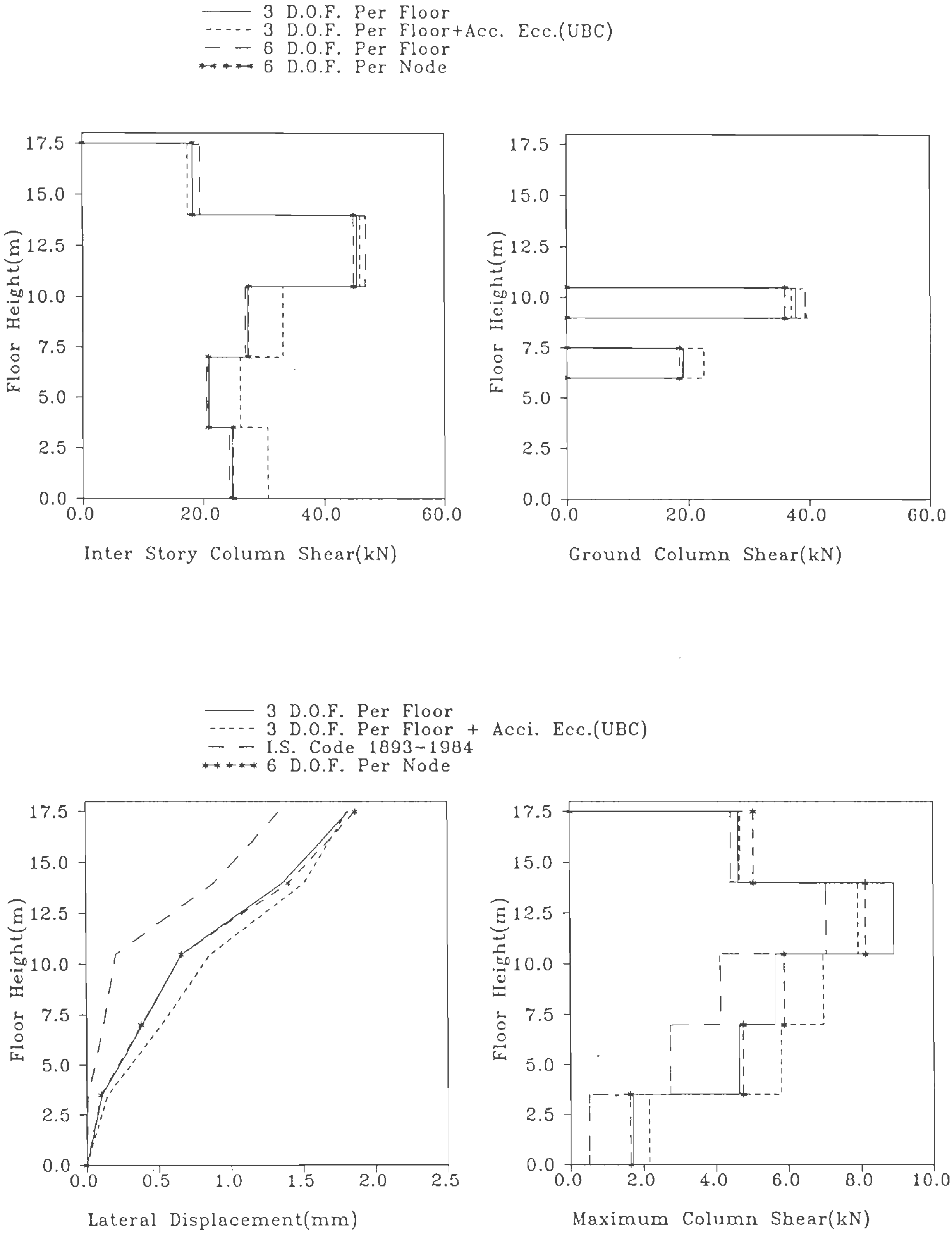


Fig. 3.13 - Comparison of Column Shears/Displacements

The time periods of vibration obtained by I.S. Code method are less as compared to the results of other methods. The time periods of vibration obtained by 3 d.o.f./floor idealisation are comparable to the 6 d.o.f./node idealisation.

Table 3.2 - Natural Time Periods(Secs)

Mode	I.S. Code Method	3 d.o.f./floor	3d.o.f./floor + accidental ecc.	6 d.o.f./floor	6 d.o.f/node with rigid floor
1	0.1719	0.1858	0.1905	0.1929	0.1898
2	0.1719	0.1719	0.1719	0.1737	0.1672
3	---------	0.1400	0.1425	0.1413	0.1639

Results of inter storey column shears and ground column shears obtained by 3 d.o.f./floor and 6 d.o.f./floor and 6 d.o.f. per node with rigid floor diaphragm are very much comparable. However the 6 d.o.f. per floor method can be used for seismic analysis of irregular tall buildings with rigid floor diaphragms under vertical excitations and to get the better results henceforth the analysis is based on 3 d.o.f./floor. The time periods of vibrations obtained considering accidental eccentricity as per UBC are higher as compared to periods obtained without considering accidental eccentricity. The difference varies up to 25%. Results of floor displacements and maximum column shears obtained by I.S. Code method are much lower as compared to 3 d.o.f. per floor method. The under estimation varies up to 70% for the maximum column shear.

3.9.2 Numerical Example 2

Figure 3.14 shows the stepback building frame. All column sizes are 350mmX350mm and plan dimensions are indicated in the figure itself. The mass has been worked out with load intensity of 7 kN/m^2. Results of time periods are presented in Table 3.3. Results of storey shears, displacements and maximum column shears are plotted in Fig. 3.15

Results of inter storey column shears indicate higher values as obtained by considering the accidental eccentricity as per UBC as compared to results obtained without considering the accidental eccentricity. The maximum difference is to the tune of 8.5%. The results of 3 d.o.f./floor are very much comparable to the results of 6 d.o.f./node. The floor displacements and maximum column shears obtained by I.S. Code method are much lower as compared to the results of 3 d.o.f./floor method. The underestimation is to the tune of 42% for maximum floor displacements and 14% to 68% for maximum column shears at various floor levels.

.Table 3.3 Natural Time Periods(Secs)

Mode	I.S. Code Method	3 d.o.f. per floor	3d.o.f./floor +acc. ecc. (UBC)	6 d.o.f./node with rigid floor
1	0.2418	0.3564	0.3814	0.3677
2	0.2418	0.2418	0.2418	0.2342
3	--------	0.1929	0.1797	0.2048

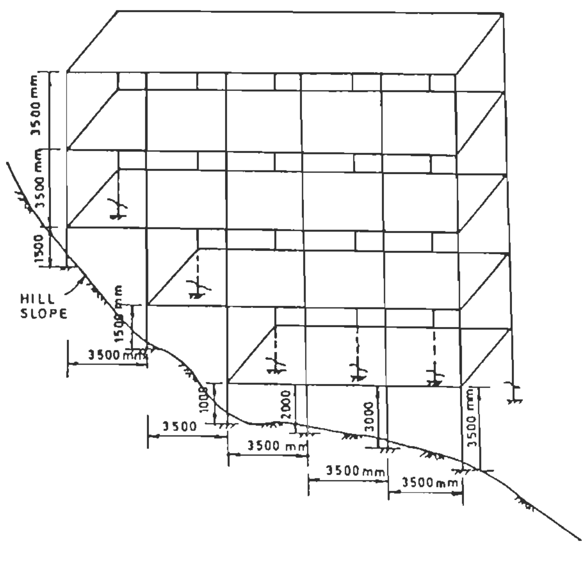


Fig. 3.14 - Stepback Building Frame

3.9.3 Time History Response of Two Examples

Time history response of storey displacements of various floors of two buildings under transverse component of Uttarkashi earthquake has been studied and the results are shown in Fig. 3.16 which shows the floor displacements in stepback building are much higher than the stepback and setback building configurations at all floor levels. Although direct comparison can not be made as the stiffness and mass distribution of the two

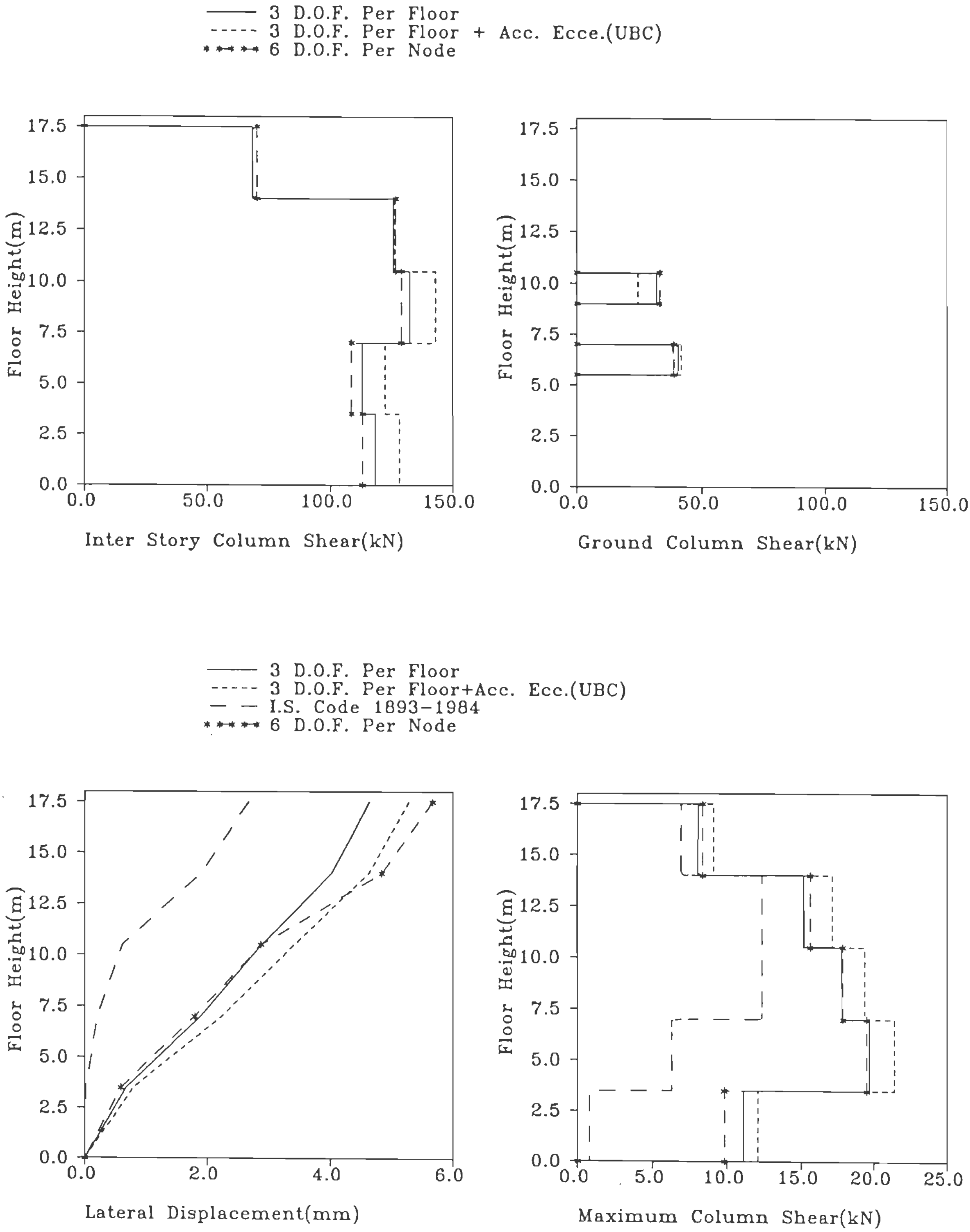


Fig. 3.15 - Comparison of Column Shears/Displacements

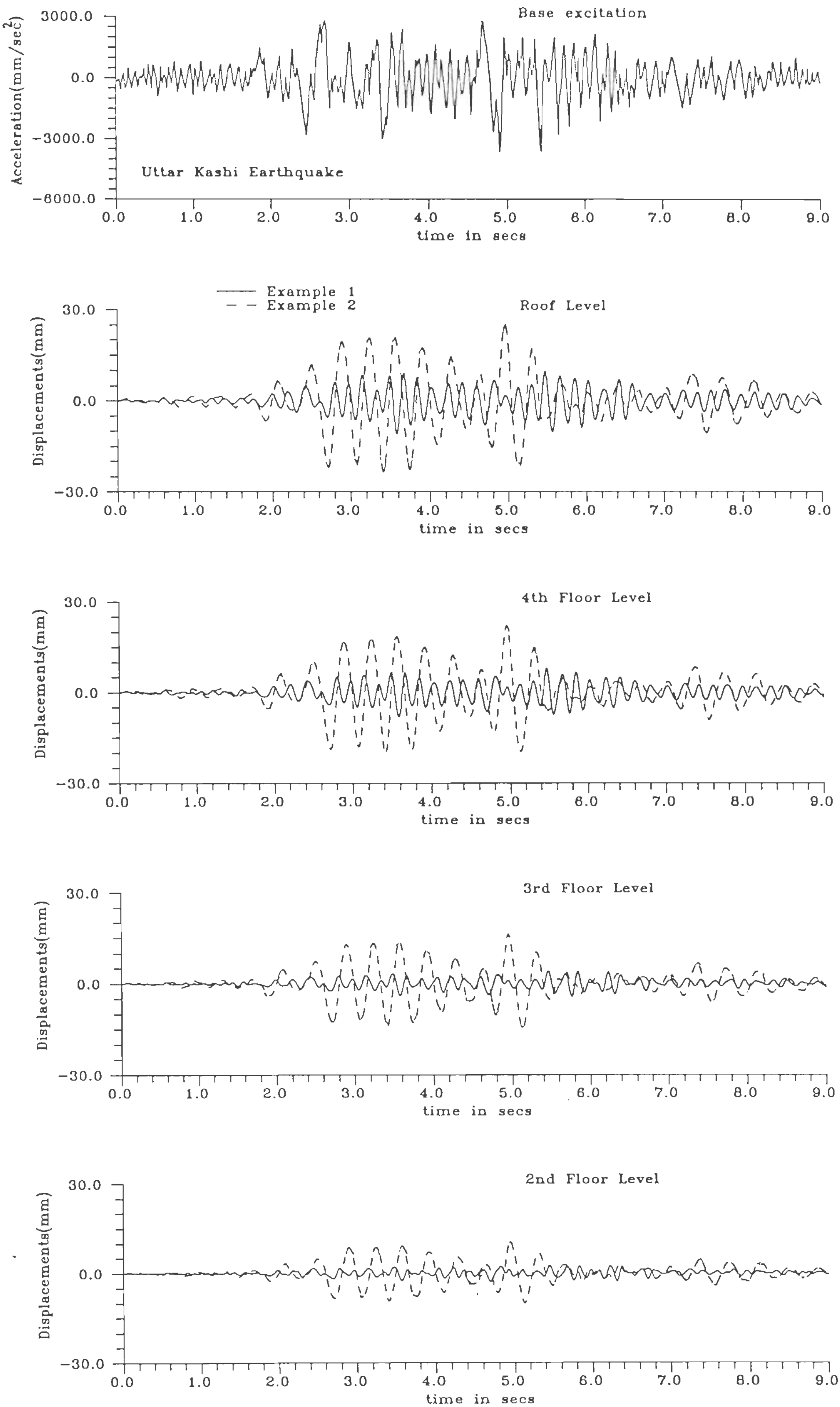


Fig. 3.16 - Comparison of Time History Displacement

buildings are different but it give qualitative idea about the response of the two cases. Uttarkashi earthquake acceleration input motion is shown in Appendix C.

The stepback buildings are more prone to torsion and therefore are more vulnerable to earthquakes than the stepback and setback buildings.

3.10 Concluding Remarks

This dynamic analysis procedure can be used for irregular buildings with rigid floor diaphragms having any configurations and supported on vertical columns, walls and diagonal bracing, especially for buildings where floors are supported on two types of columns (i) inter storey columns (ii) columns resting on the ground directly. This method takes care of any amount of asymmetry in mass and stiffness distributions. There is no need of locating centre of stiffness of various floors as it is automatically taken care of in the program by simply specifying the x and y distances of the members from vertical reference axis. The data preparation is very easy as compared to the conventional 6 d.o.f. per node analysis. The accidental eccentricity can be easily accounted for in this method. This method requires least efforts as compared to conventional 3D with 6 d.o.f. per node analysis.

With the use of this method the configurations for buildings on hill slopes can be decided by taking various trial configurations so as to get the most economical and safe design from seismic point of view. The buildings on hill slope having stepback and setback configurations are found to be better from seismic considerations rather than only stepback configurations.

The I.S.Code(1893-1984) under estimates the column shears and deflections for such irregular buildings. The closeness of the seismic response obtained by the simplified method with 6 d.o.f. per node method advocates the adoption of this simplified method in Code of Practices of different countries for dynamic analysis of highly irregular and asymmetrical buildings.

3.11 References

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BUILDING MODELLING WITH FLEXIBLE FLOOR SYSTEM

4.1 Introduction

In preliminary analysis and design of r.c.c. frame buildings, the floor system can be assumed to be rigid under the action of lateral loads. The previous Chapter describes the modelling detail of building with rigid floor/roof system. But under the action of gravity and live loads the floor system can not be taken as rigid and for detail and final analysis modelling of r.c.c. .frame building with flexible floor system would be very much appropriate. The Code of practices recommend earthquake resistant design of buildings based on weak beam/strong column philosophy. Therefore the consideration of flexibility of floor becomes important. This Chapter addresses to this problem. Irregular buildings are torsionally coupled. Three dimensional model is necessary to analyse the torsionally coupled buildings. In general the building structure is supported on columns, beams with floor slabs and the infill panels. Infill panels may be of r.c.c. or brick masonry. To get the true behaviour of the building under the action of various kinds of loading, all types of elements in the building are to be modelled carefully for its true behaviour. The mathematical model used in this study consists of r.c.c frame elements, r.c.c. plate elements, r.c.c. panel elements and brick masonry infill elements and interface elements. The linear and nonlinear modelling characterstics of these elements are described in this Chapter. The modelling of the elements and computer program developed have been validated by comparing the results of the present study with the available results in the literature for a few building frames under the action of gravity, live and earthquake loads.

4.2 Mathematical Model for Frame Elements

4.2.1 Analytical Model for 3D R/C Frame Elements

Analytical model for 3D r.c. frame elements has been idealised as shown in Fig. 4.1. It consists of three different zones, The first zone is the rigid end block located at each end of the member. The second zone is the inelastic hinge zone at each end of the member and the third is the elastic zone in between the inelastic zones. The following assumptions have been made for an inelastic element for a r.c. section.

1. The generalized force deformation relation of the element follows an elasto-plastic model, having yield strength corresponding to the ultimate capacity of the member.
2. The interaction among P_u, M_{uy}, M_{uz}, M_{ux}, V_{uy}, V_{uz} represents the generalised yield surface for 3D analysis.
3. The element retains linear elastic behaviour between nodal points, however elasto-plastic behaviour is permitted to occur over the cross section at each end for a distance dx approaching zero i.e. the inelastic behaviour is fully concentrated at the plastic hinge.

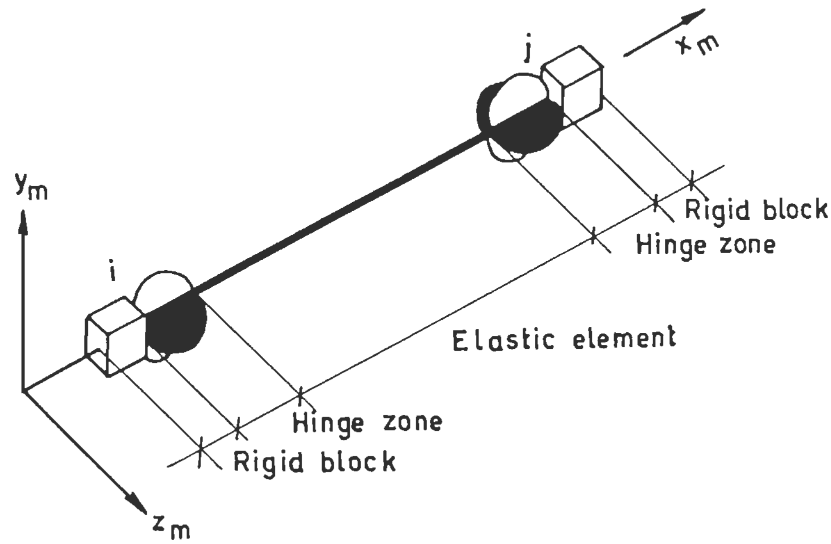


Fig.4.1 Analytical Model for 3D R/C Frame Element

4.2.1.1 Stiffness Matrix of a Frame Element

The 3D frame element ij with two unequal rigid ends of lengths a and b in space is shown in Fig. 4.2. At each end of the member there will be six displacements and there by six force components correspondingly it will yield a (12x12) stiffness matrix. The end displacements and the forces are defined as below.

$$[u_i] = [u_1, u_2, u_3, u_4, u_5, u_6]^T \tag{4.1}$$

$$[u_j] = [u_7, u_8, u_9, u_{10}, u_{11}, u_{12}]^T \tag{4.2}$$

$$[p_i] = [p1, p2, p3, p4, p5, p6]^T \tag{4.3}$$

$$[p_j] = [p7, p8, p9, p10, p11, p12]^T \tag{4.4}$$

The elastic stiffness matrix **K** of a 3D element in local coordinate system is as per Thanoon(1993). The global stiffness matrix is obtained from the local stiffness matrix through the following transformation.

$$K_e = T^T K T \tag{4.5}$$

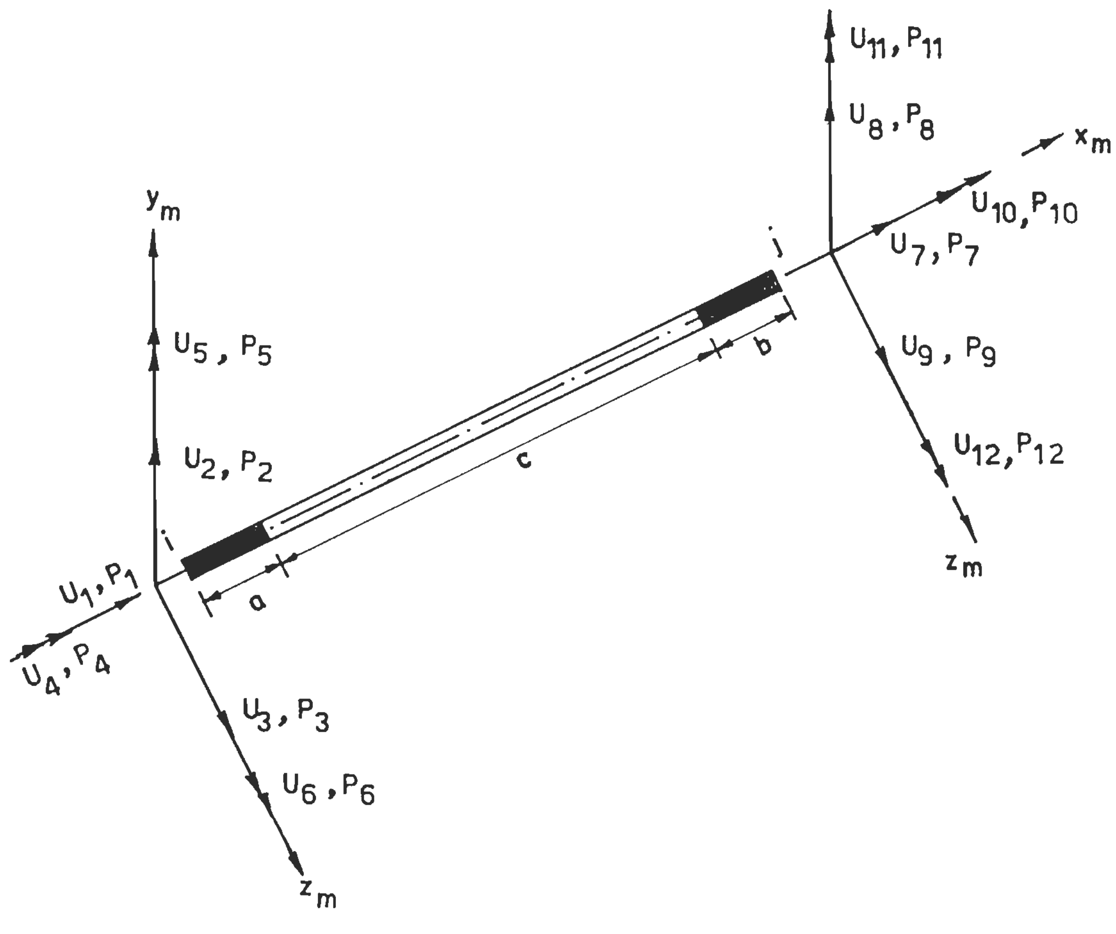


Fig. 4.2 -3D Frame Element

where K_e is the stiffness matrix of a member in global coordinate system and transformation matrix T(12x12) is defined as

$$T = \begin{bmatrix} R & 0 & 0 & 0 \\ 0 & R & 0 & 0 \\ 0 & 0 & R & 0 \\ 0 & 0 & 0 & R \end{bmatrix} \tag{4.6}$$

The sub matrix **R** (3x3) is the rotation matrix which can be obtained by successive planar rotations from reference axes(x, y, z) to the member axes(x_m, y_m, z_m). Two types of transformations are possible i.e. y-z-x transformation and z-y-x transformation [Beaufait *et al.*(1970)]. In y-z-x transformation the reference axes are rotated through angles β, γ and α succesively as shown in Fig. 4.3(a) for the reference axes to coincide with the member axes. The sub matrix **R** is defined as

$$R = \begin{bmatrix} C_x & C_y & C_z \\ \dfrac{-C_xC_yCos\alpha - C_zSin\alpha}{C_{xz}} & C_{xz}Cos\alpha & \dfrac{-C_yC_zCos\alpha + C_xSin\alpha}{C_{xz}} \\ \dfrac{C_xC_ySin\alpha - C_zCos\alpha}{C_{xz}} & -C_{xz}Sin\alpha & \dfrac{C_yC_zSin\alpha + C_xCos\alpha}{C_{xz}} \end{bmatrix} \tag{4.7}$$

$C_x = (x_j - x_i)/L$, $C_y = (y_j - y_i)/L$, and $C_z = (z_j - z_i)/L$ and $C_{xz} = (C_x^2 + C_y^2)^{1/2}$

The angle α is the angle between y_β and y_m axis or between the z_β and z_m axis measured in a counter clockwise direction when viewing the cross section of the member in negative x_m direction.

The z-y-x transformation is shown in Fig. 4.3(b) and the matrix **R** is obtained in its general form as

$$R = \begin{bmatrix} C_x & C_y & C_z \\ \dfrac{-C_xC_zSin\mu - C_yCos\mu}{C_{xy}} & \dfrac{-C_yC_zSin\mu + C_xCos\mu}{C_{xy}} & C_{xy}Sin\mu \\ \dfrac{-C_xC_zCos\mu + C_ySin\mu}{C_{xy}} & \dfrac{-C_yC_zCos\mu - C_xSin\mu}{C_{xy}} & C_{xy}Cos\mu \end{bmatrix} \tag{4.8}$$

The rotation matrix has been obtained by successive rotation of reference axes x, y, z by angle β_1, θ and μ . The $C_{xy} = (C_x^2 + C_y^2)^{1/2}$ and the angle μ required are the angle between y_θ and y_m axes or between the z_θ and z_m axes measured in a counter clockwise direction when viewing the cross section of the member in a negative x_m direction as shown in Fig. 4.3(b).

If the longitudinal axis x_m of the member corresponds to y direction of the reference coordinate system then the rotation matrix for y-z-x transformation is undefinable due to the fact that C_x and C_z becomes zero. Then rotation matrix defined by z-y-x transformation is to used. Similarly for a member whose longitudinal axis x_m corresponds to the z direction the rotation matrix through a z-y-x transformation becomes undefinable and y-z-x transformation to be used in such cases. The three direction cosines C_x, C_y, C_z defines the location of the longitudinal x_m axis and α or μ angle defines the location of the minor principle axis.

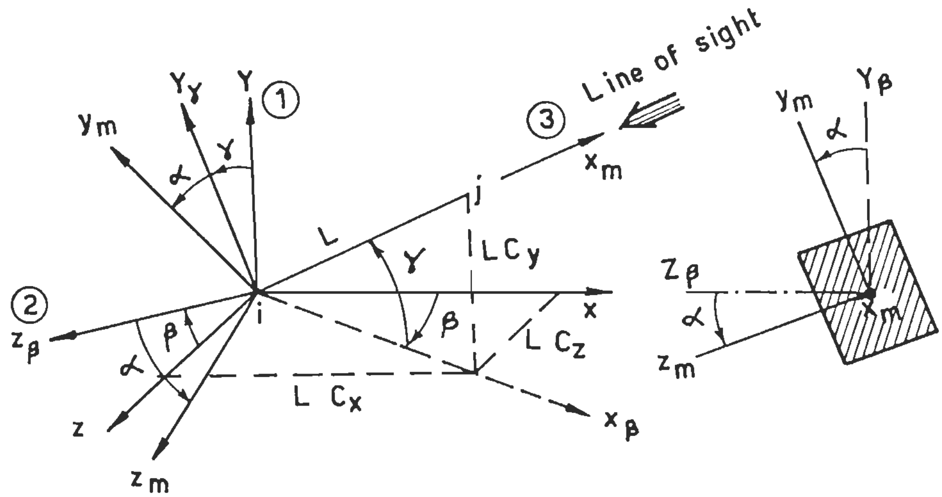


Fig. 4.3 (a) y-z-x Transformation

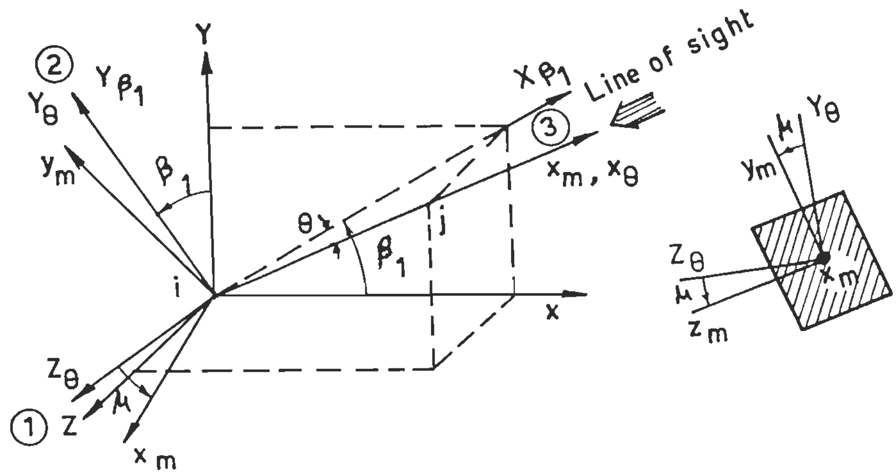


Fig. 4.3(b) z-y-x Transformation

4.2.2 Yield Surface for 3D Frame Elements

Stepback and setback buildings are highly irregular and asymmetric and therefore are subjected to severe torsional moment and lateral shears under the action of earthquakes. 3D frame elements are subjected to six components of forces(i.e. axial force, shears and moments) along the three axes. . The yielding of the r.c. frame member section takes place under the combined action of these six components. It necessitates the inclusion of torsional moment, lateral shears in the yield criteria for 3D r.c. frame elements. Determination of P_u-M_{uy}-M_{uz} surface of each reinforced concrete section of frame members require large computational effort and time because of trial and adjustment procedures used to find the inclination and depth of neutral axis. More over this approach requires storing of a large number of points(P_u - M_{uy} - M_{uz}) to define the interaction surface. In practice it is impractical to generate the yield surfaces in this manner. Therefore for analysis purposes, approximate yield surfaces have been defined, which depend only on few parameters. Various yield surfaces are available in the literature for 3D elements considering some components of actions acting on the elements. Axial force biaxial moment interaction[Hsu(1988)], biaxial moment interaction[Bresler(1960)], torsion, bending axial force interaction[Lampert and Collins(1972)], bending moment and torsional moment interaction[Chen and Powell(1982), Thanoon(1993)]. In the present study an effort has been made to develop an yield criteria for 3D r.c. frame elements considering interaction of axial force, bending moments, torsional moment and shears.

In the absence of sufficient experimental data concerning the interaction of the axial force with moments for a reinforced concrete section, an interaction surface developed by [Thanoon(1993)] has been used in the present study. The following assumptions has been made in developing the yield surface.

(i) The reinforced concrete section is symmetrical about its major and minor axes.

(ii) The interaction of all moments including the torsion is elliptical.

(iii) The interaction of M_{uy} and M_{uz} with the axial force is obtained by separate 2D analysis as shown in Fig. 4.4(a) and (b)[Thanoon(1993)].

(iv) The interaction of V_{uy} and V_{uz} with the torsional moments is obtained separately as shown in Fig. 4.5[Hsu(1988)].

The following equations are used in simulating the interaction of moments, torsion, axial forces and shear forces.

$$(m_y/m_{yp})^2 + (m_z/m_{zp})^2 + (T_x/T_{xv})^2 = 1.0 \quad (4.9)$$

where $m_{zp}/m_{zo} = a_1 + a_2(P_x/P_o) + a_3(P_x/P_o)^2 + a_4(P_x/P_o)^3$ (4.10)

$$m_{yp}/m_{yo} = b_1 + b_2(P_x/P_o) + b_3(P_x/P_o)^2 + b_4(P_x/P_o)^3 \quad (4.11)$$

$$(T_{xv}/T_{xo})^2 + (V_z/V_{zo})^2 + (V_y/V_{yo})^2 = 1.0 \quad (4.12)$$

Here m_{zo} and m_{yo} are the dimensionless ultimate bending moment capacities of the section about z and y axes respectively when the axial force is zero. T_{xo} is the dimensionless ultimate torsional moment capacity of the section about x - axis when the shear forces V_z and V_y are zero. m_{yp} and m_{zp} are the dimensionless values of the ultimate flexural strengths about z and y axes respectively for a fixed value of axial load P_u. T_{xv} is the dimensionless values of the ultimate torsional strength under the action of fixed value of V_z and V_y, shear forces along z and y axes respectively. The a_1, a_2, a_3, a_4 and b_1, b_2, b_3, b_4 are the polynomial constants combining all the above four equations gives following yield surface.

$$f(P_x, m_y, m_z, T_x, V_z, V_y, P_o, M_{zo}, M_{yo}, T_{xo}, V_{yo}, V_{zo}, ai, bi) = 1 \quad (4.13)$$

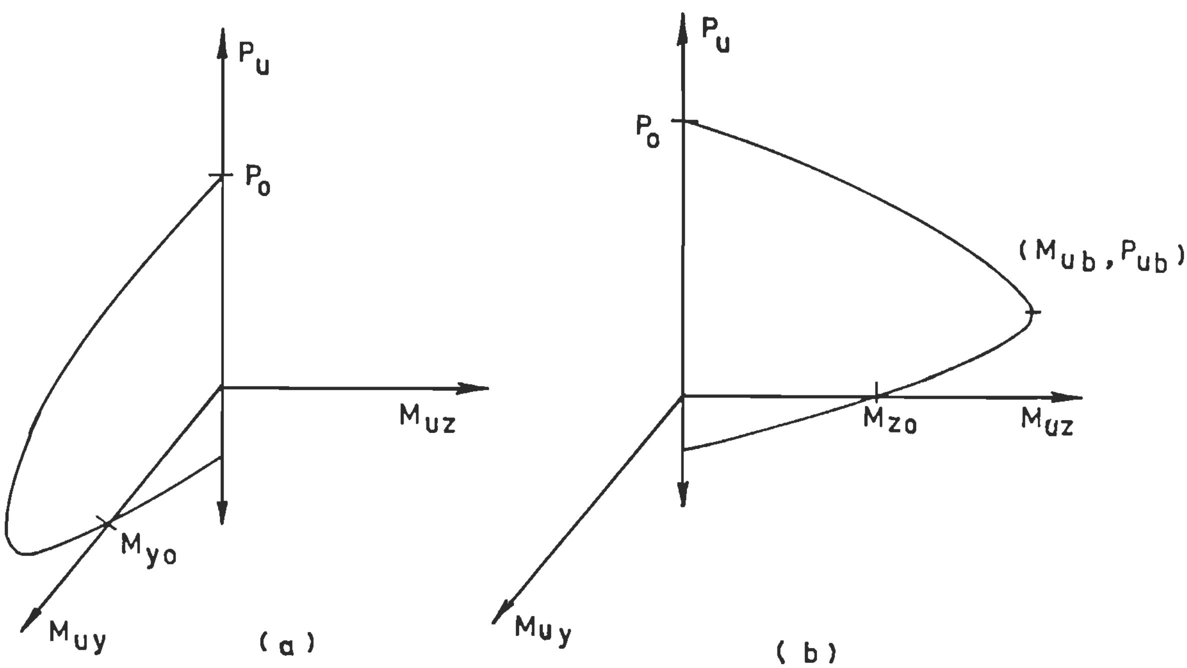


Fig. 4.4 - Moment-Axial Force Interaction Curve

This equation contains 14 constants for a section namely P_o, M_{zo}, M_{yo}, T_{xo}, V_{yo}, V_{zo}, a_1, a_2, a_3, a_4, b_1, b_2, b_3, b_4. These constants are determined in the begining for all cross sections in the frame. The complete yield function can not be shown pictorially, However for biaxial moments and axial force interaction surface in 3D space is shown in Fig. 4.6. The details of calculation for P_o, M_{zo}, M_{yo}, T_{xo}, V_{yo}, V_{zo}, a_1, a_2, a_3, a_4, b_1, b_2, b_3, b_4 are given in Appendix - A.

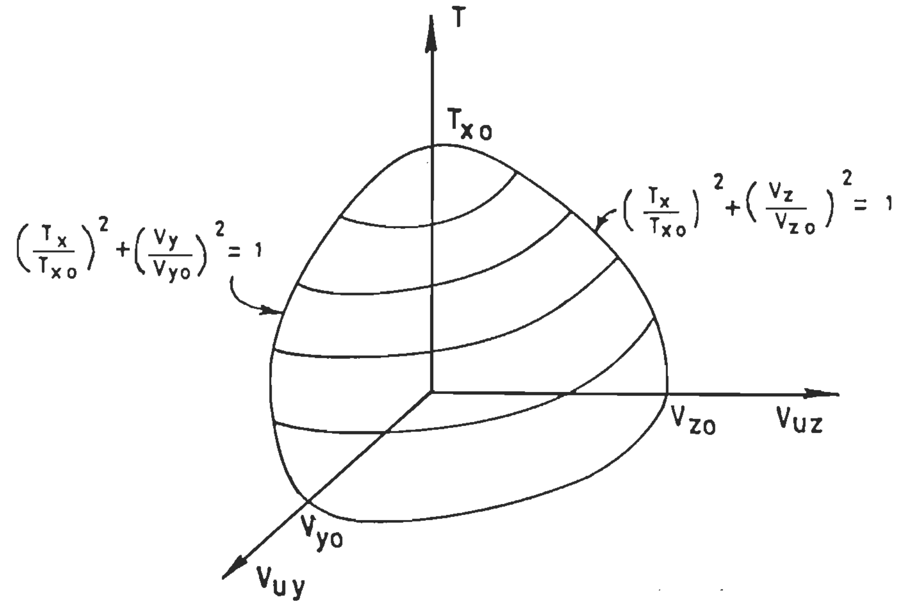


Fig. 4.5 - Torsional Moment-Shear Force Interaction Curve

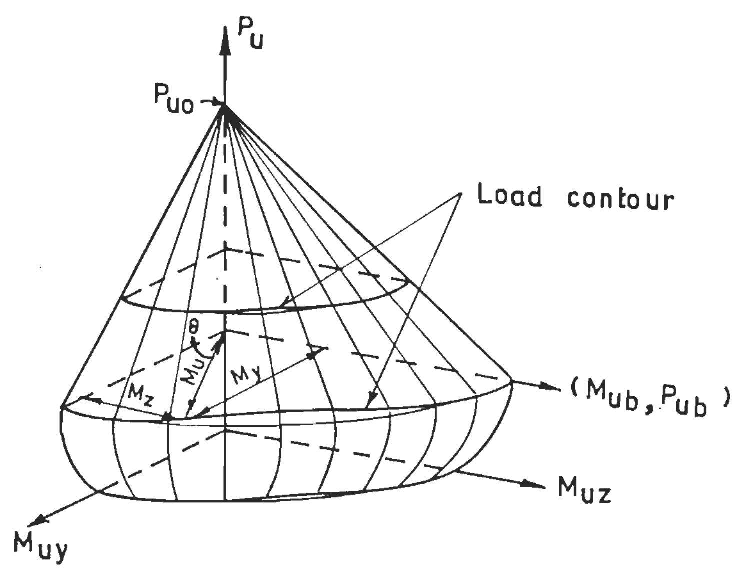


Fig. 4.6 Biaxial Moments and Axial Force Interaction Yield Surface

4.2.3 Elasto-Plastic Stiffness Property

Yielding is assumed to take place only at generalised plastic hinges of zero length and the beam between the hinges is assumed to remain elastic. In this approach, multi dimensional action deformation relationships must be specified for hinges in terms of moments, axial forces and shear forces etc. Lumped plasticity models are particularly suitable for analysis of building frames under seismic loads, because plastic action in such structures is usually confined to small regions at the beam and column ends.

Yield function can be expressed as $f(P_k) = 1.0$, where f represents the yield function and $P_k (k = i, j)$ represents the nodal forces, F <1.0 represents the elastic state, f = 1.0 represents the plastic state and f > 1.0 represents an inadmissible state. The associated flow rule can be expressed as [Chen(1982)]

$$du_{pk} = \lambda_k g_k \tag{4.14}$$

Where du_{pk} is the plastic component of the incremental nodal displacement at node k (k = i, j) of the beam column element. λ_k is the flow constant and g_k is the gradient vector of the yield surface at end k(k=i, j)

$$\mathbf{du_p} = \begin{Bmatrix} du_{pi} \\ du_{pj} \end{Bmatrix} = \begin{bmatrix} g_i & 0 \\ 0 & g_j \end{bmatrix} \begin{Bmatrix} \lambda_i \\ \lambda_j \end{Bmatrix} = \mathbf{G}\{\lambda\} \tag{4.15}$$

The g_i vector may be obtained from Eqns(4.9 - 4.12)

$$g_i(1) = -2\frac{m_y^2}{m_{yp}^2}\left(\frac{b_2}{P_0} + 2b_3\frac{P_x}{P_0^2} + 3b_4\frac{P_x^2}{P_0^3}\right)m_{y0} - 2\frac{m_z^2}{m_{zp}^2}\left(\frac{a_2}{P_0} + 2a_3\frac{P_x}{P_0^2} + 3a_4\frac{P_x^2}{P_0^3}\right)m_{z0}$$

$$g_i(2) = \frac{2T_x^2 V_y}{T_{x0}^2 V_{y0}^2 (1-(V_z/V_{z0})^2 - (V_y/V_{y0})^2)^2}$$

$$g_i(3) = \frac{2T_x^2 V_z}{T_{x0}^2 V_{z0}^2 (1-(V_z/V_{z0})^2 - (V_y/V_{y0})^2)^2}$$

$$g_i(4) = 2\frac{T_x}{T_{xv}^2}$$

$$g_i(5) = 2\frac{m_y}{m_{yp}^2}$$

$$g_i(6) = 2\frac{m_z}{m_{zp}^2}$$

(4.16)

4.2.4 Normality Condition

For an elastic perfectly plastic material, the yield surface is fixed and there is no secondary work. This implies that the increments of the nodal forces dP_k(k=i, j) corresponding to a plastic deformation at the cross-section must be tangent to the yield surface, the normality condition may be described as

$$du_{pk}^T dp_k = 0 \tag{4.17}$$

4.2.5 Plastic Stiffness Matrix

The elasto-plastic stiffness matrix for beam column element having plastic hinge at one or both ends may be derived using the associated flow rules and the normality condition described above[Tseng and Penzien(1975), Al Bermani, Kitipornchai(1990)]. The desired relation is

$$\mathbf{dp} = \mathbf{K_{ep}\ du} \tag{4.18}$$

Assuming that the element deformation increments can be decomposed into an elastic component and a plastic component as

$$\mathbf{du} = \mathbf{du_e} + \mathbf{du_p} \tag{4.19}$$

The element forces **dp** are determined by the element elastic nodal displacements

$$dp = K_e\, du_e = K_e(du - du_p) \tag{4.20}$$

where K_e is the elastic stiffness matrix. Substituting the value of du_p from Equation(4.15) results

$$dp = K_e du - K_e\, G\, \{\lambda\} \tag{4.21}$$

Pre multiplying both sides of (4.20) by G^T and using the relation (4.15) gives

$$G^T dp = G^T K_e du - G^T K_e G\{\lambda\} = \{0\} \tag{4.22}$$

The flow constant λ can be expressed as

$$\{\lambda\} = [G^T K_e G]^{-1}\, G^T K_e du \tag{4.23}$$

$$dp = K_e\, du - K_e G[G^T K_e G]^{-1} G^T K_e du = K_e du + K_p du = K_{ep} du \tag{4.24}$$

where K_p is the plastic reduction matrix, and $K_p = -K_e\, G[G^T K_e G]^{-1} G^T K_e$ (4.25)

K_p represents the loss of elastic stiffness K_e due to the development of plastic hinges at one or both ends of beam column element. For fully elastic element, K_p is a null matrix. The flexural stiffness EI which defines the behaviour of r.c. concrete beam or column in the entire elastic range prior to reaching the ultimate yield surface is assumed to be 50% of the gross section stiffness to account for loss of stiffness due to concrete cracking[Anderson and Townsend(1977), Saatcioglu(1984) and Moazzami and Bertero(1987)].

4.2.6 Stiffness Degrading Model for 3D Frame Elements

Plastic hinge is initially rigid, so that the stiffness matrix is initially infinite. After reversed loading is applied, the stiffness degrades and becomes finite. An elastic stiffness matrix for each hinge is defined as

$$K_h = \text{Diag}\,[k_x^{'}, k_y^{'}, k_z^{'}, k_{mx}^{'}, k_{my}^{'}, k_{mz}^{'}] \tag{4.26}$$

where $k_x^{'}, k_y^{'}, k_z^{'}, k_{mx}^{'}, k_{my}^{'}, k_{mz}^{'}$ are the elastic stiffnesses after unloading.

Yielding can take place only in plastic hinges concentrated at the ends. Degrading flexural stiffness are approximated by assuming that the element consists of a linear elastic beam element with non linear rotational springs at each end.

The action deformation relationship for each force component for a hinge is an extended version of Takeda's model. The extension to Takeda's model are shown in Fig. 4.7. This includes a reduction of the unloading stiffness by an amount which depends on the largest previous hinge deformations and incorporation of variable reloading stiffness which is larger than that of the Takeda's model and also depends upon the past rotation history. The unloading stiffness K_u depends on the maximum hinge deformation R_p and is controlled by the input parameter α. It must be non negative. Regardless of the value of α the unloading slope is never permitted to be less than the reloading slope, otherwise a hysteresis loop with a negative area could be produced. The reloading stiffness K_l also depends upon the previous maximum rotation and is governed by the input parameter β. The parameter β must be non negative. The initial elastic stiffness K_o is given by

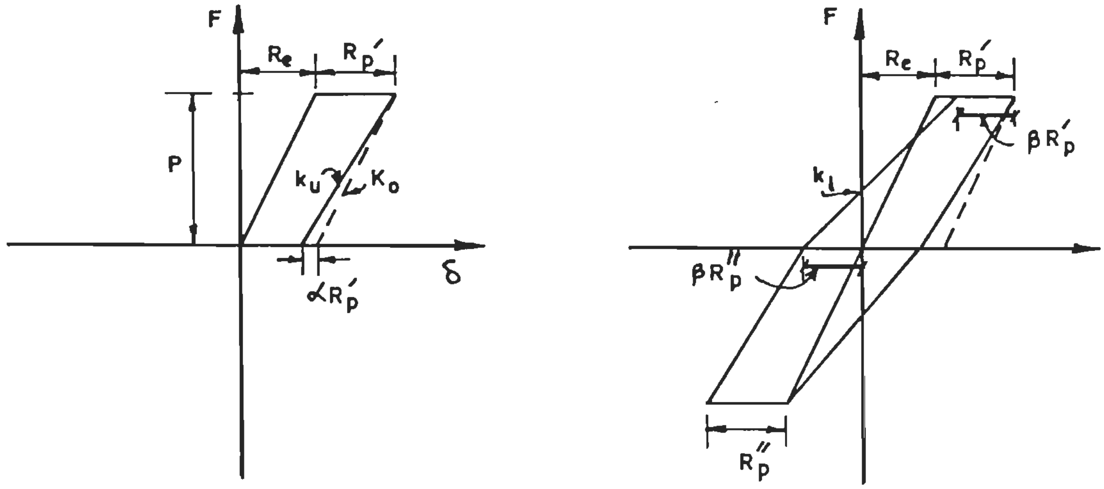
$$K_o = P/R_e \tag{4.27}$$

where P is the load and R_e is the deformation in the elastic range. The unloading stiffness K_u is given as

$$K_u = P/(R_e + \alpha R_p^{'}) \tag{4.28}$$

$$K_u = 1/(R_e/P + \alpha R_p^{'}/P) = 1/(1/K_e + 1/K_{pe}) \tag{4.29}$$

where R_e/P is flexibility of the elastic element, $\alpha R_p^{'}/P$ is the flexibility of the plastic hinge and K_e is stiffness of the elastic element and K_{pe} is stiffness of the plastic hinge.



(a) Unloading Stiffness (b) Reloading Stiffness

Fig 4.7 Action Deformation Relationship for Takeda's Model

Initially the plastic hinge was rigid but on unloading it becomes flexible and have some flexibility. Similarly for the reloading stiffness

$$K_l = P/(R_e + \beta R_p') \tag{4.30}$$

$$K_l = P/(R_e + R_p' - \beta R_p' + R_p'' - R_p''\alpha) \tag{4.31}$$

where R_p' is plastic hinge rotation in the initial cycle of loading and R_p'' is the plastic hinge rotation in the reversed cycle of loading.

$$K_l = P/(R_e + R_p'(1 - \beta) + R_p''(1 - \alpha)) = P/(R_e + \alpha R_p' + \beta R_p'') \tag{4.32}$$

$$K_l = 1/(R_e/P + (\alpha R_p' + \beta R_p'')/P) = K_e K_{pe}/(K_e + K_{pe}) \tag{4.33}$$

$$\text{where } K_{pe} = P/(\alpha R_p' + \beta R_p'') \tag{4.34}$$

In the present study $\alpha = 0.25$ and $\beta = 0.75$ have been taken.

4.2.7 Mass Matrix for 3D Frame Elements

The mass matrix of the 3D frame elements can be taken as either lumped mass matrix or consistent mass matrix. The consistent mass matrix gives better results as compared to the results using lumped mass matrix. The consistent mass matrix has been taken from[Thanoon(1993)]. The rotation matrix for transferring element mass matrix from the local to global axes is the same as that used for transferring the element stiffness matrix from local to global axes.

4.3 Mathematical Model for R.C.C. Panel and Slab Elements

R.C.C. slabs and infill panels(either of r.c.c. or brick masonry) are the essential elements in the r.c.c. framed buildings, which contributes to the stiffness and the mass to the over all structure, thereby play a vital role in the r.c.c. framed building structures. Therefore modelling of these elements is equally important as that of the beam/column elements. In the present study for the infill panels the out of plane stiffness of the panels has been assumed negligible and for r.c.c. slabs the bending stiffness has also been considered in addition to the in plane stiffness. A four noded flat shell element has been adopted to simulate the slabs and the infill panels in the structure.

4.3.1 Stffness Matrix for Flat Shell Element

A four noded flat shell element subjected to inplane and bending actions simultaneously is shown in Fig. 4.8. The element's nodal displacements and the nodal forces may be defined as

$$p_{mb} = K_{mb}\, u_{mb}$$

Where $p_{mb} = [p_{mb1}, p_{mb2}, p_{mb3}, p_{mb4}]^T$

$$u_{mb} = [u_{mb1}, u_{mb2}, u_{mb3}, u_{mb4}]^T$$

K_{mb} {24x24} is the element stiffness matrix.

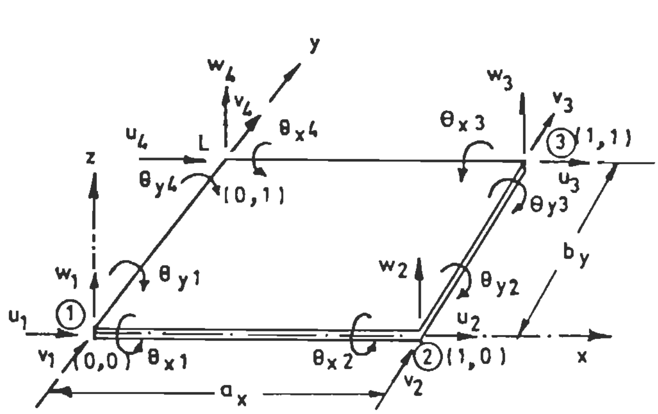


Fig. 4.8 - Mathematical Model of Flat Shell Element[Zienkiewicz(1977)]

The stiffness matrix K_{mb} consists of sixteen sub matrices each (6x6) as shown below.

$$K_{mb} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} \\ K_{21} & K_{22} & K_{23} & K_{24} \\ K_{31} & K_{32} & K_{33} & K_{34} \\ K_{41} & K_{42} & K_{43} & K_{44} \end{bmatrix} \tag{4.35}$$

The typical K_{rs} (r=1, 4 and s=1, 4) is

$$K_{rs} = \begin{bmatrix} \multicolumn{2}{c}{\multirow{2}{*}{$[K_m]$}} & 0 & 0 & 0 & 0 \\ & & 0 & 0 & 0 & 0 \\ 0 & 0 & \multicolumn{3}{c}{\multirow{3}{*}{$\begin{bmatrix} & & \\ & K_b & \\ & & \end{bmatrix}$}} & 0 \\ 0 & 0 & & & & 0 \\ 0 & 0 & & & & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \tag{4.36}$$

where K_m is the inplane stiffness matrix of the size (2x2) and K_b is the out plane stiffness matrix of the size (3x3) and last column and last row is null, and it corresponds to θz d.o.f. for the slab panel element. Zienkiewicz(1977) suggests to assign arbitrary value to the stiffness coefficients $K_{\theta z}$. This fictitious stiffness coefficient is positioned in the appropriate position in the (24x24) element stiffness matrix. The fictitious element is used only if all the elements meeting at a joint are coplanar and there is no beam column element meeting at the joint. Shape functions N_i for inplane d.o.f. and bending d.o.f. are shown in Fig. 4.9.

(a) Inplane d.o.f.	(b) Bending d.o.f.
$(1-\xi)(1-\eta)$	$1-\xi\eta-(3-2\xi)(1-\eta)\xi^2-(1-\xi)(3-2\eta)\eta^2$
$\xi(1-\eta)$	$(1-\xi)\eta(1-\eta)^2 b_y$
$\xi\eta$	$-\xi(1-\xi)^2(1-\eta)a_x$
$\eta(1-\xi)$	$(3-2\xi)(1-\eta)\xi^2+(1-\eta)(1-2\eta)\xi\eta$
	$\xi\eta(1-\eta)^2 b_y$
	$(1-\xi)\xi^2(1-\eta)a_x$
	$(3-2\xi)\xi^2\eta-\xi\eta(1-\eta)(1-2\eta)$
	$-\xi(1-\eta)\eta^2 b_y$
	$(1-\xi)\xi^2\eta a_x$
	$(1-\xi)(3-2\eta)\eta^2+\xi(1-\xi)(1-2\xi)\eta^2$
	$-(1-\xi)(1-\eta)\eta^2 b_y$
	$-\xi(1-\xi)^2\eta a_x$

Fig. 4.9 - Shape Functions for Flat Shell Element

The stiffness matrix for the flat shell element in local coordinate axes is transformed to global coordinate axes by the transformation matrix T_S

$$K_{mb} = T_s^T K_{mb} T_s \tag{4.37}$$

$$T_s = \begin{bmatrix} L & & & \\ & L & & \\ & & L & \\ & & & L \end{bmatrix} \tag{4.38}$$

$$L = \begin{bmatrix} R_s & 0 \\ 0 & R_s \end{bmatrix} \tag{4.39}$$

$\mathbf{R_s}$ is (3x3) matrix of direction cosines of the angles between the two sets of axes.

4.3.2 Mass Matrix for Flat Shell Elements

The consistent mass matrix for the flat shell element in the finite element formulation is given by

$$M_{ij} = \int_v N_i^T \rho N_j dv \tag{4.40}$$

where ρ is the material mass density and N_i and N_j is the shape function dv is the volume of the elements under consideration.

The inplane mass matrix and the out of plane mass matrix are determined separately. For the inplane mass matrix Eqn(4.40) is used while for out of plane mass matrix the term ρ is replaced by the following diagonal matrix which includes the effect of rotary inertia forces as suggested by [Zienkiewicz(1977)].

$$\begin{bmatrix} \rho & 0 & 0 \\ 0 & \rho t3/12 & 0 \\ 0 & 0 & \rho t3/12 \end{bmatrix} \tag{4.41}$$

The mass matrix of the element in the global coordinates is obtained as

$$\mathbf{M} = \mathbf{T^T M_e T} \tag{4.42}$$

where T is the transformation matrix for shell elements from local to global coordinates as defined for stiffness matrix derivation.

4.4 Mathematical Model for Interface

In r.c.c. framed structures the infill panels are of brick masonry. The interface between the r.c.c. beam/column elements and the brick masonry panel elements has been modelled by the four noded interface element as shown in Fig. 4.10. This element has been used between frame elements having 6 d.o.f..node and the infill elements having 2 translation d.o.f./node. For compatiability only two inplane translational d.o.f./node has been considered for the interface elements. The displacement vector for each node is given by

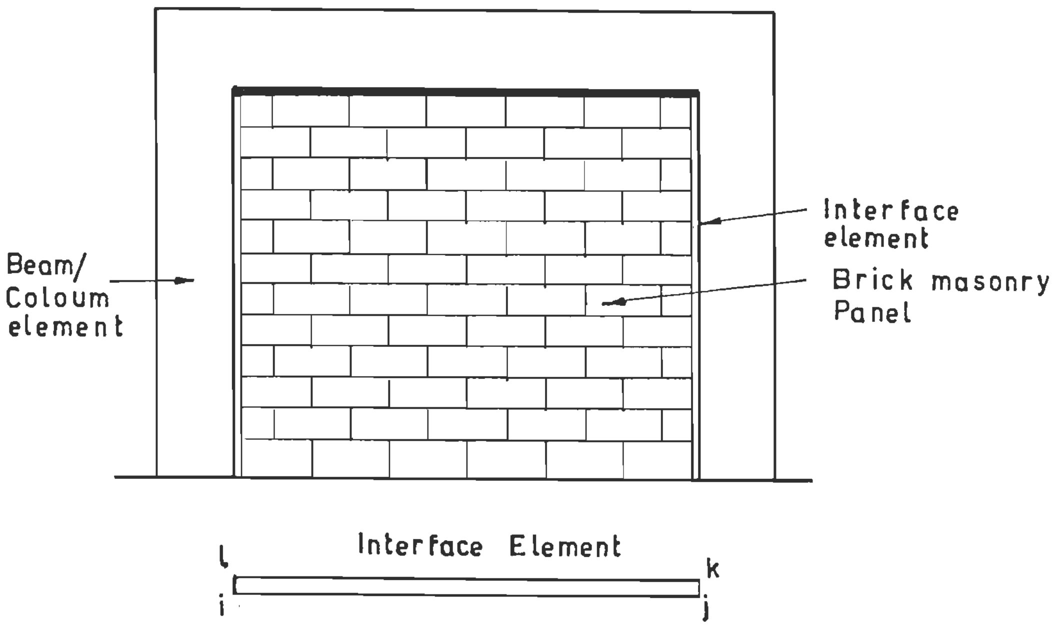


Fig. 4.10 - Interface Element

$$\delta = [u, v]^T \tag{4.43}$$

The strains are the relative displacement at the top and bottom of the elements as the thickness of the element is almost negligible. The strain vector is defined as

$$\varepsilon = [\Delta u , \Delta v]^T \tag{4.44}$$

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

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

where N_i is the shape function. For four noded line element the shape functions are expressed as

$$N_1 = (1-\xi), \; N_2 = \xi \tag{4.45}$$

The stress vector is given by

$$\sigma = [\sigma_u, \sigma_v]^T \tag{4.46}$$

The material property matrix D^j for the inter face element is given by

$$D^j = \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix} \tag{4.47}$$

where k_s and k_n are shear and normal stiffness coefficients respectively.

The strain matrix is defined as

$$\varepsilon = [-IN_1, -IN_2, IN_2, IN_1] \tag{4.48}$$

where I is a identity matrix of (2x2). The stiffness matrix for interface elements is given by

$$K_e^j = \int_L B^T D B \, dx \tag{4.49}$$

and the stiffness matrix in global coordinates can be obtained by transformation matrix **T** as

$$K^j = T_j^T K_e^j T_j \tag{4.50}$$

where T_j is the transformation matrix for interface from local to global coordinate system as defined for panel element.

4.5 Inelastic Inplane Property for R.C. Panel, Brick Masonry Infill and Interface Elements

For the r.c. panel elements concrete has been modelled as an isotropic material under biaxial stress condition and the reinforcement has been considered as a smear layer in which its properties are assumed to be distributed over the whole element. For the inelastic range, the stress strain relation and the stiffness property must be modified for concrete and steel layers through respective material property matrix D. Three conditions are to be established to evaluate the stress for a strain state in the inelastic range. These are (i) yield, (ii) cracking and (iii) crushing conditions. For steel, only yield condition is required to determine the stresses and to formulate the elasto-plastic stiffness.

Brick masonry is a complex material consisting of bricks and cement mortar. Due to the uncertainty in the properties of the bricks used, the brick masonry has got differing properties. Its behaviour becomes more complex with the cement mortar joints. The mortar joints have low tensile, shear and bond strengths. The behaviour of the brick masonry is much more complex as compared to r.c.c. as far as the uncertainty in the material properties are involved. In the present investigation the brick masonry material is assumed to be linearly elastic isotropic material till its failure. Two conditions are to be established to evaluate the stress for a strain state in the inelastic range. These are (i) compression crushing and (ii) cracking conditions.

The properties for the interface element depend upon the material which is coming in contact with each other and on the stress condition at particular instant of time. The material property matrix can be adjusted with the normal and shear stiffness coefficients for the interface elements[Sharma and Desai(1992)]. Yet no definite values for these coefficients is available in the literature. Two conditions are to be established to evaluate the stress for a (i) tensile and (ii) compression strain state.

The yield criteria, various conditions to evaluate the stress state for a strain state for the three elements are summarised in Table 4.1. The material property matrices for r.c.c. panel, brick masonry infill and interface elements in the elastic, cracking and crushing condition are given in Table 4.2.

4.5.1 Modelling of Reinforcing Steel In Shell Elements

The steel reinforcement is assumed to be an equivalent layer of uniaxial material smeared in the middle of concrete sections. Each steel layer is assumed to exhibit uniaxial

TABLE 4.1 - MATHEMATICAL EXPRESSION FOR YIELD CRITERIA, CRACKING, CRUSHING OF R.C.C. PANEL, BRICK MASONRY INFILL AND INTERFACE

Particulars	R.C.C. Panel	Brick Masonry Infill	Interface
(a) Yield Condition (i) Yield Criteria	$f(I_1, J_2) = [C_1(3J_2) + C_2(I_1)]^{1/2} = \sigma_0$ [Owen and Figueiras(1984), Cervera and Hinton(1986) and May and Naji(1991)] where C_1 **and** C_2 are material parameters, I_1 and J_2 are the first stress invariant and the second deviatoric stress invariant, σ_0 is the uniaxial compressive strength.	-------------------------------	----------------
(ii) Elasto-plastic material property matrix	$D^r_{ep} = D - [g_1^T D g_1]^{-1} D g_1 g_1^T D$ [Owen and Hinton(1980)] D is the elastic material property matrix g_1 is the gradient vector	-------------------------------	--------------
(iii) Elasto-plastic stiffness matrix	$K^r_{ep} = \int_v B^T D^r_{ep} B dv$	-------------------------------	--------------
(b) Cracking Condition (i) Maximum tensile strength	$f^r_t = 0.62(f'_c)^{1/2}$ [ACI 318-77] where f^r_t *and* f'_c are in N/mm^2	$f^b_t = f_b / 10.0$ where f_b is the yield strength of brick masonry	Tensile Condition $k_n = 0.0$ $k_s = 0.0$
(ii) Material property matrix in cracking condition	$D_{ce} = T^T_c D_c T_c$ where T_c is the transformation matrix from crack axis to element local axis	$D_{be} = T^T_b D_b T_b$ where T_b is the transformation matrix from crack axis to element local axis	-------------

(iii) shear Modulus	$G_1 = 0.25G(1-\varepsilon_1/0.004)\ for\ \varepsilon_{cr} < \varepsilon_1 < 0.004$ $G_1 = 0.0\ for\ \varepsilon_1 \geq 0.004$ $G_2 = 0.125G(1-\varepsilon_2/0.004)\ for\ \varepsilon_{cr} < \varepsilon_1 < 0.004$ where G is the shear modulus in uncracked condition, $\varepsilon_1\ and\ \varepsilon_2$ are the tensile strain in crack direction	$G_{ib} = G_b / 4.0$ where G_{1b} is shear modulus for brick masonry in cracked condition and G_b is in the uncracked condition	-------------
(c) Tension Stiffening	$\sigma_1 = \alpha_m f_t'(1-\varepsilon_1/\varepsilon_m)$ for $\varepsilon_t < \varepsilon_1 < \varepsilon_m$ or $\sigma_1 = \sigma_t \varepsilon_1 / \varepsilon_t$ for $\varepsilon_1 < \varepsilon_i$ where ε_1 is the current tensile strain, α_m is the tension stiffening coefficient, ε_m is the limiting value of tensile strain, ε_i is the maximum value reached by tensile strain at the point considered.	-------------------------------------	-------------
(d)Crushing	$C_1(3J_2') + C_2'(I_1') = \varepsilon_u^2$ where I_1' and J_2' are the strain invariants and ε_u is the ultimate strain in concrete, C_1 and C_2 are material parameter taken from Kupfer's results.	$\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau_{xy}^2 = \sigma_0$ where σ_x and σ_y are compressive stress in x and y direction τ_{xy} is shear stress and σ_0 is the crushing strength of brick masonry.	Compression condition $\lvert\sigma_s\rvert < \mu\lvert\sigma_n\rvert$ k_s = Experimental value $\lvert\sigma_s\rvert > \mu\lvert\sigma_n\rvert$ k_s = very low value when the normal strain is compressive k_n = very high value

TABLE 4.2 MATERIAL PROPERTY MATRIX FOR ELASTIC, CRACKED, CRUSHED (R.C.C., BRICK MASONRY AND INTERFACE) ELEMENTS

Particulars	R.C.C. Panel	Brick Masonry Infill	Interface	Details of Notations
Elastic Condition	$\frac{E_c}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix}$	$\frac{E_b}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix}$	$\begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix}$	$I_1 = \sigma_1 + \sigma_2 + \sigma_3$ $J_2 = 1/3(\sigma_1^2 + \sigma_2^2 + \sigma_3^2) - (\sigma_1\sigma_2 + \sigma_1\sigma_3 + \sigma_2\sigma_3)$ $C_1 = 1.355$ and $C_2 = 0.355\sigma_0$ $C_2' = 0.355\varepsilon_u$ $I_1' = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$ $J_2' = 1/3(\varepsilon_1^2 + \varepsilon_2^2 + \varepsilon_3^2) - (\varepsilon_1\varepsilon_2 + \varepsilon_1\varepsilon_3 + \varepsilon_2\varepsilon_3)$
Cracking in one direction	$\begin{bmatrix} 0 & 0 & 0 \\ 0 & E_c & 0 \\ 0 & 0 & G_1 \end{bmatrix}$	$\begin{bmatrix} tE_b & 0 & 0 \\ 0 & E_b & 0 \\ 0 & 0 & aG \end{bmatrix}$ t = 0.001, a = 0.25	$\begin{bmatrix} 0 & 0 \\ 0 & 0 \end{bmatrix}$ tensile condition	$Tc = \begin{bmatrix} Cos^2\theta_c & Sin^2\theta_c & Cos\theta_c Sin\theta_c \\ Sin^2\theta_c & Cos^2\theta_c & -Cos\theta_c Sin\theta_c \\ -2Cos\theta_c Sin\theta_c & 2Cos\theta_c Sin\theta_c & Cos^2\theta_c + Sin^2\theta_c \end{bmatrix}$
Cracking in both directions	$\begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & G_2 \end{bmatrix}$	$\begin{bmatrix} tE_b & 0 & 0 \\ 0 & tE_b & 0 \\ 0 & 0 & aG \end{bmatrix}$ t = 0.001, a = 0.25	------------	
Crushing condition	$\begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$	$\begin{bmatrix} tE_b & 0 & 0 \\ 0 & tE_b & 0 \\ 0 & 0 & tG \end{bmatrix}$ t = 0.001	------------	

behaviour only and it resists forces in the bar direction. The material matrix D_S for steel in the bar direction is given below.

$$\mathbf{D_s} = \begin{bmatrix} E_s & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{4.51}$$

where E_s is the initial modulus of elasticity for reinforcing steel bars. The incremental stress-strain relation after yielding of steel may be taken as

$$d\sigma_s = E_{ep} d\varepsilon_s \tag{4.52}$$

where E_{ep} is either the elasto-plastic modulus or the elasto-strain hardening modulus of the steel. In the first case E_{ep} is zero and in the second case it will be equal to the slope of the strain hardening portion of the stress strain curve E_{st}.

4.6 Damping Matrix of the Structure

Damping is very important property of the structure and a very limited information is available in the literature. With the higher modes, the damping in the structure increases. In the present study Rayleigh's damping has been assumed which is proportional to the mass and the stiffnesses. The Rayleigh's damping matrix is given by

$$\mathrm{C} = \mathrm{d_1 M} + \mathrm{d_2 K} \tag{4.53}$$

where **C, M** and **K** are the damping, mass and stiffness matrices for the structure and d_1 and d_2 are constants which are calculated from different modes frequencies as under

$$\begin{bmatrix} d_1 \\ d_2 \end{bmatrix} = \frac{2w_i w_j}{w_i^2 - w_j^2} \begin{bmatrix} w_i & -w_j \\ -1/w_i & 1/w_j \end{bmatrix} \begin{bmatrix} \zeta_i \\ \zeta_j \end{bmatrix} \tag{4.54}$$

where w_i and ζ_i are the frequency and the damping ratio for the ith mode respectively. The main advantage of the Rayleigh damping matrix is that it has got the same structure as that of the stiffness matrix and does not require additional computational effort while solving resulting set of equations.

4.7 Ductility

The ductility is an important parameter in earthquake resistant design of buildings. Besides the design for strength, the design for ductility is essential for safety from collapse during earthquakes. The plastic deformation in the structure in case of excessive loading are ensured by the ductility capacity of the structure. The provisions of ductility in a structure enhances the capacity of the structure to withstand unexpected overloading, ensures better behaviour during load reversals, impact and foundation settlements and leads to economical design [Labbe and Noe(1992) and Thakkar(1993)].

The ductility is the characteristic of a material that represents its capacity to undergo large strains while resisting loads. There are many ways in which the ductility can be measured, as yet there is no standard method prescribed of measuring ductility. The ductility can be defined at material, member and structure levels.

4.7.1 Material Ductility

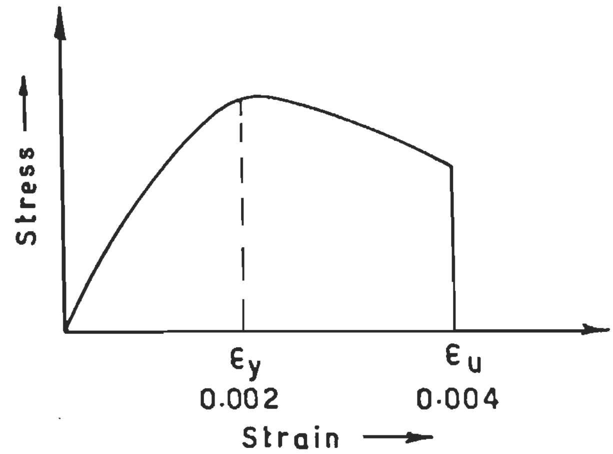
The material ductility is defined as the ratio of the ultimate strain to yield strain of the material. The stress-strain curve for concrete and steel are shown in Fig. 4.11. The material ductility for concrete and steel are given by $\varepsilon_u/\varepsilon_y$ where ε_u is the ultimate strain and ε_y is the yield strain for the material.

4.7.2 Member Ductility

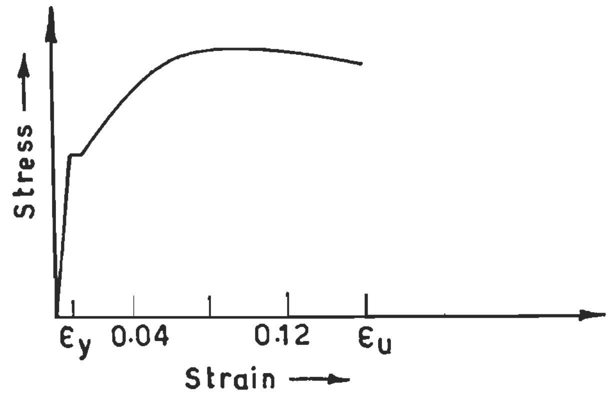
The member is subjected to various combinations of forces such as axial load, bending moments m_y and m_z, torsional moment T_x and shear forces in y and z directions of the member. The yielding of the member takes place under the action of these six components of forces in the space frames. The ductility of the member is defined as

$$\frac{du_{\max}}{du_y} \tag{4.55}$$

where $du_{\max}$ is the maximum lateral deformation reached in the member under the action of loading and du_y is deformation at which the first yield occurs.



Stress Strain curve for Conc.



Stress Strain curve for Steel

Fig. 4.11 - Stress Strain Curve for Concrete and Steel

4.7.3 Structure Ductility

The overall ductility of the structure is defined as the structure ductility and is expressed as the ratio of the maximum displacement reached at the top of the structure during the load application to the displacement at the top of the structure at the time of occurence of first yield in the structure.

In the present study, the ductility demand at member level has been evaluated. The member level ductility is evaluated by monitoring the maximum deformation in the member during the load application and the initial deformation at which the yielding takes place and is calculated as per Eqn.(4.53). The structure ductility is evaluated by monitoring the maximum deformation during the load application at the top of the structure to the deformation at the first yielding in any member of the structure.

The evaluation of the member ductility and structure ductility is implemented in the computer program.

4.8 Time Integration Scheme

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

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{q}(\mathbf{u}, \dot{\mathbf{u}}) = \mathbf{f} \qquad (4.56)$$

where $\mathbf{q}$ is the vector of internal resisting forces which depends upon the displacement 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.(4.56) has been carried out using a numerical step by step procedure. Newmark's predictor-corrector(Implicit method) has been selected for dynamic solution.

4.8.1 Newmark's Predictor-Corrector Implicit Scheme

In Newmark's scheme, the following relations are defined

$$\mathbf{M}\ddot{\mathbf{u}}_{t+\Delta t} + \mathbf{q}(\mathbf{u}_{t+\Delta t}, \dot{\mathbf{u}}_{t+\Delta t}) = \mathbf{f}_{t+\Delta t} \qquad (4.57)$$

where $$u_{t+\Delta t} = \bar{u}_{t+\Delta t} + \Delta t^2 \beta \ddot{u}_{t+\Delta t} \qquad (4.58)$$

$$\dot{u}_{t+\Delta t} = \dot{\bar{u}}_{t+\Delta t} + \Delta t \gamma \ddot{u}_{t+\Delta t} \qquad (4.59)$$

$$\bar{u}_{t+\Delta t} = u_t + \Delta t \dot{u}_t + 0.5 \Delta t^2 (1 - 2\beta) \ddot{u}_t \qquad (4.60)$$

$$\dot{\bar{u}}_{t+\Delta t} = \dot{u}_t + \Delta t (1 - \gamma) \ddot{u}_t \qquad (4.61)$$

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 $\bar{u}_{t+\Delta t}, \dot{\bar{u}}_{t+\Delta t}$ are the historical values and $u_{t+\Delta t}, \dot{u}_{t+\Delta t}$ are the corrector values. For starting the algorithm, the initial values of acceleration $\ddot{u}_o$ are obtained by solving(4.57) at time = 0.0 as

$$\ddot{u}_o = M^{-1}(f_0 - q(u_o, \dot{u}_o)) \tag{4.62}$$

where f_0 is the applied load vector at time t = 0.0

The solution of the linear case is obtained by reducing the relations(4.57) to (4.62) to a recurrence relation which involves effective static solution at intervals 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 topographical change in the structure due to formation of plastic hinges in the frame and/or post cracking and/or yielding that may occur in the infill panels.

4.8.2 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 the past history at the previous time step as

$$\begin{aligned} u^j_{t+\Delta t} &= \overline{u}_{t+\Delta t} = u_t + \Delta t \dot{u}_t + 0.5\Delta t^2(1-2\beta)\ddot{u}_t \\ \dot{u}^j_{t+\Delta t} &= \dot{\overline{u}}_{t+\Delta t} = \dot{u}_t + \Delta t(1-\gamma)\ddot{u}_t \\ \ddot{u}^j_{t+\Delta t} &= (u^j_{t+\Delta t} - \overline{u}_{t+\Delta t})/\Delta t^2\beta = 0.0 \end{aligned} \tag{4.63}$$

3. Evaluate the residual forces r^j using the following equation

$$r^j = f_{t+\Delta t} - M\ddot{u}^j_{t+\Delta t} - C^j\dot{u}^j_{t+\Delta t} - K^j u^j_{t+\Delta t} \tag{4.64}$$

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

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

$$K^{\otimes} = M/(\Delta t^2\beta) + \gamma C^j/(\Delta t\beta) + K^j \tag{4.65}$$

5. Solve for the incremental displacements

$$K^{\otimes} du^j = r^j \tag{4.66}$$

6. Update the displacements, velocities and accelerations as

$$\begin{aligned} u_{t+\Delta t}^{j+1} &= u_{t+\Delta t}^{j} + du^j \\ \ddot{u}_{t+\Delta t}^{j+1} &= (u_{t+\Delta t}^{j+1} - \overline{u}_{t+\Delta t})/(\Delta t^2\beta) \\ \dot{u}_{t+\Delta t}^{j+1} &= \dot{u}_{t+\Delta t}^{j} + \Delta t\,\gamma\,\ddot{u}_{t+\Delta t}^{j+1} \end{aligned} \tag{4.67}$$

7. if du^j do not satisfy the convergence condition then set j = j+1 and go to step 3, otherwise continue.

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

4.8.3 Elastic Analysis

The stiffness matrix for the structure is formulated from the frame elements, panels, slabs and interface elements. The load vector is calculated from the loads acting on the structure and then static analysis is carried out to find the member forces due to the dead loads and live loads on the structure. These forces are taken as the initial forces in the dynamic analysis.

4.8.4 Inelastic Analysis

An incremental iterative procedure is adopted for the static inelastic analysis and Newmark's Predictor-Corrector scheme has been used for the dynamic analysis as explained in 4.8.2. For inelastic analysis member forces for frame elements at nodes and stresses for panel elements at gauss points are first calculated assuming an elastic behaviour. Then the nodal forces in the frame elements are checked against the yield surface, if force state lies outside the yield surface then the plastic hinge is formed at the node, if the point lies inside the yield surface then the solution proceeds as an elastic solution. For panel elements the stresses and strains at the gauss points are checked against crushing, cracking and yielding. When any of the above events has occured, forces or stresses are modified accordingly and then the unbalanced forces are calculated and redistributed in the subsequent iterations. This is continued untill a specified convergence criteria is satisfied. The analysis gets terminated automatically when the structure stiffness matrix becomes non positive definite.

4.8.5 Convergence Criteria

In the present study the convergence criteria to terminate iteration process in the load step has been choosen based on the incremental displacements and is expressed as under

$$\left[\frac{\Delta u_j^T \Delta u_j}{u_j^T u_j}\right]^{1/2}_{j=1,n} \leq \varepsilon_t \qquad (4.68)$$

Where n is the number of unknown displacements in the structure and ε_t is the specified tolerance limit for the displacement criterion. In the present study ε_t has been choosen as 0.001 for static analysis and 0.0005 for dynamic analysis.

4.9 Computer Codification, Program ISABF

The previously described algorithm has been codified and the program ISABF(Inelastic Seismic Analysis of Building Frame) developed for 3D framed buildings with panels subjected to static and seismic/dynamic loads. Earlier a computer program IBRCF was developed by Thanoon(1983) in his Ph.D. Thesis which has been further modified to ISABF to take care of the additional features described below.

Brick masonry infill, interface element, truss element has been included in the ISABF. Inelastic modelling of 3D r.c. beam/column element has been modified to take care of the shear forces in the yield criteria. Stiffness degradation effect has been included. Ductility requirement for 3D r.c. frame element has been evaluated for yielded frame members.

Eigen value solution and mode superposition method has been added to evaluate the displacements and member forces due to input acceleration spectra. Provision to take care of lumped masses and additional masses corresponding to live loads and other additional loads has been made.

At present the program ISABF has the following elements. 3D r.c. frame, r.c.c. slab, r.c.c. panel, brick masonry infill, interface and truss elements. This program can analyse the static/dynamic, elastic/inelastic response of the r.c.c framed building subjected to dead loads, live loads and earthquake loads. It calculates the member end forces. stresses at gauss points , plastic deformations. It evaluates the ductility requirement of the yielded falling into inelastic range.

4.10 Validation of the Computer Program

The flow chart of the developed computer program is shown in Fig. 4.12. The validity of the proposed formulation and the developed computer program is checked in this section. Different types of building frames have been analysed taken from the available literature. The following five problems has been analysed and the results from the present study has been compared with reported results in the literature.

1. Two dimensional r.c. frame with brick masonry panel
2. Three dimensional r.c. setback frame
3. Two dimensional r.c. frame
4. Three dimensional r.c. frame

4.10.1 Two Dimensional R.C. Frame with Brick Masonry Panel

A two dimensional r.c. frame with brick masonry panel as shown in Fig. 4.13 has been choosen to validate the modelling of r.c. beam/column elements and the brick masonry panel elements for its inelastic behaviour. The problem has been studied experimentally and analytically by [Choubey(1990)]. The data for the problem and the material properties used are as given in the Figure.

Inelastic static analysis has been performed to study the behaviour of the frame. The cracking pattern shown by [Choubey(1990)] have been compared with the present study in Fig. 4.14. The load deflection curve for the frame has been compared in Fig. 4.15.

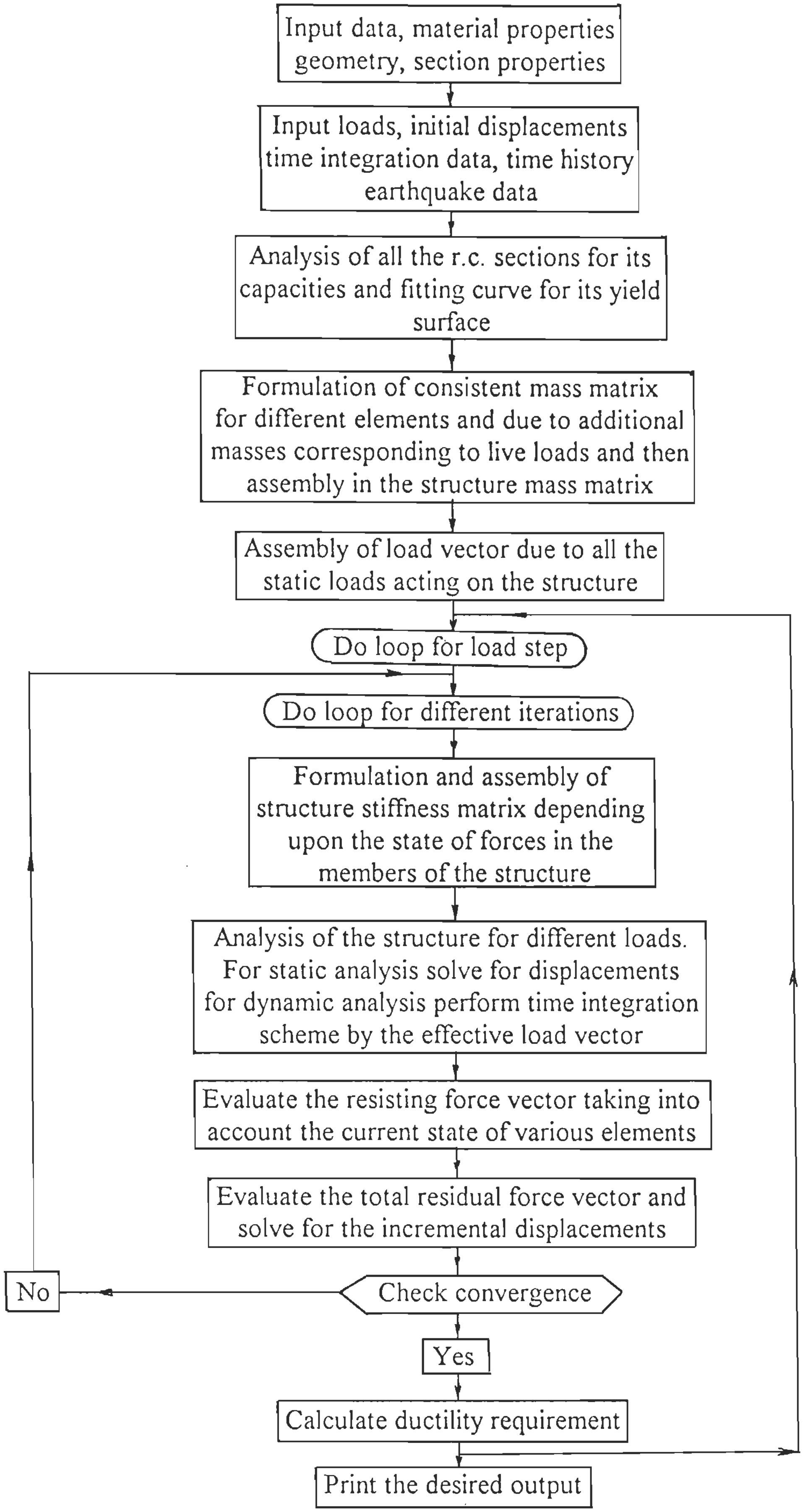


Fig. 4.12 - Flow Chart of Computer Program ISABF

The failure load obtained in the experimental study is 175.38 kN, and the failure load of 178 kN has been observed in the present study which is close to the experimentally observed failure load. The closeness of the results observed in the present study for load deflection behaviour, hinge formation and cracking pattern and failure load justifies the present modelling for the r.c. section, brick panel and interface elements.

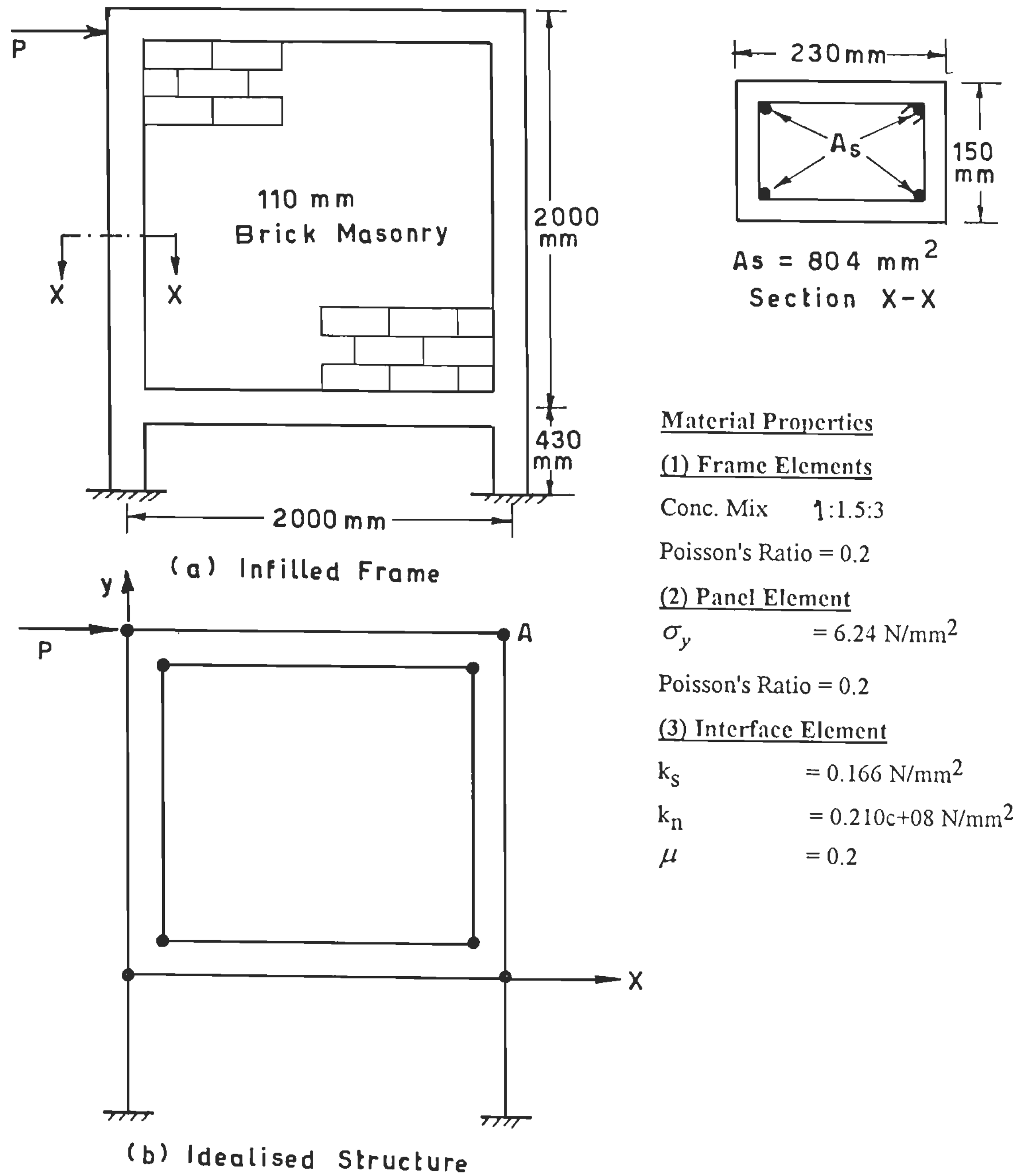


Fig. 4.13 - Two Dimensional R.C. Frame with Brick Masonry Panel

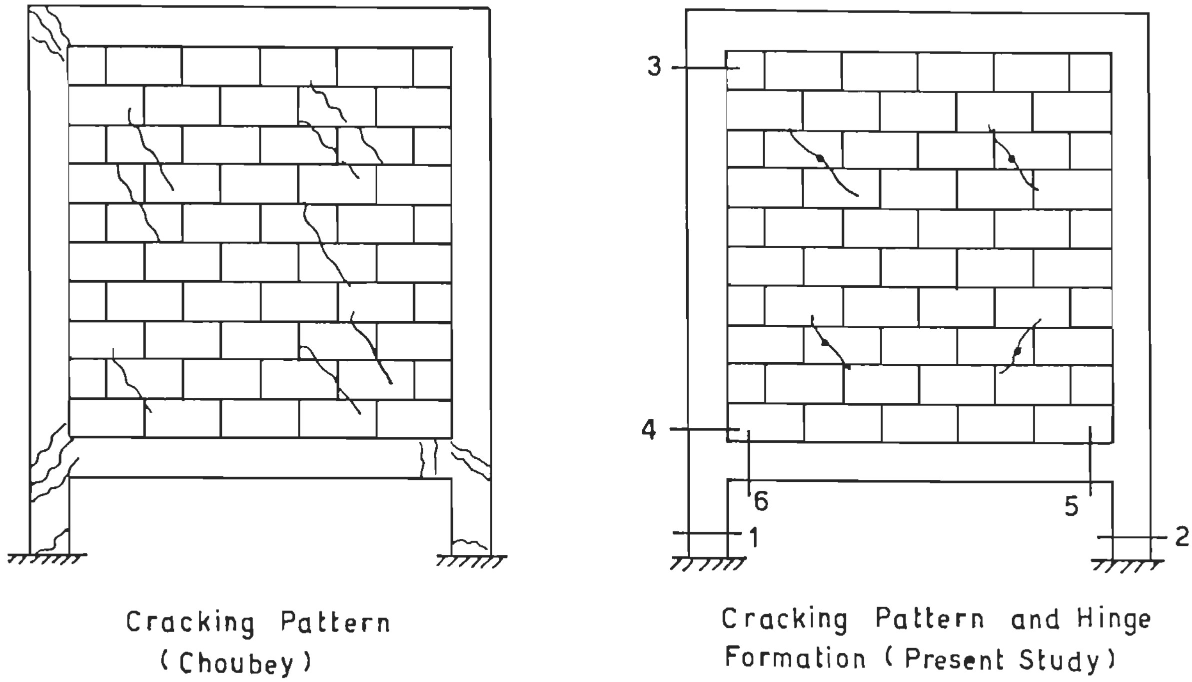


Fig. 4.14 - Cracking Pattern and Formation of Plastic Hinges

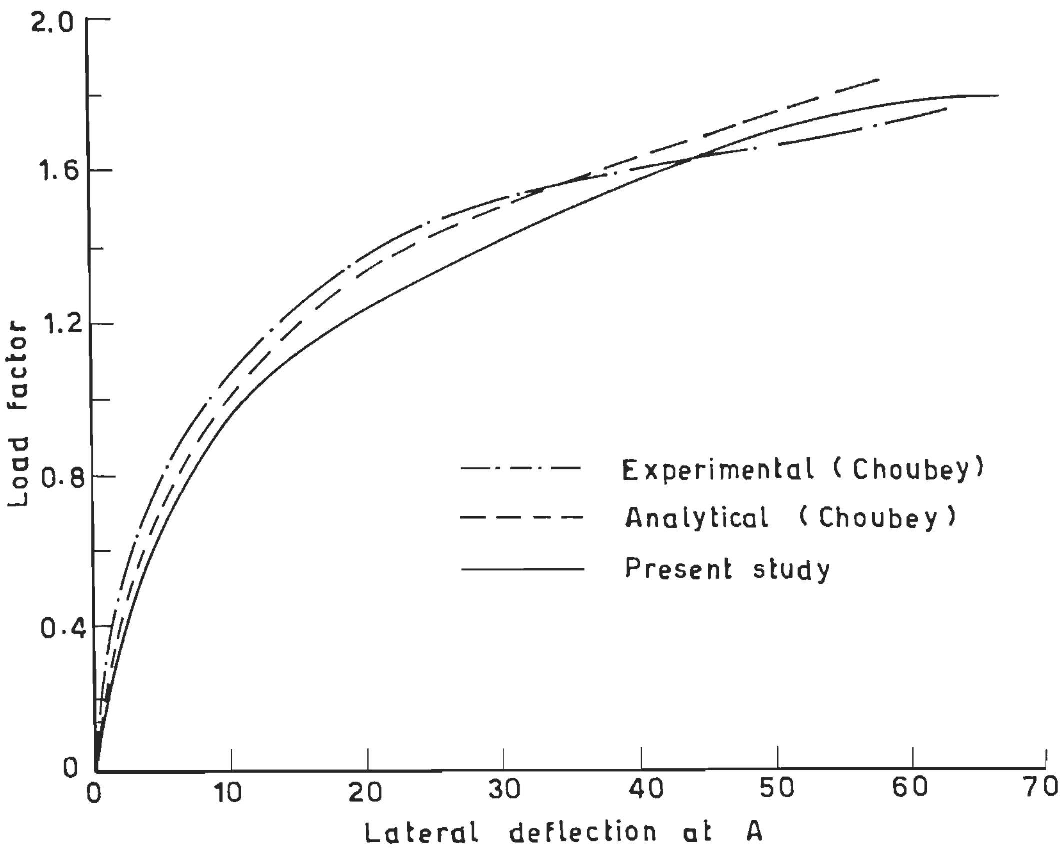
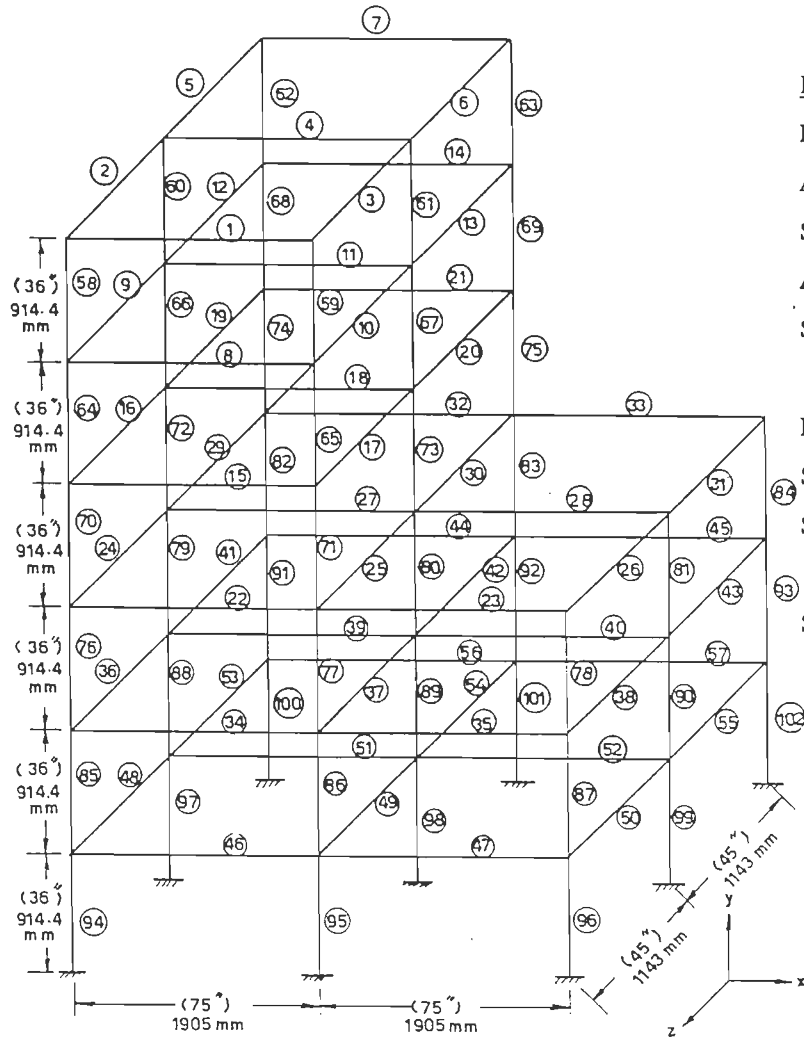


Fig. 4.15 - Comparison of Load Deflection Curve

4.10.2 Three Dimensional R.C. Setback Frame

A three dimensional r.c. setback frame shown in Fig. 4.16 with slab panels has been choosen to study the behaviour in the inelastic range due to the seismic excitation given to the structure. The problem which has been studied experimentally and analytically by [Shahrooz and Moehle] is used to validate the program. The complete data and material properties are taken for the problem as reported by the authors. First the frame has been analysed under the action of the dead loads acting on the structure. The forces in the members due to dead loads are taken as the initial forces in the structural members for studying the inelastic seismic behaviour of the building subjected to various intensities of EL-Centro earthquake.



Material Properties

Econc. = 21273.8 N/mm^2

Av. Comp.

Strength of Conc.= 28.96 N/mm^2

Av. Tensile

Strength of Conc.= 3.17 N/mm^2

σ_{sy} = 450 N/mm^2

E_{steel} = 200000 N/mm^2

Size of Columns = 127.0mmx165.1mm

Size of Beams = 127.0mmx177.8mm

= 101.6mmx190.5mm

Slab Thickness = 44.45 mm

Fig. 4.16 - Three Dimensional R.C. Setback Frame

Free vibration time periods of the framed building structure obtained experimentally and analytically by [Shahrooz and Moehle] has been compared with the time periods obtained in the present study in Table 4.3. The time periods of vibration obtained in the present study are almost the same as reported.

Table 4.3 - Comparison of Time Periods(Secs)

Results reported(Shahrooz and Moehle)		Present Study
Experimental	Analytical	Analytical
0.270	0.260	0.280
0.250	0.230	0.220
0.150	0.120	0.170
0.100	0.110	0.110
0.087	0.078	0.084
0.070	0.061	0.074

Time history response of the structure has been reported by [Shahrooz and Moehle] obtained experimentally and analytically. The results obtained in the present study have been compared with the experimental and analytical(reported) results separately. The time history response of the structure has been observed under the action of EL-Centro earthquake with varying intensities of the earthquake. The response obtained in the present study under the action of EC 7.7, EC 16.6 and EC 49.9 has been compared with the experimental results in Fig. 4.18. EC 7.7 indicates that the maximum acceleration of the earthquake is 7.7% of the acceleration due to gravity, EC 16.6 indicates 16.6% and EC 49.9 indicates 49.9% of acceleration due to gravity. The maximum acceleration of actual El-Centro earthquake is 33% of acceleration due to gravity

The present analytical results has been compared with the analytical results reported under the three intensities of the earthquake. The time history response of the structure presented in the literature takes different modelling for beam.column elements under the action of different intensities of earthquake loading. For EC 7.7 and EC 16.6 the effective cross section properties are taken as half of the gross cross section properties and for EC 49.3 the effective cross section properties has been taken as that of the fully cracked section for all the frame elements. In the present analytical investigation the effective cross section properties under the action of EC 7.7 and EC 16.6 has been taken as 1/2 of the gross section properties and under the action of EC 49.3 the effective cross section properties are taken as 1/3.5 of the gross section properties. The time history response of the structure under three intensities of the earthquake obtained in the present investigation and the analytical and experimental results as reported by[Shahrooz and Moehle(1987)] are compared in Fig. 4.17 and 4.18 respectively.

The maximum floor displacements under the action of three different intensities of the earthquake at the top floor level obtained in the present study have been compared with the experimental and analytical results(reported) in Table 4.4.

Table 4.4 - Comparison of Maximum Top Floor Displacements(mm)

Earthquake Intensity	Reported Results(Shahrooz and Moehle)		Present Study
	Experimental	Analytical	Analytical
EC 7.7	6.86	6.60	7.11
EC 16.6	15.75	15.75	15.24
EC 49.9	62.99	54.36	60.45

The results of maximum floor displacements at top floor obtained analytically in the present investigation are almost the same as the reported experimental results. It indicates that the present modelling technique is capable enough to predict the response of the structure. The little difference in the time history displacements results obtained in the present investigation from the experimental results is due to the following reasons. Uncertainty in the variation of the sizes of the members, variation in the damping in the structure during the test, variation in the cracking of the elements from the initial stage to the final loading stage. But still the response of the structure obtained in the present study is very close to the experimental reported results, thereby indicating that the present modelling of the members used in this study can be used to predict the response of a r.c.c. framed structure. The change of effective cross sectional properties with different intensities of earthquake application one after the other needs to be investigated further.

4.10.3 Two Dimensional R.C. Frame

A two dimensional r.c. frame shown in Fig. 4.19 has been analysed under the action of lateral static load to show the effect of degrading stiffness on the lateral displacement. The data is shown in the Figure.

The load on the frame has been varied from 0.0 to 0.975 times the lateral load shown in the Figure. Then the load is removed slowly and the load is applied in the

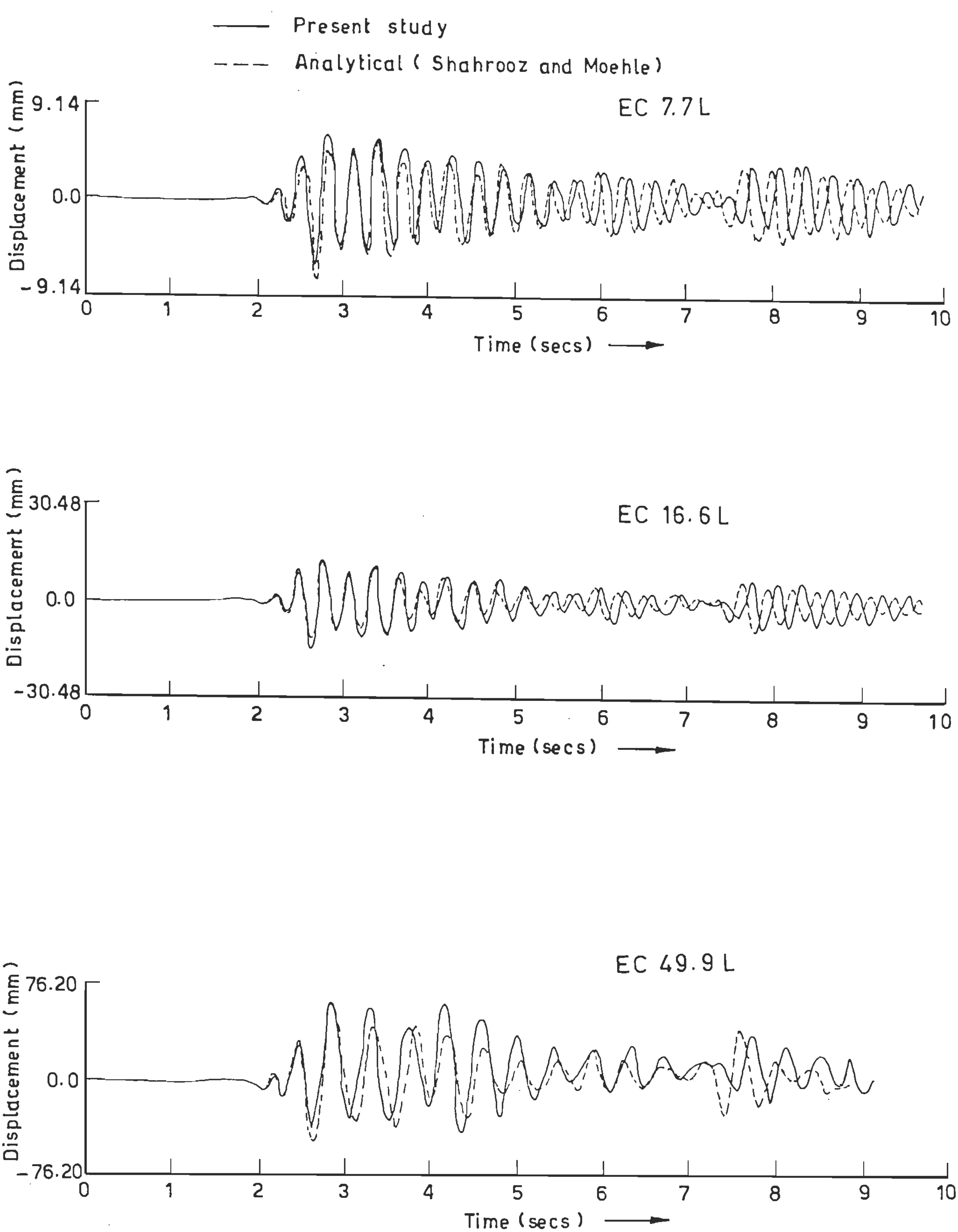


Fig. 4.17 - Comparison of Time History Displacement at Top Floor(Analytical)

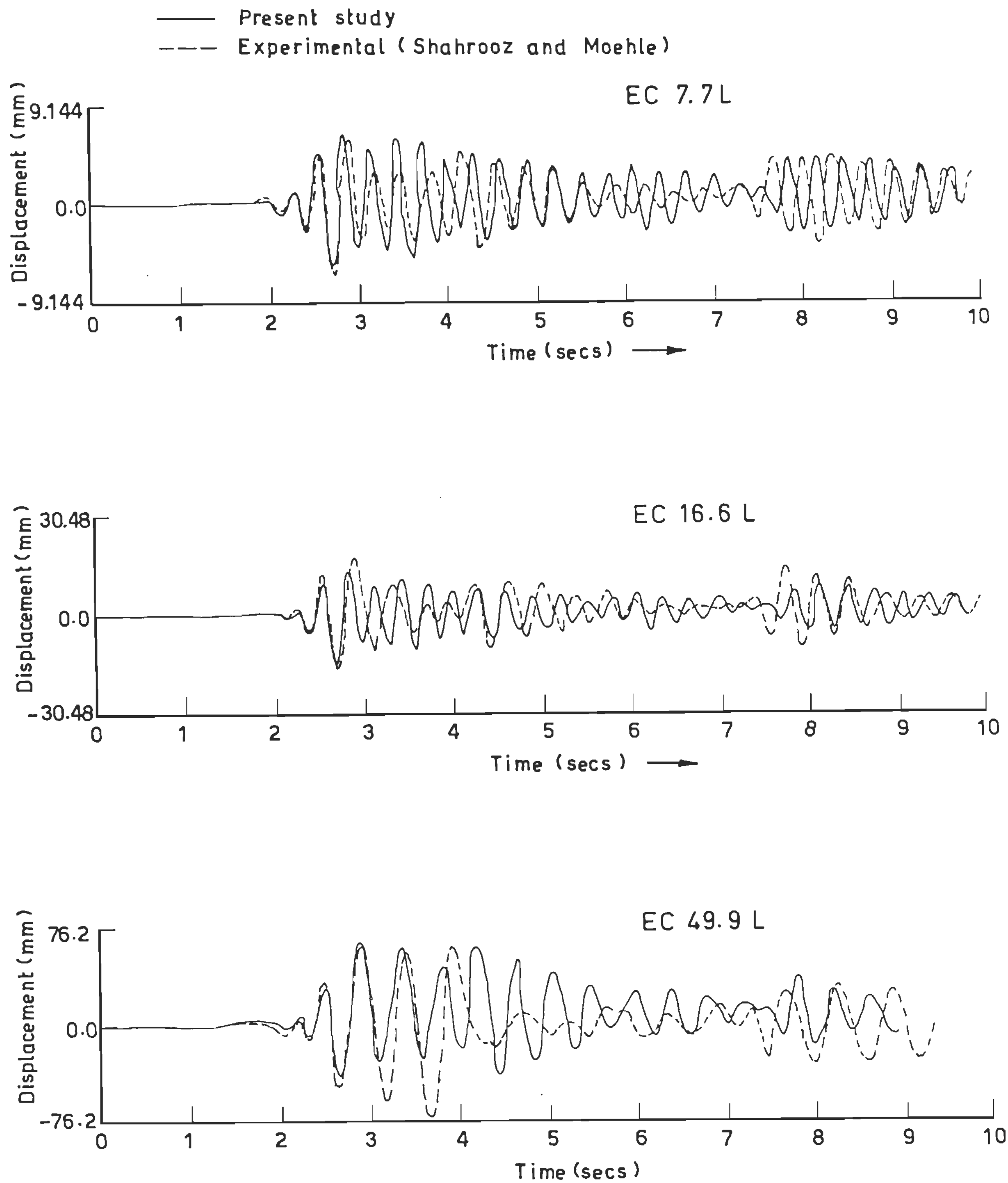


Fig. 4.18 - Comparison of Time History Displacement at Top Floor

reversed direction. A total of four cycles of load has been repeated in this manner. Load deflection curve under the cyclic load with non degrading stiffness is shown in Fig. 4.20(a) and with degrading stiffness model is shown in Fig. 4.20(b). It is observed that the maximum displacement in the non degrading stiffness model is 37.98 mm and in degrading stiffness model is 78.96mm. It is a fact that on unloading the plastic hinge the original stiffness can never be restored in r.c.c. elements and the same contributes to the larger displacements as has been observed in the model.

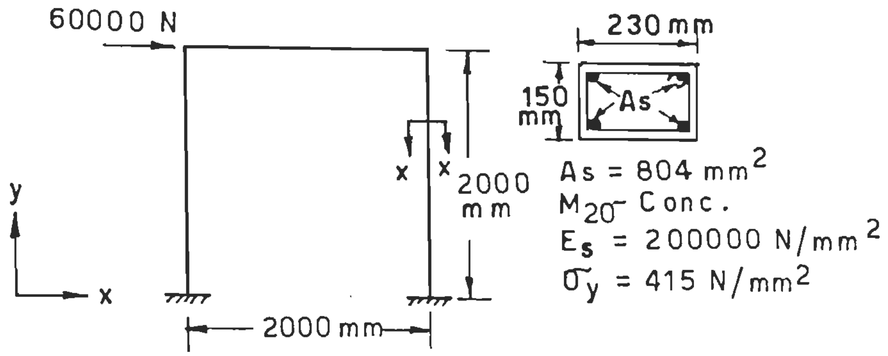
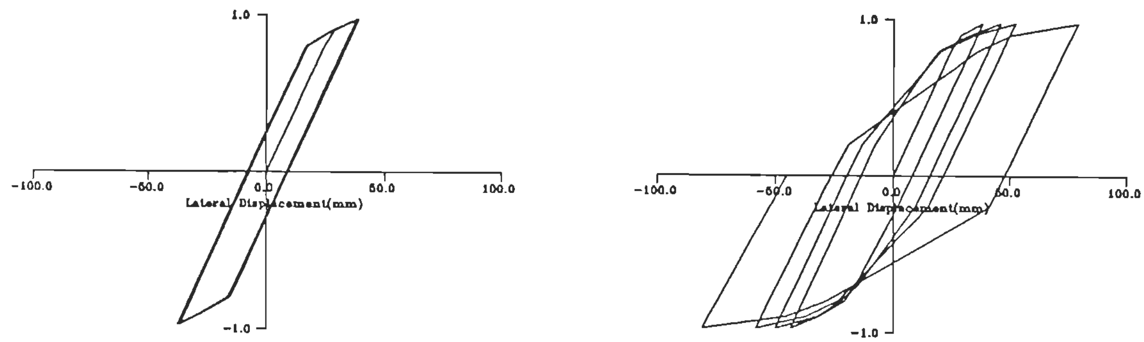


Fig. 4.19 - Two Dimensional R.C. Frame



(a) Non Degrading Stiffness

(b) Degrading Stiffness

Fig. 4.20 - Load Deflection Curve under Cyclic Load

4.10.4 Three Dimensional R.C. Frame

The two different cases of three dimensional r.c. frame under the action of different kinds of loading shown in Figs. 4.21 (a), (b) are analysed using different yield criterion with interaction of (i) axial force, biaxial moment (ii) axial force, biaxial moment, torsional moment (iii) axial force, biaxial moment, torsional moment and shears to demonstrate the importance of using different yield criteria under different conditions of loading on the different type of buildings.

Case(i) - Symmetrical frame subjected to lateral loads
Case(ii) - Unsymmetrical frame subjected to severe torsional moment and lateral loads

The cross sections, reinforcement details, material properties, dimensions of the frame, loads applied are shown in Fig. 4.21 for the two cases. The two frames have been analysed using three different yield criterion as cited above.

Case(i) -This is a symmetrical frame and the various members of the frame are subjected to bending moments and axial forces. The load deflection curve obtained from the analysis using different yield criteria is the same and is shown in Fig. 4.22. It indicates that when the members of the structure are subjected to bending moments and axial forces only then the yield criteria with interaction of axial forces and bending moments is sufficient to predict the inelastic behaviour of the building frame.

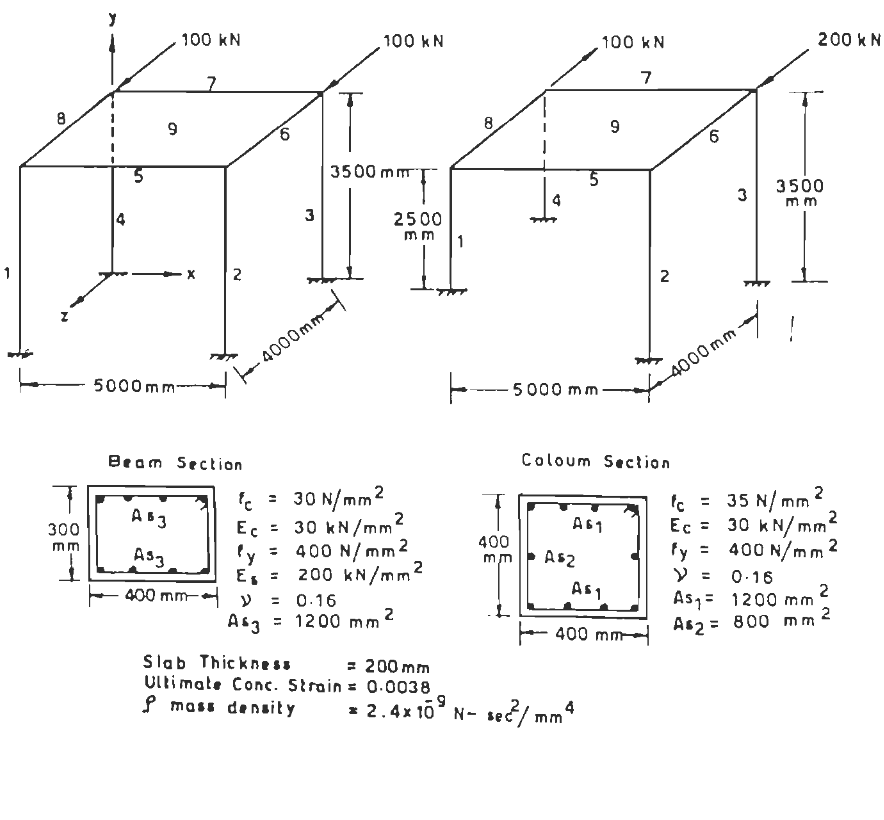


Fig. 4.21 Three Dimensional r.c. Frame

Case(ii)- This is a unsymmetrical frame subjected to heavy torsional moment and shears. The load deflection curve obtained by using three different yield criteria for this case is shown in Fig. 4.23. The failure load obtained from the analysis considering the interaction of axial force, bending moments, torsional moment and shears in the yield criteria is 78.4% of the failure load obtained using yield criteria considering interaction of axial force and bending moments only. The failure load obtained from analysis considering yield criteria with interaction of axial force, bending moments, torsional moment is 80% of the failure load obtained using yield criteria considering interaction of axial load and bending moments.

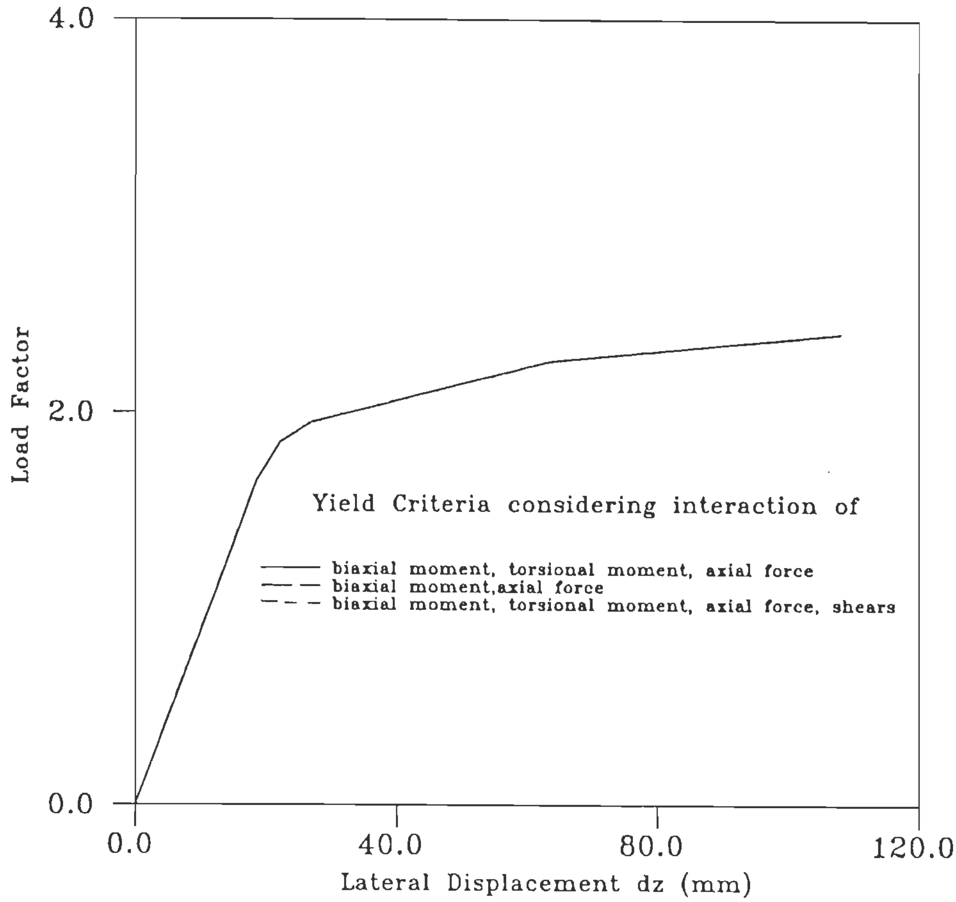


Fig. 4.22 - Load Deflection Curve for Case(i)

The unsymmetrical buildings such as stepback and setback when subjected to lateral loads alongwith gravity loads results in severe torsional moments and lateral shears in the structural components which necessitates the use of yield criteria for realistic inelastic analysis of the building structure considering interaction of axial forces, bending moments, torsional moments and shears.

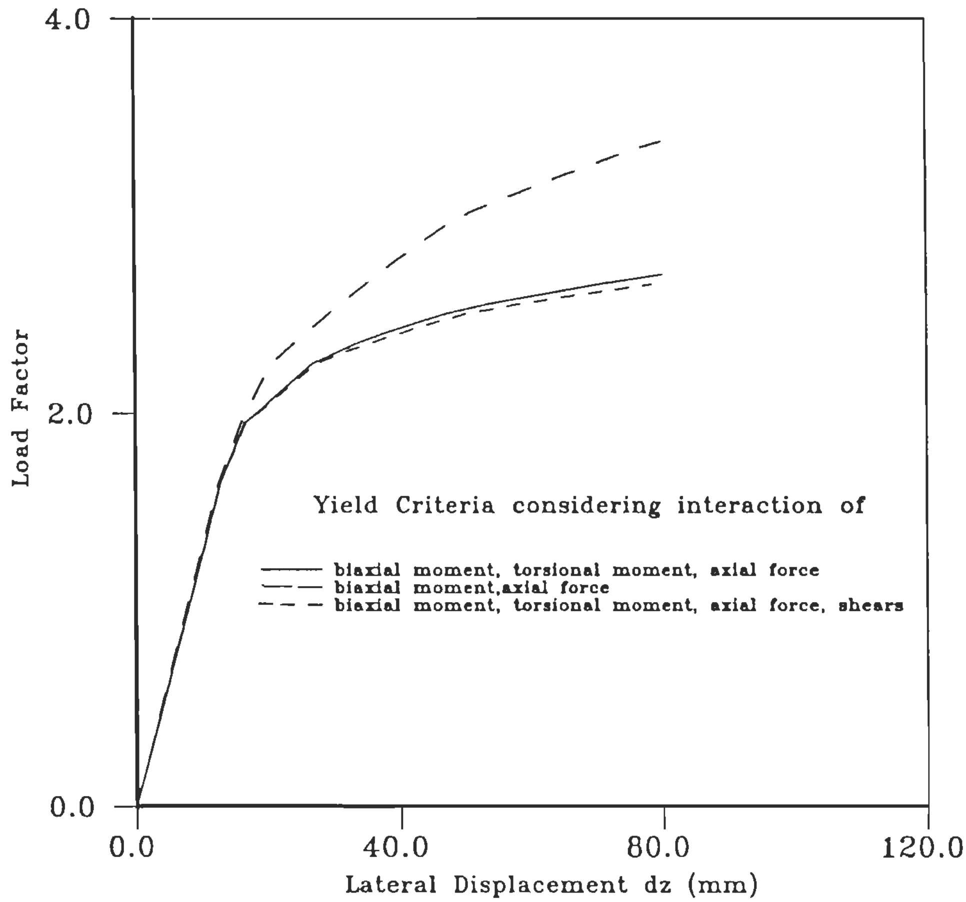


Fig. 4.23 - Load deflection Curve for Case ii

4.11 Concluding Remarks

In the present investigation for complete modelling of the real buildings six types of elements have been used. These are r.c.c. beam/column, r.c.c. slab, r.c.c. panel, brick masonry infill, interface and truss elements. Usual procedure is used to evaluate the stiffness and mass matrices of the elements. Stepback and setback buildings are subjected to severe torsional moment and shears in addition to bending moments and axial forces under the action of earthquake loads. Therefore yield surface for 3D r.c. frame element has been developed considering interaction of biaxial moments, axial force, torsional moment and shears. Degrading stiffness effects have been considered in the evaluation of stiffness matrices for the yielded frame elements. The provision has been made in the program to evaluate the ductility requirement of the yielded beam/column elements. The inelastic behaviour of the r.c.c. panel elements has been considered by taking cracking, crushing and tension stiffening into account. The brick masonry infill is modelled considering cracking and crushing. The inelastic behaviour of the interface element have been modelled considering separation and slippage.

The proposed formulation have been codified and the computer program ISABF in Fortran language have been developed modifying the existing program IBRCF. The developed computer program has the capability to study the static/dynamic, elastic and inelastic behaviour of the regular and irregular buildings. The developed computer program has been validated by comparing the results of present study with the reported experimental and analytical results in the literature.

Inelastic analysis of few frames with brick masonry infill panels subjected to lateral loads has been carried out. The results of present study compares well with experimental and analytical results. One problem of 3 dimensional setback building frame subjected to earthquake load has been analysed. The results of time periods, maximum floor displacements and time history response compares well with the reported experimental results. It warrants that the present modelling and the computer program can be used to predict the earthquake response of the irregular buildings. Two more examples have been analysed to show the effect of stiffness degradation and to show the effect of using different yield criterion. The results show that to get the realistic inelastic behaviour of the irregular buildings the torsional moment and shears are to be considered in the yield criteria.

4.12 References

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To develop a yield surface for any element, ultimate capacities of the member in various directions are required. To calculate the ultimate capacities of the section material properties, stress-strain relation for concrete and steel, section size, amount of steel in the section is needed. The following assumptions have been made to calculate the ultimate capacities of the r.c. sections

1. The stress-strain relation for concrete and steel are closely represented by uniaxial stress strain relations and are independent of the geometry of the cross section.

2. Tensile strength of concrete is negligible.

3. Plane sections before bending remains plane after bending.

4. Bond between steel and concrete is perfect.

A.1 Stress-Strain Relation for Concrete

The stress-strain relation suggested by Medland and Taylor(1971) has been chosen in this study. Taylor's expression is a single continuous function in the form of a fourth degree polynomial valid for both the ascending and descending branches of the stress strain curve as shown in Fig. A. 1.

$$\sigma_C = 0.85 f_C'[Ae_c^4 + Be_c^3 + Ce_c^2 + De_c] \qquad (a.1)$$

where σ_C is the stress in concrete corresponding to strain e_C, f_C' is the ultimate compressive strength of concrete. The constants A, B, C, D are as given by Medland and Taylor are

$$A = 0.292E+10,\ B = 0.1583E+08,\ C = -0.3229E+06,\ D = 1.0593E+03 \qquad (a.2)$$

A.2 Stress-Strain Relation for Steel

A bilinear idealization of the stress-strain diagram for mild and intermediate steel is widely accepted. The initial elastic part extends up to the yield stress, followed by strain hardening part extending up to failure.

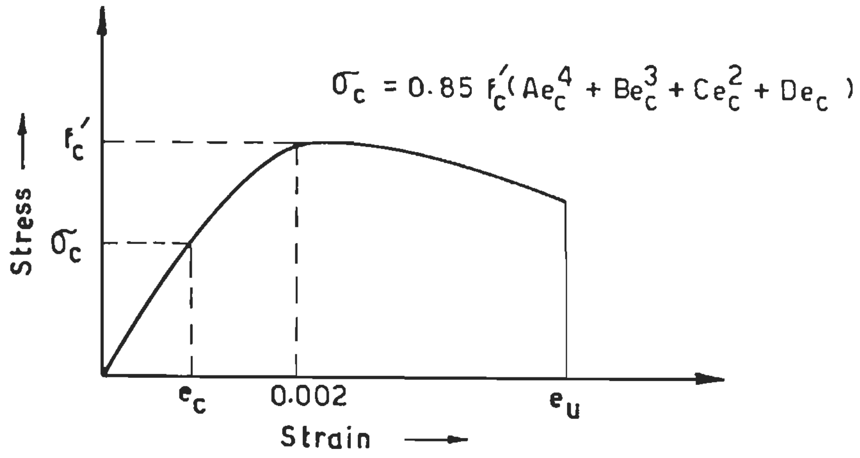

Fig. A.1 - Stress-Strain Relation for Concrete [Taylor(1970)]

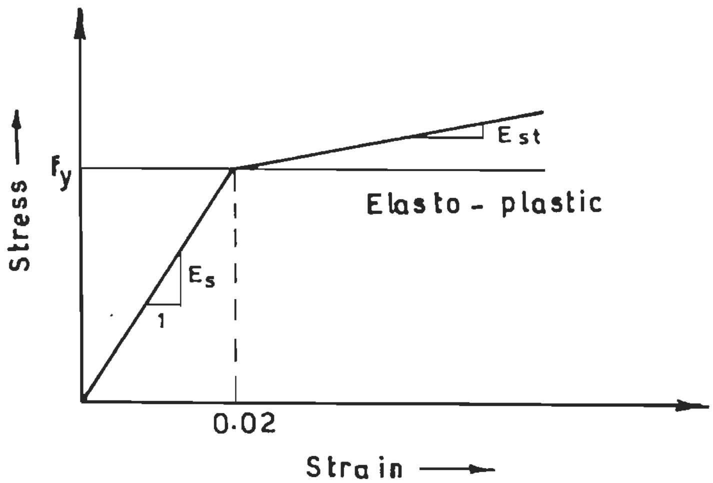

Fig. A.2 - Idealized Stress-Strain Curve for Steel

A.3 Analysis of R.C. Section

A r.c. member with cross section (bxd) as shown in Fig. A.3 is reinforced with 'n' no. of steel layers from top to bottom and the member is subjected to axial load P and moment M. The cross section, strain diagram, stress diagrams are shown in Fig A.3. The equilibrium equation of force and moment in the section at any stage of loading may be written as

$$P = \int_{A_c} b\, \sigma_c \, dy + \Sigma A_{si}\, \sigma_{si} \tag{a.3}$$

$$M + P(kd-d/2) = \int_{A_c} b\, \sigma_c \, y\, dy + \Sigma A_{si}\, \sigma_{si}\, (k-\alpha_i)d_i \tag{a.4}$$

where σ_c is the stress in concrete at a distance y from neutral axis and Ac is the area of concrete under compression. The above equilibrium equations can be written in non dimensional form as

$$p = 1/\phi \int_{e_c} \frac{\sigma_c}{f_c'} de_c + \sum_{i=1}^{n} \rho_{si}\sigma_{si} / f_c' \tag{a.5}$$

$$m = 1/\phi^2 \int_{e_c} \frac{\sigma_c}{f_c'} e_c de_c + 1/\phi \sum_{i=1}^{n} p_{si} \frac{\sigma_{si}}{f_c'} e_{si} - p(\frac{e_t}{\phi} - 0.5) \tag{a.6}$$

the p, m and ϕ have been referred to as the axial load, moment and the curvature respectively.

A.4 Axial Load Moment Interaction Curve

The two Eqns(a.5 and a.6) contains four independent variables p, m , ϕ, e_t. If two of these are known the other two can be evaluated easily. Assuming e_t equal to the ultimate concrete strain in compression e_u and by changing the value of ϕ, a set of points(m_u, p_u) is obtained. The ultimate axial force, moment curve for a given r.c. section can be generated as under.

1. Calculate the ultimate axial compression p_o and tensile load p_t for the section.

$$p_o = 1 + \rho_t (f_y - f_c')/ f_c' \tag{a.7}$$

$$p_t = \rho_t f_y / f_c' \tag{a.8}$$

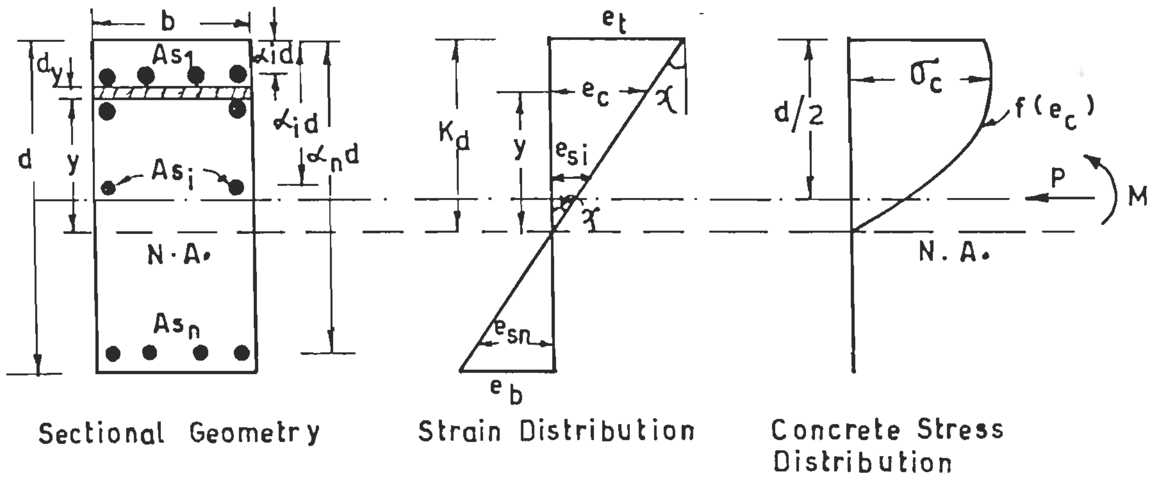

Fig. A.3 - Section Geometry and Possible Stress-Strain Distribution

where $p_o = Po/bd f_c'$ and ρ_t is the total steel ratio $(\sum_{i=1}^{n} Asi / bd)$.

2. calculate balanced axial load P_{ub}, which is the axial load when the extreme fibre concrete strain attains the ultimate compressive strain e_u simultaneously with bottom most steel layer yielding. The axial force P_{ub} may be calculated using the values

$$e_t = e_u \text{ and } \phi = (e_t + e_b)/\alpha_n \tag{a.9}$$

where α_n is the dimensionless depth of nth steel layer.

3. Calculate the ultimate moment capacity m_o of the section in the absence of axial load. Since ϕ is unknown, the following trial and error procedure is adopted.

Assuming a trial value for ϕ equal to e_t, calculate the axial load as p_c. The convergence is tested by comparing the calculated axial load p_c with the specified tolerance ε_t. If the convergence is not satisfied, a new value of ϕ is estimated by Newton- Raphson's method.

4. To get a set of points in compression failure region(between p_o and p_{ub}) ϕ is decreased below the ϕ balanced by a specified amount. The limit of this region is assumed to range between P_{ub} and 0.85Po.

5. For tension region, ϕ is increased by a specified amount above ϕ balanced and again a set of points is obtained in the region. The maximum value of ϕ used in this study is equal to $\phi_{max} = e_t/\alpha_1$, where α_1 is the distance between the first layer and top edge of the section.

A.5 Mathematical Expression for 2 D Yield Function

The following non dimensional equation is obtained by fitting a third degree polynomial to calculate set of points by least square method.

$$\frac{m_{up}}{m_o} = a_1 + a_2 \frac{p_u}{p_o} + a_3 \left[\frac{p_u}{p_o}\right]^2 + a_4 \left[\frac{p_u}{p_o}\right]^3 \qquad \text{(a.10)}$$

where m_{up} is the moment corresponding to the axial load p_u and a_1, a_2, a_3 and a_4 are the polynomial constants to be determined by least square method.

Then $f(p,m,p_o,m_o,a_1, a_2, a_3, a_4) = 1.0$ represents the relationship which defines the ultimate capacity of a reinforced concrete section under different combination of axial forces and moments.

Chapter 5

STABILITY ANALYSIS OF SLOPES WITH BUILDING LOADS

5.1 Introduction

In hill areas, landslides are frequent and hazardous. To ascertain the stability of slope, stability analysis is required. In developing hill areas many multistoreyed r.c.c. framed buildings are constructed on hill slope. The building loads are transferred to the hill slope terrain at the foundation level, which may cause hill slope failure. Therefore the stability of hill slope with building loads must be checked. The literature available on stability analysis of slope is briefly reviewed here. In this Chapter a procedure has been developed to find the factor of safety against sliding failure of slope considering building loads transferred to the slope. The computer program has been developed based on the formulation developed and is validated by solving few problems. A parameteric study has been carried out to see the effect on the factor of safety due to the variation in location of loads, distance between loads, relative difference in level of footings transferring the loads.

5.2 Literature Review

Meyerhof *et al.*(1957) has extended the theory of bearing capacity of foundation on level ground to the bearing capacity of foundation on the slopes. The theory has indicated that the bearing capacity of foundations on the face of slope or near the top edge of a slope decreases with greater inclination of the slope, especially for cohesion less soils, at greater distance from the edge of a slope, the bearing capacity becomes independent of the slope angle. The theory also indicates that the bearing capacity of foundations on the top of clay slopes decreases considerably with greater height of the slope and is frequently governed by overall slope failure.

Shields *et al.*(1975) reported in his paper that for depths greater than the footing width B, the bearing capacity was found to be constant provided the footings were located at a certain distance away from the slope. For D = B, the distance was 4 or 5B. Where D is the depth of footing from the earth surface.

Fredlund and Kralin(1977) compared six methods of slices commonly used for slope stability analysis. The method of handling the inter slice forces differentiates the normal force equation. A new derivation for the Morgenstern-Price method is presented and is called the best fit regression solution. The best fit regression solution gives the same factor of safety as the Newton-Raphson solution.

Spencer(1978) derived equations that satisfy the equilibrium conditions for both forces and moments for earth slopes to lateral acceleration. The equations are applicable to any shape of failure. Two methods, one uses the factor of safety as the variable and the other uses the critical value of the acceleration. It is shown that logarithmic spiral failure is not more critical than a circular arc, Stability is reduced if acceleration has a downward component. Taking the factor of safety and the acceleration factor as alternative criteria, the critical slip circles are fairly close.

Zangl(1978) shows that the actual stability is smaller than the calculated using the principles of mechanics. Deficiencies in the conventional methods are pointed out and it is shown that the degree of accuracy in stability calculations can be assessed more reliably. A technique is described which makes it possible to determine the upper bound for the stability coefficients with reasonable amount of calculations.

Chen(1980) presented a summary of the current advances in the applications of the theory of the plasticity in soil mechanics. A detailed description of three basic subjects (i) Idealised stress-strain models for soil, (ii) limit analysis for collapse loads, (iii) finite element analysis for progressive failure behaviour of soil mass, with particular emphasis on seismic stability of slopes by limit analysis has been presented. Some of the inter relationship between various plasticity models are demonstrated and their relative merits and limitations are evaluated with in the context of their use in the numerical analysis of soil mass involving slope failures and land slides during earthquake loading.

Tavenas *et al.***(1980)** presented a critical review of the analytical methods of slope stability analysis. The effective stress method of analysis suffer from fundamental weaknesses related to the magnitude of the effective stresses, the assumed stress path, the indiscriminate use of a Mohr-Columb criterion, definition of the factor of safety. Practical consequences as well as alternative solutions for the analysis of stability of slopes are discussed.

Spencer(1981) evaluated effective stress at the bottom of vertical inter slice boundaries along slip circles through an earth slope assuming linear stress distribution. the slope of the critical shear planes and the factor of safety on these planes are determined. The values are compared with the slope of the slip circle and the factor of safety against failure on the slip surface. The correlation between the values is found to be poor for arbitrary chosen slip circles but good for critical circles.

Chugh(1982) extended methods of slices commonly used for estimating static stability of natural slopes to include the dynamic effects due to earthquake loading. The calculations of displacement of a slide mass using the Newmark procedure are discussed.

Huang(1983) used the simplified Bishop's method to determine the factor of safety. It is assumed that the forces on the sides of each slice are in a horizontal direction. This assumption implies that there is no friction between two slices. A computer program based on this method has been developed in Fortran and Basic. It has got many features similiar to the ICES-LEASE computer program of [Bailey and Christan(1969)], however some of the features has been added to save the computer time and ensure more accurate solutions.

Prabhakar *et al.*(1982) proposed mathematical technique for analysing the stability in terms of effective stresses. Stability equations are derived based on limiting equilibrium conditions. The factor of safety is obtained in relation to the shear strength. Critical slip surface associated with the minimum factor of safety is obtained.

Daddazio(1987) described procedures for performing nonlinear dynamic slope stability analysis. This method combines updated Lagrangian Kinematic relation with a cap type strain hardening soil plasticity model in an explicit nonlinear dynamic finite element formulation. Detailed analysis of the progressive failure of an embankment is performed and comparisons with conventional methods for seismic slope stability analysis are presented. The results presented indicated the applicability and accuracy of the proposed model.

The stability analysis of free slopes has been carried out extensively[Morgenstern and Price(1965), Peck(1967), Whitman and Bailey(1967), Spencer(1967,1973), Huang and Avery(1976), Huang(1977)] using limit plastic equilibrium methods.

5.3 Stability Analysis of Free Slopes

Free slopes means that there are no superimposed loads on the slope such as due to the buildings or any other extra loads. All stability analysis are based on the concept of plastic limit equilibrium. In this a failure surface is assumed, a state of limiting equilibrium is said to exist when the shear stress along the failure surface is expressed as

$$\tau = S/F \qquad (5.1)$$

where τ is the shear stress, S is the shear strength and F is the factor of safety. In most methods of limit plastic equilibrium, concept of statics is applied. Most of the problems in slope stability analysis are statically indeterminate, some simplifying assumptions are to be made in order to determine a unique factor of safety. A variety of methods are available in the literature with different simplifying assumptions such as Fellienius, Bishop, Janbu, Morgenstern and Price, Spencer methods. The most practical methods which can be readily used by the pracitising engineers in the field are Wedge, Fellienius and simplified Bishop's method[Huang(1983), Terzaghi and Peck(1967), Fredland and Kralin(1977)]. The slope stability analysis has been carried out by finite element method also[Wong(1984)]. In the present investigation simplified Bishop's method has been used for slope stability analysis.

5.3.1 Types of Failure Surfaces

The shape of the failure surface may be quite irregular depending on the homogeneity of the material in the slope. This is particularly true in natural slopes where the relic joints and fractures dictates the locus of failure surfaces. If the material is homogeneous and a large circle can be formed, the most critical failure surface will be cylindrical because a circle has the least surface area per unit mass. If a large circle can not be developed such as in the case of an infinite slope with depth much smaller than length, the most critical failure surface will be a plane parallel to the slope. If some planes of weaknesses exist, the most critical failure surface will be a series of planes passing through the weak strata. In such cases a combination of plane, cylindrical and other irregular failure surfaces may also exist. In the present investigation the cylindrical failure surface has been taken.

5.3.2 Cylindrical Failure Surfaces

The minimum factor of safety for a cylindrical failure surface can be found by taking large number of circles to determine the most critical failure surface. Figure 5.1 shows the failure surface for which factor of safety is to be determined. The sliding mass is divided into 'n' slices. The ith slice has a weight of w_i. A length of failure surface L_i, an angle of inclination θ_i and a normal force N_i. The factor of safety is a ratio between the resisting force and the driving force. According to Mohr-Coulomb theory the resisting force in slice i is ($cL_i+N_i \tan \phi$), where c is the cohesion of the soil, L_i is the length of the slice, ϕ is the angle of internal friction of the soil. Note that N_i depends on the forces on the two sides of the slice and is statically indeterminate unless some simplifying assumptions are made. The driving force is equal to $w_i \sin\theta_i$, which is the component of the weight along the failure surface, where w_i is the weight of the slice i. The driving force is independent of the forces on the two sides of the slice because when ever there is a force on one side of the slice, there is a corresponding force, equal in magnitude but opposite in direction on the adjacent side, thus neutralizing their effect. The factor of safety can be determined by

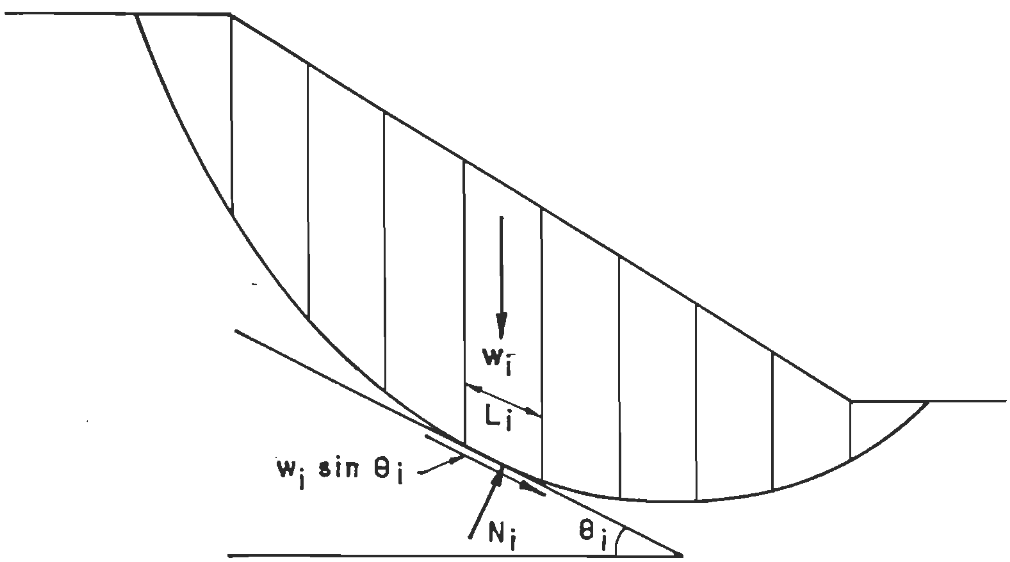

Fig. 5.1 Cylindrical Failure Surface

$$F = \frac{\sum_{i=1}^{n} (cL_i + N_i \tan\phi_i)}{\sum_{i=1}^{n} w_i Sin\theta_i} \tag{5.2}$$

where $N_i = w_i \text{ Cos } \theta_i$ (5.3)

$$\text{Therefore,} \quad F = \frac{\sum_{i=1}^{n} (cL_i + w_i Cos\theta_i \tan\phi)}{\sum_{i=1}^{n} w_i Sin\theta_i} \tag{5.4}$$

The Eqn.(5.3) does not include the effects of pore pressure. If there is any pore pressure, It can be represented by a phreatic surface. Then the effective normal force $\overline{N}_i$ is equal to the total normal force N_i minus the neutral force $\gamma_W h_{iw} b_i \sec \theta_i$. The factor of safety is then given by

$$F = \frac{\sum_{i=1}^{n}(cL_i + \overline{N}_i \tan\phi)}{\sum_{i=1}^{n} w_i Sin\theta_i} \tag{5.5}$$

where b_i is the width of slice i for the pore pressure, h_{iw} height of pheratic surface from the failure surface at slice i, and γ_w is the density of water

$$\text{or} \quad \overline{N}_i = (1-r_u)\, w_i \, Cos\, \theta_i \tag{5.6}$$

in which r_u is the pore pressure ratio, Substituting (5.6) into (5.5) then

$$F = \frac{\sum_{i=1}^{n}(cL_i + (1-r_u)w_i Cos\theta_i \tan\phi)}{\sum_{i=1}^{n} w_i Sin\theta_i} \tag{5.7}$$

To take into account the earthquake forces, a horizontal seismic force is applied at the centroid of each slice as shown in Fig. 5.2. The seismic force is equal to $c_s w_i$ where c_s is seismic coefficient and its value depends upon the seismicity of the location. It is assumed that this force has no effect on the normal force N_i, only the driving force is affected. The factor of safety is determined by

$$F = \frac{\sum_{i=1}^{n}(cb_i Sec\theta_i + N_i \tan\phi)}{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R)} \tag{5.8}$$

In which b_i is the width of slice, a_i is the moment arm and R is the radius of the slip circle. The factor of safety with pore pressure and seismic force effects is given by

$$F = \frac{\sum_{i=1}^{n}(cb_i Sec\theta_i + (1-r_u)w_i Cos\theta_i \tan\phi)}{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R)} \tag{5.9}$$

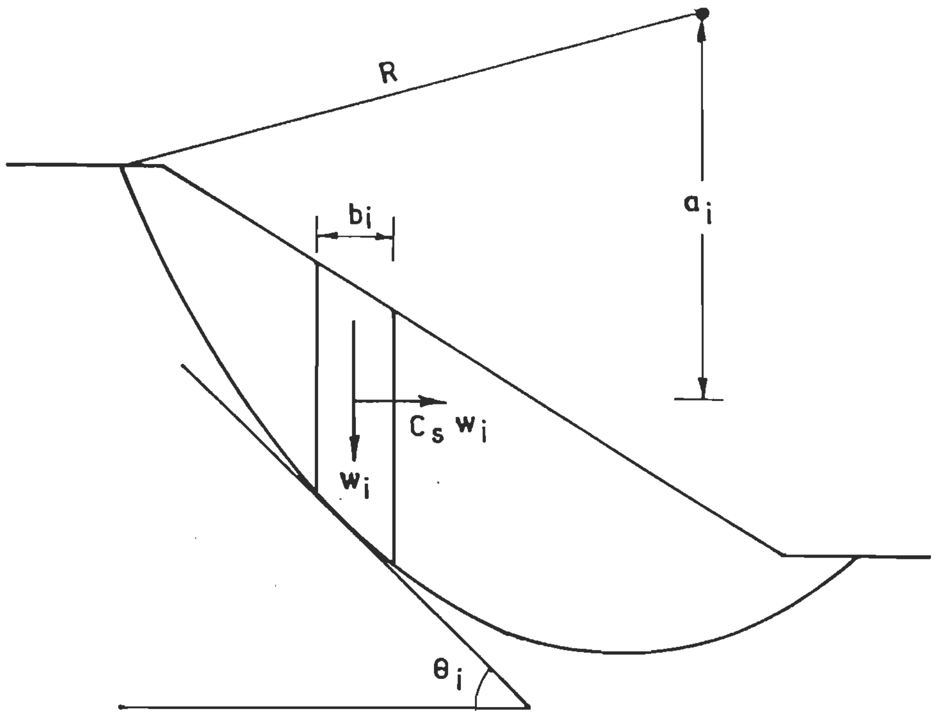

Fig. 5.2 Driving Force due to Seismic Effect [Huang(1983)]

5.3.3 Simplified Bishop's Method of Stability Analysis

In the simplified Bishop's method it has been assumed that the forces on the sides of each slice are in horizontal direction indicating that there is no friction between the slices. The forces acting on the ith slice in the simplified Bishop's method are shown in Fig. 5.3.

Equilibrium of forces in vertical direction gives

$$N_i Cos\theta_i + \gamma_w h_{iw} b_i + (cb_i Sec\,\theta_i + N_i \tan\phi) Sin\,\theta_i / F - \gamma_i h_i b_i = 0.0 \tag{5.10}$$

$$\text{or} \quad N_i = \frac{b_i(\gamma h_i - \gamma_w h_{iw}) - (cb_i \tan\theta_i)/F}{\cos\theta_i + (\sin\theta_i \tan\phi)/F} \tag{5.11}$$

Substituting value of N_i in (5.8) then

$$F = \frac{\sum_{i=1}^{n} \dfrac{cb_i + b_i(\gamma h_i - \gamma_w h_{iw})\tan\phi}{\cos\theta_i + (\sin\theta_i \tan\phi)/F}}{\sum_{i=1}^{n}(w_i \sin\theta_i + c_s w_i a_i / R)} \tag{5.12}$$

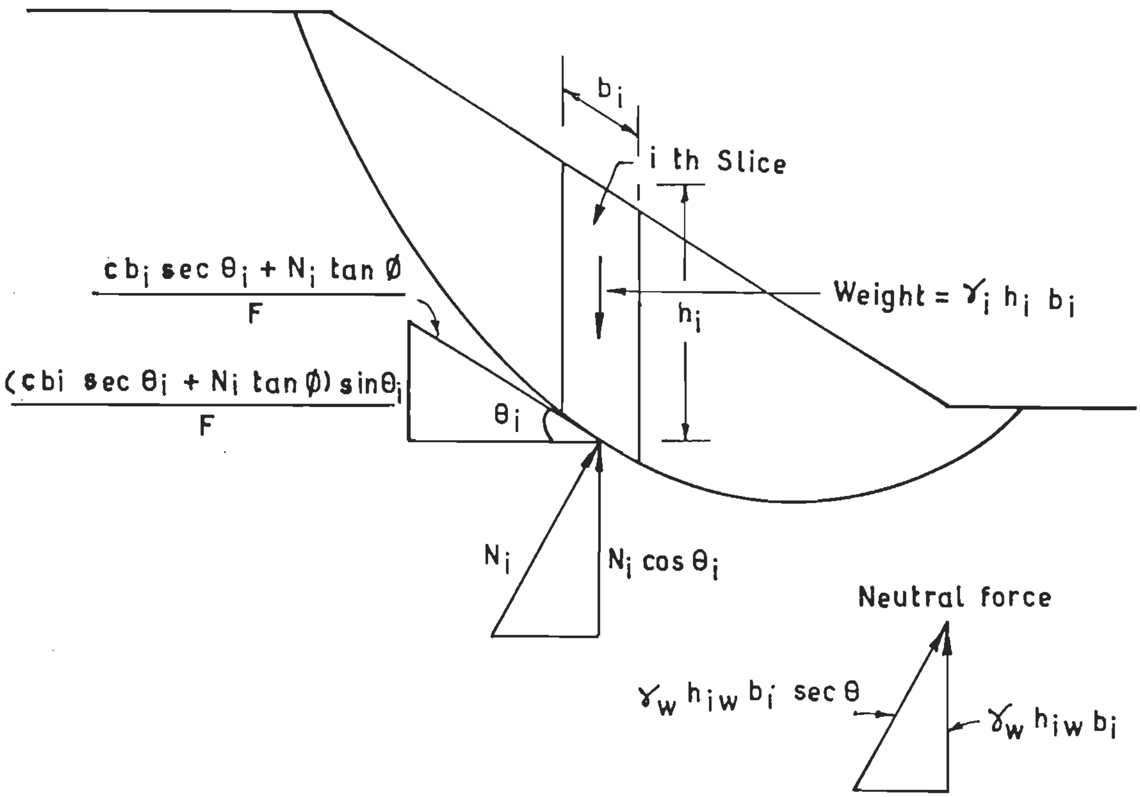

Fig. 5.3 - Stability Analysis by Simplified Bishop's Method

In terms of pore pressure ratio the Eqn. (5.12) can be written as

$$F = \frac{\sum_{i=1}^{n} \dfrac{cb_i + (1 - r_u)\gamma h_i b_i \tan\phi}{\cos\theta_i + (\sin\theta_i \tan\phi)/F}}{\sum_{i=1}^{n}(w_i \sin\theta_i + c_s w_i a_i / R)} \tag{5.13}$$

The factor of safety F appears on both sides of equation. Therefore Newton's method of tangents has been used to find out the factor of safety by an iterative process.

$$f(F) = F\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) - \sum_{i=1}^{n}\frac{cb_i + (1-r_u)\gamma h_i b_i \tan\phi}{Cos\theta_i + (Sin\theta_i \tan\phi)/F} = 0.0 \tag{5.14}$$

The intersection F_{m+1} of the tangent to the curve f(F) at $F=F_m$ is given by

$$F_{m+1} = F_m - f(F_m)/f'(F_m) \tag{5.15}$$

In which $f'(F_m)$ is the first derivative of f with respect to F. Then Eqns. (5.14) and (5.15) in combined form becomes

$$F_{m+1} = F_m\{1 - \frac{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) - \sum_{i=1}^{n}\frac{cb_i + (1-r_u)\gamma h_i b_i \tan\phi}{F_m Cos\theta_i + Sin\theta_i \tan\phi}}{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) - \sum_{i=1}^{n}\frac{(cb_i + (1-r_u)\gamma h_i b_i \tan\phi) Sin\theta_i \tan\phi}{(F_m Cos\theta_i + Sin\theta_i \tan\phi)^2}}\} \tag{5.16}$$

The first trial value of F_m is to be taken from (5.8). Then Eqn. (5.16) converges rapidly within 2 or 3 iterations.

5.4 Stability Analysis of Slopes with Building Loads

In hill areas r.c.c. framed buildings are constructed on slope itself thereby the slope is subjected to additional loads due to buildings which are transferred to the sloping ground at the foundation level. The Eqn. (5.16) gives the factor of safety of free slopes(i.e. without building loads). The building loads transferred at the foundation level are the vertical loads, horizontal loads and the bending moments. The buildings are three dimensional structures transmitting the loads at the foundation level in the 3D space. The analysis of the slope in the 3D space is required to be carried out. The stability analysis in the 3D space is cumbersome process. Generally r.c.c. framed buildings consists of frames one after the other. To simplify the problem two dimensional stability analysis has been carried out taking the loads from various columns of one frame in 2D space at a time.

These additional loads are to be taken into account while finding out the factor of safety of slope with buildings constructed on them. A hill slope with building frame is shown in Fig. 5.4, the loads from the building frame are transferred at A, B and C.

The building loads transferred at the foundation level are shown in Fig. 5.5. Let the dimension of the foundation be L x B and co-ordinates of the column point are X_c and Y_c in x and y directions respectively. Then area of foundation is A = L x B. Section modulus of foundation is $Z = B \times L^2/6$. Let the stress at extreme left end and extreme right end of the foundation due to vertical load P_1 and bending moment M_1 be σ_1 and σ_2 and are expressed as

$$\sigma_1 = \frac{P_1}{A} + \frac{M_1}{Z} \tag{5.17}$$

$$\sigma_2 = \frac{P_1}{A} - \frac{M_1}{Z} \tag{5.18}$$

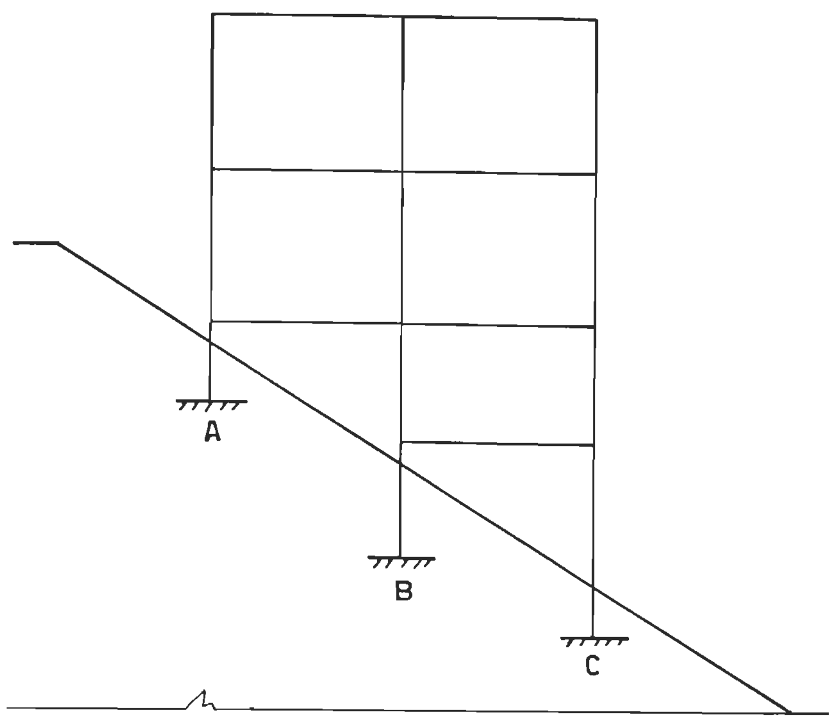

Fig. 5.4 Hill Slope With A Typical Building Frame

The stress distribution will vary from σ_1 to σ_2 from one edge of footing to the other edge of the footing. Let the total length of footing be divided into 'n_C' number of slices. Then width of each slice will be $B_s = L/n_C$. Co ordinate of two extreme ends of the Footing are (X_C - l/2, Y_C) and (X_C + l/2, Y_C). Now take slice 1 of the column load slices. Stress at one end of the slice = σ_1 and stress at other end of the slice = $\sigma_1 + \dfrac{(\sigma_2 - \sigma_1)B_s}{L}$

and let w_j is the vertical load corresponding to slice 1 of the column load and is given as

$$w_j = \frac{(\sigma_1 + (\sigma_1 + \{\sigma_2 - \sigma_1\}B_s / L\})B_s}{2} \tag{5.19}$$

and distance of c.g. of slice load from left end of the slice is given as

$$\frac{B_s(3\sigma_1 + 2\{\sigma_2 - \sigma_1\}B_s / L\})}{3(2\sigma_1 + \{\sigma_2 - \sigma_1\}B_s / L\})} \tag{5.20}$$

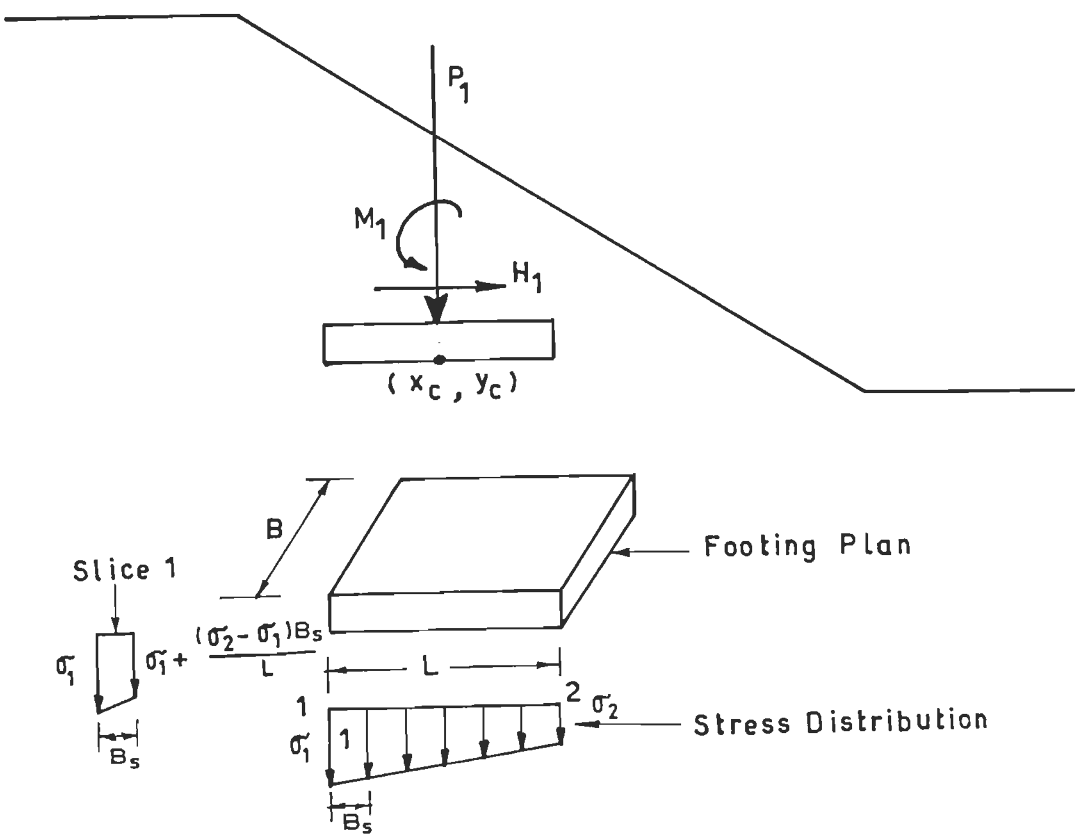

Fig. 5.5 - Building Loads Transmitted at Hill Slope

In this manner the loads of all slices of footing loads and coordinates of c.g. of slices loads are to be evaluated for the failure surface and are to be accounted for in calculating factor of safety as under.

The Eqn. (5.11) gives N_i for the soil slices weights and no building loads are considered. To consider the building loads, the N_i value should take account of the loads coming from the buildings. The building loads and the slices are shown in Fig. 5.6. Let w_j is the vertical load of slice j corresponding to column loads and N_j is the normal component of force due to slice j and θ_j is the angle of inclination of the normal to the vertical, then equilibrium of vertical forces for the jth slice corresponding to column loads

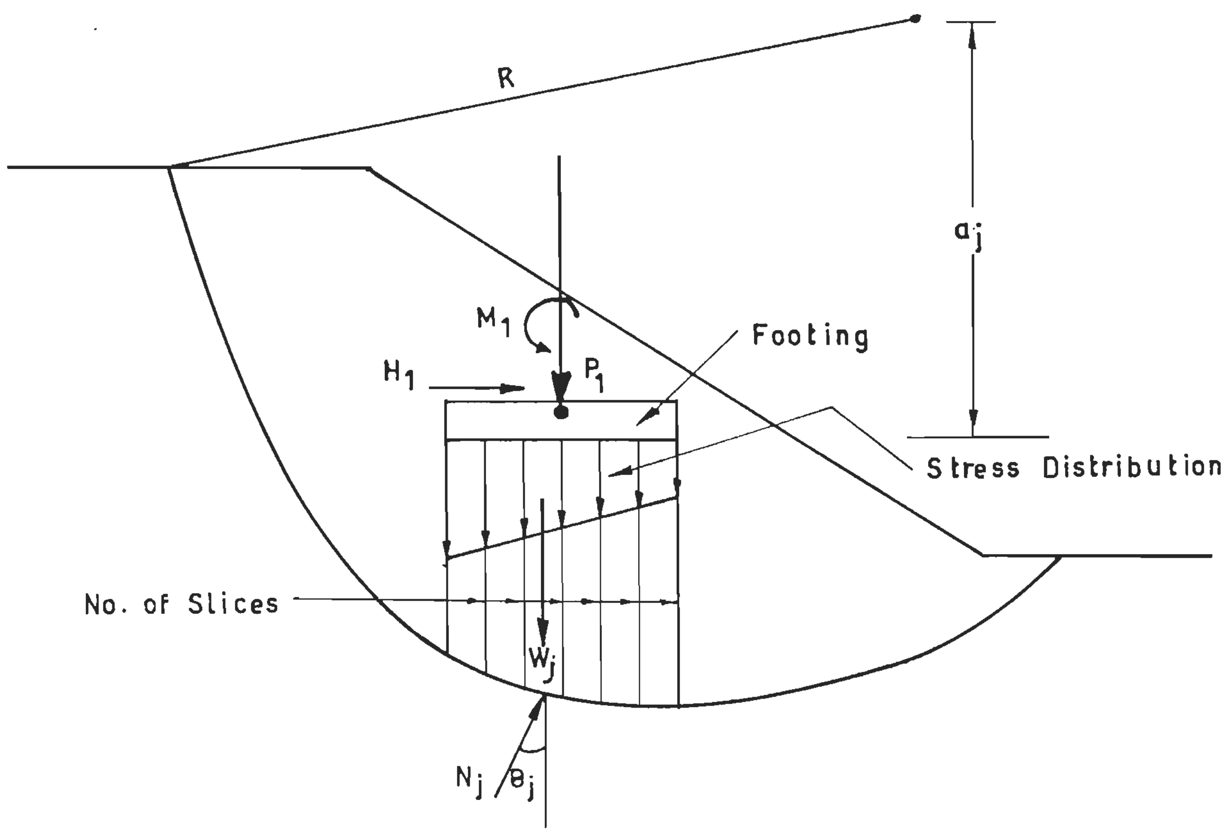

Fig. 5.6 - Building Loads and Slices of the Load

$$N_j Cos\theta_j + N_j \tan\phi \; Sin\theta_j / F - w_j = 0.0 \tag{5.21}$$

therefore, $N_j = \dfrac{w_j}{Cos\theta_j + tan\phi Sin\theta_j / F}$ (5.22)

The driving force corresponding to the building loads is

$$\sum_{j=1}^{n_c}(w_j Sin\,\theta_j + H_j a_i / R) \tag{5.23}$$

where H_j is horizontal load corresponding to jth slice of horizontal column load and a_j is the level arm. Then the factor of safety F for the slope with building load is given by

$$F = \frac{\sum_{i=1}^{n}\dfrac{cb_i + (1-r_u)\gamma h_i b_i\, tan\phi}{Cos\theta_i + (Sin\theta_i\, tan\phi)/F} + \sum_{j=1}^{n_c}\dfrac{w_j\, tan\phi}{Cos\theta_j + (Sin\theta_j\, tan\phi)/F}}{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) + \sum_{j=1}^{n_c}(w_j Sin\theta_j + H_j a_j / R)} \tag{5.24}$$

The factor of safety F comes on both sides of the expression, therefore iterative solution procedure such as Newton Raphson's is adopted.

$$\begin{aligned} f(F) &= F[\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) + \sum_{i=1}^{n_c}(w_j Sin\theta_j + H_j a_j / R)] \\ &-F[\sum_{i=1}^{n}(\frac{cb_i + (1-r_u)\gamma h_i b_i\, tan\phi)}{FCos\theta_i + (Sin\theta_i\, tan\phi)}) + \sum_{j=1}^{n_c}(\frac{w_j\, tan\phi}{FCos\theta_j + tan\phi\, sin\theta_j})] \end{aligned} \tag{5.25}$$

$$\begin{aligned} f'(F) &= [\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) + \sum_{i=1}^{n_c}(w_j Sin\theta_j + H_j a_j / R)] \\ &+[\sum_{i=1}^{n}(\frac{cb_i + (1-r_u)\gamma h_i b_i\, tan\phi)Sin\theta_i\, tan\phi}{(FCos\theta_i + (Sin\theta_i\, tan\phi))^2}) + \sum_{j=1}^{n_c}(\frac{w_j\, tan\phi(tan\phi Sin\theta_j)}{FCos\theta_j + tan\phi\, sin\theta_j})] \end{aligned} \quad 5.26)$$

$$F_{m+1} = F_m - f(F_m)/f'(F_m) \tag{5.27}$$

The final expression is

$$F_{m+1} = F_m(1 - \frac{A-B}{A-C}) \tag{5.28}$$

$$A = [\sum_{i=1}^{n}(w_i Sin\theta_i + C_s w_i a_i / R) + \sum_{i=1}^{n_c}(w_j Sin\theta_j + H_j a_j / R)] \quad (5.29)$$

$$B = [\sum_{i=1}^{n}(\frac{cb_i + (1-r_u)\gamma h_i b_i \tan\phi)}{(F_m Cos\theta_i + (Sin\theta_i \tan\phi))}) + \sum_{j=1}^{n_c}(\frac{w_j \tan\phi}{F_m Cos\theta_j + \tan\phi \sin\theta_j})] \quad (5.30)$$

$$C = [\sum_{i=1}^{n}(\frac{cb_i + (1-r_u)\gamma h_i b_i \tan\phi)Sin\theta_i \tan\phi}{(F_m Cos\theta_i + (Sin\theta_i \tan\phi))^2}) + \sum_{j=1}^{n_c}(\frac{w_j \tan\phi(\tan\phi Sin\theta_j)}{F_m Cos\theta_j + \tan\phi \sin\theta_j})] \quad (5.31)$$

In the above expression w_j, H_j, a_j, $Sin\theta_j$, $Cos\theta_j$ are corresponding to the building column load which has been divided into n_C slices and these are corresponding to jth slice. Simplified normal method formula for obtaining the factor of safety is

$$F = \frac{\sum_{i=1}^{n}(cb_i Sec\theta_i + (1-r_u)w_i Cos\theta_i \tan\phi) + \sum_{j=1}^{n_c} w_j Cos\theta_j \tan\phi}{\sum_{i=1}^{n}(w_i Sin\theta_i + c_s w_i a_i / R) + \sum_{j=1}^{c^C}(w_j Sin\theta_j + H_j a_j / R)} \quad (5.32)$$

Initially the factor of safety is evaluated from the simplified method and which is used as trial factor of safety and then the final solution is obtained from Bishop's Method using Newton Raphson's method.

To calculate factor of safety against slope failure for an area, properties of the soil of the slope plays a important role. Therefore it is necessary to get the soil parameters investigated in detail before going in for slope stability analysis. Various measures fot correcting slides have been described in Appendix B.

5.5 Computer Codification

The computer program REAME(Rotational Equilibrium analysis of Multilayered Embankments)[Huang(1983)] for the stability analysis of free slopes has been modified to take care of the building loads for calculating factor of safety against sliding failure of slope. The mathematical expressions derived above incorporating effects due to building loads has been implemented in the program. Therefore a program SASBL(Stability

Analysis of Slope with Building Loads) has emerged. The flow chart of the program SASBL is shown in Fig. 5.7. The main features of the program are as under:

(a) slopes of any configuration with larger no. of different soil layers can be handled;

(b) the static or pseudo static factor of safety can be evaluated;

(c) the building loads transferred at the foundation level can be properly considered while finding out factor of safety;

(d) pore pressure can be considered by specifying the piezometric surface or pore pressure ratio;

(e) radius control zone can be specified and one or more no. of circles can be searched in the specified radius control zone to find out the minimum factor of safety;

(f) the shallow circles can be eliminated by specifying the minimum depth of tallest slice;

(g) it takes a very little computer time as it has been designed by proper numbering of soil boundaries.

5.6 Validation of the Computer Program SASBL

There is hardly any solved problem available in the literature with building loads transmitted at the slope. Therefore to validate the computer program SASBL one problem with single column load and two column loads has been analysed.

5.6.1 Test Example 1

The column load transferred at the foundation level to the slope are shown in the Fig. 5.8. The soil properties used in the analysis are given in Figure. The factor of safety against sliding failure of slope has been evaluated for single column load. The four conditions have been considered (i) no external load (ii) only vertical load, (iii) vertical and horizontal load (iv) vertical load, horizontal load and bending moments. Horizontal loads acting in outward direction with respect to slope will give destabilising effect and moments acting in anticlockwise direction as shown in Fig. 5.8 will give the destabilising effect. Therefore in the present study the horizontal loads acting in outward direction and moments in anticlockwise direction has been considered for finding the factor of safety of the slope. The loads and the factor of safety for single column are shown in Table 5.1. The slope failure surface for the four conditions of single column load are shown in Fig. 5.9.

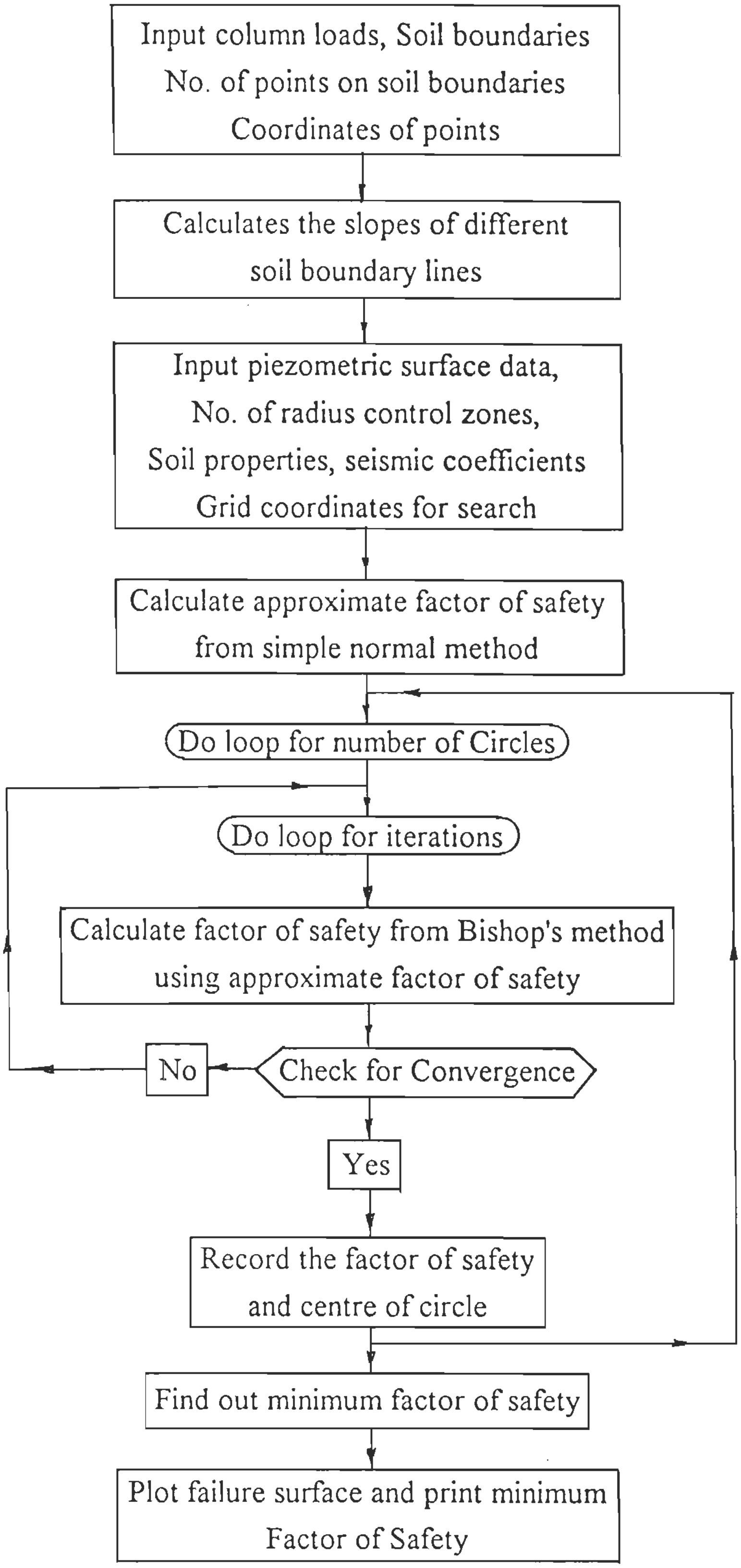

Fig. 5.7 - Flow Chart of SASBL

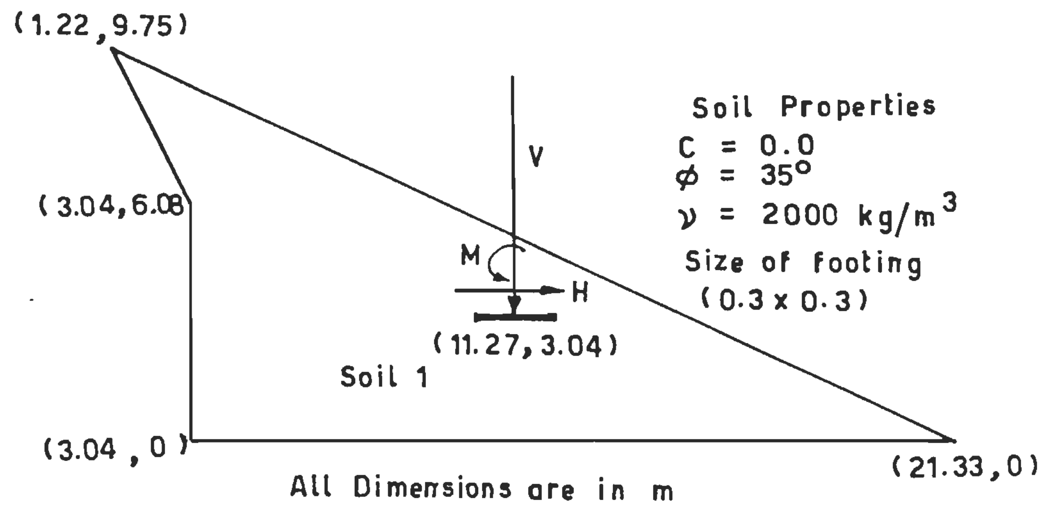

Fig. 5.8 - Cross Section of the Slope with Single Column Load

Table 5.1 - Single Column load and Factor of Safety

Condition	Vertical load	Horizontal load	Bending Moment	Factor of Safety
1	0	0	0	1.499
2	2500 lbs (1133 Kg)	0	0	1.435
3	2500 lbs (1133 Kg)	1000 lbs (453 Kg)	0	1.279
4	2500 lbs (1133 Kg)	1000 lbs (453 Kg)	5000 lb-ft (691 Kg m)	1.262

It is observed that the factor of safety decreases as the column load on the slope increases. The factor safety decreases from 1.499 to 1.435 when only vertical load is applied to the slope, it further decreases to 1.279 when in addition horizontal load is applied and further decreases to 1.262 when bending moment is also applied in addition to the above loads. The factor of safety 1.499 when column loads are zero matches exactly with the reported factor of safety[Huang(1983)].

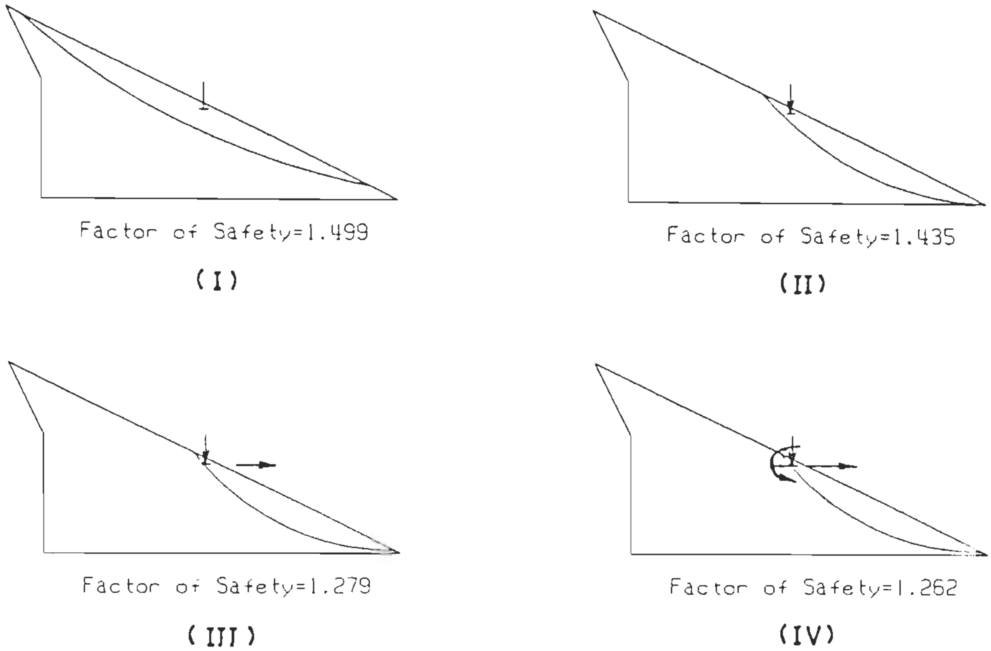

Fig. 5.9 - Slope Failure Surface with Single Column Load

5.6.2 Test Example 2

The cross section of the slope and the two column loads transferred to the slope are shown in Fig. 5.10. In this also four conditions have been considered as in Example 1. The factor of safety against slope failure are shown in Table 5.2. The slope failure surface is shown in Fig. 5.11.

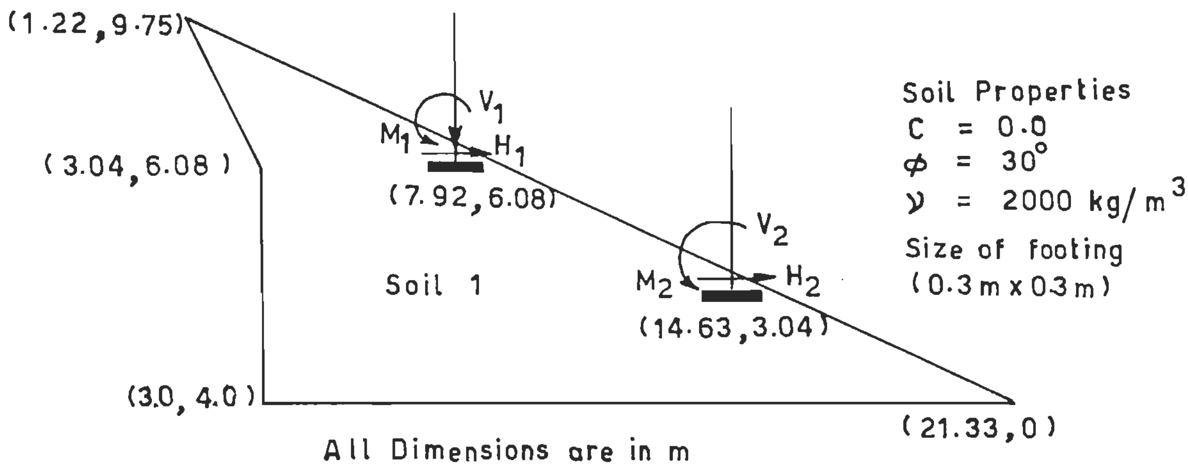

Fig. 5.10 - Cross section of Slope with two Column loads

Table 5.2 - Two Column loads and Factor of Safety

Condition	Column 1			Column 2			Factor of Safety
	Vertical load lbs(Kg)	Horizontal load lbs(Kg)	Bending moment lb-ft (Kg-m)	Vertical load lbs(Kg)	Horizontal load lbs(Kg)	Bending moment lb-ft (Kg-m)	
1	0	0	0	0	0	0	1.499
2	2500 (1133)	0	0	2500 (1133)	0	0	1.481
3	2500 (1133)	1000 (453)	0	2500 (1133)	1000 (453)	0	1.254
4	2500 (1133)	1000 (453)	5000 (691)	2500 (1133)	1000 (453)	5000 (691)	1.199

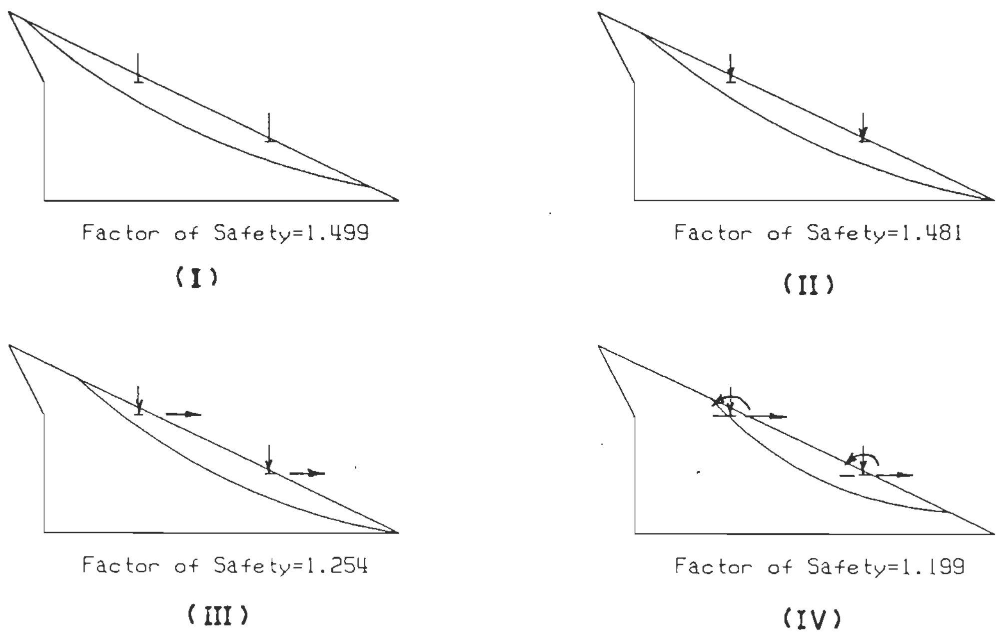

Fig. 5.11 - Slope Failure Surface with Two Column Loads

It is observed that the factor of safety decreses as the column loads on the slope increases. The factor safety decreases from 1.499 to 1.481 when only vertical loads are applied to the slope, it further decreases to 1.254 when in addition horizotal loads are applied and further decreases to 1.199 when bending moments are applied in addition to the above loads.

5.7 Parameteric Study of Stability of Slope with Two Column Loads

A parameteric study has been carried out to study the effect of varying positions of loads with respect to distance from the free edge of the slope, distance between the column loads, difference in levels of footings of the column loads on the factor of safety against sliding failure of slope. In this study two column loads has been taken as given in Table 5.3 in the form of vertical loads, horizontal loads and bending moments. The cross section of the slope, location of column loads, soil properties taken for the study are shown in Fig. 5.12. The distance from edge of the slope has been varied from zero to 3 m, distance between the two column loads has been varied from 3 to 5 m and the level difference in the location of footing for the two columns has been varied from 0 to 3 m. The minimum depth of tallest slice has been varied from 1 to 5 m. The factor of safety against sliding failure of slope has been evaluated with various positions of loads.

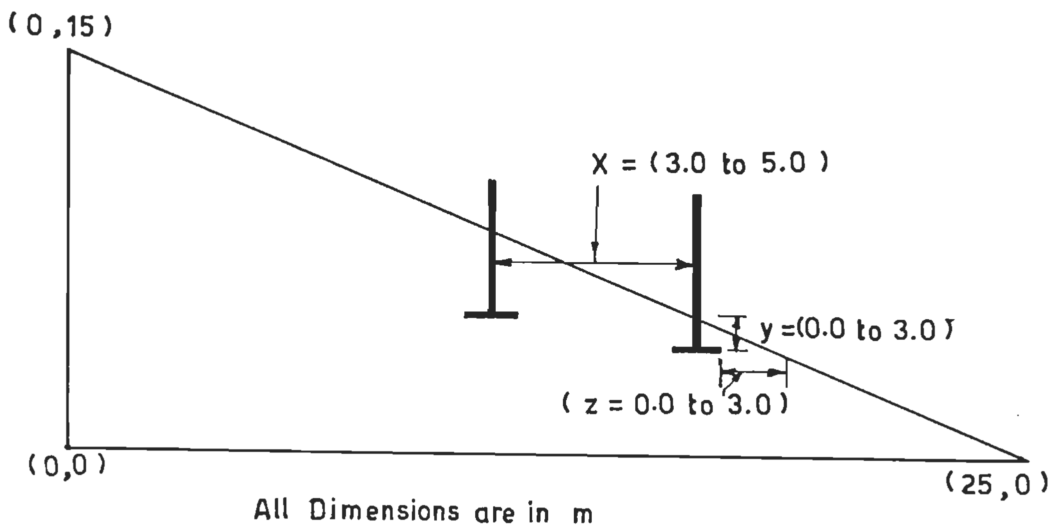

Fig. 5.12 - Cross Section of Slope and Varying Position of Column Loads

Table - 5.3 Column Loads

Column no.	Vertical load(Kg)	Horizontal load(Kg)	Moment(Kg-m)
1	10000	2500	2500
2	10000	2500	2500

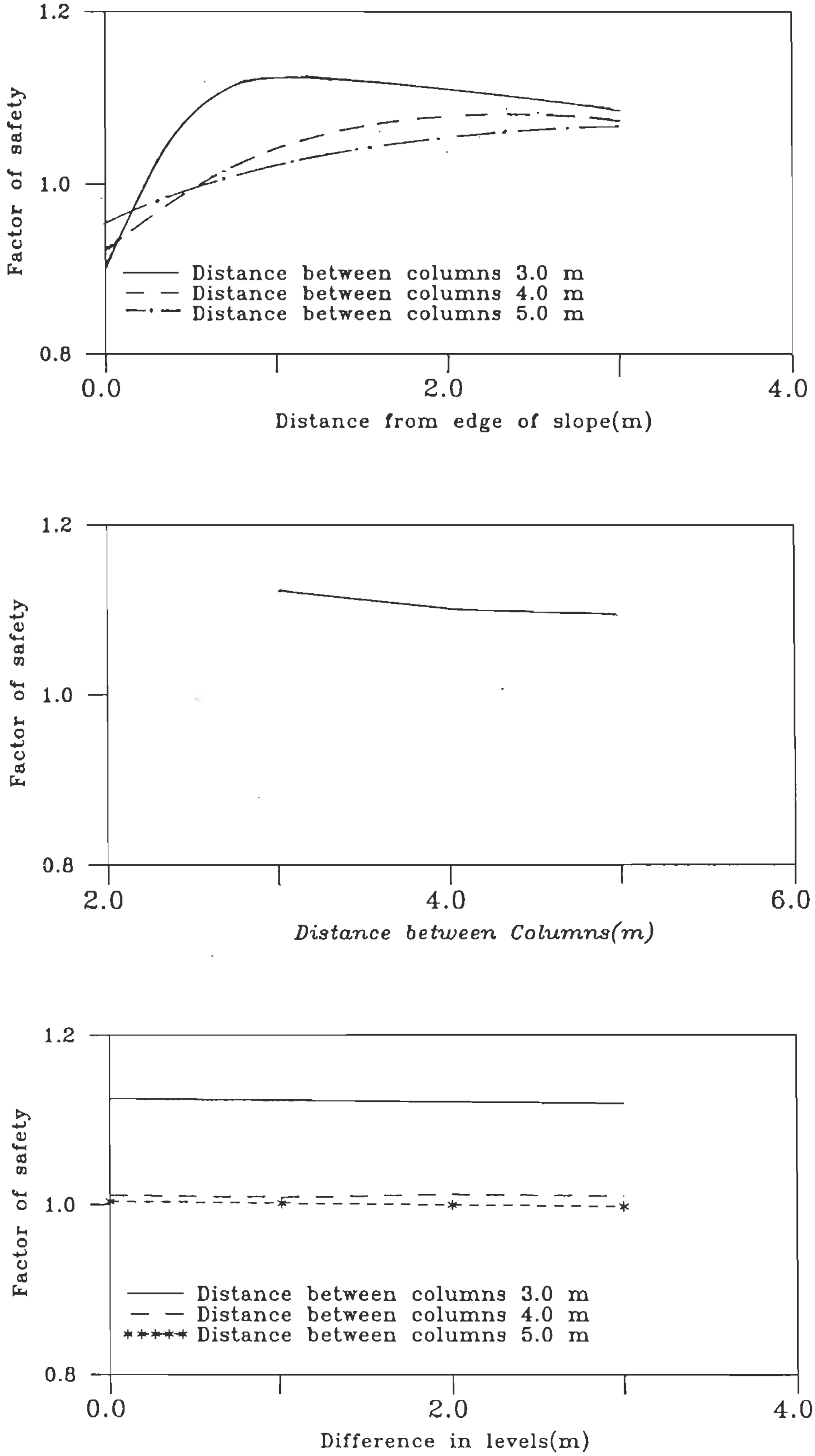

Fig. 5.13 - Variation of Factor of Safety with Column Loads Location

It has been observed from the study of the present cases that

(i) The factor of safety increases with increase in distance from free edge of slope of location of footing of the column. Increased distance between the column loads requires that the distance of location of footing from free edge of the slope should be increased to achieve the desirable factor of safety. It has been observed in the study that when the distance between two columns is 3 m, the distance of 1 m from free edge of the slope is sufficient from the column footing, when the distance between two columns is 3 to 4 m the free edge distance of 2 m gives the maximum factor of safety and when the distance between the two columns is up to 5 m the free edge distance of 3 m gives the maximum factor of safety and is shown in Fig. 5.13(a).

(ii) Increase in distance from 3.0 m to 5.0 m between the columns does not show much difference in the factor of safety as shown in Fig. 5.13(b).

(iii) The variation in the factor of safety against sliding failure of slope is very small when the angle between the extreme adjacent edges of the footings of two adjacent column loads lies between 0 to 45°.

5.8 Stability of Slope with Different Configuration of Building

The two differently configured buildings such as regular framed building and setback framed building have been choosen to study the stability of the slope. The column loads obtained from analysis under static loads and seismic loads are applied to find the factor of safety of the slope. The loads transferred to the ground under static and seismic load conditions are given in Tables 5.4 and 5.5 respectively.

Table 5.4 - Column Loads Transferred at Fondation Level (Static)

Regular fame				Setback frame			
Column	Axial load(N)	Shear force(N)	Moment(N-mm)	Column	Axial load(N)	Shear force(N)	Moment(N-mm)
1	0.214+06	0.131+05	0.650+07	1	0.270+06	0.132+05	0.664+07
2	0.427+06	0.125+04	0.636+06	2	0.483+06	0.137+04	0.641+06
3	0.427+06	0.125+04	0.636+06	3	0.376+06	0.133+04	0.726+06
4	0.214+06	0.131+04	0.650+07	4	0.160+06	0.132+05	0.652+07

Table 5.5 - Column Loads Transferred at Foundation Level(Static + Seismic)

Regular fame				Setback frame			
Column	Axial load(N)	Shear force(N)	Moment(N-mm)	Column	Axial load(N)	Shear force(N)	Moment(N-mm)
1	0.227+06	0.701+04	0.851+06	1	0.287+06	0.742+04	0.468+08
2	0.428+06	0.111+05	0.990+07	2	0.484+06	0.109+05	0.960+07
3	0.428+06	0.862+04	0.863+07	3	0.377+06	0.819+04	0.823+07
4	0.227+06	0.192+05	0.138+08	4	0.170+06	0.191+05	0.136+08

The factor of safety obtained under static loads for free slope, slope with regular frame and slope with setback frame are 1.469, 1.411 and 1.446 respectively. The sliding failure surface of slope is shown in Fig. 5.14 for static loads.

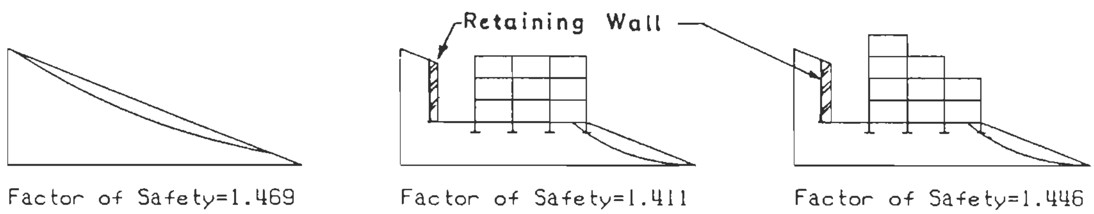

Fig. 5.14 - Sliding Failure Surface of Slope Under Static Loads

The factor of safety under (static + seismic) loads for free slope, slope with regular frame and slope with setback frame are 1.130, 1.132 and 1.147 respectively. The sliding surface of slope is shown in Fig. 5.15 for seismic condition.

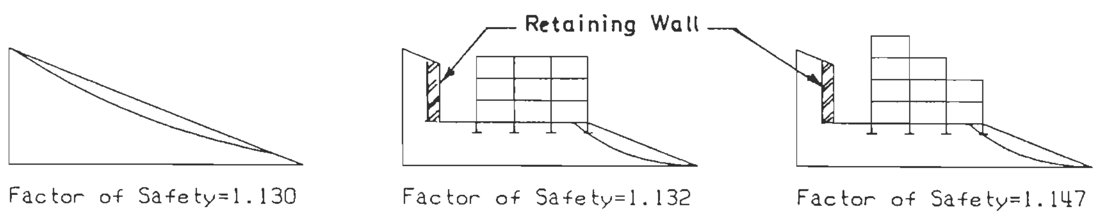

Fig. 5.15 - Sliding Failure Surface of Slope Under (Static+Seismic) Loads

The results shows that the factor of safety against sliding failure of slope is higher for setback type building as compared to regular frame building. In such cases where the building is constructed adjacent to hill slope local failure of slope may take place. Therefore to avoid the local failure the building load transferred near to the edge of slope should be minimum. This can be achieved by constructing setback type configuration of the building adjacent to hill slope. The I.S. 13063(1991) also says that building should have setback type configuration when constructed adjacent to slope. The present study also proves the same thing. Hence it is recommended that when the building is to be constructed adjacent to hill slope, it is preferable to have setback type of configuration from stability consideration.

5.9 Provisions for Stability of Slope for Stepped Foundations: The following measures will be beneficial for reducing/avoiding slope failures and damage to the buildings when such buildings are constructed on hill slope.

1. The plinth beam system should be as heavy as possible. The r.c.c. slabs are to be provided at the plinth level so that the load at the plinth level from the users should be transferred to the slopes through the column or wall supporting system rather than transferring the loads directly under the plinth area. This will reduce the lateral pressure on the retaining wall corresponding to the users loads, thereby avoid the failure of the retaining wall as well as the overall lateral forces on the structure gets reduced. The heavy plinth beam system will help in reducing the cracks and failure of the structure in case some local failure of slope takes place at some point under the building foundation.

2. The foundation system of the structure should be taken deeper into the slope thereby the local failure of slope is avoided as the deeper foundation system gives a lateral support to the soil and avoids any landslides or slope failures.

3. The drainage system should be very good, in addition a pucca apron must be constructed around the whole building area to avoid any water seepage into the soil system under the building area.

4. The foundation across the slope for columns in one row should be continuous strip type instead of isolated footing. The combined/continuous type of footing will distribute the load uniformly and the pressure intensity on the slope will be less and avoid slope failures

5. If a building is to be constructed adjacent to hill slope , then the building should be so planned that the heavier part of the building should be located on the up hill side to provide better stability as shown in Fig. 5.16 [I.S. 13063(1991)] and the same is observed in the present study.

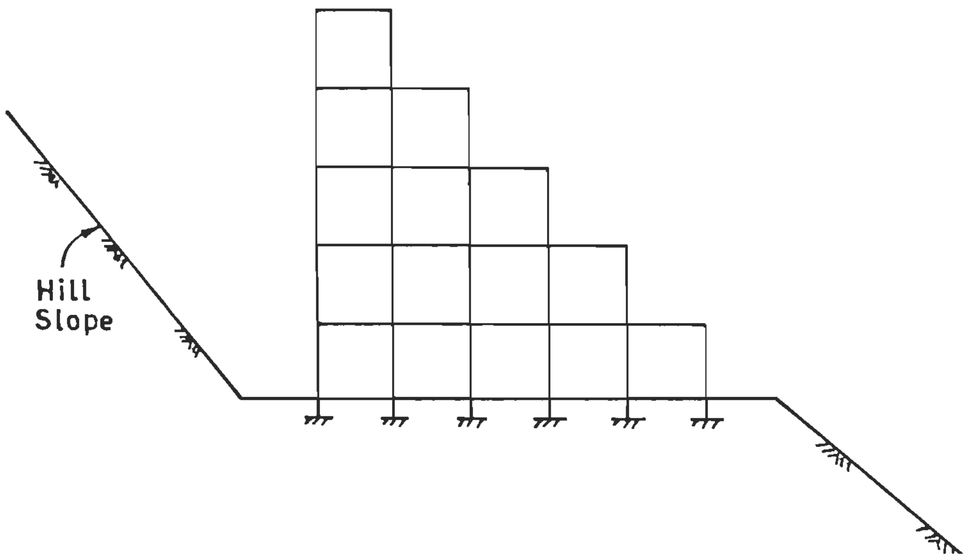

Fig. 5.16 - Building Adjacent to Hill Slope

6. When the footings are adjacent to sloping ground or where the bottom of the footing of a structure are at different levels or at levels different from those of the footings of the adjoining buildings, the provision as per I.S. 1904(1986) should be followed.

5.10 Concluding Remarks

The importance of the stability analysis for slope with building loads has been presented. A method has been developed to analyse the slope stability using simplified Bishop's method considering building loads in the form of vertical loads, horizontal loads and bending moments and the same has been incorporated in a computer program. Numerical examples show that the factor of safety against sliding failure of slope reduces due to the application of column loads and moments.

Investigation has indicated that it is important to check the stability of the slope including building loads under seismic conditions. Two types of failures may occur((i) local failure under the column footing near the slope and (ii) overall failure of slope including the building loads). The various parameters which influence the stability of slope have been studied and these parameters are footing loads, their spacing, height difference and distance from the edge of the slope.

As expected the factor of safety is found to decrease under seismic conditions. It is found that buildings should have setback type configuration from seismic stability considerations when constructed adjacent to hill slope. Remedial measures have been suggested to avoid slope failures with building loads by properly designing and constructing the buildings and its foundations.

5.11 References

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B.1 Remedial Measures for Correcting Slides

There are various methods with which the sliding of slopes can be avoided. As the factor of safety is the ratio between the resisting force and the driving force, therefore the factor of safety can be improved either by decreasing the driving force or by increasing the resisting forces or a combination of both. There are various ways of improving the factor of safety of slope as outlined below.

B.1.1 Slope Reduction: The slope can be reduced either by direct reduction, flattening by cutting berms and flattening without hauling the material away. All the above three methods of slope reduction are shown in Fig. B.1.

B.1.2 Surface Drainage: Proper drainage of water for stabilising the slope is most important. The surface water is to be carried away from the slopes by providing proper drainage system. Cracks and fissures are to be sealed and the surface depressions are to be eliminated which will avoid seepage of water into the soil slope. Proper surface drainage keeps the material in dry condition which will help in avoiding the slope failures.

B.1.3 Sub Surface Drainage: Ground water is one of the major causes of the slope instability. Therefore sub surface drainage is essentially required to avoid the slope failures. Horizontal drains, vertical drainage wells and drainage tunnels are to be installed at proper location to avoid the surface erosion due to the ground water.

B.1.4 Vegetation: Forestation is most suitable for avoiding the slope failures particularly for shallow slides. Vegetation can lower the infiltration of surface water into the slope and this contributes indirectly to the stabilisation of the slide. As the trees draws the water from slope for their growth, therefore the most suitable species such as deciduous trees are to be planted on the slopes. Certain species of plants helps in holding the soil by its root system and this help in stabilising the slope.

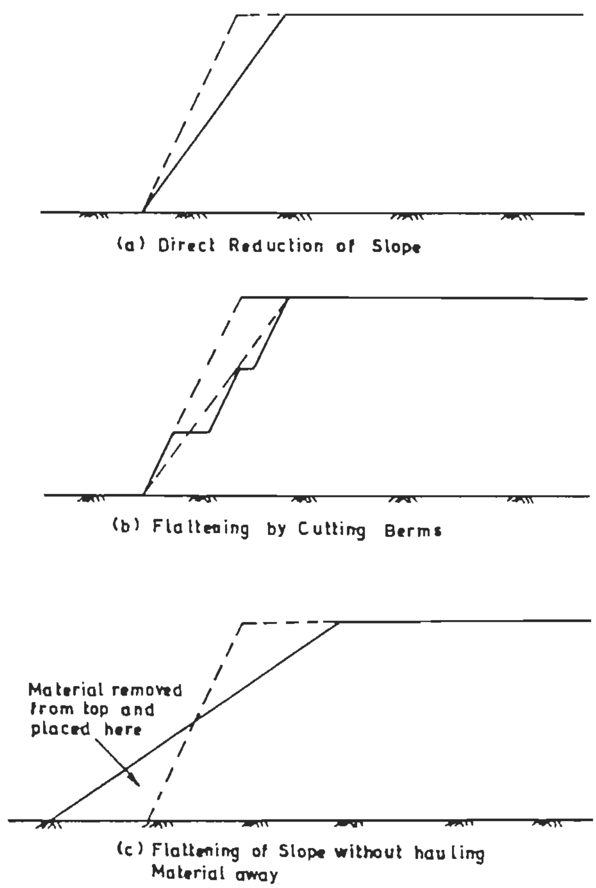


Fig. B.1 - Methods for Slope Reduction

B.1.5 Buttress or Retaining Walls: Stabilizing berms, buttresses and the retaining walls are the other ways of stabilizing the slope. Fig. B.2 shows retaining wall and buttress to stabilize the slope.

B.1.6 Pile System: The use of driven steel or wooden piles of nominal diameter are suitable for shallow landslides, because the piles can be driven to an adequate depth, otherwise they may tilt from the vertical position thus disturbing the adjacent material and material underneath the piles and causing the development of a slip surface below the pile cap.

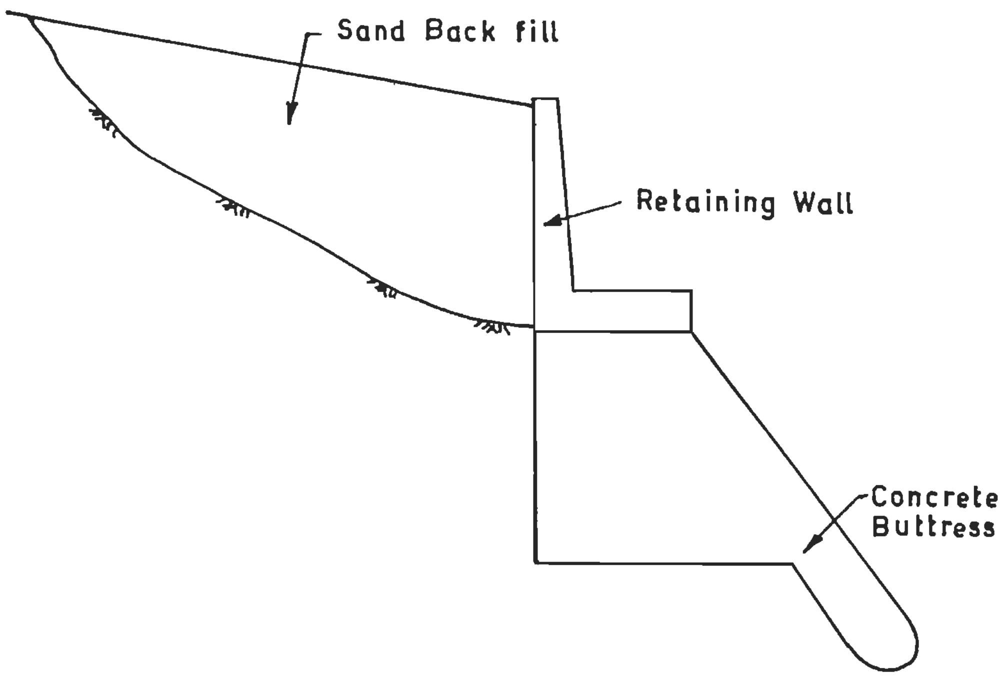


Fig. B.2 - Retaining Wall and Buttress to Stabilise the Slope

B.1.7 Anchor System: Anchor system is the back wall which carries the back fill forces on the wall by a tie system to transfer the imposed load to an area behind the slide mass where satisfactory resistance can be established. The ties may consists of pre or post tensioned cables, rods or wires and some form of dead mass or other method to develop adequate passive earth pressure.

B.1.8 Stabilization of Soils: If the sub surface drainage method are inadequate to drain the water of the slope, then the methods such as chemical treatments, electro osmosis and thermal treatments are to be used. Chemical treatment are the lime, lime soil mixture and cement grout. The electro osmosis has the same effect as the sub surface drainage , the difference is that water is drained by electric field than by gravity. The use of thermal treatments for preventing landslides causes a permanent drying of the embankments and cut slopes.

Chapter 6

SEISMIC ANALYSIS OF BUILDINGS ON HILL SLOPES

6.1 Introduction

Buildings Constructed in hilly areas are irregular and asymmetric due to their varied configurations such as stepback buildings, setback buildings and combination of stepback and setback buildings. These buildings are subjected to severe torsion under the action of earthquakes, which necessitates that a 3D analysis of the buildings is to be carried out so as to take care of the torsional coupling effects. Code of Practices such as (UBC, NBCC, NZS etc) recommend 3D dynamic analysis for such highly irregular buildings. Buildings constructed on hill slope also necessitates the stability analysis of the slope considering building loads transferred at the foundation level. The stability analysis of slopes are to be carried out considering building loads in static and earthquake conditions and under dry or wet soil situations. In the present Chapter few real building problems have been studied for seismic behaviour using the simplified method presented in Chapter 3 and with the rigorous method presented in Chapter 4. The stability analysis of the slope for these problems has been carried out considering building loads. Inelastic behaviour has also been studied using the nonlinear modelling of various components described in Chapter 4.

6.2 Comparison of Seismic Response using Simplified and Rigorous Method

Few real building problems choosen are analysed for its seismic behaviour using the simplified method(i.e. 3 d.o.f per floor with floor diaphragm idealised as rigid) and the rigorous method(i.e. 6 d.o.f. per node considering flexibility of floor/roof). The objective of this study is to compare the results of the time periods, mode shapes, lateral deflections, column shears, storey shears obtained from two methods of analysis and to see the reliability of the simplified method to be adopted in the design practice.

6.2.1 Illustrative Example 1

This example is of a hostel building for which the complete architectural and structural design data was made available. This building problem has been analysed for seismic response by the two methods of analysis. The infill panels have been modelled as equivalent struts. The structural idealisation of the frame and the material properties are shown in Fig. 6.1. The number of beam/column elements, slab panels, infill panels used by two methods of analysis are shown in Table C.1 in Appendix C. The cross section details and the conc. mix used for the members are shown in Table C.2 in Appendix C. The dead loads, live loads and earth pressure load for the analysis are given in Appendix C in C.1. The results of seismic response obtained from two methods of analysis are presented as under.

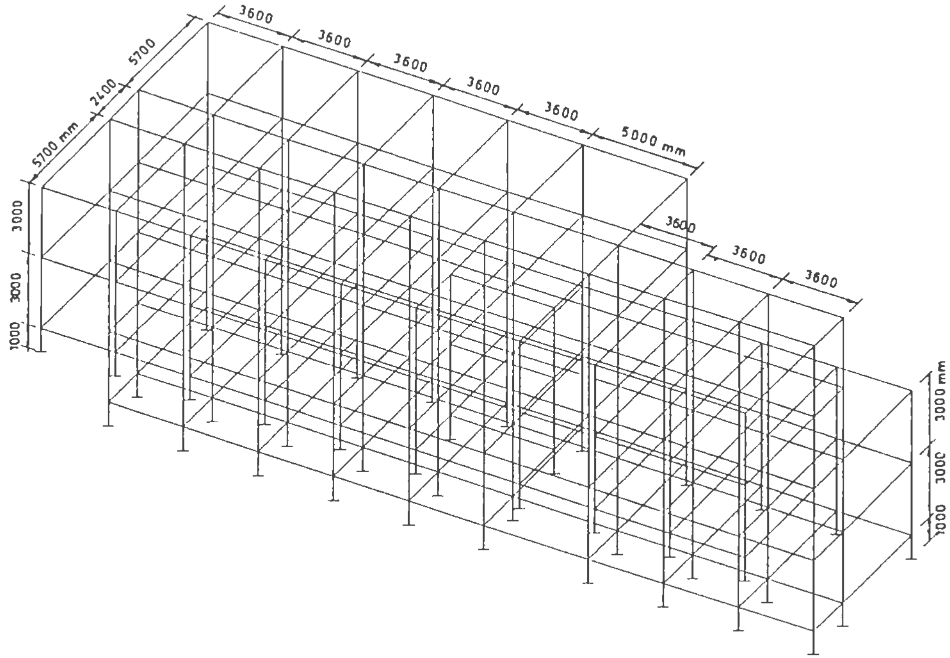
(a) Free Vibration Time Periods: The time periods of vibration obtained from two methods of analysis are shown in Table 6.1. The time periods obtained from two methods of analyses shows that the time periods are very close to each other for all the six modes.

Table 6.1 - Comparison of Time Periods

Mode	Time periods obtained by	
	Rigorous method	Simplified method
1	0.4032	0.3868
2	0.3215	0.2768
3	0.2590	0.2538
4	0.1350	0.1502
5	0.1238	0.1103
6	0.1131	0.0991

(b) Mode Shapes: Mode shapes for the first six modes have been plotted in Figs. 6.2 and 6.3 for rigorous and simplified methods respectively. Comparison of mode shapes shows that the first mode is vibration across the slope, second mode is a combination of vibration along the slope and torsional vibration, third mode is pure torsional, fourth mode is a combination of lateral and torsional vibration, fifth and sixth modes in both the cases are torsionally coupled modes. All the mode shapes obtained by two methods of analysis shows the similar trends in both the methods of analysis.

(c) Floor Displacements and Inter Storey Column Shears: Floor displacements and inter storey column shears have been obtained at various floor levels for excitation given along the slope and across the slope using I.S. Code 1893 - 1984 Code Spectra as shown in Fig. 6.4 with 5% damping. Floor displacements for excitation along the slope and across the slope are shown in Fig. 6.5. Inter storey column shears at various floor



Concrete

Conc. Mix	= M_{15}
$E_{concrete}$	= 22076 N/mm^2
μ	= 0.2
Ultimate Conc. Strain	= 0.0038
Yield Conc. Strain	= 0.002
Maximum tensile strength of Conc.	= 2.5 N/mm^2

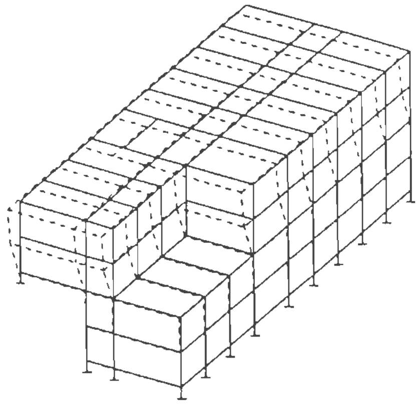
Steel

E_{steel}	= 200000 N/mm^2
Fy	= 415 N/mm^2
μ	= 0.2

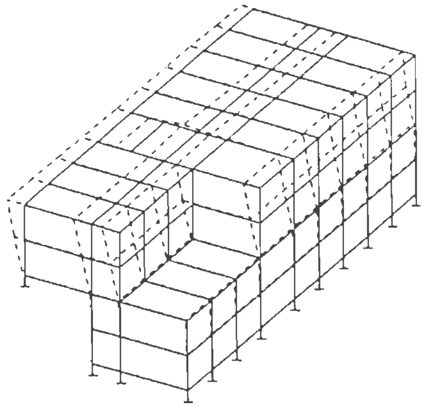
Brick Masonry

$E_{brick\ masonry}$	= 700 N/mm^2
σ_y	= 6.5 N/mm^2

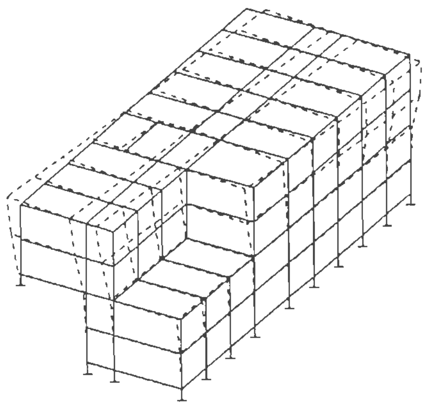
Fig. 6.1 - Mathematical Model of Frame (Example 1)



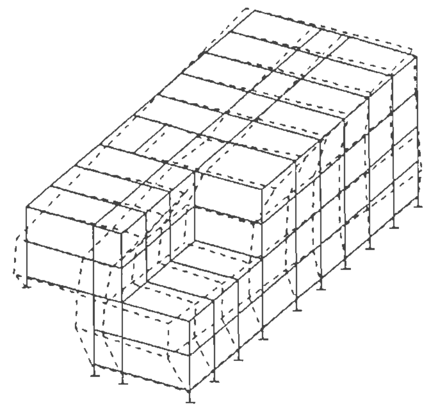
First Mode Shape T= 0.4032 Sec



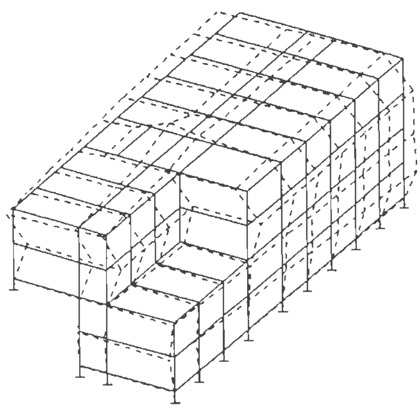
Second Mode Shape T= 0.3215 Sec



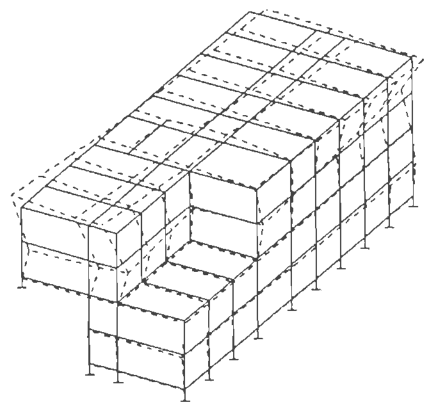
Third Mode Shape T= 0.2590 Sec



Fourth Mode Shape T= 0.1350 Sec

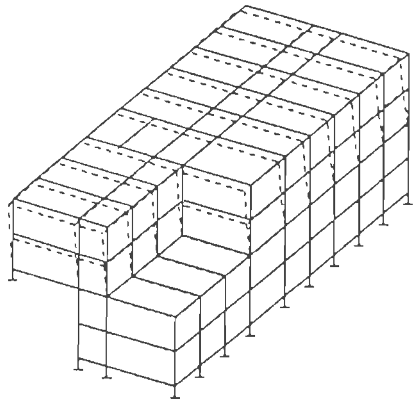


Fifth Mode Shape T= 0.1238 Sec

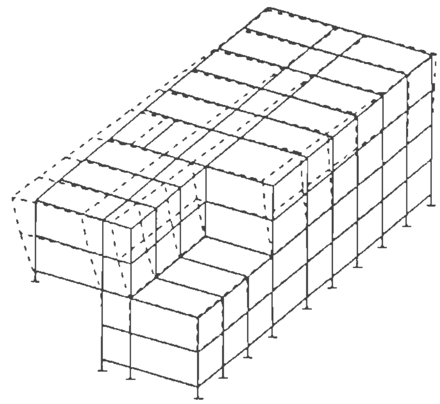


Sixth Mode Shape T= 0.1131 Sec

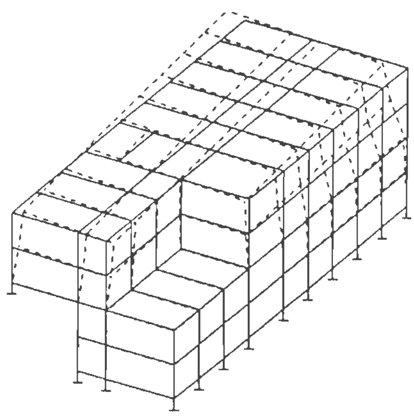
Fig. 6.2 - First Six Mode Shapes(Rigorous Method - 6 D.O.F./Node)



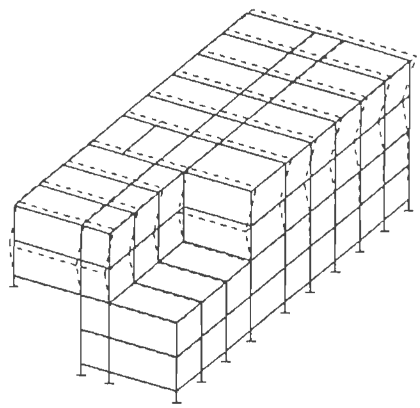
First Mode Shape T= 0.3868 Sec



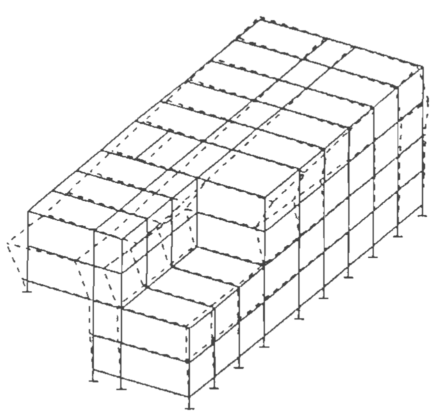
Second Mode Shape T= 0.2768 Sec



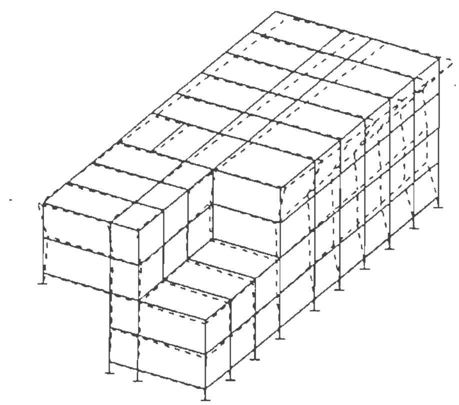
Third Mode Shape T= 0.2538 Sec



Fourth Mode Shape T= 0.1502 Sec



Fifth Mode Shape T= 0.1103 Sec



Sixth Mode Shape T= 0.0991 Sec

Fig. 6.3 - First Six Mode Shapes(Simplified Method - 3 D.O.F./Floor)

levels under excitation along the slope and across the slope are shown in Fig. 6.6 obtained from two methods of analysis. Simplified method with rigid floor diaphragm gives the stiffer behaviour in both directions of excitation, but the excitation across the slope is governing and the results by two methods of analysis are closer for this excitation direction. The results of inter storey column shears obtained by two methods of analysis are almost close to each other.

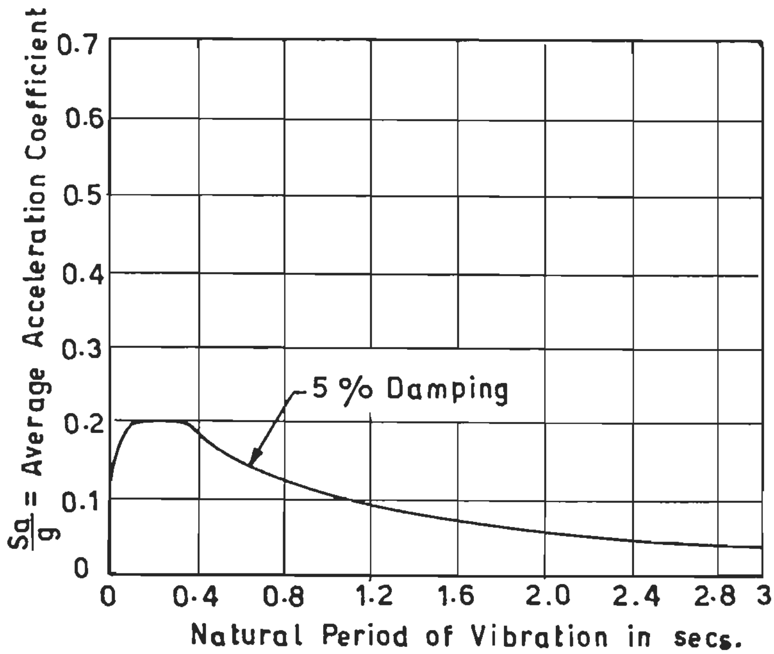


Fig. 6.4 - Average Acceleration Spectra of I.S. 1893 - 1984

(d) Ground Column Shears and Infill Shears: The ground column shears and infill shears are obtained from two methods of analysis for both directions of excitation and are shown in Figs. 6.7 and 6.8 respectively. The results are close to each other.

The results of time periods, mode shapes, floor displacements, inter storey column shears, ground column shears, infill shears are close to each other obtained from two methods of analysis, indicating that the simplified method with rigid floor idealisation can be used in the design offices at least for preliminary design purposes requiring much less efforts.

Excitation Along the Slope Excitaion Across the Slope

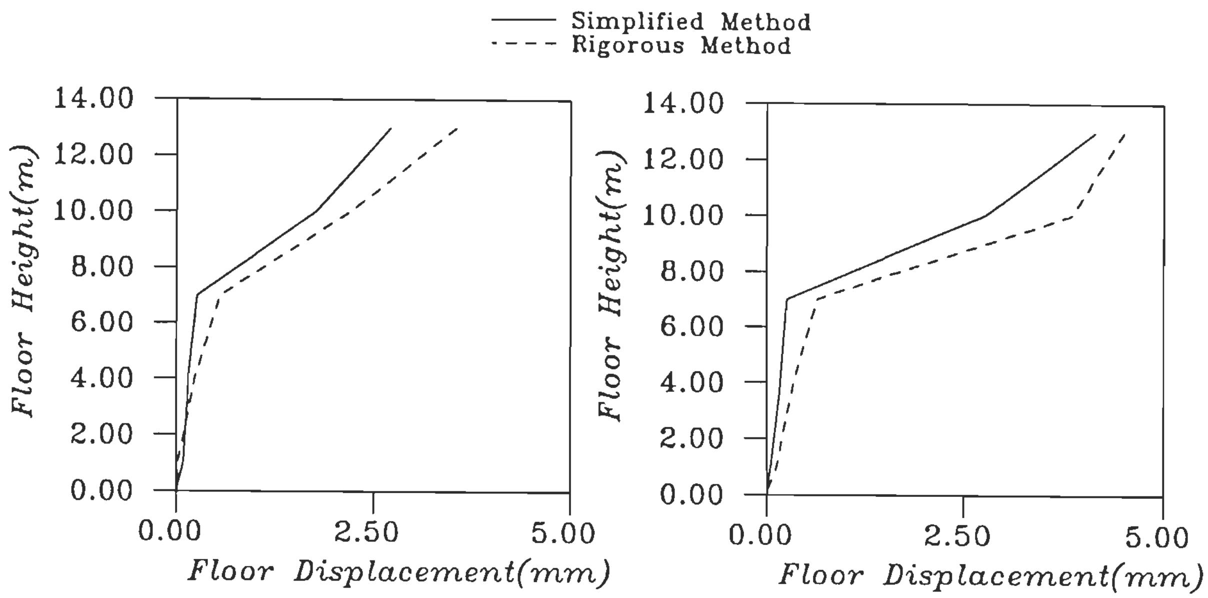


Fig. 6.5 - Comparison of Lateral floor Displacement

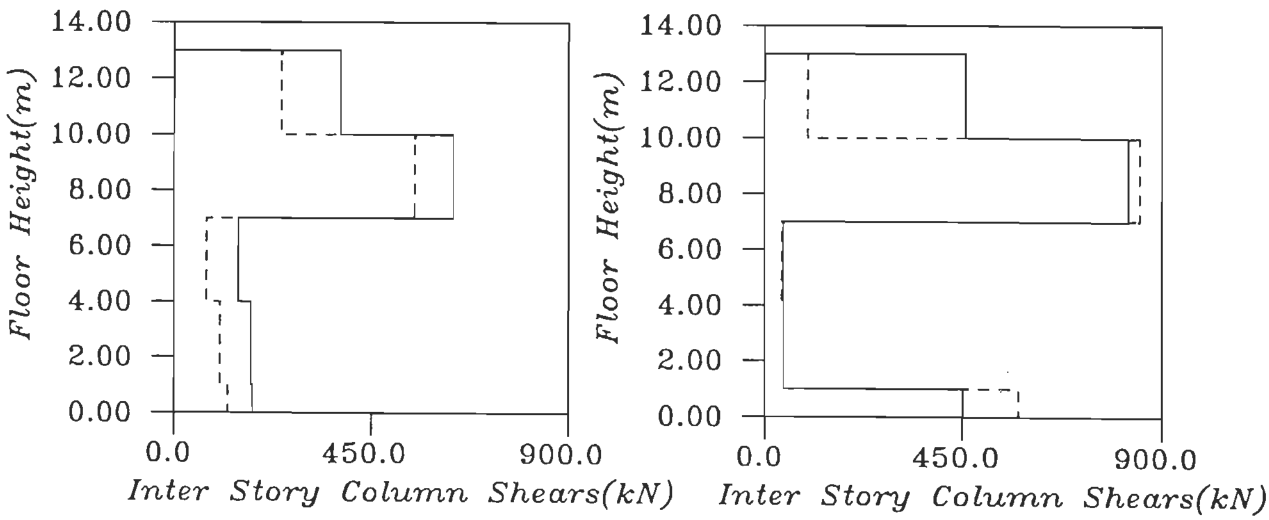


Fig. 6.6 - Comparison of Inter Storey Column Shears

Excitation Along the Slope

Excitation Across the Slope

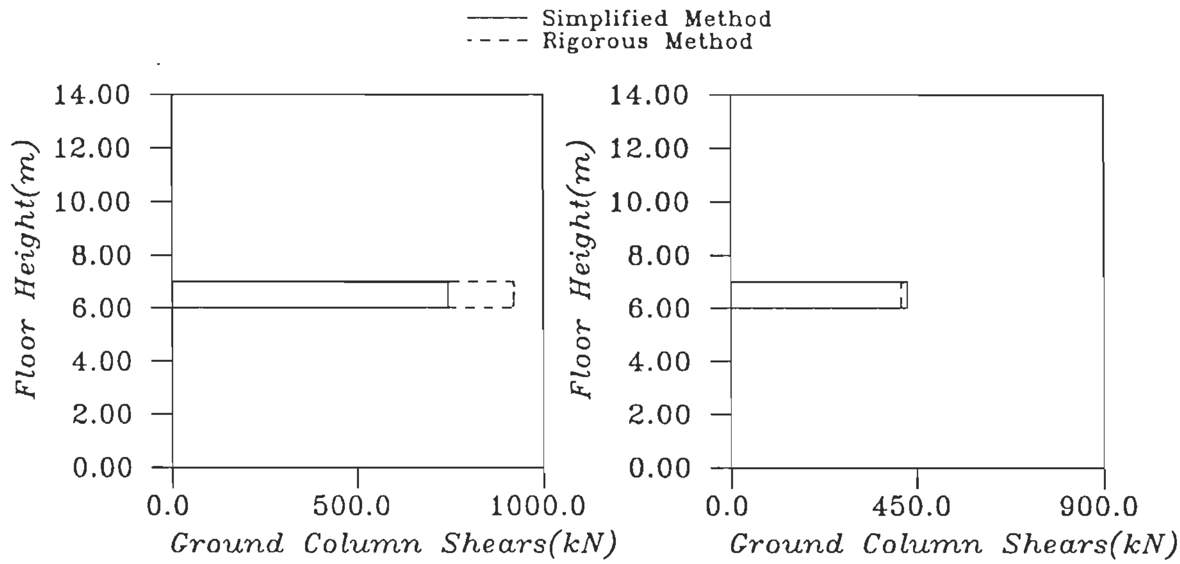


Fig. 6.7 - Comparison of Ground Column Shears

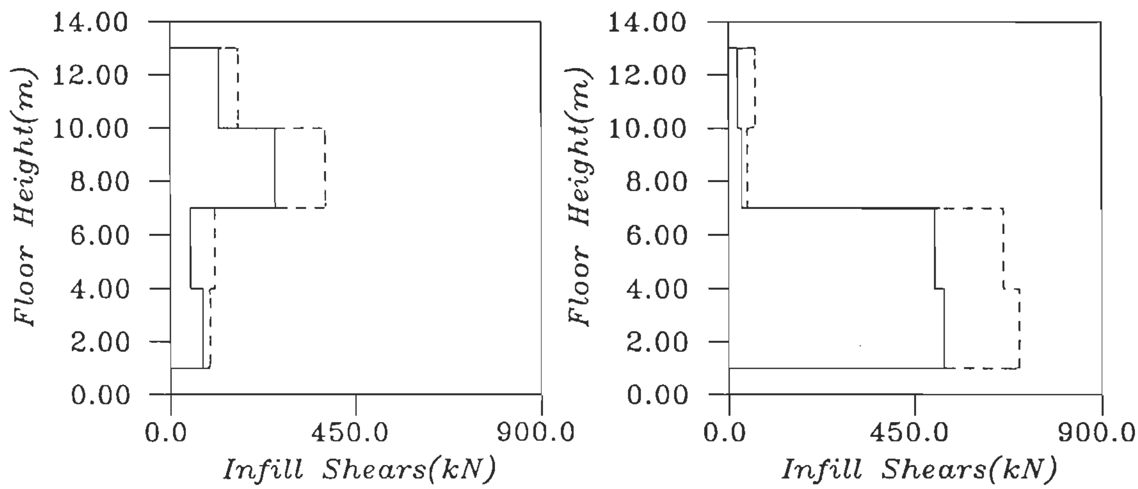


Fig. 6.8 - Comparison of Infill Shears

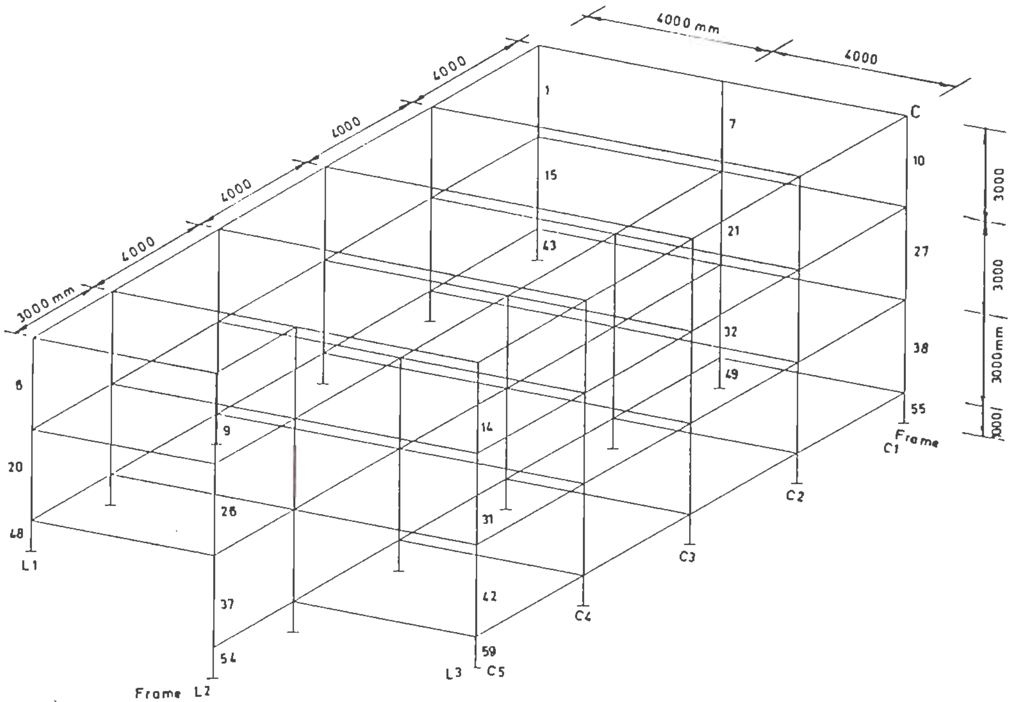
6.2.2 Illustrative Example 2

A hill building already constructed has been selected for seismic analysis. The structural design and drawings were made available. This building structure has been idealised for seismic analysis as per the two methods of analysis. The structural idealisation of the frame and the material properties used in the analysis are shown in Fig. 6.9. The number of beam/column elements, slab elements, strut elements used in the seismic analysis for the two methods are shown in Table C.3 in Appendix C. The member cross section and the reinforcement details used in the analysis are shown in Figs. C.1(a) and (b). The loads taken in the analysis are given in C.2 in Appendix C. The results of seismic response for two methods of analysis are presented as under:

(a) Mode Shapes: Figure 6.10 shows the first six mode shapes obtained from rigorous method and Fig. 6.11 shows the mode shapes obtained using simplified method. Comparison of mode shapes obtained from two methods of analysis shows that the first mode indicates the vibration is across the slope , second mode is a combination of vibration along the slope and torsional vibration, third mode is the torsional mode, fourth mode is vibration across the slope, fifth mode is vibration across the slope and sixth mode is torsional mode in both the methods of analysis. All the six mode shapes obtained by two methods of analysis shows similar modes of vibration indicating that the simplified method of analysis can be employed for free vibration analysis.

(b) Deflected Shapes: The deflected shape for this problem has been plotted obtained by two methods of analysis under excitation along the slope and across the slope separately and are shown in Figs. 6.12 and 6.13 respectively. The deflected shapes obtained by two methods of analysis show similar trends under both types of excitations but for excitation along the slope the simplified method shows a stiffer behaviour.

(c) Floor Displacements and Column Shears: Floor displacements and maximum column shears have been obtained at various floor levels for excitation along the slope and across the slope using I.S. 1893-1984 Code spectra for 5% damping. Floor displacements for excitation along the slope and across the slope are shown in Fig. 6.14, maximum column shears at various floor levels for excitation along the slope and across the slope are shown in Fig. 6.15 obtained from two methods of analysis. Floor displacements under excitation along the slope shown in Fig. 6.14 show that the simplified method gives stiffer results and under excitation across the slope the floor displacements obtained from two



Concrete

Conc. Mix	$= M_{15}$
$E_{concrete}$	$= 22076\ N/mm^2$
μ	$= 0.2$
Ultimate Conc. Strain	$= 0.0038$
Yield Conc. Strain	$= 0.002$
Maximum tensile strength of Conc.	$= 2.5\ N/mm^2$

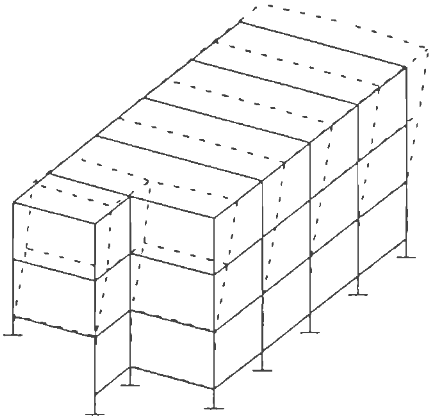
Steel

E_{steel}	$= 200000\ N/mm^2$
Fy	$= 415\ N/mm^2$
μ	$= 0.2$

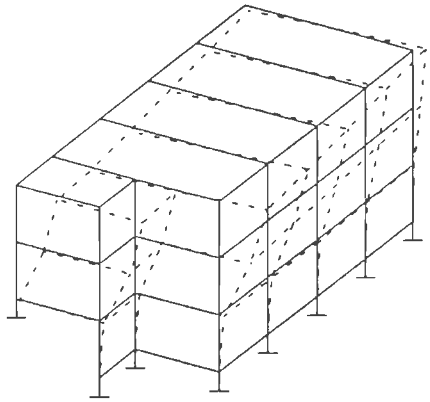
Brick Masonry

$E_{brick\ masonry}$	$= 700\ N/mm^2$
σ_y	$= 6.5\ N/mm^2$

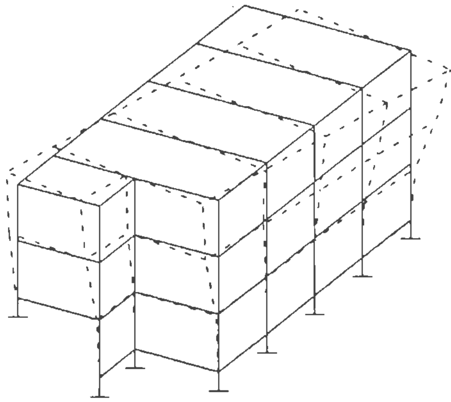
Fig. 6.9 - Mathematical Model of Frame (Example 2)



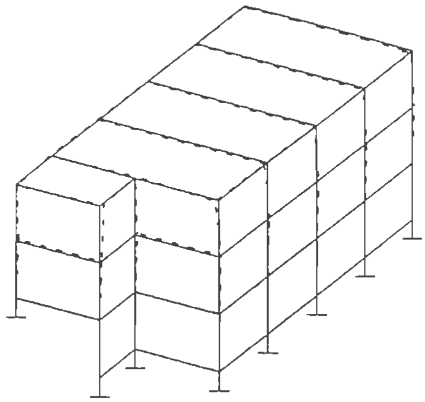
First Mode Shape T= 0.2425 Sec



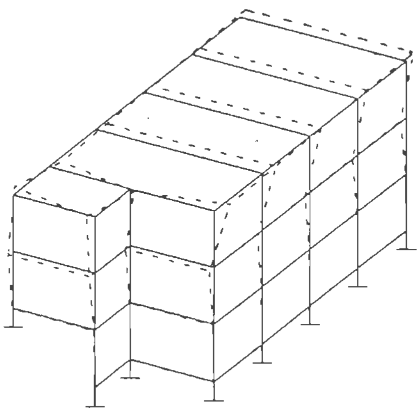
Second Mode Shape T= 0.1914 Sec



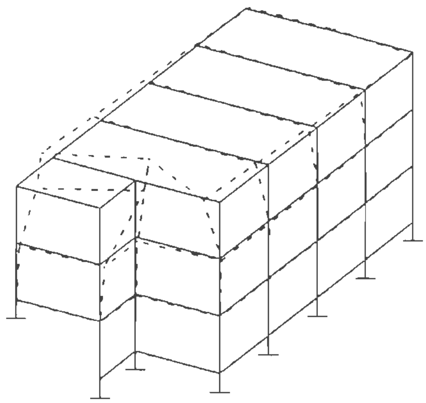
Third Mode Shape T= 0.1664 Sec



Fourth Mode Shape T= 0.1072 Sec

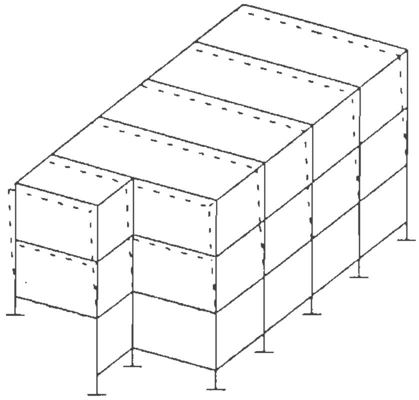


Fifth Mode Shape T= 0.0992 Sec

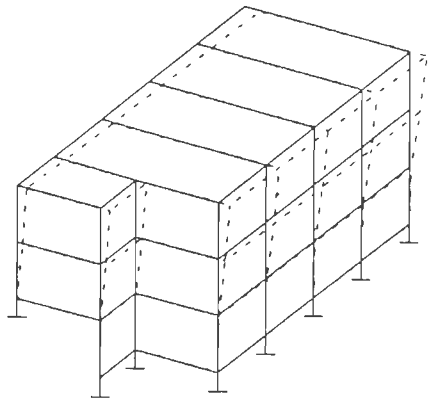


Sixth Mode Shape T= 0.0794 Sec

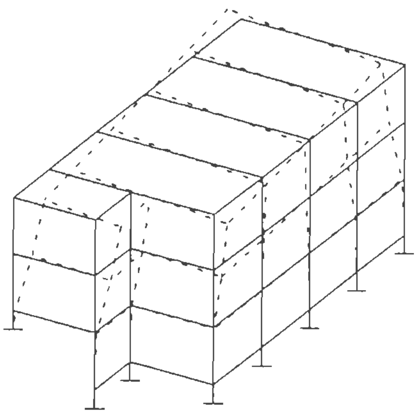
Fig. 6.10 - First Six Mode Shapes(Rigorous Method - 6 D.O.F./Node)



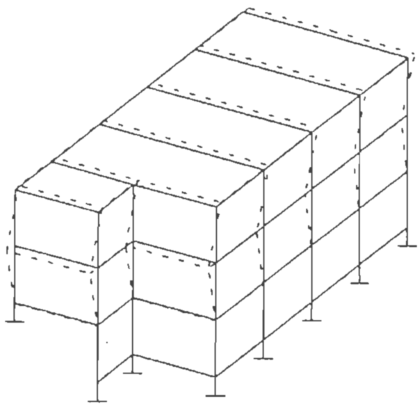
First Mode Shape T= 0.2561 Sec



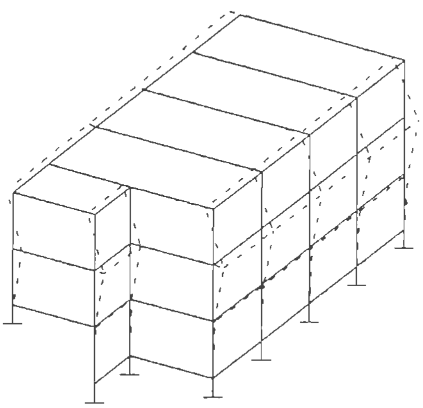
Second Mode Shape T= 0.1597 Sec



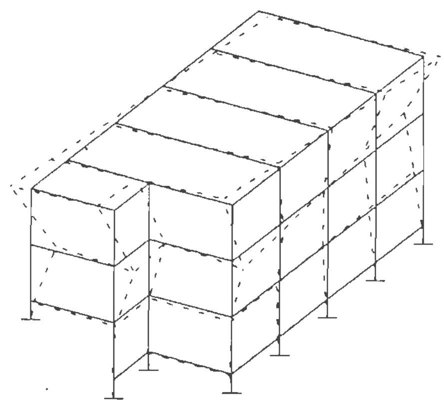
Third Mode Shape T= 0.1248 Sec



Fourth Mode Shape T= 0.1054 Sec



Fifth Mode Shape T= 0.0654 Sec



Sixth Mode Shape T= 0.0503 Sec

Fig. 6.11 - First Six Mode Shapes(Simplified Method - 3 D.O.F./Floor)

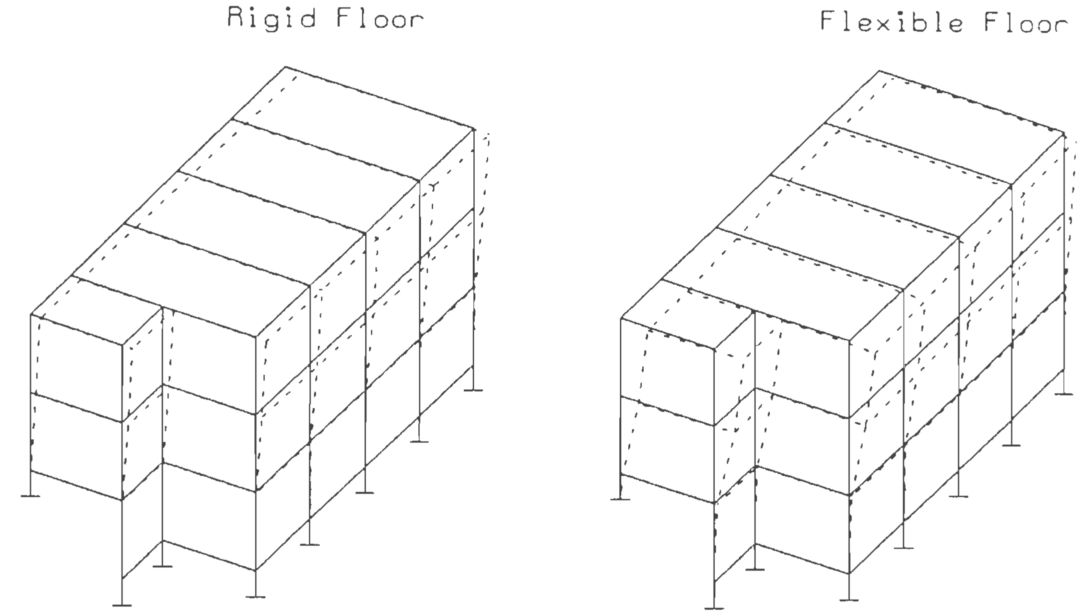


Fig. 6.12 - Deflected Shape under Excitation along the Slope

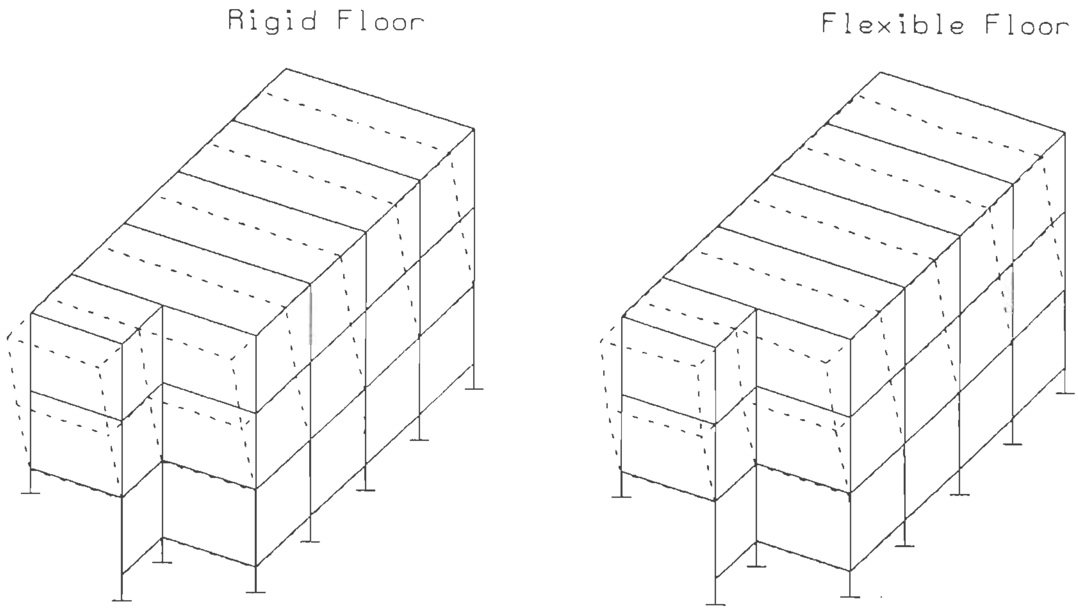


Fig. 6.13 - Deflected Shape under Excitation across the Slope

Excitation Along the Slope   Excitation Across the Slope

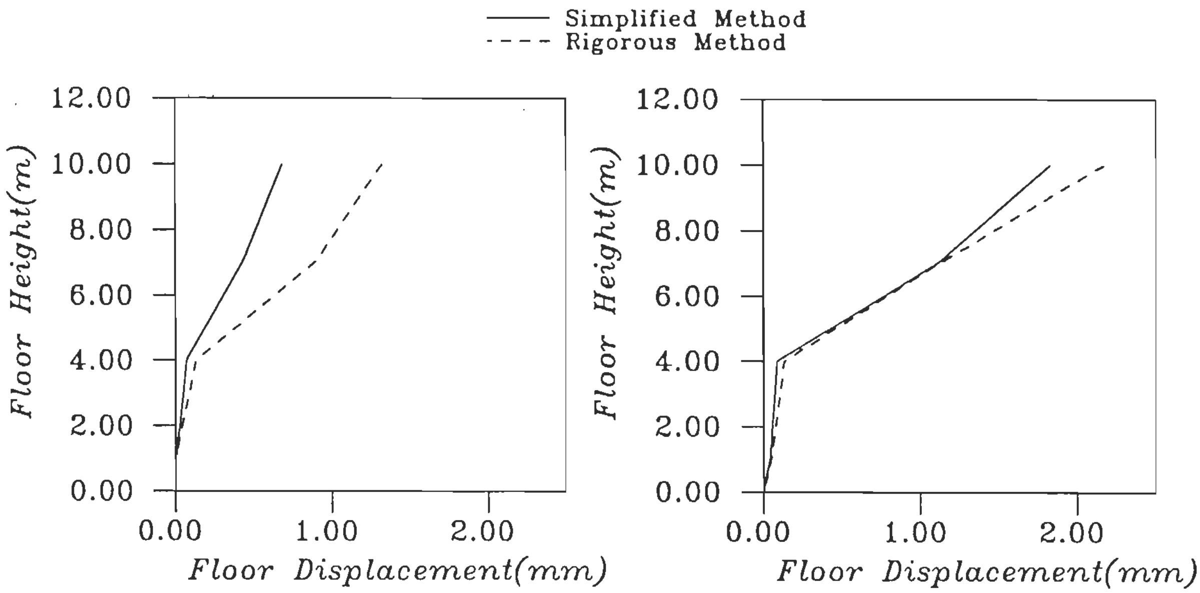


Fig. 6.14 - Comparison of Lateral floor Displacement

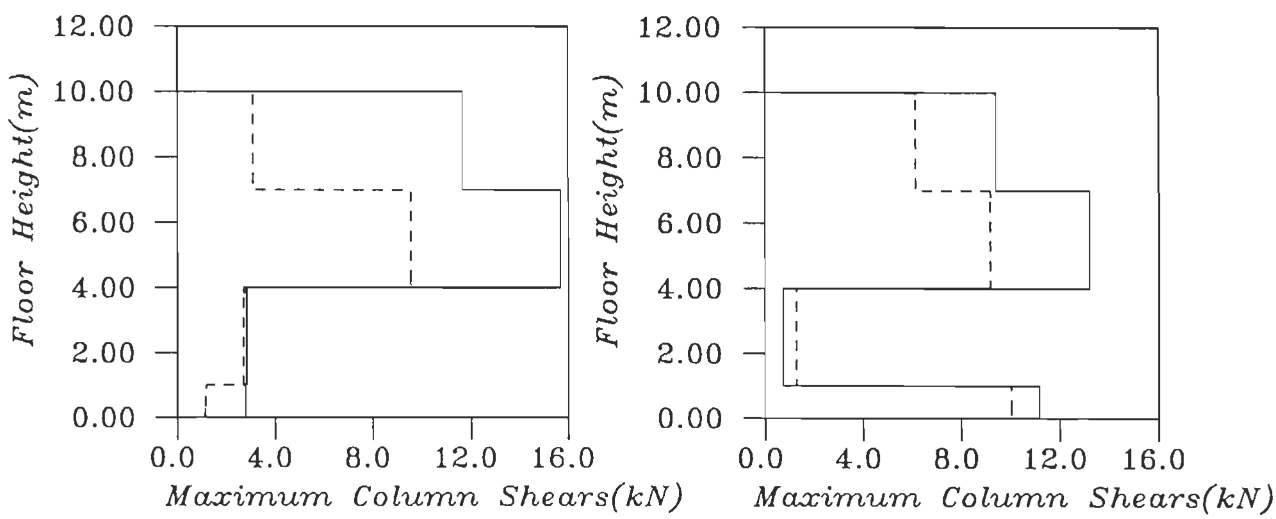


Fig. 6.15 - Comparison of Maximum Column Shears

methods are almost similar and are governing also. The results of maximum column shears at various floor levels obtained using the two analyses shows that the results shown by simplified method are on the conservative side, however the results for excitation across the slope obtained from two methods are close to each other and governing also. This shows that the simplified method of analysis can be used for estimating the floor displacements and maximum column shears which give results very much close to the results given by rigorous method.

(d) Inter Storey Column Shears, Ground Column Shears and Infill Shears: The inter storey column shears, ground column shears and infill shears are obtained separately from two methods of analysis and are shown in Fig. 6.16. The results given by simplified method are on the conservative side for the two excitations. The results for excitation across the slope are governing and are close to each other obtained from two methods of analysis.

The results of time periods, mode shapes, deflected shapes, floor displacements, maximum column shears, inter storey column shears, ground column shears, infill shears obtained from two methods of analysis show that the results obtained from simplified method of analysis are on conservative side and closer to rigorous method of analysis. Hence the simplified method of analysis can be used for design purposes requiring much less computation efforts and data preparation is very easy. The number of equations to be solved in the simplified method are very less as compared to the rigorous method.

6.3 Simplified Method vs Rigorous Method

The seismic response of these buildings obtained using two methods of analysis shows the similar trend and the results of floor displacements, column shears, infill shears are close to each other. Various Codes of Practices (UBC, NBCC, NZS etc.) recommend that 3D dynamic analysis is to be carried out for irregular buildings characterised by centre of mass of different floors lying on different vertical axes and so is its centre of stiffness of various floors. Although many computer codes are available for 3D dynamic analysis of the buildings to take care of any amount of asymmetry, still a simplified method of analysis of analysis is required in the design offices which should give insight into the real seismic behaviour of the irregular buildings such as on the hill slopes characterised by centre of mass of various floors lying on different vertical axes and so is the stiffness of various floors. The simplified method developed in this thesis fulfills this requirement. The

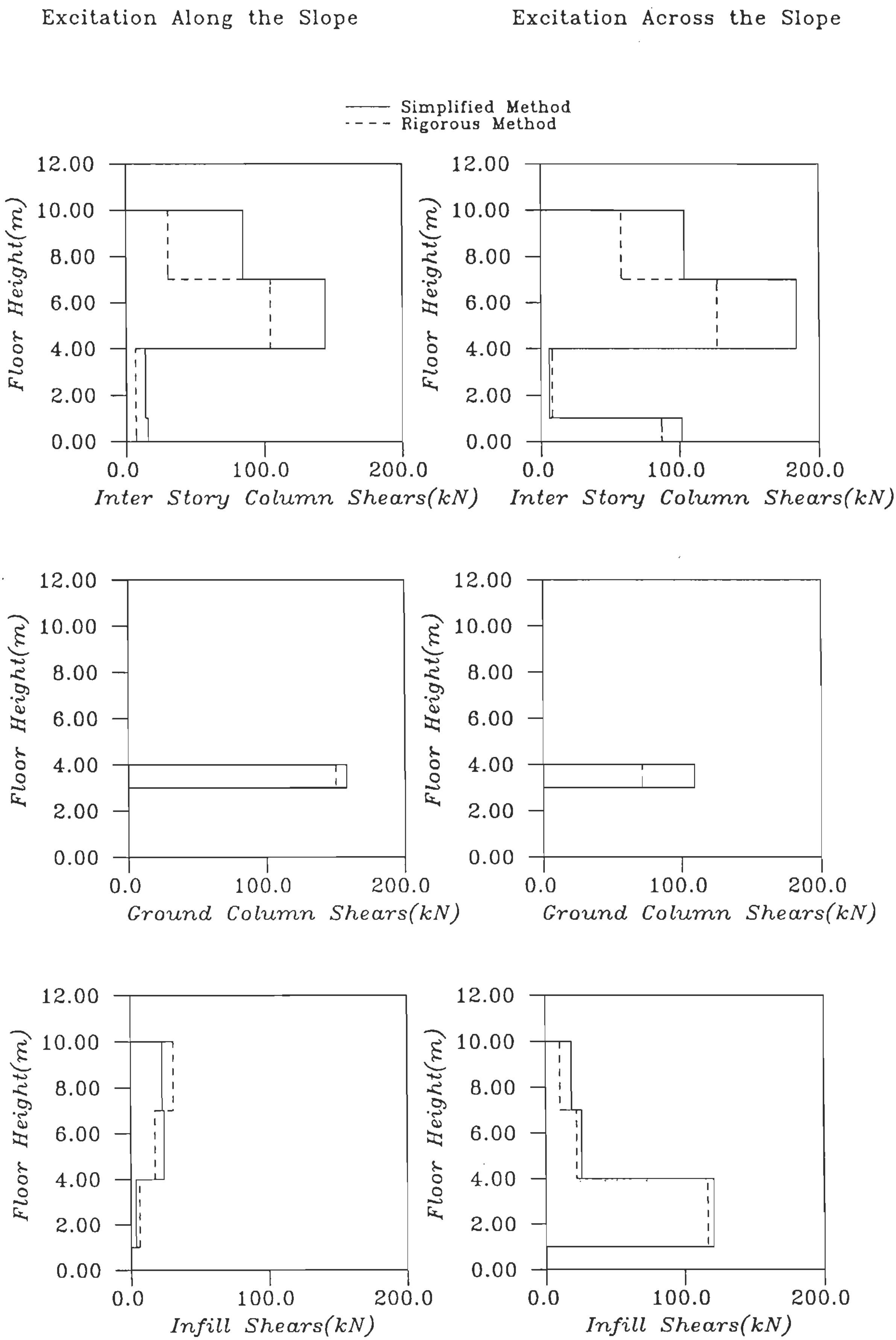


Fig. 6.16 - Comparison of Storey Shears

closeness of the results of seismic response obtained by simplified model with the rigorous model justify the use of this model in the design practice. It is recommended that a simplified 3D analysis developed in the present study for stepback and setback buildings on hill slopes based on transformation of mass and stiffness of various elements about a common vertical reference axis be adopted in the Code of Practices. It is also seen from the shear distribution pattern at various floor levels that the base shear concept of Code of Practices is not applicable in stepback and setback types of buildings. The benefits of using such a simplified analysis is to gain a deeper understanding of interaction between individual structural components and of behaviour of the building as a whole.

6.4 Stability of Slopes with Building Loads

The two buildings analysed in this Chapter are located on hill slope. The stability of the slope is of major concern. Therefore the stability of natural slope and the human created slope with building loads is to be checked under static and earthquake conditions. The methodology presented in Chapter 5 has been used to determine the factor of safety of natural slope and the slope with building loads. The loads are transferred from 3D frame to the soil slope through column at the foundation level. The framed buildings consists of various frames one after the other along the slope. Therefore the maximum load coming on the slope for the plane frame along the slope has been taken to find the factor of safety of slope.

6.4.1 Illustrative Example 1

(a) Natural Slope: The slope stability analysis of the natural slope and with the building load are carried out for illustrative Example 1 already solved in this Chapter. The cross section of the natural slope is shown in Fig. 6.17.

(b) Hill Slope with Building Frame:

The cross section of hill slope with building frame is shown in Fig. 6.18.

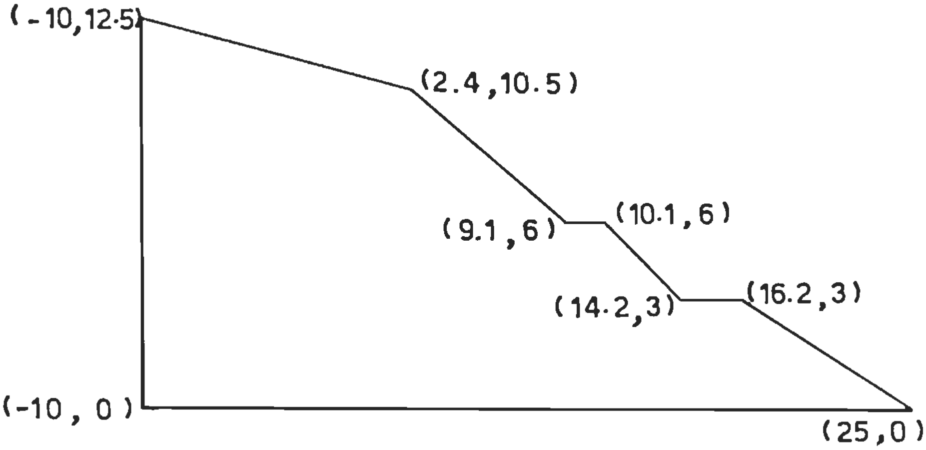


Fig. 6.17 - Cross Section of Natural Slope

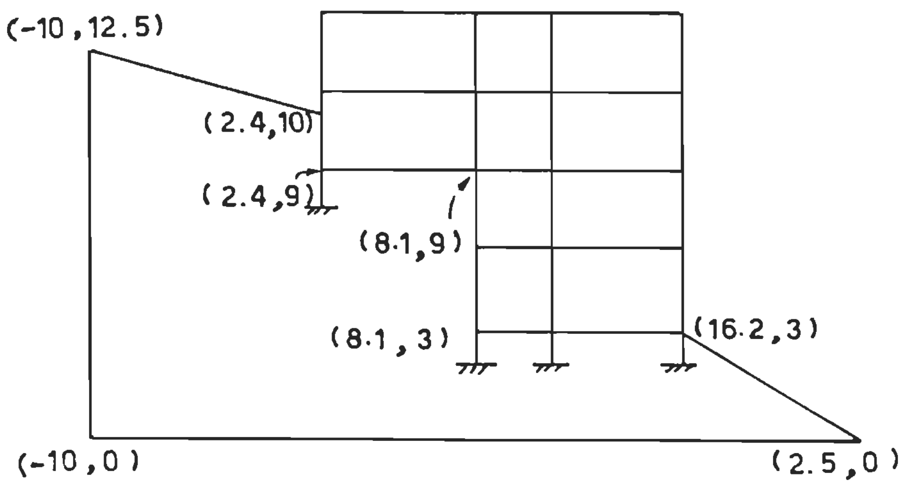


Fig. 6.18 - Cross Section of Slope with Building Frame

The coordinates of key points on the slope are shown in Figs. 6.17 and 6.18 in (m). The column loads obtained from analysis at the foundation level are taken for finding the factor of safety against sliding failure of slope. The building loads transmitted at the foundation level at locations A, B, C, D in the static condition and (static+earthquake) condition are shown in Table 6.2.

Table 6.2 - Building Loads(Static and Earthquake) [Example 1]

Location and size of Footing	Static Condition			(Static+earthquake) Condition		
	Vertical Load(Kg)	Lateral Load(Kg)	Bending Moment(Kg-m)	Vertical Load(Kg)	Lateral Load(Kg)	Bending Moment(Kg-m)
A2.45x2.35	35000	7120	6120	41700	17900	1500
B3.75x3.70	61300	4320	3550	63200	5100	2610
C3.75x3.70	82200	1610	1380	85600	246	239
D3.60x3.30	74200	8790	4360	83700	9380	3450

The factor of safety against sliding of slope is worked out for the three cases(i.e. natural slopes, slope with static building loads, slope with (static+earthquake) building loads and are shown in Table 6.3.

Table 6.3 - Factor of Safety Against Sliding

Minimum Depth of Slice(m)	Natural Slope	With static building load	With static and earthquake loads
1	1.017	0.923	0.738
2	1.190	0.923	0.738
3	1.411	0.934	0.746
4	1.631	1.462	1.231

The slope sliding surface for three cases with different minimum depths of slices are shown in Fig. 6.19. The first column of figures (a-d) shows that the factor of safety against sliding for free slope is greater than 1 and hence safe. The second column of figures (e-h) shows the sliding surface for slope with static building loads. The first three cases are not possible as the failure surface is passing through the building frame and the fourth is the possible case and the factor of safety is 1.462 and hence safe. The third column of figures (i-l) shows the sliding surface for slope with building loads in earthquake condition. Again the first three cases are not possible as the sliding surface is passing through the frame and only the fourth case is possible and the factor of safety is 1.231 and hence safe.

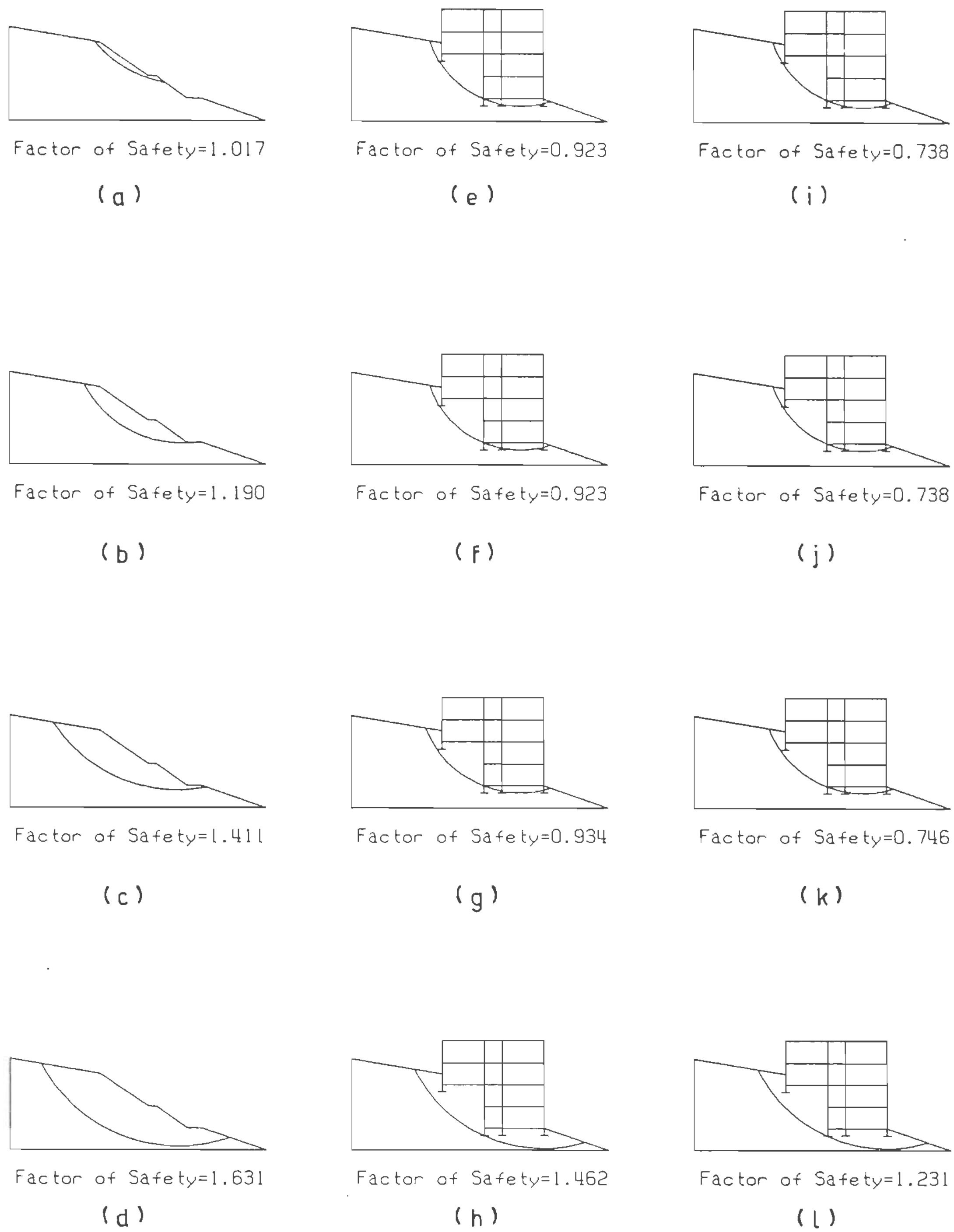


Fig. 6.19 - Slope failure Surface

6.4.2 Illustrative Example 2

(a) Natural Slope: The cross section of natural slope before construction of building is shown in Fig. 6.20.. The stability analysis of slope is carried out for different minimum depths of the slices.

(b) Hill Slope with Building Frame: The cross section of the hill slope with building frame is shown in Fig. 6.21.

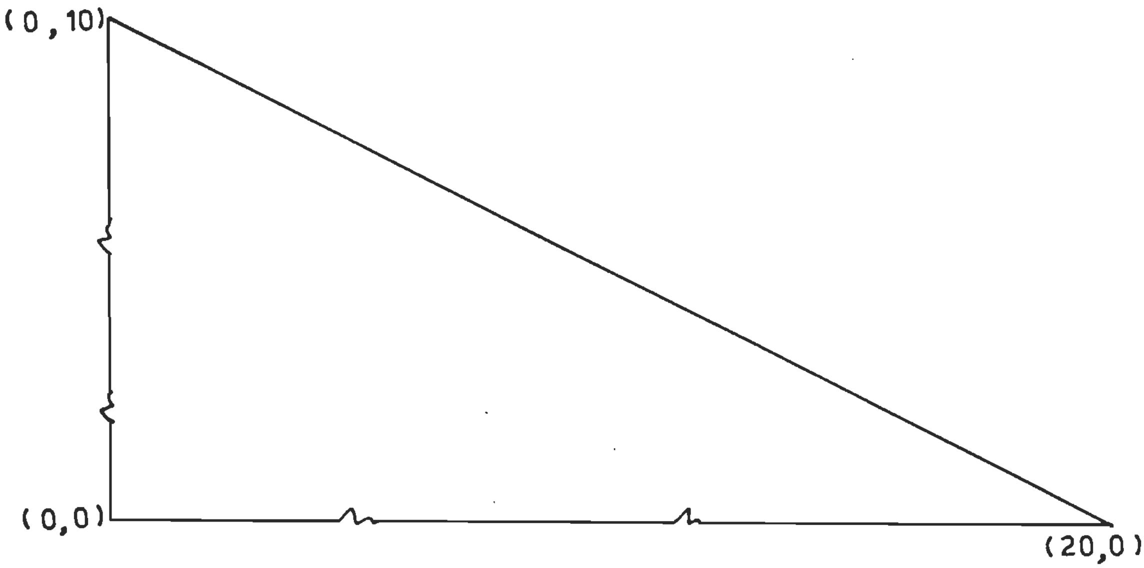


Fig. 6.20 - Cross Section of Natural Slope (Example1)

The coordinates of all the points on hill slope are shown in Figs 6.20 and 6.21. The density and angle of internal friction of soil are taken as 2000 kg/m^3 and 30^0. The cohesion of the soil is zero. The loads from building at A, B and C location of footings in the static and earthquake condition are shown in Table 6.4.

The factor of safety against sliding has been worked out and is shown in Table 6.5 for the three cases(i.e. natural slope, hill slope with static building loads and hill slope with earthquake loads.

The sliding surface for three cases with different minimum depths of slices are shown in Fig. 6.22. The first column of figures (a-d) shows that the factor of safety in all the cases is greater than 1 and hence the natural slope is stable. The second column of figures (e-h) shows sliding surface under the action of static building loads. The slope sliding surface in first three figures is not possible as it cuts the building frame and the fourth is possible and the factor of safety is 1.202 which is safe. The third column of figures (i-l) shows the sliding surface with building loads in earthquake conditions. In this case also the first three surfaces are not possible as these are cutting through the building frame and the fourth is possible and the factor of safety is 1.053.which is also safe.

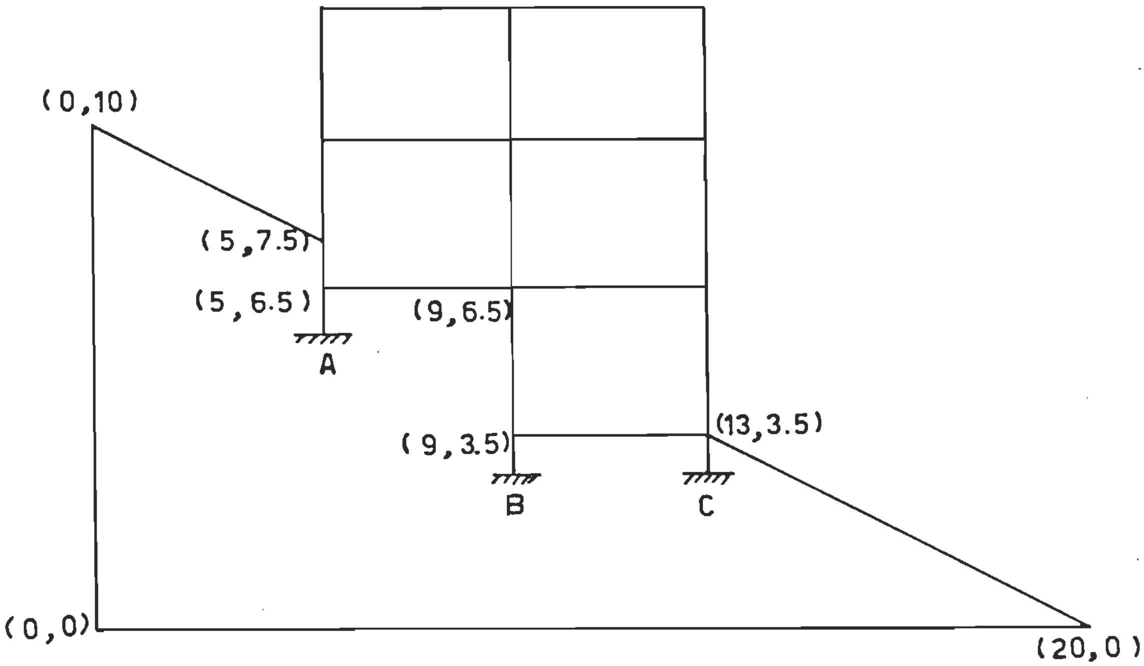


Fig. 6.21 - Cross Section of Hill Slope With Building Frame (Example 1)

Table 6.4 - Building Loads(Static and earthquake) [Example 2]

Location and size of Footing	Static Case			(Static+earthquake) Case		
	vertical load(Kg)	lateral load(Kg)	bending moment (Kg-m)	vertical load(Kg)	lateral load(Kg)	bending moment (Kg-m)
A(2.0x2.30)	38200	7630	7040	39100	12200	-10900
B(2.45x2.6)	52800	3020	87	53300	2880	-73
C(2.7x3.0)	57400	9950	4930	59000	10500	5350

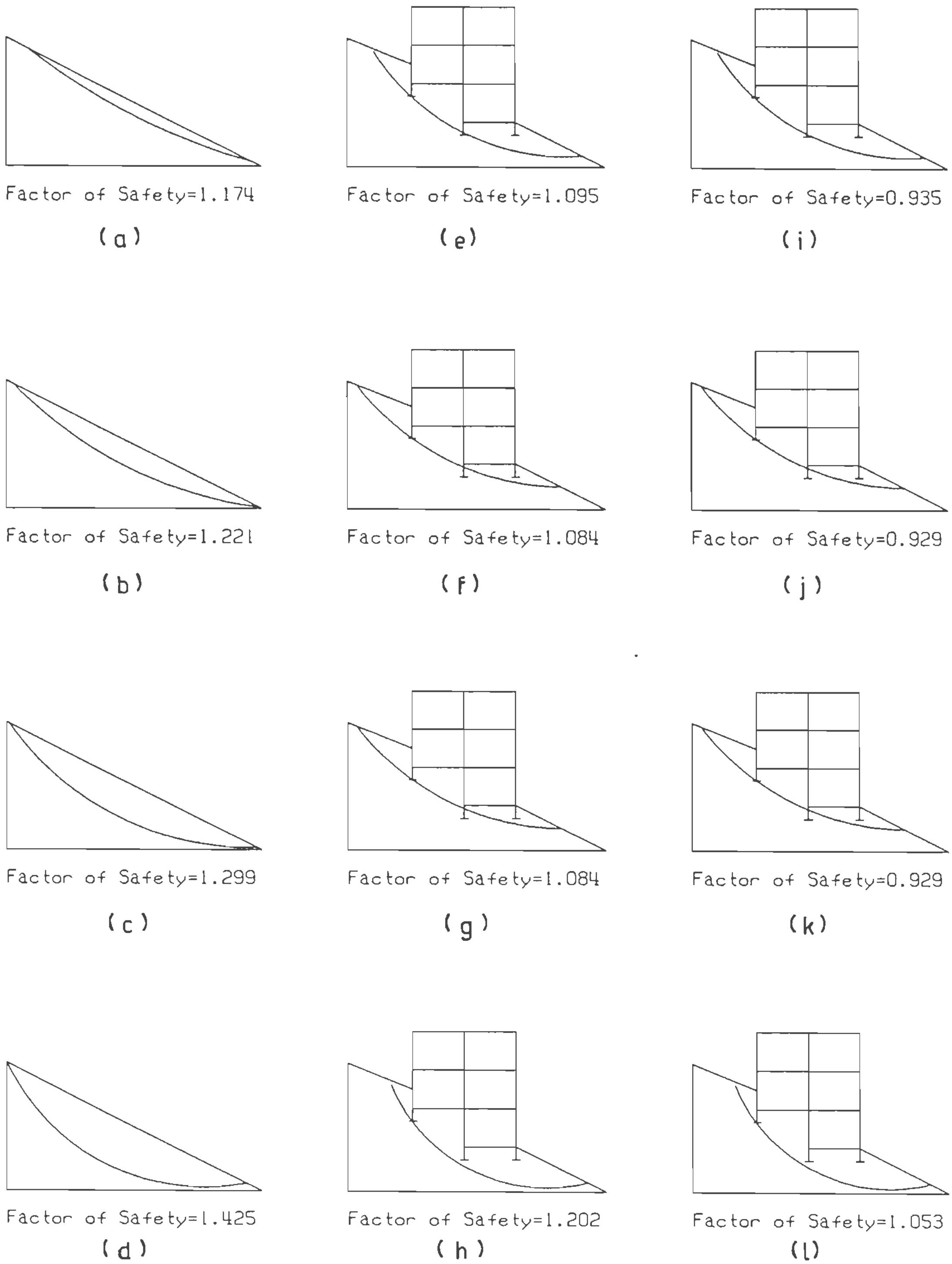


Fig. 6.22 - Slope failure Surface (Example 2)

Table 6.5 - Factor of Safety Against Sliding

Minimum Depth of Slice(m)	Natural Slope	With static building load	With static and earthquake loads
1	1.174	1.095	0.935
2	1.221	1.084	0.929
3	1.299	1.084	0.929
4	1.425	1.202	1.053

6.5 Elastic and Inelastic Time History Seismic Analysis

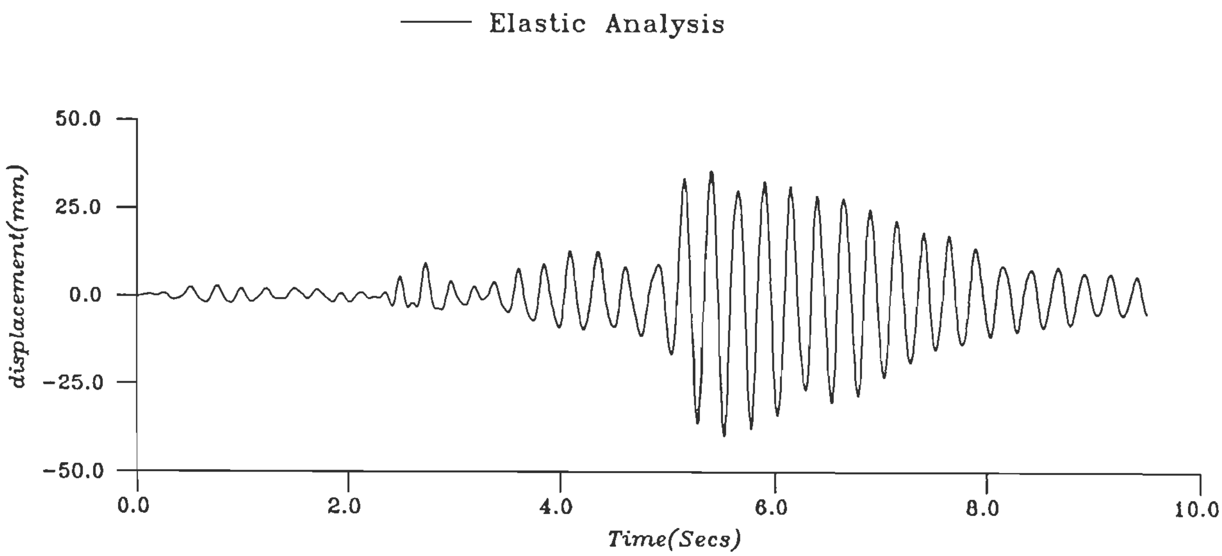
The Example 2 analysed already for seismic response using response spectrum method has been analysed for elastic and inelastic time history seismic analysis. The objective of this analysis is to investigate the probable failure zones in the building structure under the action of combined dead loads, live loads and the earthquake loads. In the present study Uttarkashi earthquake acceleration record has been taken as the base input motion for the analysis. The details of the Uttarkashi earthquake acceleration records are given in Appendix B.

6.5.1 Static Analysis: The building in Example 2 has been idealised using beam/column elements, 4 noded slab elements, 4 noded r.c.c. panels and 4 noded brick masonry infills. The dead loads and live loads used for static analysis is given in Appendix C. The building is analysed statically under the action of dead and live loads. The member forces are recorded due to dead and live loads.

6.5.2 Elastic Earthquake Analysis: The member forces obtained in the static analysis case are taken as the initial forces in the members of the structure and the elastic time history analysis is carried out under the action of Uttarkashi earthquake input motion. Rayleigh's damping has been assumed as 5% in the structure. The time step length of 0.005 secs has been taken. The time history displacement of the node c has been recorded and the member forces time history response has been recorded for member no. 10 as shown in Fig. 6.9. This building is a irregular structure having stepback configuration and is torsionally coupled.

6.5.3 Inelastic Earthquake Analysis: The inelastic time history analysis of the same building structure has been carried out under the action of Uttarkashi earthquake motion. The time history displacement of node c has been recorded and the member forces time history has been recorded for member no. 10.

The time history displacement of node c has been plotted obtained from elastic and inelastic seismic analysis in Fig. 6.23 and the bending moment at end i of the member 10 has been plotted obtained from elastic and inelastic seismic analysis in Fig. 6.24.



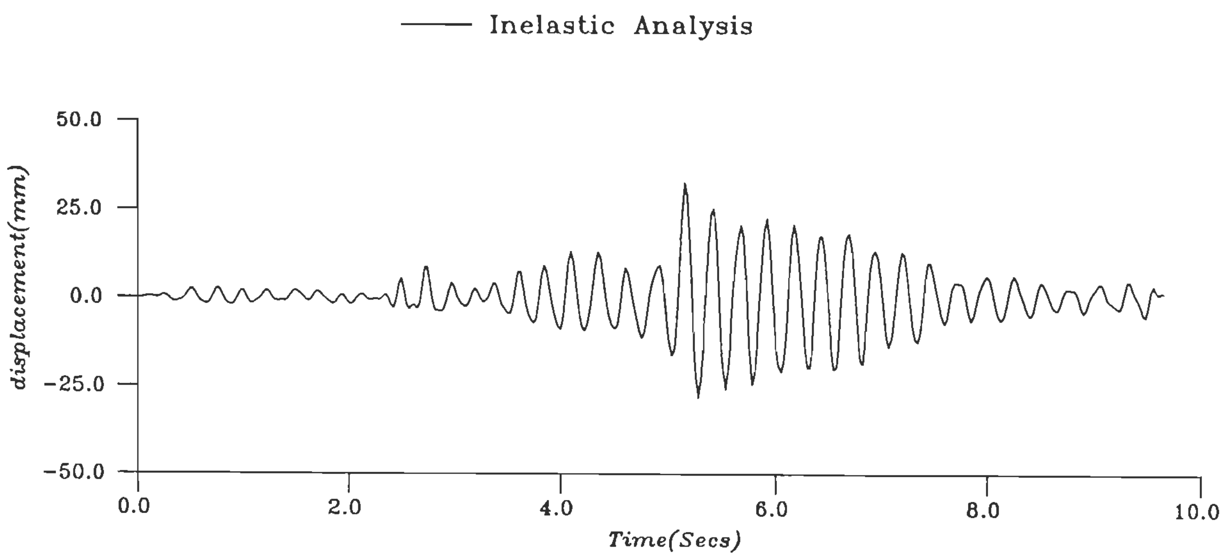


Fig. 6.23 - Time History Displacement of Node C in Elastic and Inelastic Analysis

Comparison of time history displacement of node c in z direction shows that the displacement of node c in the inelastic analysis is less than the displacement in the elastic analysis. The first peak deflection observed by elastic and inelastic analysis is almost at the same time. The time period of vibration gets elongated in the inelastic analysis due to yielding effects. The plastic hinges form for a very short period and disappear when the direction of vibration gets reversed. The comparison of bending moment in elastic and inelastic analysis for the member 10 shows that the bending moment predicted by inelastic analysis is less than that predicted by elastic analysis.

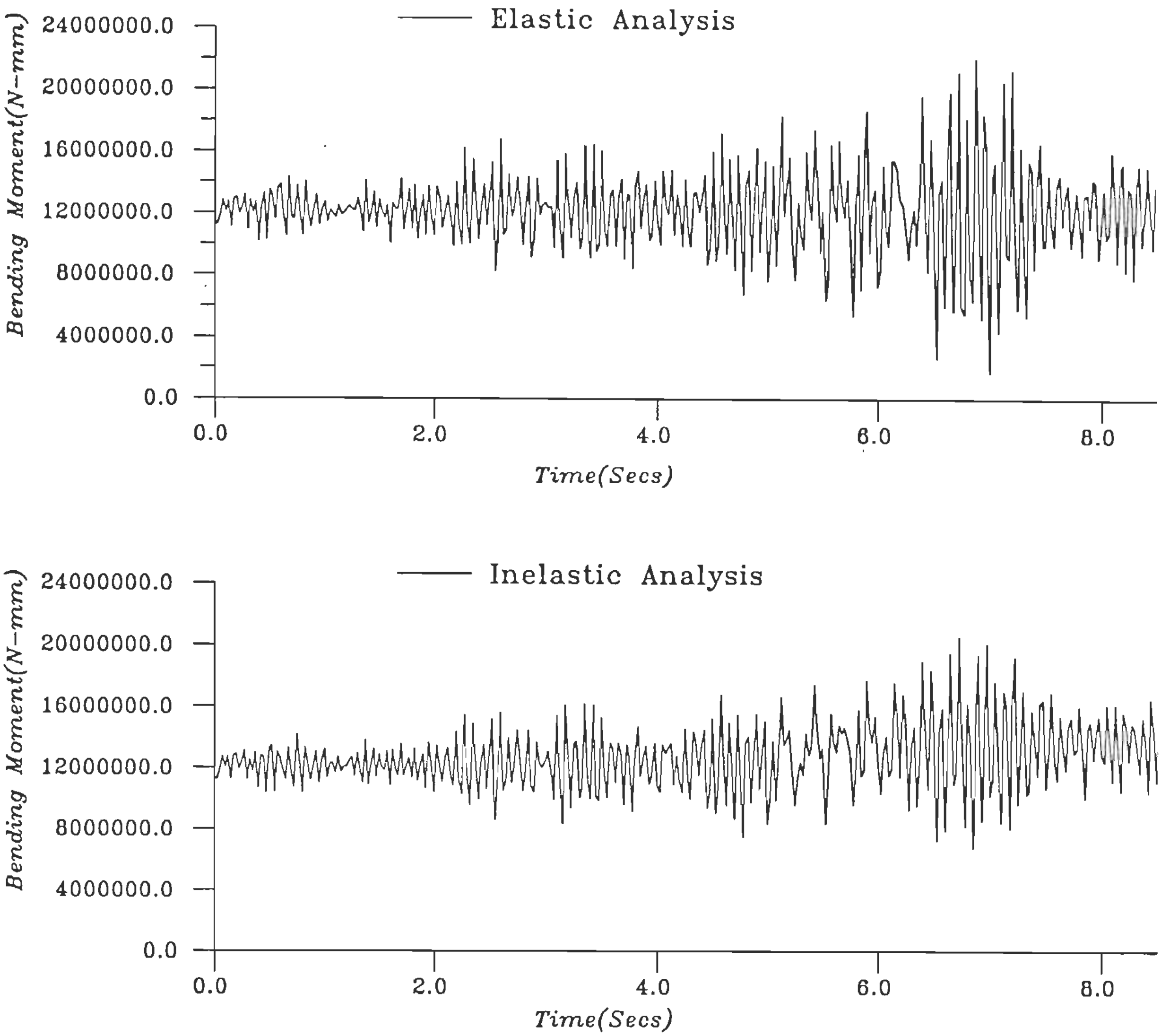


Fig. 6.24 - Time History Bending Moment of Member 10 at Node i in the Elastic and Inelastic analysis

6.5.4 Sequence of Hinge Formation: The member forces obtained from static loads + earthquake forces due to Uttarkashi earthquake excitation are obtained at various instant of time and checked against the yield criteria and the formation of hinges in the members at various instant of time has been recorded and are shown in Figs. 6.25-6.27. The formation of hinges, time of occurrence and the displacement at node c has been given in Table 6.6. The formation of cracks in the infill are presented in Table 6.7. The ductility requirement of the members are shown in Table 6.8. The formation of cracks at the gauss points in the infill panels are shown in Figs. 6.27. The variation of action-deformation for all the six components for the member 10 have been shown in Fig. 6.28 during the entire 10 secs of records.

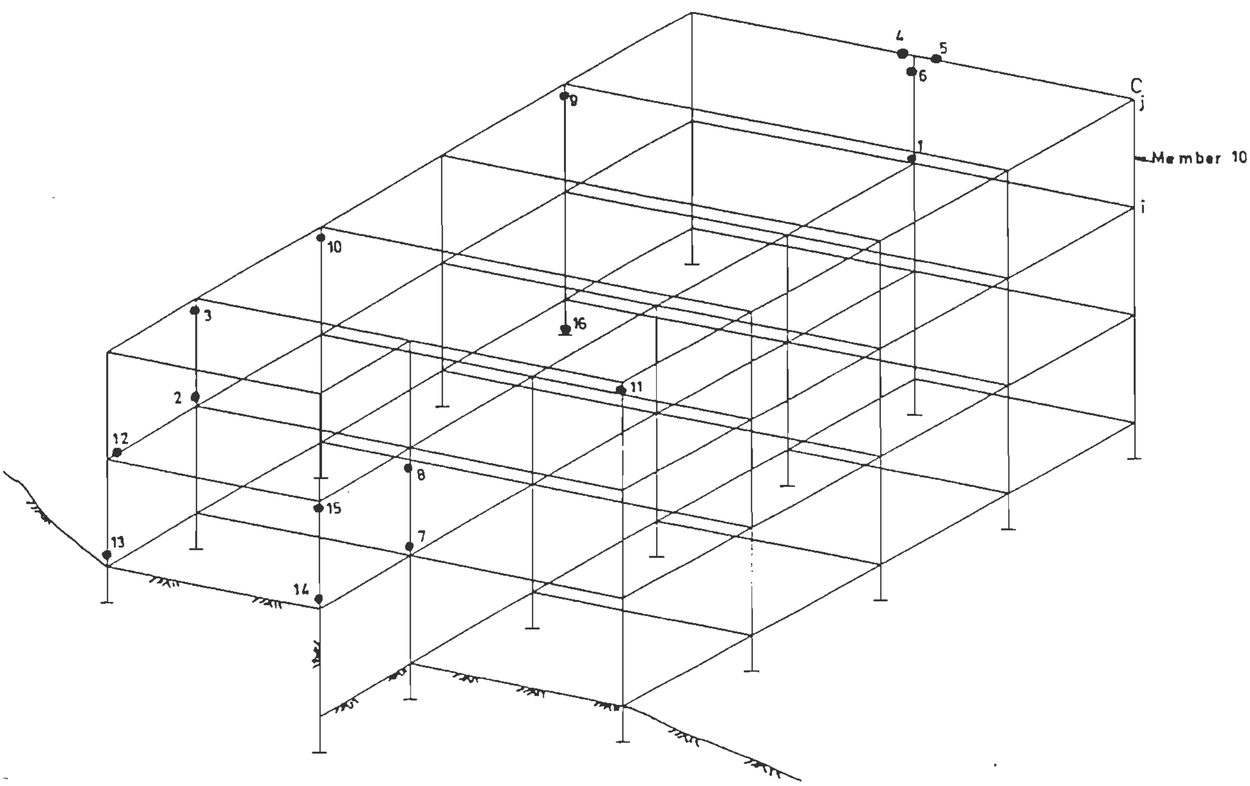


Fig. 6.25 - Location of Hinges Formed

Table 6.6 - Details of Plastic Hinge Formation

Plastic hinge no.	Member no.	Node	Time of occurence	Displacement of node c in z direction(mm)
1	7	1	5.045	-15.688
2	5	1	5.135	28.436
3	5	2	5.135	28.436
4	60	2	5.135	28.436
5	66	1	5.135	28.436
6	7	2	5.140	30.249
7	25	1	5.140	30.249
8	25	2	5.140	30.249
9	2	2	5.145	31.613
10	4	2	5.145	31.613
11	14	2	5.145	31.613
12	105	1	5.145	31.613
13	20	1	5.160	32.616
14	26	1	5.260	-26.207
15	26	2	5.260	-26.207

Table 6.7 - Formation of Cracks at Gauss Points

Crack No.	Gauss point	Member no.	Time of occurence	Crack angle
1	101	167	5.025	-43.196
2	102	167	5.030	-43.196
3	103	167	5.110	44.335
4	104	167	5.115	44.335

The Fig. 6.25 shows that the hinges are formed in the outer peripheral members of the structure and are formed mostly in columns. The Figs. 6.28(d) and (c) show that the column member 10 is subjected to severe torsional moment and lateral shears respectively under the action of earthquake load. The axial force and bending moments in the member are predominantly due to gravity loads. It shows that the irregular building components are subjected to severe torsion and shears under earthquake excitations, which in combination with axial forces, bending moments due to gravity loads leads to yielding of the members. It shows that the building behaves predominantly under the influence of torsional effects. The buildings on hill slope having stepback configurations are highly torsionally coupled and the yielding of columns takes place predominantly under the torsional coupling effects in such buildings. The torsional moment and shears in stepback building has led to high ductility demand in the columns located at the outer periphery of the stepback building. The ductility demand is defined as the ratio of maximum deformation to the yielding deformation during the whole time history acceleration input motion. The ductility requirement of yielded members shows that the member no. 2 and 7 will be damaged as the ductility demand of these members is around 8 but the building will not collapse when subjected to Uttarkashi earthquake.

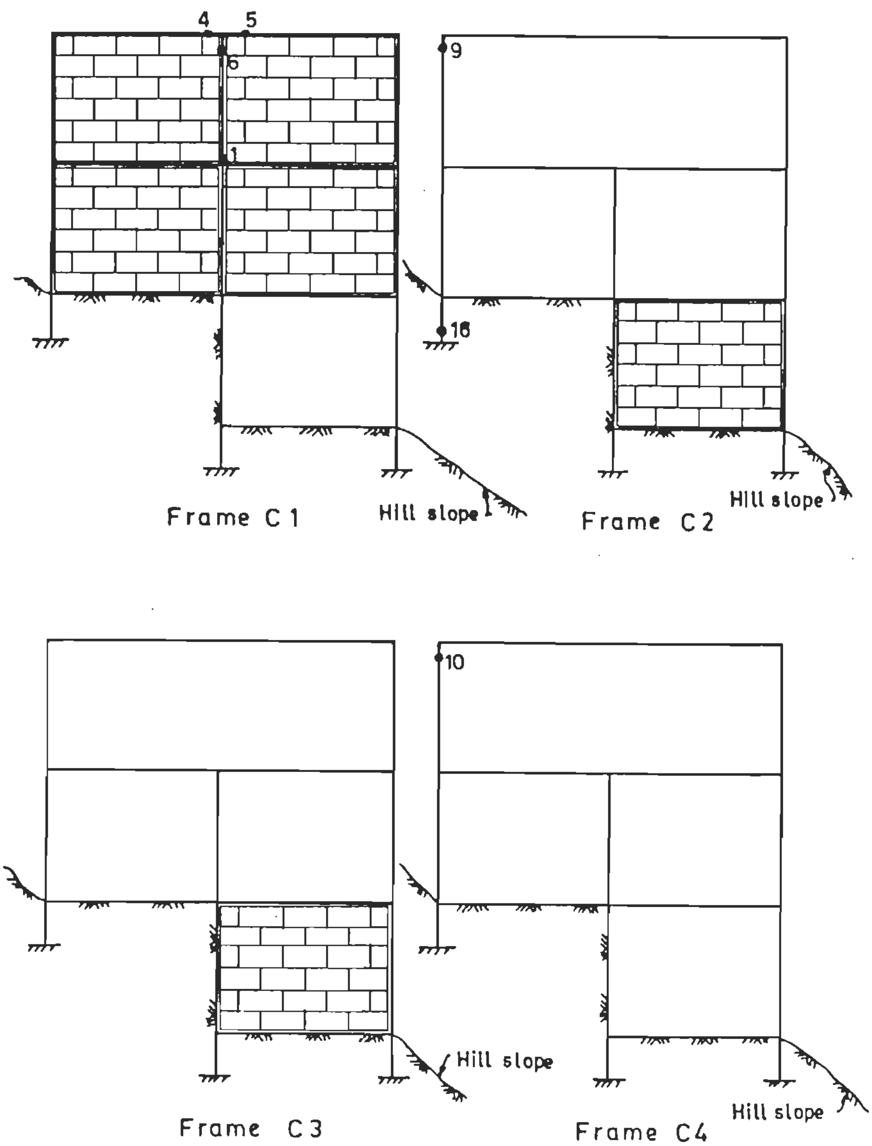


Fig. 6.26 - Location of Plastic Hinges Formed in Various Frames

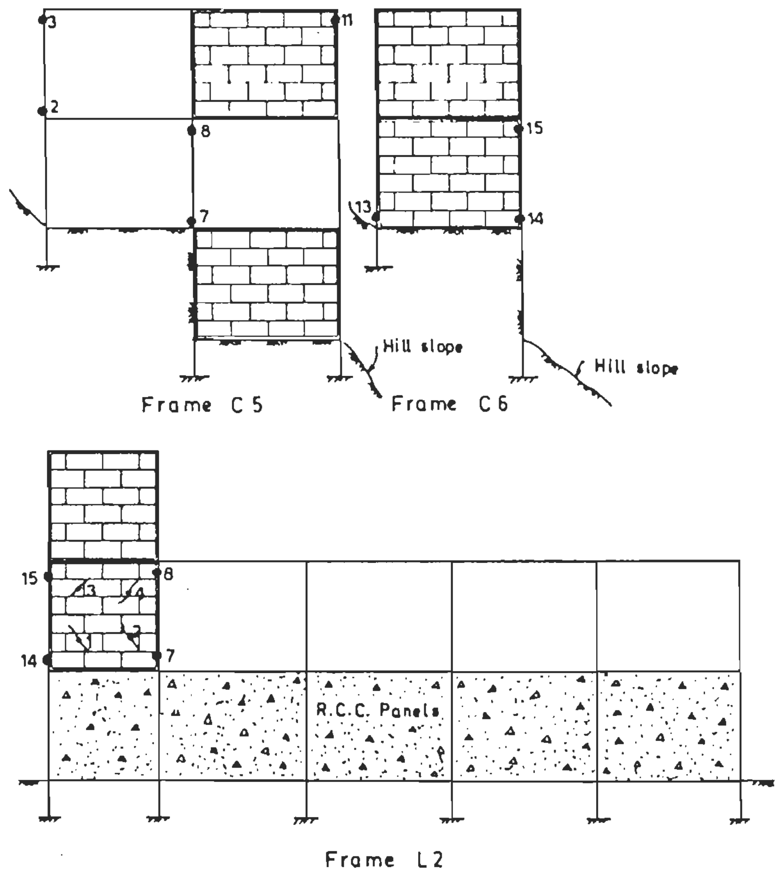


Fig. 6.27 - Location of Plastic Hinges and Cracks in Masonry Infills

Table 6.8 - Ductility Requirement of Various Members

Sr. No.	Member no.	Ductility demand
1	2	8.02
2	4	2.35
3	5	3.26
4	7	7.88
5	14	1.79
6	20	1.22
7	25	4.39
8	26	1.47
9	60	4.07
10	66	3.46
11	105	1.81

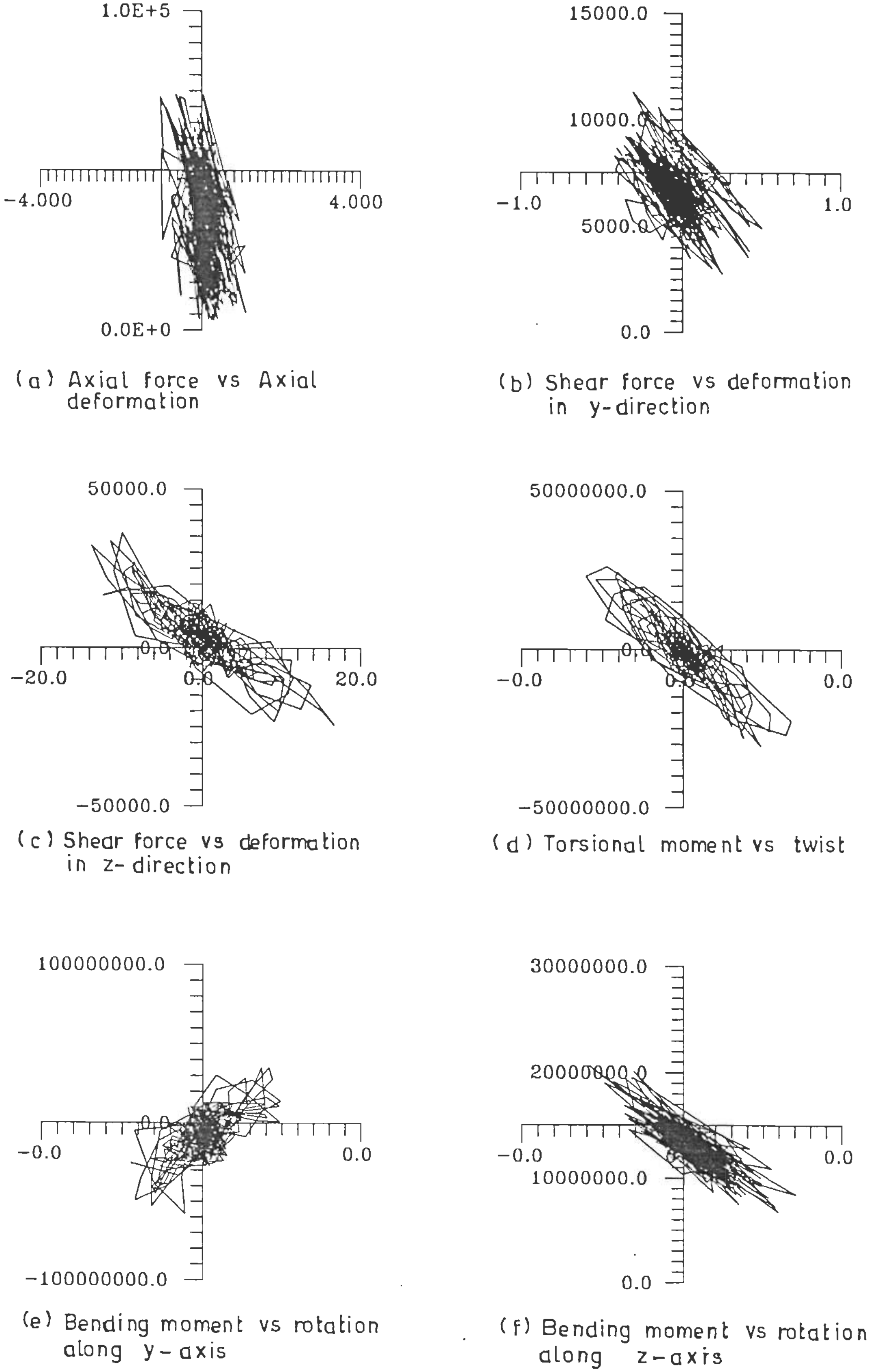


Fig. 6.28 - Variation of Action Deformation for Member 10 for Time History of 10 Secs

6.5.5 Ductility Demand: The inelastic analysis of the stepback building show that the maximum ductility demand of the yielded members is 8.02 under Uttarkashi earthquake excitation. The achievable ductility of r.c.c. frame members is about 10. Therefore the ductility demand of 8.02 can be met out of the ductility capacity of the r.c.c. members. The design ductility of 4 to 6 is preferable to be adopted in the design practice to avoid the severe damage of the yielded members.

6.6 Concluding Remarks

The two real building problems of hilly region has been analysed for the seismic response with two methods of analysis i.e. simplified method having rigid floor diaphragm idealisation with 3 d.o.f. per floor and rigorous method having 6 d.o.f. per node considering flexibility of floor/roof. It is found that the results obtained from simplified method is of the same order as the results obtained from rigorous method of analysis. It is recommended that simplified method be adopted in the Code of Practices (UBC, NBCC, NZS, I.S.: 1893) for 3D dynamic analysis of highly irregular buildings such as stepback, setback and a combination of stepback and setback.

The slope stability analysis for the two problems shows that the stability analysis is also of great importance because all the slopes may not be suitable from stability consideration for buildings. The slope may exhibit local or overall failure which is essentially required to be checked before going in for any kind of construction.

Inelastic earthquake analysis of the real building problem shows that the building on hill slope with stepback type configuration is more prone to torsion and results into yielding of the outer peripheral resisting elements of the building and demands greater ductility under earthquake excitations. Therefore using simplified method, various trial configurations of the buildings on hill slope may be made so as to get the minimum torsional coupling effects and then finally the structure may be analysed by rigorous method of analysis and designed.

6.7 References

1. NBCC(1990), National Building Code of Canada.
2. UBC(1991), Uniform Building Code, International Conference of Building officials, 5360 South Workman mill road, Whiitter, California 90601.
3. **Earthquake resistant regulations A World list-1992**, compiled by International Association for Earthquake Engg.
4. **New Zealand Standard NZS 4203(1992)**.
5. **I.S. 1893(1984)**, Criteria for Earthquake Resistant design of Structures, Indian Standard Institution, New Delhi, INDIA.

C.1 Loads for Analysis

(i) Floors

Live loads = 2000 N/m^2 for rooms
= 3000 N/m^2 for kitchen/dining hall
Dead loads due to slab = 2880 N/m^2
Dead load due to floor finish = 2400 N/m^2
Dead load due to plaster finish = 240 N/m^2
Total dead load = 5520 N/m^2

(ii) Roof

Dead loads due to slab = 2880 N/m^2
Dead loads due to floor finish = 2000 N/m^2
Dead load due to plaster finish = 240 N/m^2
Total dead load = 5120 N/m^2
Dead loads corresponding to self wt of beams and columns are taken separately.

(iii) Infill Panels

Dead load due to solid walls = 13500 N/m
Dead load due to walls with openings = 6750 N/m

(iv) Lateral Earth Pressure

Lateral earth pressure on the structure at the first floor and second floor level has been taken into account while analysing the structure with static loads. The density and angle of internal friction of soil are taken as 2000kg/m^3 and 30^0 respectively.

Masses for seismic analysis corresponding to full dead load and 25% of live loads has been taken into consideration.

Table C.1 - Comparison of Size of Simplified and Rigorous Models(Example 1)

Sr. No.	Description	Rigorous Method	Simplified Method
1	Total no. of nodes	222	222
2	Total no. of d.o.f.	1080	15
3	No. of beam elements	289	assumed rigid
4	No. of column elements	180	180
5	No. of slab elements	114	assumed rigid
6	No. of strut elements.	46	46

Table C.2 - Cross Section Details of Members

Particulars	Column/beam Size(bxd)	Concrete mix
All columns in third Floor	300mmx400mm	M_{15}
All columns in second Floor	300mmx400mm	M_{20}
All columns in first and ground floor	300mmx600mm	M_{20}
Cross beams in end Frames at plinth level	300mmx500mm	M_{15}
All other cross beams at plinth level	300mmx550mm	M_{15}
All beams in first Floor	300mmx500mm	M_{15}
All other beams	300mmx450mm	M_{15}

The slab thickness for roof, third floor, second floor and first floor and Plinth level is 120 mm and in the depressed portion it is 175 mm and it is reinforced with 8 mm ϕ @ 200mm c/c and 8 mm ϕ @ 250 mm c/c in two directions respectively. The thickness of the diaphragm wall is 190 mm and is reinforced with 8 mm ϕ @ 150 mm c/c. The thickness of solid filler brick masonry wall is 225 mm.

C.2 Loads for the Analysis

The dead loads and live loads for static analysis are taken as under

(i) Floors:

Dead load due to slab = 2880 N/m^2

Floor finish = 2400 N/m^2

Plaster finish = 240 N/m^2
Live load = 3000 N/m^2
Total load = 8520 N/m^2

(ii) Roof:

Dead load due to slab = 2880 N/m^2
Floor finish = 2000 N/m^2
Plaster finish = 240 N/m^2
Total load = 5120 N/m^2

(iii) Infill Panels:

Solid walls = 16200 N/m
Wall with opening = 8100 N/m

(iv) Lateral Earth Pressure:

Lateral earth pressure corresponding to soil having density as 2000 N/m^3 and ϕ = 30 has been taken into account for static analysis.

The mass corresponding to dead loads and 25% of live load has been taken.

Table C.3 - Comparison of Size of Simplified and Rigorous Models

Sr. No.	Description	Rigorous method	Simplified method
1	Total no. of nodes	76	76
2	Total no. of D.O.F.	354	12
3	No. of Beam elements	82	rigid
4	No. of column elements	59	59
5	No. of slab elements	27	rigid
6	No. of struts	17	17

The slab thickness for roof, second floor, first floor and ground floor is 120 mm and reinforcement is 8mm ϕ @ 180 mm c/c both ways. The r.c.c. Diaphragm wall is of 165 mm thickness and the reinforcement is 8 mm ϕ @ 220 mm c/c both ways. The thickness of brick masonry wall is 225 mm.

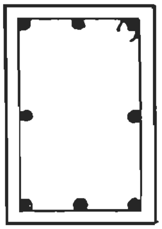
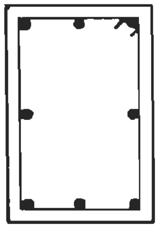
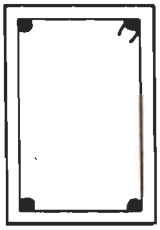
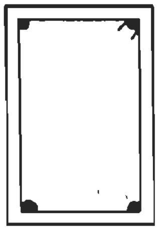
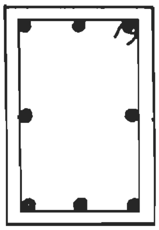
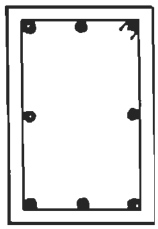
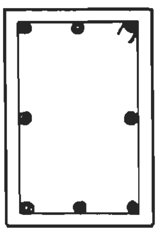
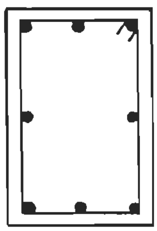
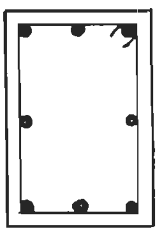
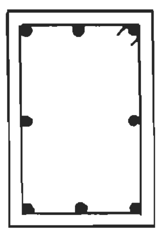
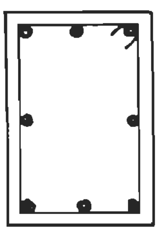
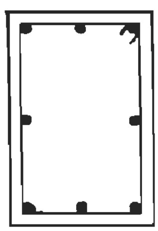
Member Nos.	Member Size and Reinforcement	Member Nos.	Member Size and Reinforcement
2,4,6 25,26	 300 x 450 8 – 16 mm ϕ	12	 300 x 450 4 – 20 mm ϕ 4 – 12 mm ϕ
1,5,60 64,65 66,67	 300 x 450 4 – 20 mm ϕ	15,19,43 47	 300 x 450 4 – 25 mm ϕ
3,17,22 23,24,29	 300 x 450 4 – 20 mm ϕ 4 – 16 mm ϕ	16,18,20	 300 x 600 4 – 18 mm ϕ 4 – 16 mm ϕ
7,8,9,21	 300 x 450 4 – 18 mm ϕ 4 – 12 mm ϕ	28,30	 300 x 600 4 – 20 mm ϕ 4 – 12 mm ϕ
10,14	 300 x 450 4 – 16 mm ϕ 4 – 12 mm ϕ	27,31 38,42	 300 x 600 4 – 18 mm ϕ 4 – 16 mm ϕ
11,13	 300 x 600 4 – 20 mm ϕ 4 – 18 mm ϕ	44,46, 48	 300 x 600 8 – 18 mm ϕ

Fig. C.1 - (a) Member Cross-Section and Reinforcement Details

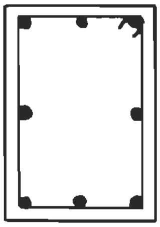
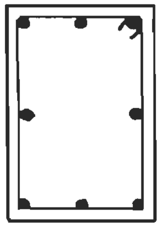
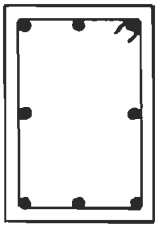
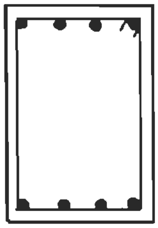
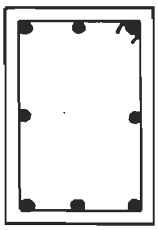
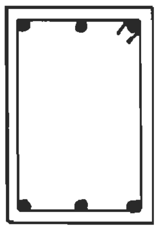
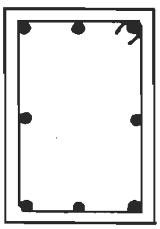
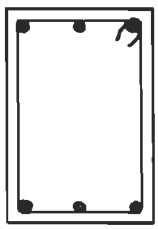
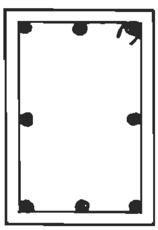
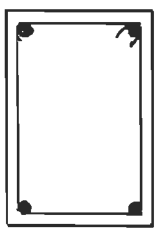
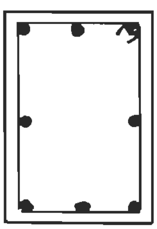
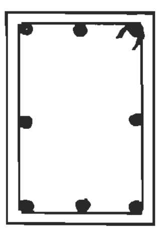
Member Nos.	Member Size and Reinforcement		Member Nos.	Member Size and Reinforcement	
36, 37, 45		300x 450 8- 20mm Φ	56, 58		300x600 4-25 mm Φ 4-20 mm Φ
32, 55, 59		300 x 450 8-18 mm Φ	80, 81, 82 86, 87, 88 90, 94		300x 450 8-20mm Φ
33, 34, 35 53, 54		300x 450 4-25mm Φ 4-20mm Φ	83, 84, 85 89, 79, 91 92, 93, 110 111, 112, 113 114, 119, 120 121, 122, 123 133, 134, 135 136, 137, 138 139, 140, 141		300x 450 6-20mm Φ
39, 41		300x 600 8-20 mm Φ	61, 62, 63		300 x 550 6-16mm Φ
40, 50, 51 52, 57		300 x 450 8-25mm Φ	95, 96, 97 98, 99, 100 101, 102 103, 104		300 x 350 4-20 mm Φ
49		300 x450 4-20mm Φ 4-18 mm Φ	68, 69, 70 71, 72, 73, 74 75, 76, 77 78, 105, 106 107, 108, 109 115, 116, 117 118, 124, 125, 126, 127 128, 129, 130 131, 132		300x 450 8-16mm Φ

Fig. C 1 - (b) Member Cross-Section and Reinforcement Details

C - 3 Ground Acceleration Record

The three translational components of ground acceleration records for the Uttarkashi Earthquake of Oct. 20, 1991 having magnitude of 6.6 on the Richter scale used in this investigation have been presented in Fig. C 2. The peak acceleration of Longitudinal component was 5209.12 mm/sec^2, of Transverse component was 3656.15 mm/sec^2 and of vertical component was 1895.98 mm/sec^2.

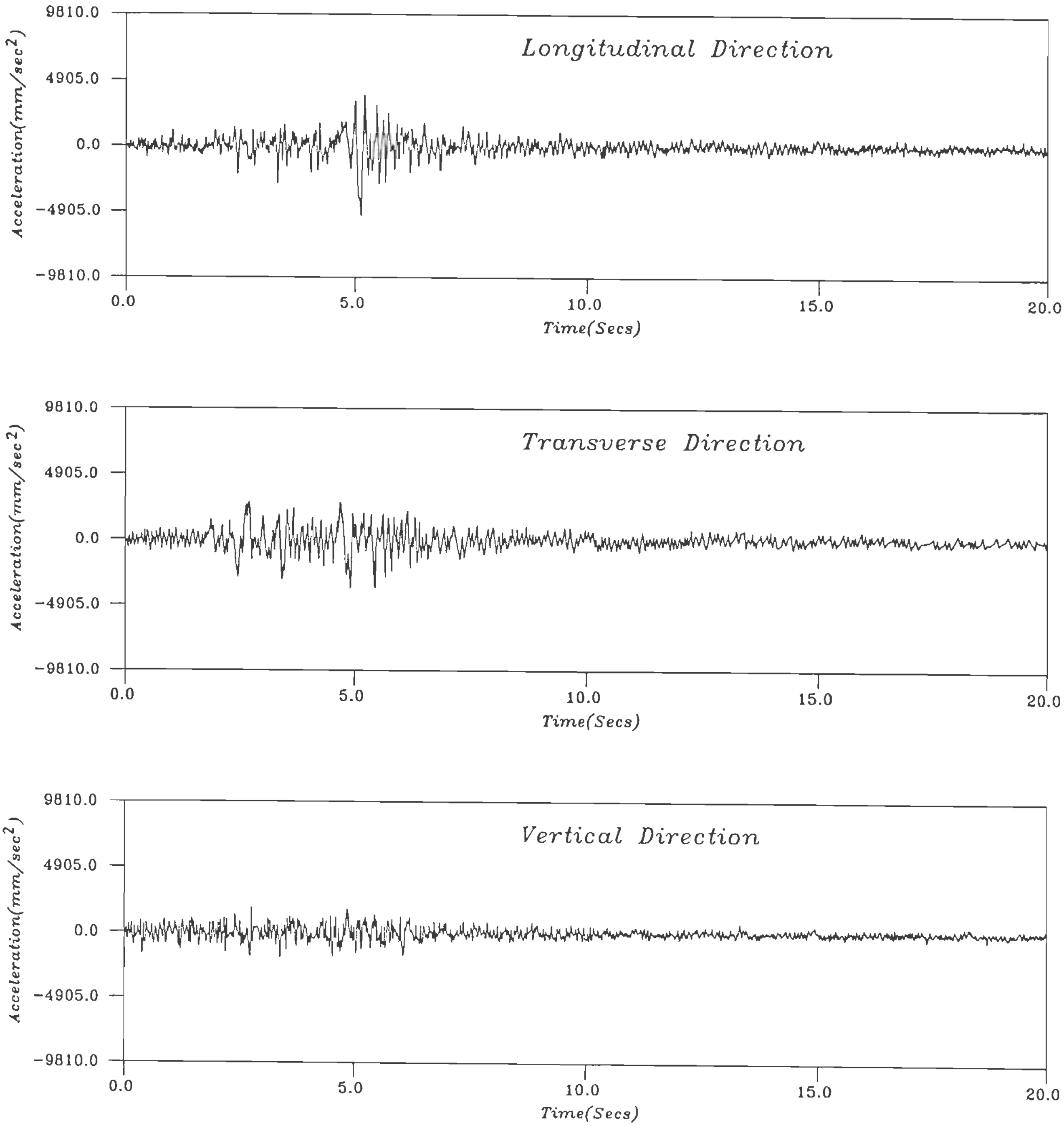


Fig. C.2 - Uttarkashi Earthquake Acceleration Record

STEPBACK AND SETBACK BUILDINGS

7.1 Introduction

Hilly areas are under going rapid changes due to economic development, which have marked effects on the buildings in terms of style, material and method of construction. Stone, load bearing and wooden building structures are common. The loss of lives and property are mainly due to the damage of these buildings during earthquakes. Due to harsh weather conditions and durability of concrete over bamboo and timber, reinforced concrete construction although very expensive is becoming more and more popular. However very few buildings receive the careful planning of Architect and Engineer's rigorous analysis and design. People construct their houses any way they like without thinking about the structural safety aspects. Most of the hilly regions of India are highly seismic, normally buildings are not designed for earthquake forces except for a very few government buildings. In the present Chapter, various types of differently configured r.c. framed buildings are described and studied from structural/seismic safety point of view under the action of dead, live and earthquake loads. The merits and demerits of one over the other has been highlighted. Soil structure interaction studies has been carried out for few problems of stepback and setback buildings.

7.2 Various Configurations of R.C.C. Framed Buildings

The r.c.c. framed buildings having different configurations can be constructed on a flat and sloping ground. The buildings on a flat ground may have regular or setback configurations as shown in Figs. 7.1 (a) and (b). The buildings on a sloping ground may have stepback or a combination of stepback and setback configurations as shown in Figs. 7.2 (a) and (b).

Buildings on a flat ground are constructed first by cutting the slope and levelling the ground and on sloping ground are constructed with little cutting and filling of the hill slope. The earth on one side of the building may be touching with the building at various levels which will be supported by r.c.c. panels retaining walls or by separating the earth from the building by providing stone masonry retaining walls at different levels.

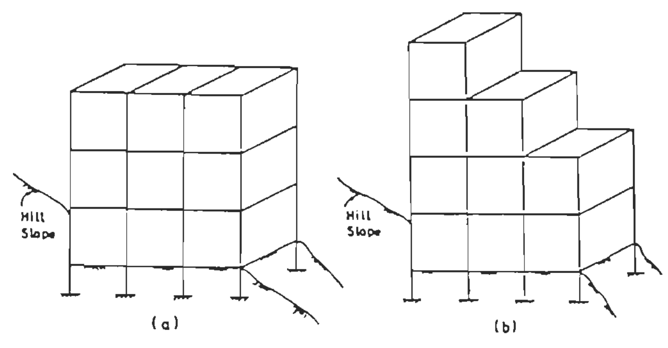


Fig. 7.1 - Buildings on Flat Ground

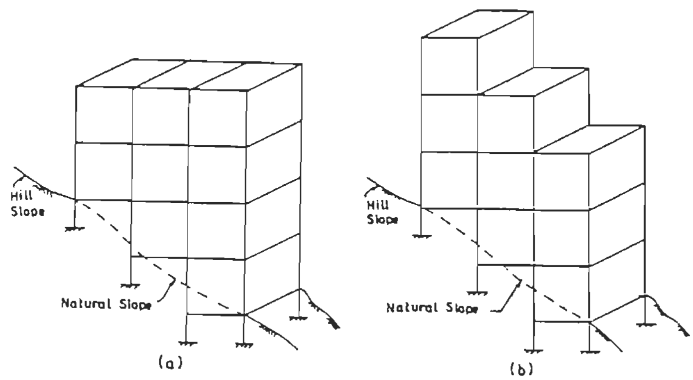


Fig. 7.2 - Buildings on Sloping Ground

Such buildings needs to be analysed completely for the super structure and the sub structure. The super structure means the framed structure and the sub structure means the foundation which has to be checked for bearing capacity and stability of slope with building loads. Super structure of a buildings on flat ground can be analysed in the same manner as is done for buildings in plains. In this Chapter stepback and setback buildings are completely analysed from structural safety point of view for the super structure as well as the sub structure. Firstly the buildings are analysed under the action of dead and live loads, then under the combined action of dead, live and the earthquake loads. The stability analysis of the slope is carried out with building loads for different configurations of the buildings. The overall merits and demerits of the different configurations of the framed buildings are evaluated.

7.3 Structural/Seismic Behaviour of Stepback and Setback Buildings

To evaluate the structural/seismic behaviour two differently configured buildings are selected such that the plinth and the usable covered floor area are the same. The dead and live loads for both the buildings are taken as under.

(i) Roof

Earth fill 100 mm thick = 1800 N/m^2
Tile terracing 40 mm thick = 800 N/m^2
Self wt. of slab 150 mm thick = 3750 N/m^2
Live load on roof = 1500 N/m^2
Total load on roof = 7850 N/m^2

(ii) Floors

Screed 50 mm thick = 1000 N/m^2
Surface topping 40 mm thick = 1000 N/m^2
Self wt. of slab 150 mm thick = 3750 N/m^2
Live load on floors = 2000 N/m^2

The seismic coefficient for calculating the horizontal seismic force acting at the centroid of slices of soil for checking the stability of slope has been taken as 0.1. The material properties used for all the problems studied in this Chapter are as under:

Concrete Mix M15

Compressive strength of concrete = 15 N/mm^2
Ultimate concrete strain = 0.0038
Yield strain = 0.0020
Young's modulus of elasticity = 22076 N/mm^2
Density of concrete = 24000 N/m^3
Ultimate tensile strength of concrete = 2.43 N/mm^2

Steel Fe 415

Yield Strength of steel = 415 N/mm^2
Young's modulus of steel = 200000 N/mm^2

Soil Properties

Density of soil = 2000 Kg/m^3

Angle of internal friction = 30^0
Cohesion = 0.0

7.3.1 Structural/Seismic Behaviour of Super Structure of R.C.C. Framed Buildings

The two building frames shown in Figs. 7.3(a) and (b) having stepback and a combination of stepback and setback configurations respectively are taken for studying the seismic response. In both the problems plinth area and the usable covered floor area are kept the same.

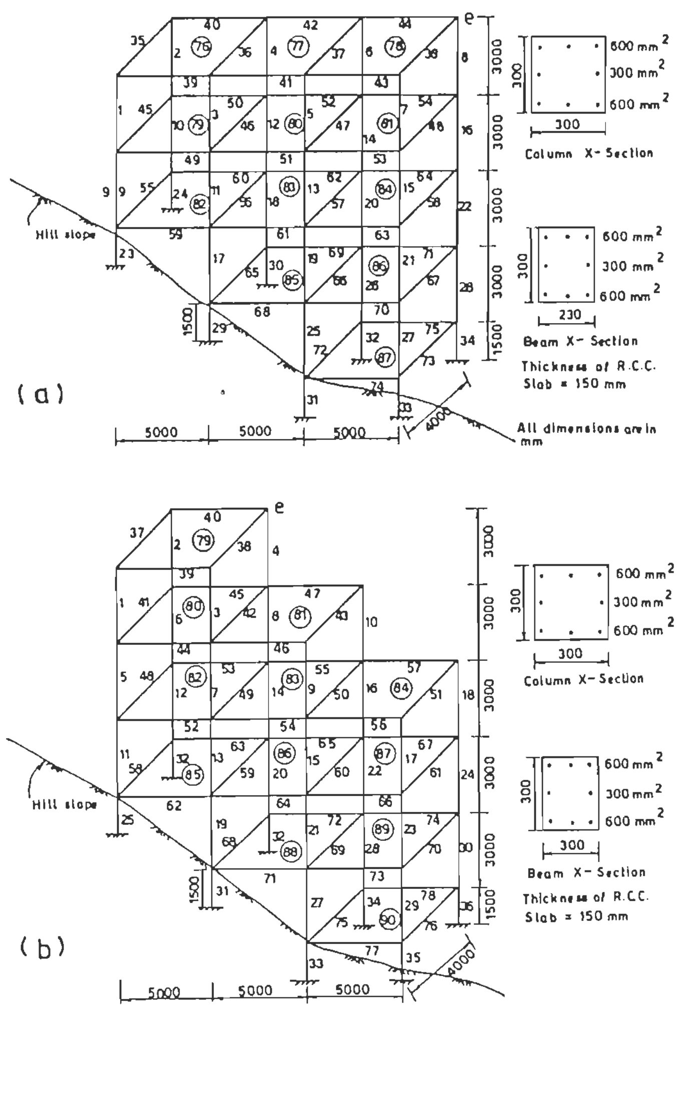


Fig. - 7.3 Stepback and Setback Building Frames

(a) Static Analysis

Both the problems are first analysed under the action of dead and live loads. The maximum axial force and bending moments obtained for columns and beams for two types of buildings are shown in Table 7.1.

Table 7.1 - Comparison of Axial Forces, Bending Moments for Stepback , Stepback and Setback Buildings

Sr. no.	Particulars	Forces	Stepback frame	Stepback and setback frame
1	Column	Axial force Bending moment	0.469+06 N 0.249+08 N-mm	0.430+06 N 0.247+08 N-mm
2	Beam	Bending moment	0.352+08 N-mm	0.351+08 N-mm

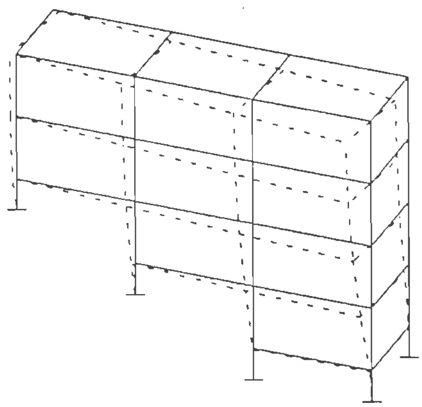
The results of maximum axial forces, bending moments for columns and beams under the action of dead and live loads obtained from static analysis does not show much difference for the two buildings for design purpose. The members of the two buildings are designed for these forces and moments and same size of beams and columns are used for both the problems. The cross section and reinforcement details for beams and columns are shown in Fig. 7.3.

(b) Free Vibration Analysis

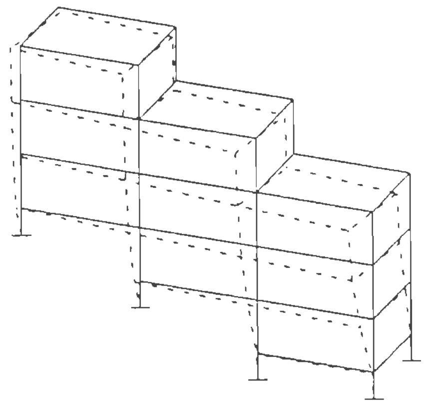
These two buildings are analysed for studying the seismic response. The free vibration time periods obtained for two buildings are shown in Table 7.2. The first three mode shapes for both the buildings are shown in Fig. 7.4.

Table 7.2 - Free Vibration Time Periods(Secs)

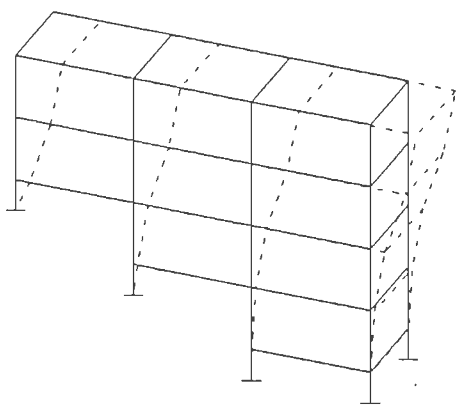
Mode	Stepback frame	Stepback and setback frame
1	0.57934	0.54415
2	0.35183	0.38493
3	0.28202	0.37839
4	0.23345	0.21472
5	0.17924	0.20022
6	0.16662	0.16815



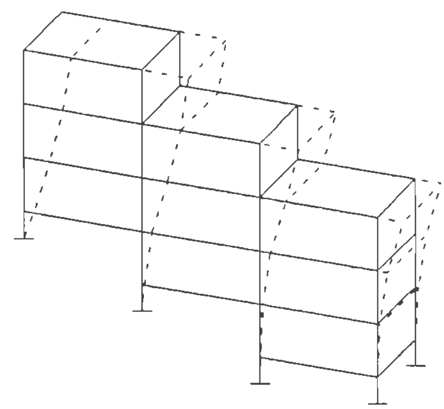
First Mode Shape T= 0.5793 Sec



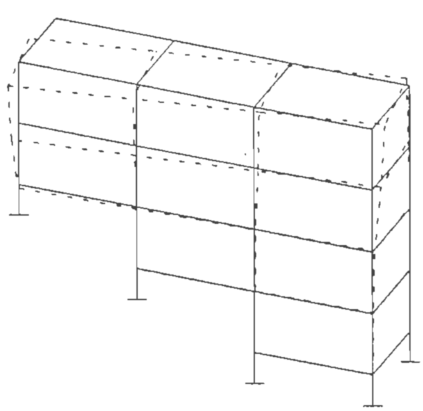
First Mode Shape T= 0.5442 Sec



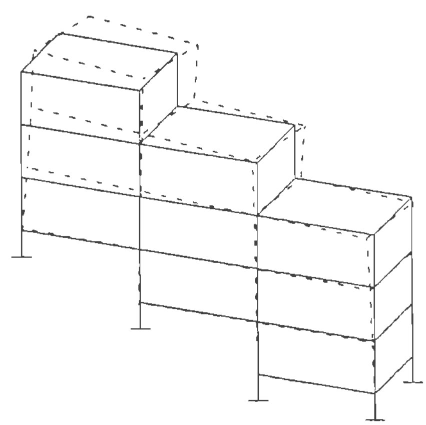
Second Mode Shape T= 0.3518 Sec



Second Mode Shape T= 0.3849 Sec



Third Mode Shape T= 0.2820 Sec



Third Mode Shape T= 0.3784 Sec

Fig. 7.4 - First Three Mode Shapes

The first mode of vibration for stepback building is a combination of translational and torsional vibration, second mode is pure translational and the third mode is again a combination of translational and torsional vibration. The first mode of vibration of stepback and setback building is combination of translational and torsional vibration but the torsional coupling effect is less in this building as compared to stepback building. Second mode of vibration is pure translational and third mode is predominantly torsional. The free vibration time period in the fundamental mode obtained for stepback frame is 0.57934 and for a combination of stepback and setback frame is 0.54415. The mode shapes show that the stepback type building is more prone to torsion as compared to combination of stepback and setback type building.

(c) Inelastic Seismic Analysis

Both the problems are further analysed for inelastic seismic behaviour under the action of Uttarkashi earthquake applied across the slope. The static forces in the members due to dead and live loads are taken as the initial forces. It is combined with seismic forces at various instant of time and then finally checked against the yield criteria. The sequence of formation of hinges, their time of occurence and ductility requirement of the yielded members in the stepback building are shown in Table 7.3 and for a combination of stepback and setback building are shown in Table 7.4. It is expected that the excitation across the slope will result substantial torsion in the buildings.

Table 7.3 - Sequence of Plastic Hinge Formation in Stepback Building

Plastic hinge No.	Excitation Across the Slope			
	Member No.	Node	Time of occurence	Ductility requirement.
1	23	1	4.775	1.98
2	24	1	4.790	2.03
3	30	1	5.140	1.53
4	29	1	5.145	1.45
5	67	1	5.165	1.42
6	66	1	5.180	1.35
7	65	1	5.205	1.16
8	57	1	5.215	1.51
9	58	1	5.225	1.63
10	48	1	5.265	1.13

Table 7.4 - Sequence of Plastic Hinge Formation in Stepback and Setback Building

Plastic hinge No.	Excitation Across the Slope			
	Member No.	Node	Time of occurence	Ductility requirement.
1	25	1	4.800	2.26
2	26	1	4.810	2.18
3	32	1	5.130	1.72
4	31	1	5.135	1.59
5	70	1	5.165	1.53
6	69	1	5.180	1.27
7	60	1	5.220	1.35
8	61	1	5.225	1.53
9	50	1	5.275	1.04

Inelastic seismic analysis show that the yielding takes place in 4 columns and 6 beams in stepback building and in 4 columns and 5 beams in stepback and setback building. The sequence of formation of plastic hinges in both the buildings are shown in Figs. 7.5 (a) and (b). It shows that the more number of outer peripheral resisting elements are yielded in case of stepback building as compared to stepback and setback indicating that the stepback buildings are more prone to torsion as compared to a combination of stepback and setback.

The actual earthquake produces motion in all the three directions. Therefore to study the inelastic seismic behaviour, these buildings are subjected to simultaneous action of three components of Uttarkashi earthquake excitation i.e. along the slope, across the slope and in vertical directions. The sequence of formation of plastic hinges and the ductility requirement of yielded members for stepback building are given in Table 7.5 and for stepback and setback building are given in Table 7.6.

Inelastic seismic behaviour shows that in the stepback building 14 members yielded whereas in the stepback and setback building only 10 members yielded. The sequence of formation of plastic hinges in various members are shown in Figs. 7.6 (a) and (b) for both the buildings. The maximum ductility requirement of yielded members in stepback building is 5.87 and that in stepback and setback building is 4.38. It shows that the stepback type buildings will be damaged more as compared to a combination of stepback and setback buildings under Uttarkashi earthquake motion.

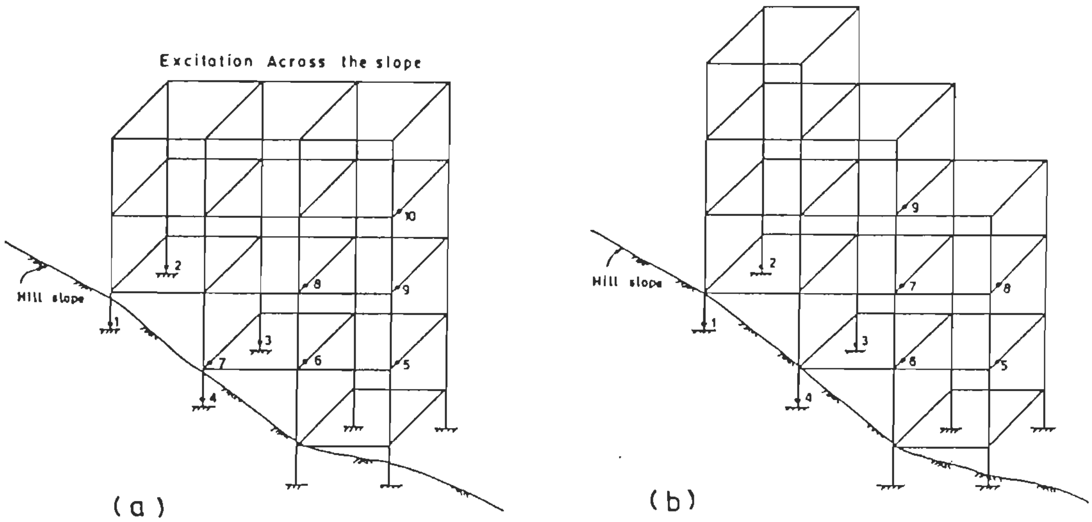


Fig. 7.5 - Location of Plastic Hinges

Table 7.5 - Sequence of Hinge Formation Under Excitation in Three Directions

	Excitation Across the Slope			
	Stepback Frame			
Plastic hinge No.	Member No.	Node	Time of occurence	Ductility requirement.
1	58	2	2.815	1.67
2	57	2	2.835	1.75
3	23	1	2.840	4.47
4	29	1	2.875	2.30
5	30	1	3.155	3.62
6	67	1	3.215	2.21
7	66	1	3.240	1.63
8	56	1	3.265	1.57
9	57	1	3.290	1.78
10	24	1	3.310	5.12
11	24	2	3.675	5.83
12	9	1	5.145	5.87
13	23	2	5.145	2.20
14	69	2	5.155	1.30

Table 7.6 - Sequence of Hinge Formation Under Excitation in Three Directions

Plastic hinge No.	Excitation Across the Slope			
	Stepback and setback Frame			
	Member No.	Node	Time of occurence	Ductility requirement.
1	25	1	2.720	3.87
2	61	2	2.810	1.43
3	60	2	2.815	1.39
4	26	1	2.830	4.38
5	31	1	3.190	2.37
6	32	1	3.200	3.17
7	70	1	3.210	1.69
8	69	1	3.240	1.57
9	26	2	5.140	3.08
10	45	2	5.295	2.03

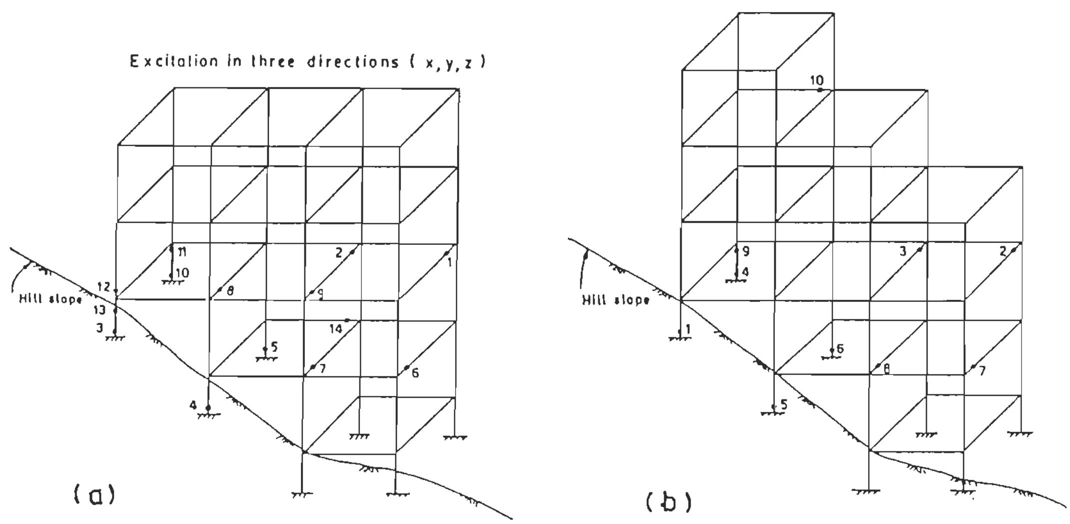


Fig. 7.6 - Formation of Plastic Hinges Under Three Component Earthquake Motions

One more configuration of building has been analysed by keeping the usable covered floor area same and by increasing the plinth area as shown in Fig. 7.7 (a). The free vibration time periods for first six modes are 0.46876, 0.27881, 0.27283, 0.25934, 0.20743, 0.19418 secs. The comparison of time periods with the other two configurations of building analysed earlier in this Chapter shows that this configuration gives stiff behaviour. The inelastic seismic behaviour has been studied for the configuration under Uttarkashi earthquake excitation applied in z direction(i.e. across the slope). The sequence of formation of hinges in this configuration of building is shown in Fig. 7.7 (b). In this configuration of building only 8 members yielded and the maximum ductility requirement of the yielded members in this case is 2.47. This configuration again gives better response as compared to the stepback type of configuration.

The inelastic seismic behaviour under the action of Uttarkashi earthquake excitation in three directions show that the number of yielded members and ductility requirement of yielded members are more as compared to uni-directional excitation. The inelastic seismic behaviour, ductility requirement of yielded members, free vibration time periods and mode shapes indicates that the stepback building is more vulnerable to earthquakes than a combination of stepback and setback building. It has been observed that the more number of outer peripheral members are yielded in case of stepback building than a combination of stepback and setback indicating that stepback building is more torsionally unbalanced as compared to stepback and setback building. It shows that the stepback and setback configuration neutralizes the torsional coupling effect under the action of earthquakes. Incidentally the outer profile of the stepback and setback buildings follows the natural slope of the ground, which is architecturally more acceptable in hill areas. Moreover climatic conditions of the hill areas demands that the buildings should so designed that these should be energy efficient. The stepback and setback buildings fulfills these requirement in a way that the natural light and direct sun heat will be available at various floor levels, which will help in reducing the requirement of conventional sources of energy. Keeping in view the structural safety of the buildings under earthquake loads, it is recommended that a combination of stepback and setback buildings are more suitable on sloping grounds. It has also been observed that the short columns are damaged in both types of buildings. Therefore the design details of short columns needs more attention. The remedial measure for the short columns is that either these are to be avoided altogether or to be designed adequately.

7.3.2 Stability Analysis of Slopes with Building Loads

The configuration of building has a great impact on the stability of slope. The super structure of the building(i.e. framed structure) on a flat ground and on a sloping ground can be designed for all the configurations catering to the needs of stresses induced in the members using proper designs, rich quality materials etc. The stability of slope is more of a natural phenomena, depending upon the properties of the soil, building loads transferred to the slope, location of loads, type of loads, drainage conditions of the area, climatic conditions of the area. The overall safety of the building on hill slope depends upon the stability of slope. If the slope is not stable, a sound building properly designed and constructed, still may collapse. Therefore the stability analysis of the slope with building loads under static and earthquake loads has been carried out to see the suitability of the different configurations of building. The different configurations on a flat ground and a sloping ground described earlier are taken for calculating the factor of safety against sliding failure of slope i.e. (i) buildings on a flat ground((a) regular framed buildings, (b) setback framed buildings) (ii) buildings on sloping ground((a) stepback framed buildings, (b) stepback and setback framed buildings).

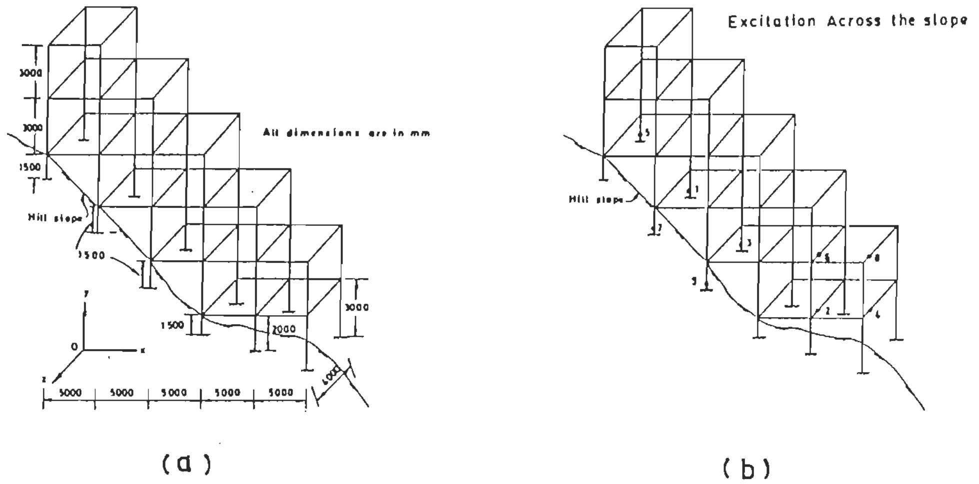


Fig. 7.7 - Stepback and Setback Building and Formation of Plastic Hinges

(i) Buildings on Flat Ground

The two buildings on flat ground adjacent to hill slope are (i) regular framed building, (ii) setback framed buildings are taken for studying the stability of slope. The column loads at foundation level obtained from static and seismic analyses are applied to the slope. The minimum factor of safety against sliding failure of slope for the two buildings under the static and seismic conditions are evaluated. The loads transferred at foundation level are shown in Table 7.7 for both the configurations. The factor of safety obtained in static load condition for free slope, slope with regular framed building, slope with setback framed building are 1.469, 1.411, 1.446 and the sliding surface are shown in Fig. 7.8. The sliding failure shows that chances of local failure are more when buildings are constructed adjacent to hill slope.

Table 7.7 - Column Loads Transferred at Foundation Level

Regular framed building				Setback framed building			
Column	Axial load(N)	Shear force(N)	Moment (N-mm)	Column	Axial load(N)	Shear force(N)	Moment (N-mm)
1	0.214+06	0.131+05	0.650+07	1	0.270+06	0.132+05	0.664+07
2	0.427+06	0.125+04	0.636+06	2	0.483+06	0.137+04	0.641+06
3	0.427+06	0.125+04	0.636+06	3	0.376+06	0.133+04	0.726+06
4	0.214+06	0.131+04	0.650+07	4	0.160+06	0.132+05	0.652+07

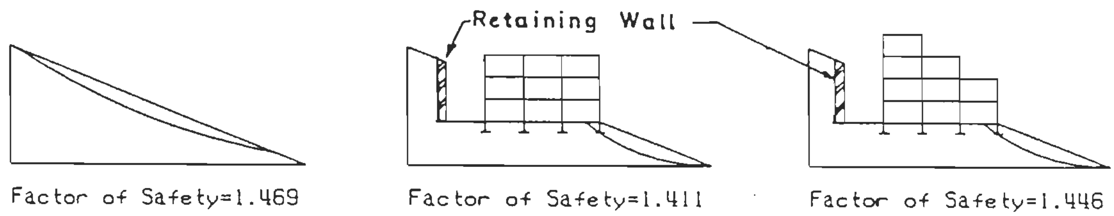


Fig. 7.8 - Slope Sliding Surface(Static Condition)

The loads transferred at the foundation level for both the buildings in seismic condition are shown in Table 7.8. The factor of safety obtained for seismic condition for free slope, slope with regular building, slope with setback building are 1.130, 1.132, 1.147.

The sliding failure surfaces are shown in Fig. 7.9 for seismic condition. The results show that the sliding of slope with building load can take place under the column adjacent to hill slope. The factor of safety against sliding of slope decreases under earthquake conditions. The factor of safety is higher in case of setback building as compared to regular building. It indicates that the heavier part of the building is to be located on the uphill side to provide better stability. The buildings constructed on flat ground adjacent to hill slope having setback type configuration is better from stability consideration than the regular framed building.

Table 7.8 - Column Loads Transferred at Foundation Level(Seismic Condition)

Regular framed building				Setback framed building			
Column	Axial load(N)	Shear force(N)	Moment (N-mm)	Column	Axial load(N)	Shear force(N)	Moment (N-mm)
1	0.227+06	0.701+04	0.851+06	1	0.287+06	0.742+04	0.468+08
2	0.428+06	0.111+05	0.990+07	2	0.484+06	0.109+05	0.960+07
3	0.428+06	0.862+04	0.863+07	3	0.377+06	0.819+04	0.823+07
4	0.227+06	0.192+05	0.138+08	4	0.170+06	0.191+05	0.136+08

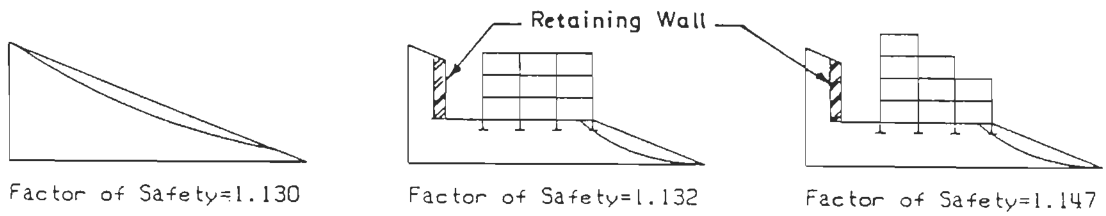


Fig. 7.9 - Slope Sliding Surface(Seismic Condition)

(ii) Buildings on Sloping Ground

The two buildings analysed earlier in this Chapter for seismic response are taken for study of stability of slope. The loads transferred to the slope from building in static and seismic conditions are taken. The factor of safety for stepback building, stepback and setback building under static loads are 1.518 and 1.486 respectively. Under earthquake condition the factor of safety reduced to 1.242 and 1.125 for the two buildings respectively. The sliding surface for the slope with building loads is shown in Figs. 7.10 and 7.11 for static and seismic conditions respectively. It is observed that the factor of safety in both static and seismic conditions is more in case of stepback building as compared to stepback and setback building. It is due to the fact that the heavier load transferred by stepback building at the down stream edge of the slope gives stabilising effect.

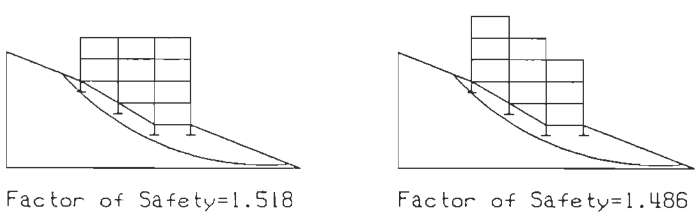


Fig. 7.10 Slope Sliding Surface (Static Loads)

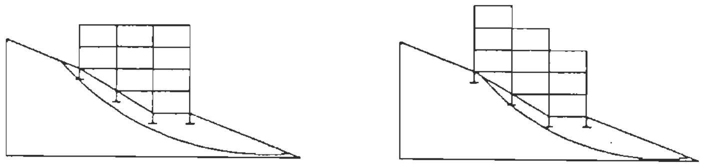


Fig. 7.11 Slope Sliding Surface (Earthquake Loads)

The above study indicates that the setback buildings on flat ground adjacent to hill slope gives better stability to slope than the regular building whereas the stepback buildings on sloping ground gives better stability than the stepback and setback buildings.

To see the effect of taking foundation deeper on upstream side of slope, the above two problems on sloping ground have been analysed for stability. The location of foundation of the columns on the upstream side of slope are lowered by 1.5 m. The factor of safeties obtained under the static condition for stepback, stepback and setback building is 2.027 and 2.007 and for seismic condition reduced to 1.630 and 1.604 respectively. The sliding surface are shown in Figs 7.12 and 7.13 for static and seismic conditions.

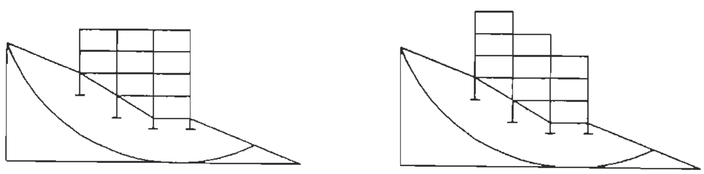


Fig. 7.12 Slope Sliding Surface (Static Loads)

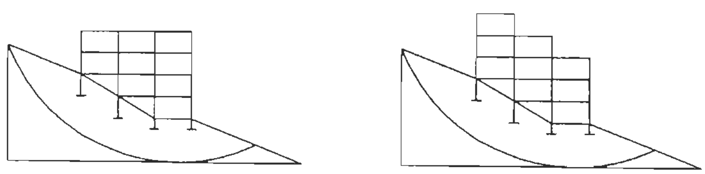


Fig. 7.13 Slope Sliding Surface (Earthquake Loads)

It indicates that by taking the foundation deeper on the sloping ground, higher factor of safety against sliding failure of slope can be achieved.

7.4 Soil Structure Interaction

The analysis carried out for stepback and setback buildings so far is based on the assumption that the foundation base of the building is fixed. This assumption may not be always true because the soil at different places may be loose or dense. Therefore the flexibility of the soil is to be taken into account for more realistic results. Keeping this in view, soil structure interaction study has been carried out for the stepback and setback buildings. Four different soil conditions have been considered with shear wave velocity ranging from 150, 300, 600 and 1200 m/sec. The densities for loose, medium and dense soil have been taken as 1280, 1600 and 1920 kg/m^3. The shear modulus of the soil has been evaluated using the relation $G = \rho v_s^2$ where G is the shear modulus of the soil and ρ is mass density of the soil and v_s is the shear wave velocity. The soil at the foundation base is modelled as elastic springs lumped at the foundation level. A linear stress-strain relation for the soil has been taken and the spring constants have been evaluated using the formulae given in [Richart *et al.*(1970)]. The spring constants evaluated for different shear wave velocities are given in Table 7.9.

Using the spring constants for the different soils with shear wave velocity of 150, 300, 600 and 1200 m/sec, the free vibration analysis of the stepback and setback building shown in Fig. 7.14(a) has been carried out and the free vibration time periods obtained for different soil conditions are shown in Table 7.10.

Table 7.9 - Spring Constants for different Shear Wave Velocities

Motion	Spring constant for shear wave velocity			
	150 m/sec	300 m/sec	600 m/sec	1200 m/sec
Vertical	2.20e+05	1.10e+06	5.30e+06	2.12e+07
Horizontal	1.67e+05	8.37e+05	4.02e+06	1.61e+07
Rocking	1.93e+11	9.68e+11	4.65e+12	1.86e+13
Torsion	2.32e+11	1.16e+12	5.58e+12	2.23e+13

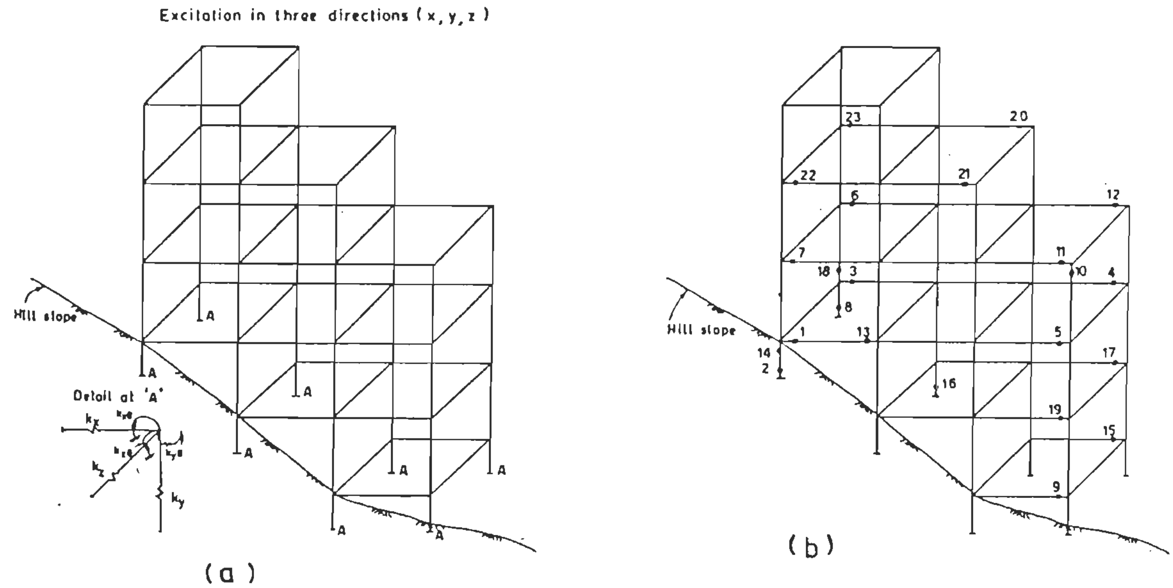


Fig. 7.14 - Stepback and Setback Building on Flexible Base

Table 7.10 - Free Vibration Time Periods Considering Soil Structure Interaction

Mode	Time Periods with Shear wave velocity of				
	150m/sec	300m/sec	600m/sec	1200m/sec	Fixed Base Condition
1	0.56794	0.54939	0.54556	0.54480	0.54454
2	0.39624	0.38721	0.38541	0.38505	0.38493
3	0.38927	0.38048	0.37885	0.37851	0.37839
4	0.21757	0.21531	0.21484	0.21475	0.21472
5	0.20907	0.20206	0.20061	0.20032	0.20022
6	0.16996	0.16851	0.16823	0.16817	0.16815

The results of free vibration time periods show that with the increase of the flexibility of the base of foundation of the building, the time period of vibration increases. The variation of time periods is up to 5% for a shear wave velocity of 150m/sec as compared to fixed base condition.

The inelastic seismic analysis of the stepback and setback building has been carried out considering the flexibility of the foundation base with shear wave velocity of 600m/sec. The sequence of formation of hinges in the frame are shown in Figs. 7.14 (b). A total of 25 plastic hinges formed in the building structure and the maximum ductility demand for the yielded members of the frame is found to be 7.55.

The results of seismic analysis shows that with the flexible base condition the time periods of the building increases. The more number of resisting elements yielded and the ductility requirement of the yielded members increases as compared to the fixed base condition. More detailed analysis is required to study the inelastic seismic behaviour of the stepback and setback buildings founded on flexible base foundations. With the finite element modelling of the soil along with structure modelling the variation of base input motion at different levels of building foundation should also be accounted. In the present study the base input motion has been taken as same at all the foundation levels of the building.

7.5 Concluding Remarks

The structural behaviour of different configurations of buildings under gravity loads does not show much difference. But seismic behaviour of different configurations of buildings under earthquake excitation is quite different. It is found that a combination of stepback and setback buildings are less affected by torsion as compared to stepback buildings. The damage and ductility requirements of r.c. members of stepback and setback type buildings are less as compared to stepback type buildings. Incidentally the outer profile of the stepback and setback type buildings follows the natural slope of the ground which is architecturally more acceptable in hill areas. It has also been observed that the short columns are worst affected. The problem of short columns can be avoided either by eliminating the use of short columns or by properly designing these members.

The study shows that it is important to carry out the stability analysis of slope with building loads. It has been found that taking foundations deeper into the slope on the upstream side increases the stability of the slope. Stability analysis of slope suggests that the buildings on flat ground adjacent to hill slope should have setback type configurations and buildings on sloping ground should have stepback type configurations to achieve better stability.

Soil structure interaction studies show that the buildings founded on loose and medium soil are liable to be damaged more and ductility demand in the yielded members under earthquake excitations is more as compared to the fixed base conditions.

A combination of stepback and setback configuration of building is seismically better and therefore recommended for construction on sloping ground provided the stability of the slope is ensured.

7.6 References

1. **Richart, F.E., J.R. Hall and R.D. Woods(1970)**, Vibrations of soils and foundations, Prentice Hall Inc., Englewood Cliffs, New Jersy.

SUMMARY AND CONCLUSIONS

8.1 Introduction

The main aim of the thesis is to study the seismic behaviour of r.c. frame stepback and setback buildings common in hill areas. The buildings in hilly areas are entirely different class of structures as compared to buildings in plain areas posing different problems at analysis, design and construction stages. These buildings are highly irregular and asymmetric. Stepback and setback buildings are subjected to severe torsional moment and lateral shears under earthquake excitations. During the course of the study, literature review, structural modelling, material modelling, inelastic algorithm, seismic response by simplified method and rigorous method, stability of slopes and overall response of various configurations of buildings in hill areas have been examined. An attempt has been made in this study to develop a simplified method of dynamic analysis for stepback and setback buildings on hill slopes to get the realistic seismic response of the buildings. The inelastic seismic behaviour of the buildings on hill slope having different configurations, the real problems associated with these buildings and further stability problems of hill slope with building loads has been studied. The various problems and the remedial measures for hill buildings has been presented. The significant conclusions based on the present study are summarised in the following sections for various aspects of these buildings.

8.2 Literature Review

A critical review of literature on the irregular asymmetric buildings, modelling of 3D r.c. beam column elements, r.c.c. panel elements, brick masonry elements, interface elements, inelastic modelling, stiffness degradation, ductility and stability of slope are summarised. Based on the review following points emerge.

(i) The 3D dynamic analysis is required for irregular asymmetric framed buildings subjected to earthquake excitations to capture the real behaviour using simplified models or rigorous models. Code of Practices (UBC, NBCC, NZS etc.) recommend 3D dynamic analysis for such irregular buildings to capture the real behaviour.

(ii) For inelastic dynamic analysis of 3D framed buildings, the beam-column element models based on lumped plasticity concept at the ends have been used extensively, as these are computationally efficient and sufficiently accurate for dynamic problems.

(iii) The r.c.c. panel have been modelled as either 4 noded or 8 noded shell elements and their inelastic behaviour have been studied considering cracking, crushing, interlocking etc., For brick masonry elements cracking and crushing have been considered.

(iv) Slope stability analysis has been extensively carried out based on limit plastic equilibrium using slip circle method for free slopes.

(v) There is little literature available on seismic response of buildings on hill slope.

8.3 Structural Modelling

The stepback and setback buildings are highly irregular and unsymmetrical due to their varied configurations. These buildings are characterised by the location of centre of mass of different floors lying on different vertical axes and so is its centre of stiffness. These buildings are supported on two types of columns (i) columns resting on the ground, (ii) columns resting on the floor below. The eccentricity is caused by (i) due to irregular configuration, (ii) due to the unequal column lengths at the same floor. These buildings undergo severe torsion in addition to the lateral shears under earthquake loads. In such irregular buildings, it is essential to take care of the stiffness and mass contributed by various components such as beams/columns, slabs, panels, infills and affect of interface.

To capture the realistic behaviour of such irregular buildings, 3D modelling of the building is required. The structural modelling of the members should be such that, it is practicable to handle the size of the problem and predict reasonably accurate results. In the present study, rigorous and simplified 3D structural modellings of the building are presented for dynamic analysis.

To carry out the preliminary analysis and design, a simplified 3D structural model has been developed assuming floor diaphragm as rigid under lateral loads having 3 d.o.f. per floor at the centre of gravity of the floor. The flexibility in the structure is only due to columns and infills. The columns are modelled as 2 noded frame elements and infills are modelled as strut elements. The stiffness and mass matrices are formulated at a common vertical reference axis by transferring the stiffness and mass of all the members through the transformation matrices. In this simplified method the mass of different floors lying on different vertical axes gets transferred to common vertical reference axis and so is the stiffness of various floors. This algorithm reduces the size of the problem tremendously for computation purposes. In this modelling accidental eccentricity can be taken into account by simply adding the accidental eccentricity to the distance between the centre of mass of floor to the common vertical axis. The proposed algorithm and the computer program developed has been validated by comparing the results of present study with the reported results in the literature and the results are found to be comparable. The results of the present study has also been compared with the results of 6 d.o.f/node analysis with floor diaphragm considered as rigid and are found to be very close.

In order to consider the flexibility of beams and slabs in addition to columns and infills 3D rigorous analysis has also been carried out. In this analysis, beam/column members are modelled as 3D frame elements, r.c.c slabs are modelled as plate elements, r.c.c panels and brick masonry infills are modelled as 4 noded inplane elements and interfaces are modelled as 4 noded line elements. The computer program has been validated by comparing the results of present study with the reported results in the literature.

8.4 Material Modelling

The r.c. beam/column element cross section has been analysed using nonlinear stress-strain relation for concrete and an elasto-plastic model for steel. The regression analysis is used to fit a third degree polynomial to the points obtained from the actual analysis of the r.c. cross sections. This modelling reflects the actual behaviour of the r.c. cross section and at the same time is computationally efficient. It requires analysis of different r.c. cross sections only once and storing of a few constants resulting from the analysis for use whenever required during different stages of analysis. The stepback and setback buildings on hill slope are subjected to severe torsion in addition to lateral shears. Therefore the yielding of the frame elements takes place under the combined action of bending moments, axial force, torsional moment and shear forces. The yield surface considering the interaction of bending moments, axial force, torsional moment and shear forces has been developed.

In case of r.c.c. panel elements, plane stress condition is assumed to study the response of frame panel system. Concrete is modelled as an isotropic material under biaxial stress condition and the reinforcement is considered as a smeared layer assuming full bond between the concrete and the reinforcement. Material modelling for different phenomena's such as cracking, yielding and crushing of concrete and yielding of steel are adequately modelled using available models from literature. Yield surface for concrete in compression includes the interaction of different stresses(σ_x, σ_y, σ_{xy}) in terms of first and the second deviatoric stress invariants. Maximum tensile stress at a gauss point governs the formation of crack. The shear transfer across a crack due to aggregate interlock and dowel action is incorporated by taking a reduced value of shear modulus. The effects of tension stiffening is included using a available model.

In case of brick masonry panel elements under compression, the material has been assumed to be linearly elastic untill failure and on crushing the stiffness has been assumed to reduce to almost zero and under tension on cracking the stiffness normal to crack has been assumed to be zero. However a partial shear transfer due to inter locking between the particles has been maintained. The stiffness and stresses along the crack has also been maintained.

For interface elements the tangential stress-strain relation has been assumed to be elastic perfectly plastic. In case if normal stress is tensile, a separation has been assumed, otherwise contact has been maintained. If the normal strain is compressive and tangential strain exceeds the coefficient 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 points.

8.5 Inelastic Analysis

The yield surface for beam-column element developed in the present study is used for inelastic analysis. It adopts the concept of a point plastic hinge so that the beam-column element is assumed to remain elastic with lumped plasticity at the ends. This algorithm predicts formation of plastic hinges, disappearence of plastic hinges and the solution is obtained up to failure point. In the present study, it has been found that stepback and setback buildings subjected to severe torsion and lateral shears under earthquake loads fails at lower load as compared to the failure load obtained using the available yield criteria where torsion and shears are not considered. There is a gradual deterioration of the stiffness of the structure due to plastic hinges formed in the frame members, cracking, yielding and crushing of r.c.c. panels, cracking, crushing of brick masonry infills and separation and slip of the interface elements. Ductility is an important parameter in earthquake resistant design of buildings. Ductility of a yielded member cannot be realistically determined unless appropriate inelastic degrading stiffness model is used. Therefore stiffness degradation model has been implemented in the present algorithm considering the maximum plastic deformations. The stiffness degradation effects are worked out independently for all the six components of forces using proposed yield criterion. The ductility requirement of yielded beam members is evaluated by monitoring the rotations and of column members by monitoring the lateral deflection at the ends at which the yielding takes place and then finding the maximum rotation/deflection occured in the member during the excitation period. The ductility requirement calculation gives an idea about the severity of damage in the structural members.

The proposed algorithm of modelling of various elements in the structure for elastic and inelastic analysis and the computer program developed has been validated by comparing the results obtained from the present study with the results reported in the literature(experimental and analytical) for 2D and 3D framed buildings under static and earthquake loadings. Good agreement of results has been observed. It is observed in the present study that the buildings which are subjected to severe torsion and lateral shears in addition to bending moments and axial loads, the yielding of r.c. members takes place at a lower load factor as compared to buildings which are not subjected to severe torsion and lateral shears.

8.6 Simplified Method Vs. Rigorous Method

The results of the simplified and rigorous method have been compared for a few real buildings having stepback and setback configurations on hill slope. The results of free vibration time periods, mode shapes, lateral floor displacements, maximum column shears, inter storey column shears, ground column shears, infill shears obtained by two methods of analysis are found to be close to each other. Rather in most of the cases the results of shears are on the conservative side obtained by simplified method of analysis and is acceptable for preliminary design purpose. The actual size of the problem gets reduced in simplified method tremendously as compared to the rigorous method of analysis.

Code of Practices(UBC, NBCC, NZS etc.) recommends 3D dynamic analysis for irregular buildings such as on hill slope. Although many sophisticated computer codes are available for seismic analysis of irregular buildings. A simplified method for seismic analysis of stepback and setback buildings on hill slope gives an insight into the real behaviour of buildings under seismic conditions. A simplified method can be used in design offices for dynamic analysis. For complicated and important buildings this method can be used for preliminary analysis and design purposes so as to get the least torsion effect by taking various trial configurations of the buildings. Finally the building may be checked using the rigorous method of analysis. It is suggested to adopt the simplified method for 3D dynamic analysis of irregular buildings in the Code of Practices.

8.7 Inelastic Analysis of hill Buildings

The inelastic analysis of a few real hill buildings has been carried out using rigorous method of analysis. It has been observed that in inelastic analysis the time periods of the structure gets elongated due to the formation of plastic hinges. The forces in the members in the inelastic analysis gets reduced as compared to that in the elastic analysis. It is due to the fact that the energy gets absorbed in the yielded members in the structure. It has been found that the plastic hinges form in the resisting members located on the outer periphery of the building and mostly are in columns. It shows that stepback buildings are more prone to torsion. The ductility requirement of the yielded members has been evaluated and it is found that ductility requirement is higher for members located at the outer periphery of the buildings. The too short and too long columns at the same floor level in these buildings are worst affected and are to be avoided at the planning stage of the building itself.

8.8 Soil Structure Interaction

Soil structure interaction study has been carried out for few cases of hill buildings. It is observed that for loose and medium soil with shear wave velocity up to 300m/sec, the free vibration time periods increases as compared to fixed base condition. For dense soil with shear wave velocity of 600m/sec and above, the results of free vibration time periods are almost the same as that of fixed base condition. It is also observed that ductility demand of the yielded members increases, where the buildings are located on the loose and medium soil bases.

8.9 Stability of Slope with Building Loads

A procedure for stability analysis of slope with building loads has been developed based on limit plastic equilibrium using simplified Bishop's method. The building loads in the form of vertical loads, horizontal loads and moments transferred at the foundation level to the hill slope has been considered in addition to self wt. of the sliding mass of the soil. The dry/wet condition of the soil can be considered in the analysis. Different layers of the soil can be taken into account considering different properties of the soil. Earthquake effects can also be taken into account.

A computer program has been developed modifying the existing program considering all the above provisions and the minimum factor of safety is evaluated by taking various trial slip circles automatically in the computer program. The developed computer program has been validated by solving some problems.

It is found from the study that the stability of slope depends upon the type of loads, location of loads, configuration of the building transferring the load, drainage condition of the area. The stability of slope decreases with earthquake loads. It has been found from the study that in hill areas setback type configuration of the buildings on flat ground adjacent to hill slope are better from stability considerations. Taking foundation deeper into the slope on upstream side of slope increases the stability of slope. The reduction in pressure due to building loads enhances the stability of slope and can be achieved by providing strip foundation across the slope for all the columns in one row. It has been observed that factor of safety against sliding failure of slope increases with increase in distance of location of footing from free edge of the slope. The distance between the two columns transferring the loads also affects the factor of safety. Proper drainage arrangement is required to be made around the building complexes so as to avoid soil erosion and land slides.

8.10 Seismic Behaviour of Stepback and Setback Buildings

Different configurations of r.c. framed buildings in hill areas have been described. It has been found from the study that under the action of gravity loads the structural behaviour and maximum forces for design purposes does not show much difference. But under the action of earthquake loads the behaviour of these buildings is quite different, which advocates the use of different configurations in different situations.

It has been found from the study that buildings constructed on flat grounds adjacent to hill slope should have setback type configurations to achieve better stability of slope. Stepback buildings on sloping ground gives better stability to slope as compared to stepback and setback buildings.

It has been found from the study that the building on sloping ground having stepback and setback configuration are seismically better and are less prone to torsion as compared to stepback type configuration. The outer profile of the stepback and setback building follows the natural slope of the ground and are architecturally more acceptable. Therefore a combination of stepback and setback configuration of buildings are recommended for construction in hill areas, provided the stability of the slope is ensured.

8.11 Suggestions for Further Research

Due to the increase in construction activity in hill areas the design of r.c.c. framed buildings from seismic point of view is to be carried out. The loss of lives and property during the occurence of many eathquakes in the past in hill areas has increased the awareness among the masses for the structural safety of buildings. More detailed studies are needed to understand the practical feasibility of designing buildings in hill areas, so that the safety of the buildings in hill areas can be achieved at affordable prices. Some suggestions for further research in this area are made as under.

(i) Case studies of reinforced concrete framed buildings on hill slope during the past earthquakes should be conducted to throw light on the causes of damages/failures.

(iv) Experimental studies are needed to study the seismic behaviour of buildings on sloping ground so as to validate the theoretical results.

(ii) Detailed soil structure interaction studies are needed for buildings on sloping ground in hill areas.

(iii) Stability of rock slopes needs to be investigated under the action of building loads.