13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2799

SEISMIC EVALUATION OF R/C MOMENT RESISTING FRAME STRUCTURES CONSIDERING JOINT FLEXIBILITY

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SUMMARY

Seismic strength evaluation of existing building is assuming increasing importance in the field of earthquake engineering. Recent earthquakes all over the world, have demonstrated the disastrous consequences and vulnerability of inadequate structure. In Reinforced Concrete (R/C) moment resisting frames, the joint integrity is a requisite for adjoining flexural components to mobilise their strength and deformation capacity. In seismic hazard assessment, identification of vulnerable joints is very important, because joint failures would result in the collapse of the structure.

In literature, various assessment techniques have been published which advocate the use of nonlinear static and dynamic procedures. Computational tools, which comprise of several mathematical models to reproduce the cyclic behavior of components, are used to perform seismic evaluation task, where the joints could usually be modeled as rigid link elements. This assumption shows their inability to predict the possible shear failure within the joints. Use of such packages, in the seismic evaluation of frame structures with inadequate detailing in the joints, could be misleading.

A computational tool (IDARCFJ – Inelastic damage analysis for reinforced concrete frame with flexible joints) has been developed incorporating the necessary analytical models to perform the task of seismic hazard assessment of frame structures. The component model constitutes flexural element with joint elements at the ends. The shear characteristics of joint such as shear strength capacity and shear deformation has been estimated by establishing shear stress-shear strain relationship based on softened truss model theory.

The load-deformation history and joint shear stress levels predicted analytically have been compared with the reported test results. The importance of joint modeling during seismic evaluation of R/C moment resisting frame has also been briefly illustrated in a case study, where a 4 storey building designed as per Indian standards IS456-1978 [1] has been tested for the impact of non-ductile detailing aspects in seismic performance and evaluation.

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Key words: Buildings, Beam-column-joints, Concrete, Nonlinear dynamic analysis, Seismic evaluation, Structural response, damage assessment

INTRODUCTION

Earthquakes all over the world, have been demonstrating very frequently the disastrous consequences and vulnerability of inadequate structures. The lessons from the aftermath of earthquakes and the research efforts have resulted in upgrading of seismic code provisions. Hence, many existing reinforced concrete buildings may not conform to the current code requirements in terms of lateral strength and ductility. Although they possess inherent lateral strength, however, the deficient detailing practices adopted lead to poor structural performance. They represent seismic risk to the occupants and this fact explains the need for identification of such buildings, their expected seismic performance, and if needed, their seismic strengthening.

Preservation of gravity load carrying capacity and lateral load strength in reinforced concrete frame structures under earthquake action is linked to the integrity of the beam-column joints. Experimental literature confirms that joint performance appears to be particularly sensitive to the magnitude of joint shear stress and drift history.

Many of the computational tools used for seismic evaluation, perform sophisticated non-linear dynamic analysis with implicit assumption of joint panel zone as rigid, neglecting the possible identification of shear failure in the joints. But, the integrity of the interlinking element ‘joint’ is very crucial for satisfactory performance of the reinforced concrete structures. Thus, in the process of seismic evaluation of reinforced concrete frames, predicting the behavior of beam column joint becomes very essential. This paper presents a new analytical model for joint panel zone to establish the shear strength and shear deformation history envelope, enabling the model to be incorporated in non-linear dynamic analysis program. The validity in the application of the model has been illustrated by comparing the predicted results with published results on experimental specimens.

PAST RESEARCH WORK

A detailed review has been carried out on the past research work on the behavior of joints both on experimental and analytical sides. The experimental investigations conducted by Agbabian et al [2] to study shear behavior of joint panel zone are limited as the design of sub-assemblages demands specific efforts to ensure a pure shear failure rather than coupled flexural shear failure mode in the joint panel. Bonacci and Pantazopoulou [3] attempted to identify the parametric dependence of joint behavior from studies conducted on interior connections but observed no consistent trend due to the diversity in the experimental techniques used and the large number of influencing parameters.

Paulay [4] reported theoretical explanations to joint shear transfer mechanisms and to determine the maximum joint shear stress by considering force equilibrium conditions only. Pantazopoulou and Bonacci [5] developed analytical formulations, by satisfying, both equilibrium and compatibility conditions to compute average joint shear stress and joint panel deformation.

Experiments on beam column joints have shown inferior behavior when the bond deterioration of beam bars passing through the joint occurs. Kitayama et al [6] have studied the influence of bond condition on the joint behavior. As a design recommendation, they proposed an index of bond stress to approximately indicate the severity of bond within beam column joint by restricting the ratio of beam bar size to column depth. Also a limit to input shear into joint has been suggested for better performance of the joint. Leon

[7] investigated the performance of interior joints with respect to joint shear stress and beam bar anchorage lengths.

Shiohara [8] studied the interaction of joint shear with bond strength of beam bars, two different modes of deformations are identified in reinforced beam column joints, i.e., (i) deformation due to the shear force in the joint core and (ii) deformation due to the opening of the crack at the end of the beam resulting from bond-slip effect. Appropriate models were proposed to calculate the lateral resistance of joint under each mode. From a collective review of experimental evidence, particularly focusing on old-type joints, Lehman [9] concluded that joint performance appears to be particularly sensitive to the magnitude of joint shear stress and drift history.

RESEARCH SIGNIFICANCE

In the light of preceding discussion it is evident that an analytical model is required for systematic and quantitative assessment of the basic response aspects of poorly detailed joints, namely strength, stiffness, and deformation capacity. Various parameters have been evaluated in experimental literature with regards to the sequence of failure in beam-column connection, where joint damage is associated with shear distortion and slip of the primary reinforcement. In order to properly assess the likely hierarchy of failure and distribution of anticipated damage in inadequate connection, it is necessary to represent in the analytical model of the connection the joint flexibility resulting from joint shear function.

In this paper, an analytical shear model for joint has been proposed, which essentially accommodates the effect of all these variables in establishing the shear stress-shear deformation characteristics of panel zone considering the uniform distribution of average bond stress within the joint [3]. The averaging of concrete stresses over the panel zone will be valid for the joint that is going to be considered in the scope of the present study, where the joints are not completely lack of transverse reinforcement.

Apart from all these one more significant parameter is the bond stress condition of longitudinal bars within the joint, which affects the response of beam column joint severely and the interaction with joint shear behavior is reported to be highly complex. Bond deterioration with bar slippage results in the degradation of strength and stiffness of joint. In addition to this pinching or crack closing effect deteriorates the beam column joint behavior and is generally reflected in the hyseteretic response curves.

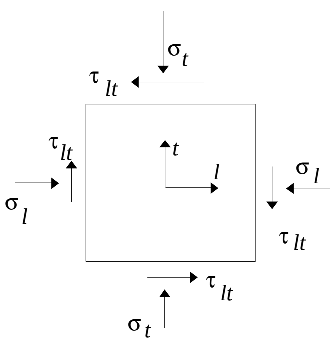
ANALYTICAL SHEAR MODEL FOR JOINTS

The performance of beam column joint is influenced by many parameters such as column axial load, the amount and detailing pattern of main reinforcement, volumetric ratio of lateral ties and its confinement effect in the joint core, and the characteristics of steel and concrete. The joint model should be capable of reflecting the effect of all such parameters in depicting the behavior under cyclic loads. In this paper, a model is proposed for the joints idealising it as 2D plane element, subjected to inplane forces. To predict the behavior of such elements subjected to inplane and normal stresses, a softened truss model is used. The model considers equilibrium of stress resultants, satisfies Mohr's compatibility conditions for deformations within the joint. The algorithm to establish the shear stress-shear strain relationship of the joint takes into account the constitutive law for softened concrete.

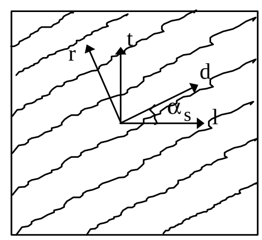
Joint Behavior and Idealisation

Behavior of joints is commonly characterised by an average shear stress (horizontal/vertical) introduced to the joint by adjacent beams and columns. As the forces at the joint boundary increase, the relevant response such as yielding of transverse reinforcement, crushing of concrete along the diagonal or yielding of column reinforcement can happen. Only by establishing the shear stress-shear strain relationship for the joint, it is possible to monitor the deformation of joints throughout the progress of response to establish the sequence in which the performance would occur.

Beam column joint has been idealised as two dimensional (2D) element subjected to only in-plane forces such as normal and shear stresses and is shown in Fig.1.a. Lateral loads are considered in one principal direction (in the direction of longitudinal beam) and vertical loads are along the column. Since contradictory opinions do prevail regarding the role of transverse beams on joint in literature[10,11], the influence of transverse beam in confinement of core is neglected in modeling and only in-plane effects for the 2D joint panel is considered. In the present study, to establish the shear stress-shear strain curve, rotating angle softened truss model theory is used. Joint reinforcements in orthogonal directions are column reinforcement in vertical direction and beam and stirrup reinforcement in horizontal direction. On the application of the normal stresses (σ_l, σ_t) and shear stresses (τ_{lt}) diagonal cracks are formed as shown in Fig. 1.b. A truss action is formed between the concrete struts subjected to compression and the steel bars act as tension links. The compression struts are oriented in the *d*-axis, which is inclined at an angle α_s to the longitudinal steel bars. Taking the direction perpendicular to the *d*-axis as *r*-axis, we have *d-r* co-ordinate system in the direction of the principal stresses and strains. The normal principal stresses are designated as σ_d in compression and σ_r in tension.



(a) *Stress Conditions within the joint*



(b) *diagonal cracks and orientation*

Fig. 1 **Inplane stresses and cracking assumptions**

It is assumed that the steel bars can resist only axial 'smeared steel stresses' in *l* and *t* directions respectively. Hence, the state of stress in the reinforced concrete joint panel can be considered as the superposition of steel stresses and concrete stresses (σ_l and σ_t). More information on the basic theory could be obtained from Hsu [12] .

Development of the Model

The joint model proposed establishes shear stress – shear strain envelope, which forms the backbone of the primary envelope in non-linear dynamic analysis. It is ensured that both equilibrium and kinematic conditions are satisfied. The model adopts non-linear constitutive laws of concrete considering the softening effect. Also, the confinement effect of joint core due to stirrup reinforcement has been considered in the formulation.

The constitutive equations are based on non-linear stress-strain relationship of concrete and steel. Confinement in the joint core is essential to hold the joint core concrete and is generally provided by ties and closed stirrups. The stress-strain relationships proposed by Sheikh and Uzumeri [13] for confined concrete of tied columns are appropriately adopted for joints with stress strain curves for confined and unconfined concrete. However, if the stirrup yielding takes place before peak stress is attained, the confinement is reduced and it is assumed conservatively that the concrete struts will subsequently follow the stress-strain relationship of unconfined concrete.

Diagonal cracking of concrete results in the softening of concrete. Hence, the softening coefficient ζ has been defined as a function of tensile strain ε_r, in *r* direction and ε_d in *d* direction. In this joint model, the one proposed by Vecchio and Collins [14] is used for proportional softening of both stress and strain. A minor modification is incorporated in the expression by Pantazopoulou and Bonacci [5] to take into account confinement effect on softening, in terms of hoop volumetric ratio within the joints.

The stress-strain relationship for the longitudinal and transverse steel bars is assumed to be elastic-perfectly plastic. The stress-strain curve forms the backbone of primary envelope for the purpose of dynamic analysis. The shear capacity of joint is estimated from the ultimate shear stress and the effective joint area.

Shear Capacity

The joint shear performance deteriorates significantly after the initiation of yielding of hoops. The effectiveness of closed stirrups in providing confinement to the joint core by restraining the volumetric expansion will be reduced when the stirrups yield. Once hoop/stirrup yields, failure may occur by, either yielding of longitudinal column reinforcement or crushing of concrete in the direction of principal stress by attaining enhanced peak strength [5].

For the present validation study, the experimental results obtained by Agbabian, *et al.* [2] and Otani, *et al.* [15] were used. Validation study includes two main objectives: (i) establishment of shear stress-shear strain curve and (ii) estimation of shear capacity of joints.

Test Specimens

From experimental study, test specimens that were reported to have failed under joint shear failure by hoop yielding, were chosen. The properties of test specimens and detailing of the joint panel zones are presented. The effective dimensions of joints were reckoned from column and beam dimensions as per ACI-ASCE 352 recommendations. Agbabian *et al*. [2] tested three one-third scale models to study the effect of axial load on joint shear capacity. The specimens (SA1, SA2, SA3) were identical in all aspects except the axial load applied on column. The joint panel zone was of dimension 127mmx178mm. The concrete mix had an average strength of 27.56MPa at 28 days. Reinforcing steel bars of Grade 60 was used in all specimens.

Table 1. Specimen Details (Agbabian *et al*., [2])

Design Strength		Beam Reinf.		Joint Reinf.	Col. Reinf.
f'_c, Mpa (ksi)	f_y, Mpa (ksi)	Top	Bottom		
27.56 (4.0)	413.4 (60.0)	2 # 3	2 # 3	2 # 2	4 # 2

Agbabian *et al.* [2] reported shear capacities of three subassemblages (designated as SA1, SA2, SA3), designed to exhibit a failure mode entirely controlled by the panel zone. The axial load applied on the column was varied from 0 to 10% of the squash load. They arrived at the capacities by proposing an analytical method, which is based on a simple mechanical model. The joint model proposed in the present study, identified the ultimate failure mode of these specimens as yielding of steel in both direction and the corresponding ultimate shear capacities were compared with reported results in Table 2. A difference of about 2-7 percent between the experimentally acquired strength and the analytical strength is observed which shows that the model used is capable of predicting reasonably representative of strength estimates.

Table. 2 Evaluation of Shear Capacity (kN) (Agbabian *et al*., 1994)

Axial Load	Agbabian Results		Present Study
	Analytical	Experimental	
SA2 (0%)	92.25	98.17	100.20
SA1 (5%)	95.77	107.45	106.50
SA3 (10%)	106.40	121.11	118.30

Test Specimen C1 (Otani et al., 1985)

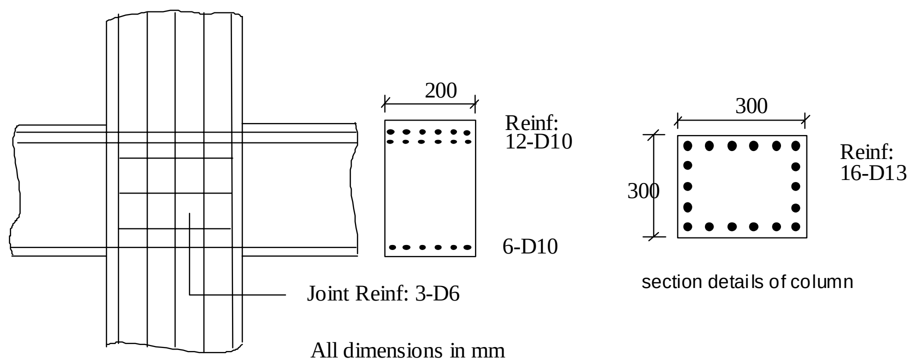


Fig. 2. Details of Test Specimen C1 (Otani et al., 1985)

The specimen C1 from Otani, *et al*. [15] was considered for validating the shear deformation of joints with reported values of Bonacci and Pantazopoulou (1993). The specimen details are given in Fig. 2. The concrete compressive strength was 25.6 MPa (3,713 psi). The yield strength of beam reinforcement was 317 MPa (46 ksi) and that of hoop reinforcement was 331 MPa (48 ksi). The axial load on the column was 181.5 kN (40 kips). Table 3 compares the shear stress and shear strain values obtained from proposed model with the experimental and theoretical values reported by Otani et al [15] and Bonacci et al. [3] respectively.

Table. 3 Joint Response at and beyond Yield (Specimen C1)

Reference	at yield $\varepsilon_l = \varepsilon_y$		beyond yield $\varepsilon_l = 2\varepsilon_y$	
	Shear Stress (MPa)	Shear Strain	Shear Stress (MPa)	Shear Strain
Proposed Model	5.456	0.00331	5.994	0.00526
Bonacci et al., 1993	5.016	0.00322	5.788	0.00531
Experiment, Otani *et al*. (1985).	5.532	--	--	--

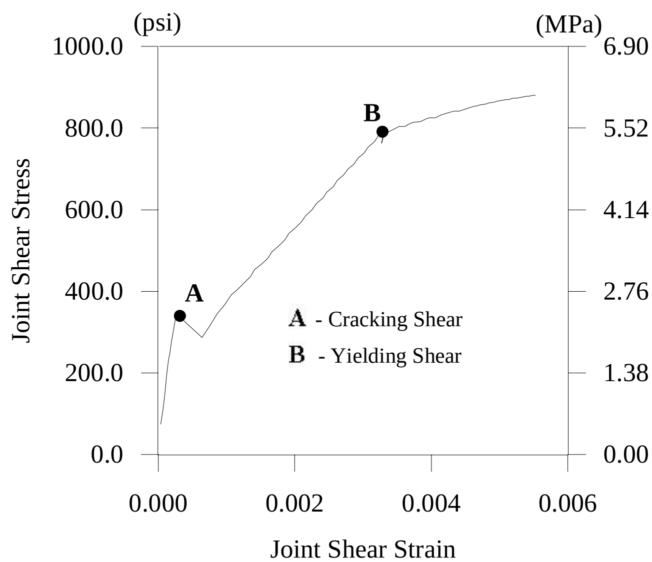


Fig. 3 Establishment of Joint Shear Stress-Strain Curve for Specimen C1,(Otani et al [15])

Given the applied constant stresses for a particular joint panel, the shear stress - shear strain curve is established. In addition, in this curve it was necessary to identify the critical milestones of joint response observed experimentally such as cracking shear, yielding of hoops and shear capacity. Experimental studies have indicated that first significant diagonal cracking in the joint panel occurred at the instant when the measured strain in joint hoops began to increase substantially. The corresponding shear stress was referred as cracking shear. In Fig. 3, the point 'A' marked on the curve indicates the diagonal cracking point and the corresponding cracking shear stress, After the diagonal cracking of concrete, the strain in hoops, ε_l increased as the compression strain in strut, ε_d was increased. The corresponding increase in the shear stress, τ_{lt} was observed due to the effective confinement of the core up to yielding. The shear stress corresponding to the initiation of hoop yielding was referred as yielding shear τ_y. This point was identified and marked as B in Fig. 3 which shows the shear stress-shear strain curve established for the joint of specimen C1. The shear strain is the measure of joint deformation and its performance.

IMPLEMENTATION OF JOINT MODEL

The present investigation is targeted on a class of structures as multi-storey lightly reinforced concrete moment resisting frames, which are generally non-ductile, the mathematical models of elements should essentially reflect the corresponding behavior. The non-ductile detailing aspects such as lack of rotational capacity at plastic hinges, lack of transverse reinforcement in joint and slip are constructed in addition to the regular inelastic frame analysis. New analytical models have been formulated to capture the effects of local joint shear failure and pull out failure of bars. The damage model includes effect of lack of confinement in plastic hinging regions affecting the deformation ductility.

Computational Tool - IDARCFJ

IDARCFJ is based on the macro modeling scheme formulation in line with IDARC 2.0 and the details of various modules viz., component model, strength-deformation model, hysteretic model and damage model are given here. The major features of the tool in detail could be found elsewhere, Uma [16].

Component Model

The need for considering the inelastic behavior of flexural components along with that of joint element has been already emphasised. Hence, a suitable basic component model is proposed which is shown in Fig. 4. This consists of a flexure element to represent the beam/column with joints at ends acted upon by a vertical shear force V and a moment M. The flexural element is modeled as an equivalent shear-flexure spring in which the shear deformation effects are implicitly included in the flexural rigidity term as reported by Kunnath et al.[17] . Interaction of axial deformation with bending moment in columns is ignored. The joint is idealized as a shear beam element and is assumed to be flexurally rigid (i.e. EI is infinite). The joints are acting in series with flexural element.

The flexibility matrices for joints at both ends (AB and CD), which are idealized as shear beam elements, are derived for element forces V and M. The shear rigidity G, is to be reckoned from the joint shear stress-shear strain ratio. Since the joint is flexurally rigid, the flexibility coefficients with (1/EI) terms vanish. The flexibility matrix for the component model of size (2x2) is obtained after combining the element flexibility matrices with appropriate transformation matrix.

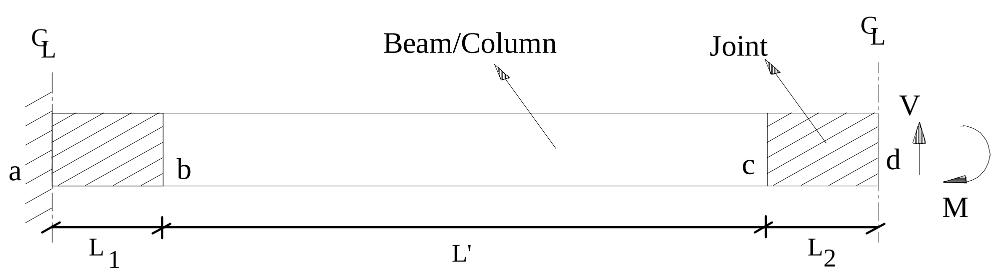


Fig. 4 **Component Model of a Typical Flexural Element in a Frame**

The incremental force-deformation relationship for the component element can be written in terms of flexibility as:

$$\begin{pmatrix} \Delta\, v_d \\ \Delta\, \theta_d \end{pmatrix} = [\ f_s] \begin{pmatrix} \Delta\, V_d \\ \Delta\, M_d \end{pmatrix} \tag{1}$$

where

Δv_d , $\Delta\theta_d$ = incremental deformations

ΔV_d , ΔM_d = force increments

The force-deformation relationship with flexibility matrix $\left[f_s \right]$ given in Eq. 1 can also be expressed in the form with inverted flexibility matrix $\left[k_s \right]$ and expressions for beam and column can be derived using $\left[k_s \right]$ and necessary transformation matrices for global stiffness assembly.

Strength-Deformation Model

The force-deformation relation of the component model is described by trilinear curve, indicating three branches with two turning points identified by cracking point, yielding point and corresponding curvatures respectively. Strength and deformation refer to moment - curvature for flexural elements and shear stress - shear strain for joint elements. As the component model comprises of flexural (beam/column) and shear (joint) elements, it is necessary to model the non-linear behavior of these elements exclusively.

Hysteretic Model

A multi-linear hysteretic model, the Three Parameter model (IDARC21), is used to idealize the irreversible physical behavior of the components, with three parameters that control stiffness degradation (HC), strength deterioration (HB) and pinching (HS) behavior.

Damage Model

A damage model, adopting the damage indexing procedure for the components, is used to provide a physical qualitative interpretation for the response obtained from the analysis module. Knowing the seismic demand and capacity for each structural member, the damage index is computed. This measure of damage enables to ascertain the system vulnerability in terms of serviceability, reparability and/or collapse. A modified damage index model proposed by Park *et al*.[18] is used in the program.

Effect of joint failure on damage index

Once the joint fails, the elements framing into the joints loose their capacity to reach their flexural strength. In such cases, the curvature of these flexural elements cannot increase further and hence their component damage index is set to 1.0 irrespective of energy dissipation, indicating extensive element level damage and the need for retrofitting.

Modeling of Bar Slippage within a Joint

During the cyclic loading of beam-column sub-assemblages, the beam and column main reinforcement is pulled on one-side of the joint and is pushed simultaneously from the opposite side. An important parameter related to the slip of continuous bars through a beam-to-column joint is the ratio of appropriate joint dimension to reinforcing bar diameter. The bond failure is identified using Bond Index, proposed by Otani, *et a* [15]. This is the average bond stress that must develop over the column depth when beam bars yield in tension and compression at both column faces, normalized by $\sqrt{f'_c}$ in appropriate units. The effect of slip has been incorporated in terms of increased pinching in the hysteretic response of the components.

DYNAMIC ANALYSIS

An incremental dynamic response analysis under horizontal and vertical earthquake excitations can be performed in IDARCFJ. The procedure illustrated in the form of flow chart in Fig. 5, involves the following dynamic equation of equilibrium:

$$\left[M \right] \left\{ \Delta \ddot{u} \right\} + \left[C \right] \left\{ \Delta \dot{u} \right\} + \left\{ R\left(u_t \right) \right\} = \left\{ F(t) \right\} \qquad (2)$$

in which

$[M]$ = lumped mass matrix

$[C]$ = the viscous damping matrix

$\{R(u_t)\}$ = the restoring force vector at the start of time step

u = the relative displacement

$\{F(t)\}$ = the effective load vector

The element stiffness matrix get reassembled and updated whenever there is a change in the stiffness either in the flexural element or in joint or in both.

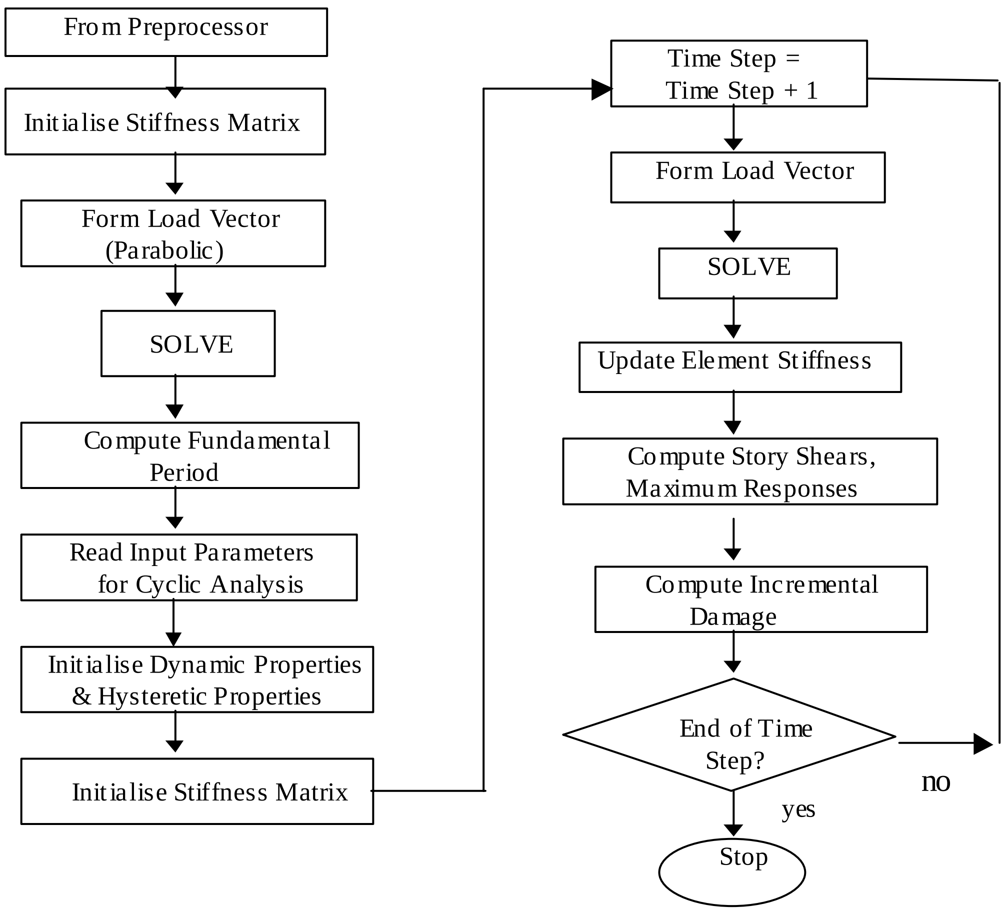


Fig. 5 Flow Chart for Dynamic Analysis

EXAMPLE STUDY

A typical a four storey GLD frame building has been considered in the present study. The building considered is regular in its configuration to enable meaningful interpretation of results. The presence of vulnerable joints and their effect on the total response of the structure is illustrated in this paper.

Design and Detailing

The buildings are designed for gravity loads (1.5(DL + LL)) and no lateral loads are considered. Proportioning of structural elements are performed as per IS 456: 1978 [1] and detailed as per SP 34 (S&T): 1987 [19], which are meant for gravity load design. The grade of concrete considered is M20 and that of reinforcement steel is Fe415. Dead loads are computed considering the unit weight of concrete as 25 kN/m^3. Live loads on the floors are taken as 2.5 kN/m^2 and on the roof as 1.5 kN/m^2 assuming office occupancy from IS 875 (Part 2): 1987 [20].

Inadequacies of GLD Buildings under Seismic Loads

Pertinent details are given regarding the possible non-ductile detailing aspects which could lead to inadequate performance of the building and discussed in the following sections.

Anchorage requirement

The anchorage requirements are fulfilled by limiting the bond index 1.66 (in units $\sqrt{MPa}$). The maximum bond index, given the sectional and reinforcement details in the beams against the limiting value in a typical floor amounts to be 3.70. Hence, slippage of bars is quite likely.

Confinement Requirement in Plastic Hinge Zones

The column and beam plastic hinge zones are not provided with sufficient amount of transverse reinforcements as required for ductile detailing. Table 4 compares the transverse reinforcement provided for some of the columns against that required for ductile detailing as per IS: 13920-1993 [21].

Table 4. Comparison of Transverse Reinforcements in Plastic Hinge Zones

Frame	Member	Ties Provided (IS 456)	Ties Required (IS 13920)
Ext. Frame	Ext. Columns	8 @ 300 mm c/c	8 @ 100 mm c/c
Int. Frame	Ext. columns	8 @ 250 mm c/c	8 @ 100 mm c/c

Joint Shear Reinforcement

As per SP 34 (S&T): 1987, clause 7.6 the column ties are extended through the joints if a) beams do not frame into the column on all four sides b) beams do not frame into the column by approximately the full width of the column. Hence, the column ties are extended for all the joints for the buildings under consideration. However, considering the spacing of ties in the columns, the transverse reinforcement provided may not be adequate to resist the shear developed in the joint, which makes them vulnerable under seismic loads.

Lap Splices

Lap splices of column reinforcements are generally located near the floor levels just above the joints. Seismic design codes call for closer spacing of ties in splicing regions to provide better confinement which will avoid splice failure. Studies by Panahshahi *et al*., [21] have shown that the required splice length is comparatively shorter (35 d for M20 concrete and Fe 415 steel) under inelastic cyclic loading than that required by detailing practice code for GLD structures (47 d), but with closer stirrup/tie spacing (150 mm). Hence, by providing splice lengths as per code requirements, slightly liberal spacing of ties can be resorted to, without encountering splice failure and hence this particular failure mode is not considered in the present study.

It is verified that the shear failures in beams and columns for the chosen buildings under the selected earthquakes are not likely.

SEISMIC EVALUATION

The lateral strength of the building and demands of earthquake motions on the structural response are evaluated using IDARCFJ. The performance of the building under non-linear dynamic analysis is performed to study the behavior under typical Elcentro earthquake record. To study the significance of joint modeling, analyses are carried out with joints assumed as rigid zones (i.e. without joint model) and with joint model . The responses are compared for both cases.

Assumptions in Analytical Modeling

The building was idealized as a series of planar frames having a common lateral degree of freedom at each storey level. Engineering approximations were made to arrive at the initial stiffness and hysteretic parameters. Accordingly the initial stiffness for beams and columns are taken as $0.6EI_g$ and $0.35EI_g$ and the values adopted for hysteretic parameters are given in Table 5 below.

Table 5 Member Properties for Analytical Modeling

Initial Stiffness		Hysteretic Properties				
Column	Beam	HC	HB	HS	Crack Closing Point	Post Yield Stiffness Ratio
0.6	0.35	1.5	0.15	0.50	1.0	1.0

Dynamic analysis

This section presents the computed dynamic response of the buildings under Elcentro earthquake record. Raleigh damping was used to specify 5% critical equivalent viscous damping.

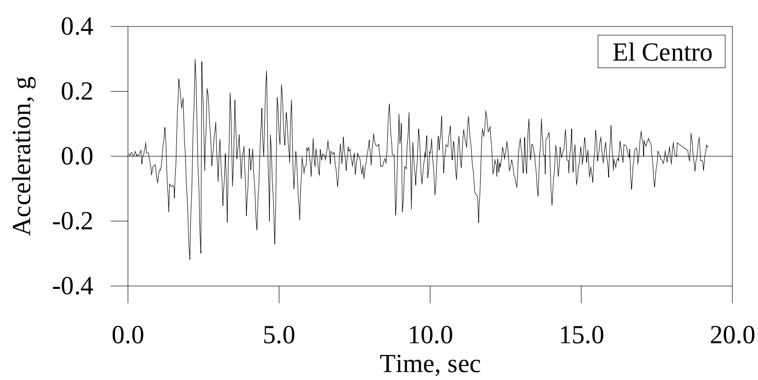


Fig. 6 Earthquake accelerogram record, Imperial Valley, 1940

The building was subjected to Elcentro earthquake record as shown in Fig. 6 and the performance of the building was studied. The failure mechanism for four storey building is soft-storey mechanism initiated by the failure of the interior column joint failure as shown in Fig.7.

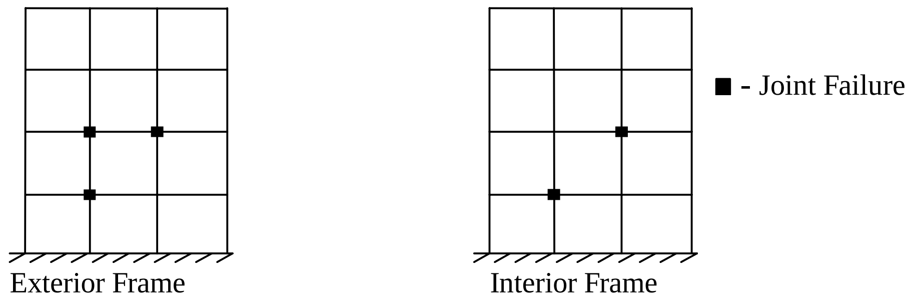


Fig. 7. Joint Failure Pattern in the Frames

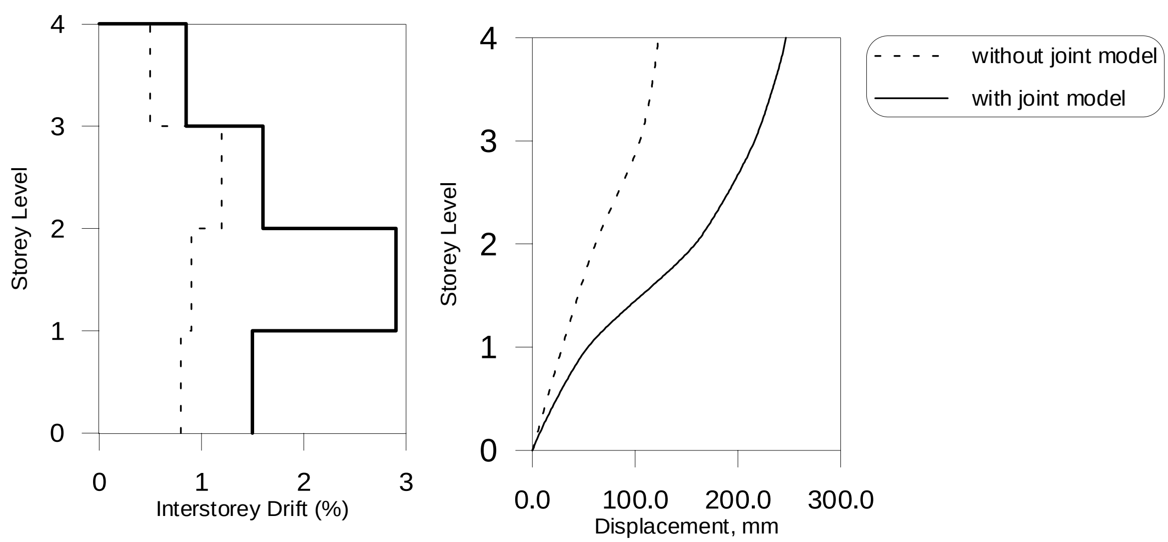


Fig. 8 Response envelope of the building

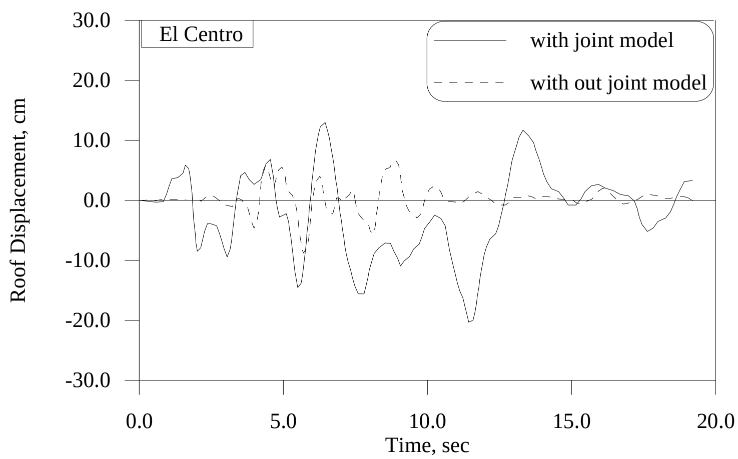


Fig. 9. Roof Displacement Time History of Roof Displacement

The results presented correspond to those obtained from the dynamic analyses of four storey building under the earthquake records chosen. The responses of the structure with and without joint model are computed and compared. In order to illustrate the effect of joint flexibility on the responses, typical results for El Centro earthquake is given in Fig 8 with respect to maximum responses of inter storey drift. The time history response of roof displacement is given in Fig 9. The inelastic drift computed with rigid joints assumption are contributed predominantly by column hinging. The joint failures and the resulting large lateral drifts observed for the record suggested that the structure would have probably collapsed under the event.

CONCLUSION

This paper discusses on the inadequate performance of the poorly designed and detailed structure under seismic conditions. Specific attention is given in the modeling aspects of beam column joints to capture the shear effects within the panel zone and all other salient features considered to lead to non-ductility. The above features are implemented in IDARCFJ, computational tool to perform inelastic dynamic analysis. The interaction of bond deterioration resulting in the slippage of bar with joint shear behavior essentially reflects pinching in the hysteretic curves. The validity of the joint shear model and formulations proposed are illustrated with the good comparison of analytical results with reported experimental behavior.

The dynamic analysis of a typical GLD building is carried out for a typical earthquake record. Joint failures are observed in the structure these joint failures do not show any discernible trend, except that they are confined to interior columns of lower stories. The critical responses like inter storey drift and overall displacements are significantly influenced by joint failures. The possibility of joint failures in GLD structures designed and detailed as per the code, is brought out and its effect on the overall responses is depicted.

REFERENCES

1. IS 456: 1978 Code of practice for plain and reinforced concrete, Bureau of Indian Standards, New Delhi.
2. Agbabian, M.S., E.M. Higazy, A.M. Abdel-Ghaffar and A.S. Elnashai. “Experimental observations on the seismic shear performance of RC beam-to-column connections subjected to varying axial column force.” Earthquake Engineering and Structural Dynamics 1994; 23: 859-876.
3. Bonacci, J. and S. Pantazopoulou. “Parametric investigation of joint mechanics.” ACI Structural Journal 1993; 90: 61-71.
4. Paulay, T. “Equilibrium criteria for reinforced concrete beam column joints.” ACI Structural Journal 1989; 86: 635-643.
5. Pantazopoulou, S. and J. Bonacci. “Considerations of questions about beam column joints.” ACI structural Journal 1992; 89: 27-36.
6. Kitayama, K., Otani., S., Aoyama., H. “Design of beam-column joints for seismic resistance.”, ACI SP –123, American Concrete Institute, Michigan, 1991; 97-123.

7. Leon R.T. “Shear strength and hysteretic behavior of interior beam column joints.” ACI Structural Journal 1990; 87: 3-11.

8. Shiohara, H., “New model for shear failure of RC interior beam-column connections.” Journal of Structural Engineering Division, ASCE 2001; 127: 152-160.

9. Lehman D. “State-of-the-Art review on exterior and interior beam column joints” 2002 PEER report.

10. Ehsani M.R., Wight J.K., “Exterior reinforced beam to column connections subjected to earthquake type loading.” ACI Journal 1985; 82: 492-499.

11. Zerbe H.E., Durrani A.J. “Seismic response of connections in two bay reinforced concrete frame subassemblies with a floor slab.” ACI Structural Journal 1990; 87: 406-415.

12. Hsu T.T.C. “Unified Theory of Reinforced Concrete”, CRC Press, Inc., Florida, 1993

13. Sheikh, S.A. and S.M. Uzumeri “Analytical model for concrete confinement in tied columns.” Journal of Structural Engineering Division, ASCE 1982; 108: 2703-2722.

14. Vecchio, F.J. and M.P. Collins. “The modified compression-field theory for reinforced concrete elements subjected to shear.” ACI Journal 1986; 83: 219-231

15. Otani, S., K. Kitayama and H. Aoyama. “Beam bar bond stress and behavior of reinforced concrete interior beam-column connections.” Proceedings, 2nd U.S.-N.Z.-Japan Seminar on Design of Reinforced Concrete Beam-Column Joints 1985, Department of Architecture, University of Tokyo, 1-40.

16. Uma, S.R. “Seismic Evaluation of Lightly Reinforced Concrete Frames considering Joint Details”, Ph.D. Dissertation 1998, Indian Institute of Technology Madras, India.

17. Kunnath, S.K., A.M. reinhorn and Y.J. Park. “Analytical modeling of inelastic seismic response of RC structures.” Journal of Structural Engineering Division, ASCE 1990; 116: 996-1017.

18. Park, Y.J., A.H.-S. Ang and Y.K. Wen. “Mechanistic seismic damage model for reinforced concrete Journal of Structural Engineering Division, ASCE 1985; 111: 722-739.

19. SP: 34 (S&T) :1987 “Handbook on concrete reinforcement and detailing” Bureau of Indian Standards, New Delhi.

20. IS 875 : 1987 Code of practice for design loads (other than earthquake) for building and structures - Part 2 : Imposed loads; Bureau of Indian Standards, New Delhi.

21. IS 13920 :1993 Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces; Bureau of Indian Standards, New Delhi.

22. Panashahi, N., R.N. White and P. Gergely. “Reinforced concrete compression lap splices under inelastic cyclic loading” ACI Structural Journal 1992; 89: 164-175.