Comparative Study of Beam Column Junction Subjected To Specified Ground Motion

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Abstract- *Reinforced concrete moment resisting frames (RCMRF) are structural systems that should be designed to ensure proper energy dissipation capacity when subjected to seismic loading. In this design philosophy the capacity design approach that is currently used in practice demands "strong-column / weak-beam" design to have good ductility and a preferable collapse mechanism in the structure. When only the flexural strength of longitudinal beams controls the overall response of a structure, RC beam-column connections display ductile behavior (with the joint panel region essentially remaining elastic). The failure mode where in the beams form hinges is usually considered to be the most favorable mode for ensuring good global energy-dissipation without much degradation of capacity at the connections. Though many international codes recommend the moment capacity ratio at beam column joint to be more than one, still there are lots of discrepancies among these codes and Indian standard is silent on this aspect.*

Keywords- Linear & Non Linear analysis, RCC & Precast beam-column connections, ground motion.

I. INTRODUCTION

Precast concrete systems have many advantages like speed in construction, good quality due to factory production, economy in mass production. Despite many advantages of precast concrete, it is not widely used throughout the World, especially in regions of high seismic risk. The reason behind this is a lack of confidence and knowledge base about their performance in seismic regions as well as the absence of rational seismic design provisions in major model building codes (Priestley, 1991). High storey precast frame panel buildings performed poorly in the 1988 Spitak, Armenia earthquake due the lack of adequate seismic design considerations such as ductility in precast joints (Hadjian, 1993). A significant number of parking structures suffered extensive damage and a number of precast concrete parking structures collapsed in the 1994, Northridge earthquake. One of the reasons for the collapse was lack of proper diaphragm connections (Mitchell et al., 1995). In the 1995 Kobe earthquake, most of the precast prestressed concrete structures performed well, only three sustained severe structural damage. The structural damage was due to insufficient connection detailing (Muguruma et al., 1995). The lessons learnt from the past earthquakes are that the connections are the weakest link. Hence more research is required in the study of connections.

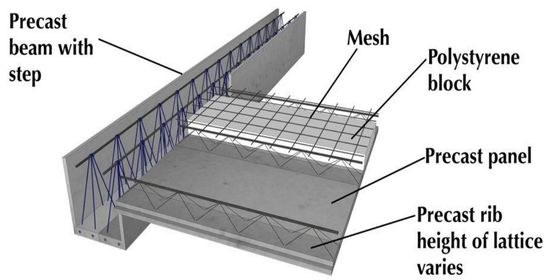


Fig 1: Precast beam

AIM: To study of beam column junction subjected to specified ground motion.

II. OBJECTIVE

- To study precast element and compare its aspect with RCC.
- To study and collect data of specified ground motion for time history analysis.
- To check and compare parameters like bending stress, shear stress and principal stress for linear and non-linear analysis.

III. LITERATURE REVIEW

EhsanNoroozinejadFarsangi**[1]** studied finite element analysis on 4 types of precast connections which are pinned, rigid, semi rigid and a new proposed connection. The stiffness of the new connection was obtained from the slope of the total load versus deflection graph in the elastic range. Then the seismic loading from El Centro earthquake modified with 0.15g and 0.5g were applied to the whole structure.He concluded from the analysis results that new connection has sufficient stiffness, strength and also higher ductility. Meanwhile, the whole structure analysis results showed that the new connection behaves as semi rigid connection. LUSAS and SAP2000 were used for analysis.

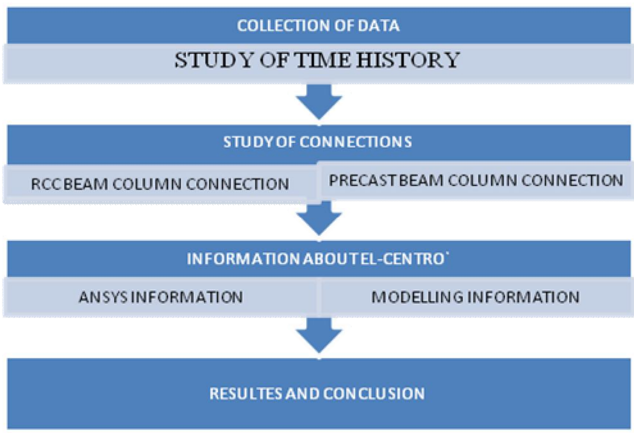
AkashLanke*et al***[2]** had taken one building as a case and designed the same building as a precast building & Traditional Cast in-situ building. He made cost analysis as well as feasibility check on basis of costing & Duration.From his analysis he concluded that the cost of precast building significantly reduces the duration of construction which is much lesser than traditional method. Also he concluded that the precast concrete system is economical than conventional cast in situ method taking into consideration some conditions like good supervision while using precast, quantity of construction, distance of site from manufacturing unit, and type of building etc.For standard and Repetitive work precast is the best option to choose. Precast construction technique is time effective and it require less time to construct. It requires skilled worker and qualified contractor, Lower initial cost especially for large project. Better concrete quality control and lighter concrete unit. The main limitation of precast is transportation from place of manufacturing to place of site where it is to be fixed.

R.A. HawilehLanke*et al***[3]** Studied nonlinear finite element analysis and modeling of a precast hybrid beam–column connection subjected to cyclic loads. A detailed three-dimensional (3D) nonlinear finite element model was developed to study the response and predict the behavior of precast hybrid beam–column connection subjected to cyclic loads that was tested at the National Institute of Standards and Technology (NIST) laboratory.The model had taken into account the pre-tension effect in the post-tensioning strand and the nonlinear material behavior of concrete. The model response was compared with experimental test results and yielded good agreement at all stages of loading. Fracture of the mild-steel bars resulted in the failure of the connection. In addition, the magnitude of the force developed in the post-tensioning steel tendon was also monitored and it was observed that it did not yield during the entire loading history.He concluded that successful finite element modeling will provide a practical and economical tool to investigate the behavior of such connections.

Vidjeapriya. R *et al***[4]**had developed a 3-D nonlinear FE model to study the response of an exterior precast beam to column connection subjected to reverse cyclic loading. Tests of a one-third scale exterior beam column precast concrete connections was conducted. Two types of connections were compared. The connections included a monolithic connection and two precast beam - column connections (i) using J-bolt (ii) using Cleat Angle. ANSYS finite element software was used for the non-linear analysis of the precast beam column connection. For the nonlinear analysis, one-third scale model was developed. Two types of elements were used including solid elements and contact elements.The finite element analysis results compared well with the experimental data. It is concluded that if the material constitutive relation and failure criterion can be selected suitably, the finite element model can accurately predict the overall seismic behavior and the inelastic performance of these two kinds of joints.

Prof. Dr. Khalid S. Mahmoud *et al***[5]** studied the nonlinear response of composite concrete beams, a finite element analysis was presented. Material nonlinearities as a result of nonlinear response of concrete in compression, crushing and cracking of concrete, strain softening and stiffening after cracking, yielding of reinforcement, bond- slip, shear-slip, and dowel action between the precast concrete beams and the cast-in-situ slabs were considered. A biaxial concrete model was adopted. A two-dimensional plane stress finite element type was used to model the concrete. Reinforcement was represented by one-dimensional bar elements. Bond-slip and dowel action was modeled by using fictitious linkage elements with two springs at right angles. Shear-slip was modeled by using shear transfer interface elements with appropriate stiffness values.Comparison between the results obtained by the finite element and available experimental results of composite concrete beams aremade. The results compare satisfactorily with the experimental ones.

IV. METHODOLOGY



V. GROUND MOTIONS AND LINEAR TIME HISTORY ANALYSIS

Dynamic analysis using the time history analysis calculates the building responses at discrete time steps using discredited record of synthetic time history as base motion. If three or more time history analyses are performed, only the maximum responses of the parameter of interest are selected. Time history analysis is the study of the dynamic response of the structure at every addition of time, when its base is exposed to a particular ground motion. Static techniques are applicable when higher mode effects are not important. This is for the most part valid for short, regular structures. Thus, for tall structures, structures with torsional asymmetries, or no orthogonal frameworks, a dynamic method is needed.

In linear dynamic method, the structure is modeled as a multi degree of freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modeled utilizing time history analysis, the displacements and internal forces are found using linear elastic analysis. The playing point of linear dynamic procedure as for linear static procedure is that higher modes could be taken into account. In linear dynamic analysis, the response of the building to the ground motion is computed in the time domain, and all phase information is thus preserved. Just linear properties are considered. Analytical result of the equation of motion for a one degree of freedom system is normally not conceivable if the external force or ground acceleration changes randomly with time, or if the system is not linear. Such issues could be handled by numerical time-stepping techniques to integrate differential equations.

In order to study the seismic behavior of structures subjected to low, intermediate, and high-frequency content ground motions, dynamic analysis is required. The STAAD Pro [1] software is used to perform linear time history analysis. Two, six, and twenty-story regular as well as irregular RC buildings are modeled as three-dimension. Material properties, beam and column sections, gravity loads, and the six ground motions listed in Table 4.3 are assigned to the corresponding RC buildings and then linear time history analysis is performed. The linear time-history analysis results for regular and irregular RC buildings are shown in chapter 5 and 6 respectively.

VI. ANSYS

Material modelling:

The definition of the proposed numerical model was made by using finite elements available in the ANSYS code default library. SOLID186 is a higher order 3-D 20-node solid element that exhibits quadratic displacement behavior. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. The element supports plasticity, hyper elasticity, creep, stress stiffening, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyper elastic materials. The geometrical representation of is show in SOLID186 fig 3.5

This SOLID186 3-D 20-node homogenous/layered structural solid were adopted to discretize the concrete slab, which are also able to simulate cracking behavior of the concrete under tension (in three orthogonal directions) and crushing in compression, to evaluate the material non-linearity and also to enable the inclusion of reinforcement (reinforcement bars scattered in the concrete region).The element SHELL43 is defined by four nodes having six degrees of freedom at each node. The deformation shapes are linear in both in-plane directions. The element allows for plasticity, creep, stress stiffening, large deflections, and large strain capabilities. The representation of the steel section was made by the SHELL 43 elements, which allow for the consideration of non-linearity of the material and show linear deformation on the plane in which it is present. The modelling of the shear connectors was done by the BEAM 189 elements, which allow for the configuration of the cross section, enable consideration of the non-linearity of the material and include bending stresses as shown in fig 3.4. CONTA174 is used to represent contact and sliding between 3-D "target" surfaces (TARGE170) and a deformable surface, defined by this element. The element is applicable to 3-D structural and coupled field contact analyses. The geometrical representation of CONTA174 is show in fig 3.1. Contact pairs couple general axisymmetric elements with standard 3-D elements. A node-to-surface contact element represents contact between two surfaces by specifying one surface as a group of nodes. The geometrical representation of is show in TARGET 170 fig 3.2.

The TARGET 170 and C0NTA 174 elements were used to represent the contact slab-beam interface. These elements are able to simulate the existence of pressure between them when there is contact, and separation between them when there is not. The two material contacts also take into account friction and cohesion between the parties.

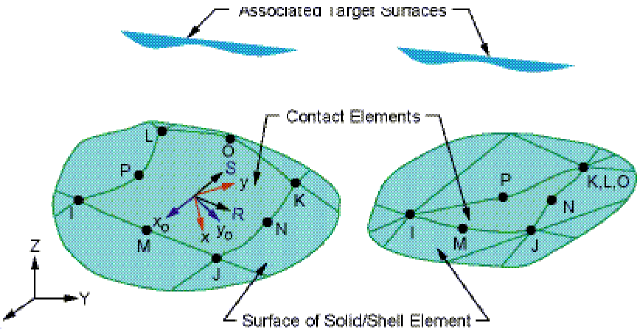


Fig.1: CONTA 174

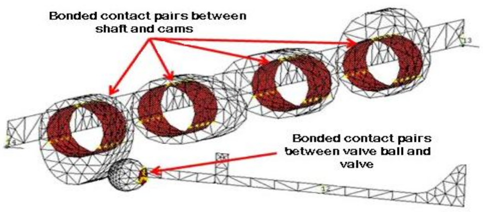


Fig.2: TARGET 170

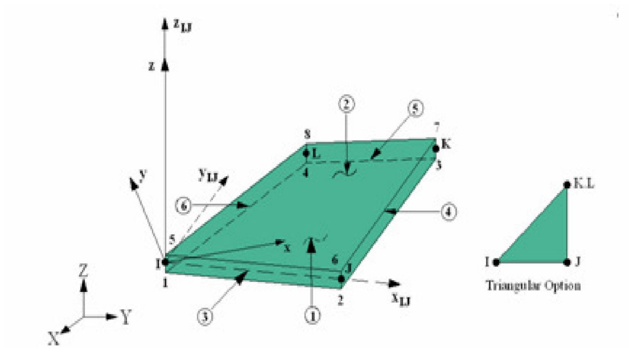


Fig.3: Shell 43

VII. MATERIAL PROPERTIES

Sr.No.	Material	Property	Value
1	Structural steel	Yield stress f_y (MPa)	265
		Ultimate strength f_u (MPa)	410
		Young's modulus Es(MPa)	205×10^3
		Poisson's ratio u	0.3
		Ultimate tensile strain et	0.25
2	Reinforcing bar	Yield stress f_y (MPa)	250
		Ultimate strength f_u (MPa)	350
		Young's modulus Es(MPa)	200×10^3
		Poisson's ratio u	0.3
		Ultimate tensile strain et	0.25
3	Concrete	Compressive strength f_c (MPa)	42.5
		Tensile strength f_t (MPa)	3.553
		Young's modulus E_c(MPa)	32920
		Poisson's ratio u	0.15
		Ultimate compressive strain e_c	0.045

VIII. PROBLEM STATEMENT

A G+9 RCC Commercial building is considered.

Plan dimensions: 12 m x 12 m
Location considered: Zone-IV
Soil Type considered: Hard Strata.

General Data of Building:

- Grade of concrete: M 25
- Grade of steel considered: Fe 250, Fe 500
- Live load on roof: 2 KN/m2 (Nil for earthquake)
- Live load on floors: 4 KN/m2
- Roof finish: 1.0 KN/m2
- Floor finish: 1.0 KN/m2
- Brick wall in longitudinal direction: 240 mm thick
- Brick wall in transverse direction: 140 mm thick
- Beam in longitudinal direction: 230X350 mm
- Beam in transverse direction: 230X350 mm
- Column size: 300X750 mm
- Density of concrete: 25 KN/m3
- Density of brick wall including plaster: 20 KN/m3
- Plinth beam (PB1): 350X270 mm
- Plinth beam (PB2): 270X300 mm

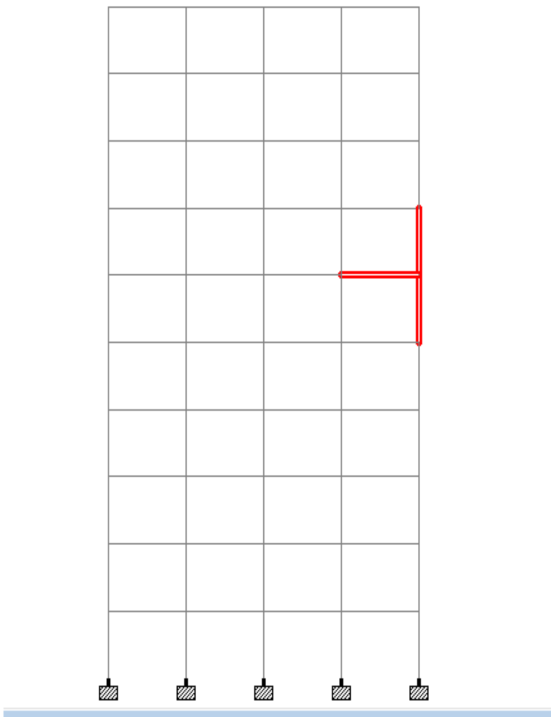


Fig 2 Mathematical Model of Frame structure

IX. ANALYSIS OF MODEL

Following are the analysis of RCC beam column connection subjected to point load in ANSYS

MANUAL CALCULATION OF DESIGN OF COLUMN

1.DATA:

Axial load P =	715	KN	
Size of column			
B=	230	Mm	
D=	230	Mm	
Eff length of column L=	1.95	M	
f_{ck}=	20	N/mm^2	
f_y=	415	N/mm^2	

2. SLENDERNESS RATIO:

(L/D)=	8.48	<12

Hence the column is short column

3.FACTORED LOAD:

P_u= 1.5*P =	1072.5	KN

4.MINIMUM ECCENTRICITY:

e_{min}=(L/500+ D/30)=	7.67	Mm	<20mm
also 0.05D =	11.5	Mm	<20mm

5.MAIN REINFORCEMENT:

$P_u = 0.4f_{ck}*A_g+(0.67f_y-0.4f_{ck})A_{sc}$

A_{sc}=	2404.37	mm^2
Ast_{min}=0.8* A_g/100 =	423.2	mm^2

hence provide 6 nos of 22mm diameter with 3 bars distributed on each face

6.LATERAL TIES:

Dia = dia of main bar /4=	3	<6mm	dia of tie bar =	6mm

hence provide 6mm dia lateral ties

7.PITCH:

Least lateral dimension =	230	mm
16 times of dia of main bar=	192	mm
minimum 300 mm		
whichever is less		
hence adopt 6mm dia bar at	230	mm c/c

MANUAL CALCULATION OF DESIGN OF BEAM:

Design of Rectangular Beam

1. GIVEN DATA:

B =	300
D = span/10	450
d =	575
clear span =	4500
effective cover =	35

2. DESIGN CONSTANT:

fck =	20
fy =	415
Qu =	2.759

3. EFFECTIVE SPAN:

i)clear span+effective depth	5075

ii)c/c distance of support = whichever is less	4730
	$L_{eff} =$ 4.73

4. LOAD CALCULATION:

live load =	16.555
load from slab =	17.7375
self weight of beam =	3.375
total load =	37.6675
design load =	56.50

5.CALCULATION OF BM:

$M = WL^2/12$ =	105.34

6.EFFECTIVE DEPTH REQ:

MR = 2.759*bd^2

d =	√(M/Qb)
d =	356.7492495

7.CALCULATION OF AST:

astreq =o.5 fck/fy[1-√1-(4.6mu/(fck bd^2))]bd

astreq =	543.1572993
Ast min	
0.85bd/fy =	353.313253
Ast max	
0.04bD =	6900

hence we provide steel for astreq

8.SHEAR REINFORCEMENT:

vu =WL/2	133.63
τv = vu/b	0.45
pt = 100ast/bd	0.31
τc =	0.45

9.DESING OF SHEAR REINFORCEMENT:

$Vus = v_u - \tau_c bd$	56000.46

10.SPACING :

i) Sv = 0.87fy $A_{sv}d/V_{us}$	186.25
ii)3d =	1725
iii)300 mm	

hence provide 2 legged vertical stirupps @300 c/c

11. CHECK FOR MIN REINFORCEMENT:

i) Sv = 0.87fy $A_{sv}/0.4b$

= 151.1596

X. CONCLUSION

- In this project the beam column connections are studied subjected to dynamic load.
- In literature review so far concluded that Seismic Performance Factors for Precast Buildings with Hybrid Beam-Column Connections and Comparison between cast-in-situ and precast solutions subjected to seismic load.
- Results confirmed its good structural performances in terms of strength and ductility in case of precast beam column connections.

XI. FUTURE SCOPE

- On this project the various precast members are studied subjected to static load and dynamic load.
- It's observed that precast members are more effective than RCC members for both static load and dynamic load.
- However same comparison can be made for vibration analysis.

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