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Original Research Article

Effect of Nonlinear Modeling of Beam-Column Joint on Pushover Analysis

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Abstract

The present paper is concerned with the seismic risk assessment of buildings in the Kingdom of Saudi Arabia. A critical review of the existing literature is presented to identify the shortcomings of extant studies. None of the extant studies considered nonlinear action of the beam-column joint (BCJ) but rather they dealt with BCJ as a rigid element for simplicity and the only plastic hinging has been considered in beams and/or columns. Hence the main focus of this paper is to demonstrate the significant effects of the nonlinear action of BCJ in the pushover analysis and in turn the inadequacy of all previous studies which overlooked such effect. In this study, nonlinear static pushover analysis is performed on two-dimensional RC frames of existing buildings in Jeddah city, with and without using macro node elements and pushover curves are compared. The beam-column joint modelling approach adopted in this study is through macro node element which accounts for failure due to shear collapse of the joint, concrete crushing, flexural and/or shear plastic damage of the beams or columns connected and bond-slip failure. The results clearly indicate that the RC frame in which the beam-column joints were modeled using a macro node element, tends to have lesser base shear values and higher displacement capacity when compared to the RC frame modeled without using the macro node. Furthermore, the status of plastic hinges developed in building frames modeled without using macro node element was found to be within the Immediate occupancy (IO) performance level, but this hinge status drastically changed to Collapse prevention (CP) performance level when BCJ was modeled using macro node. Hence, the results highlight that the nonlinear action of beam-column joint has a significant effect on the nonlinear response of a structure.

Keywords: Beam-column joint, Macro modeling, Pushover analysis, Saudi Arabia.

1. INTRODUCTION

It is well known that attempting to conduct performance-based evaluation of existing buildings in low to medium seismic activity areas such as Jeddah city in Saudi Arabia is a big challenge. On one hand due to scarcity of data and on the other hand most of extant studies are not readily applicable to such case. For example, pre-1970s buildings in Jeddah city are characterized by being gravity designed and contain many deficient reinforcement details mainly related to beam column joints such as lack of splice and development length as well as use of smooth bars whose bond characterization are inferior to the present deformed bars.

Beam column joints of reinforced concrete (RC) framed structures are one of the key structural elements, especially which are non-seismically designed as they greatly affect the response of structures exposed to seismic loading. In RC framed structures, it is usually assumed that all plastic rotations occur in the beams and columns and that the joint core is rigid. This assumption is acceptable for the structure subjected to gravity loads, but greatly deceptive for structures exposed to earthquakes. Under the action of earthquake, the joint core, which is initially rigid, slowly starts to soften due to the large amount of shear forces acting on it. The beam-column joint experiences brittle shear failure during an earthquake, and they contribute substantial portion as much as 75% to overall structural drift *(Walker et al.*, 2004). Hence, modeling the inelastic behavior of joints is of great significance to evaluate the accurate performance of the structures.

2. LITERATURE REVIEW

2.1 Critical Review of Seismic Risk Assessment of Buildings in Saudi Arabia

S.No.	Author	Structural System	Idealized/Actual Building	Seismic Analysis	Beam Column Joint Modelled
1.	Tarek M. Alguhane et.al	Five story existing old Reinforced Concrete building	Actual	Pushover Analysis	Rigid
2.	Mohammed Ismaeil et.al	6 storeyed residential building in Haql city. The selected building is an Ordinary Moment Resisting Frame (OMR)	Actual	Equivalent Lateral force Method (SBC 301)	Rigid
3.	Yasser Alashker et.al	Five storied Reinforced Concrete School building in Saudi Arabia	Actual	Equivalent Lateral force Method (SBC 301)	Rigid
4.	Mohammed Ismaeil et.al	Eight-Storey Existing RC Building in Abha City, Saudi Arabia.	Actual	Equivalent Lateral force Method (SBC 301)	Rigid
5.	A.E. Hassaballa et.al	10 storeyed reinforced concrete building located in the Jazan city.	Actual	Equivalent Lateral force Method (SBC 301)	Rigid
6.	Abo El-Wafa et.al	The studied building is a six-storeyed hollow block slab type reinforced concrete office building.	Actual	Pushover Analysis & Linear Time History Analysis	Rigid
7.	M.K. Rahman et.al	8 Storeyed RC Concrete frame-shear wall building in Madinah constructed in 1996	Actual	Pushover Analysis	Rigid
8.	Suwondo et.al	An existing typical six storeyed residential building assumed to be located in four different regions Makkah, Jeddah, Gizan and Haql.	Actual	Pushover Analysis	Rigid
9.	Yasser Alashker et.al	Four buildings of same area but with different aspect ratios of 1, 1.5, 2 & 4 have been analyzed.	Idealized	Pushover Analysis	Rigid
10.	Yasser E. Ibrahim	12 storeyed moment resisting RC concrete frame designed as per Saudi Building Code was analyzed considering the seismic factors of three cities; Abha Jazan and Al-sharaf.	Idealized	Incremental Dynamic Analysis	Rigid
11.	R. A. Hakim et.al	Four RC frame structures having 3, 6, 9, and 12 stories intended for regular residential building in Haql were adopted in the study.	Idealized	Pushover Analysis	Rigid
12.	Riza Ainul Hakim et.al	This study considered a six storeyed reinforced concrete building in Saudi Arabia.	Idealized	Pushover Analysis	Rigid
13.	Riza Ainul Hakim et.al	A 3 storeyed RC gravity designed building was investigated for its resistance to expected seismic loading in different regions (Makkah, Jeddah, Gizan and Haql).	Idealized	Pushover Analysis	Rigid
14	Abdulelah	Five different buildings School, Residential (2) , Mosque, Commercial were considered in this study.	Actual	Pushover Analysis	Rigid
15	Ayed Eid Alluqmani	Four-storey reinforced concrete building (Faculty of Engineering –E1 building) which is located at Islamic University in Medina was chosen in this study.	Actual	Pushover Analysis and Linear Dynamic Analysis	Rigid

The above summary highlights that Non-linear static pushover analysis is most commonly being adopted to perform the seismic assessment of buildings in Saudi Arabia. It is also seen in literature that in every work, the beam-column joint is modelled as a rigid joint during the seismic analysis. This leads to unconservative and less accurate results. None of the existing studies in Saudi Arabia considered the nonlinear action of the beam-column joint (BCJ) but rather they dealt with BCJ as rigid element for simplicity and only plastic hinging has been considered in beams and/or columns. For accurate nonlinear analysis of a structure, it requires an elaborate modeling of beam column joint as well as it controls the behavior of a structure significantly. Hence the main focus of this paper is to demonstrate the significant effects of the nonlinear action of BCJ in the pushover analysis and in turn the inadequacy of all previous studies which overlooked such effect.

2.2 Critical Review of Existing Beam-Column Joint Modeling Methods

Many researchers have proposed different modeling methods for the beam column joint such as models based on experimental data, spring models (multiple and rotational) and finite element models. Each model has its own strengths and limitations. For example, some model requires large database of experimental results in order to calibrate it, some models may be suitable only to specific type of joint such as joints which have reinforcement in the core, joints having lesser strength etc. There are some models which are very thorough and can be applied for any type of joint theoretically, but they are impractical to use and very expensive computationally. Few models with their limitations are discussed below.

In 1973, Townsend & Hanson proposed polynomial expressions to model BCJ, but its limitation is that, as it is established on the physical understanding of the behavior of the joint and not on mechanics of the joint, it couldn’t be used to model all types of joints. Otani (1974) suggested a rotational hinge model but it was based on the assumption that bond stresses are constant throughout the development length of the rebars and there is sufficient reinforcement embedment length. Anderson & Townsend (1977) and Soleimani et.al(1979) also suggested models in the but both these models were not based on mechanics of the joint and cannot be applied to all types of joints. In 1983, Fillipou suggested a model with rotational spring, but its limitation is that it does not take into consideration the shear developed in the joint and diagonal cracking of the joint. El-Metawally & Chen (1988) proposed a model based on the assumption that during an earthquake, the inelastic joint action is controlled by the anchorage failure of longitudinal reinforcement and the total energy dissipation due to this failure is approximately constant for all joints. This model is applicable to only those beam column joints which are adequately designed and have proper shear strength. Alath & Kunnath, in their model altered the flexural capacities of the beams and columns to implicitly model the inadequate anchorage and deficient joint shear capacity. Furthermore, the joint shear deformation was modeled with a rotational spring with degrading hysteresis. Rigid links were used to develop the finite size of the joint model.

Altoontash and Lowes (2003) proposed a four node with twelve degree of freedom joint element. To represent the bond-slip response of the beam and longitudinal reinforcement of column, 8 translational springs of zero length were used. To simulate the deformation of the joint under shear, a panel zone with rotational springs were adopted. Later on based on experimental data, Lowes *et al*. (2004) modeled the shear along the interface. However, the model is not applicable for the joints with no transverse reinforcement. This model was further modified by Altoontash(2004) by introducing four rotational springs of zero-length at beam column joint interfaces. These springs simulated the member end rotations because of bond slip. The panel zone component remained the same as used in previous model.

The beam-column joint modelling approach adopted in this study is a model with macro node element proposed by (Panto, 2017) as its accounts for failure due to shear collapse of the joint, due to concrete crushing, flexural and/or shear plastic damage of the beams or columns connected and bond-slip failure. This model has some resemblances to the joint model initially suggested by (Lowes, L.N., Altoontash, 2003) and later on modified and calibrated by (Nilanjan Mitra, 2004). In comparison to other models, this approach is better as the element interfaces are discretized according to a detailed fibre discretization reporting for concrete and steel contributions. Furthermore, this macro node approach can be easily implemented during building modeling (2D) in any commercially available analysis softwares such as (SAP2000, 2005), ETABS etc. The macro node element in simple terms consists of quadrilateral with flexible interfaces. These interfaces are consistent with fibre discretization to account for concrete and steel bars effect. These nonlinear interfaces along-edge and in-plane deformability govern the mechanical behavior of the node. Separate nonlinear links are assigned in the model for concrete and steel bars affect namely concrete contact Nlinks and steel bars Nlinks respectively as shown in Figure 1. To account for the failure due to shear in beams or columns connected to the joint, a single longitudinal non-linear link is added between the core and beam/column interface which is named as concrete shear Nlink. Two diagonal nonlinear links Nlinks govern the shear failure of the central core of the node. Hence by using this model, any framed structure can be modelled and simulated as a combination of nonlinear macro nodes and elastic frame elements as shown in Figure 2.

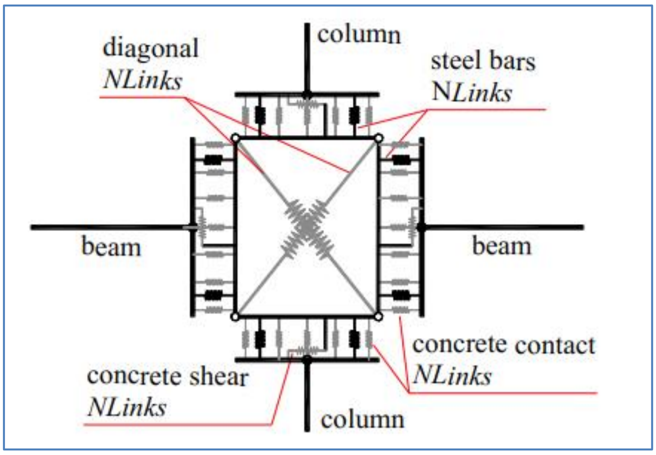

Fig-1: Beam column joint macro node modeling (Panto, 2017)

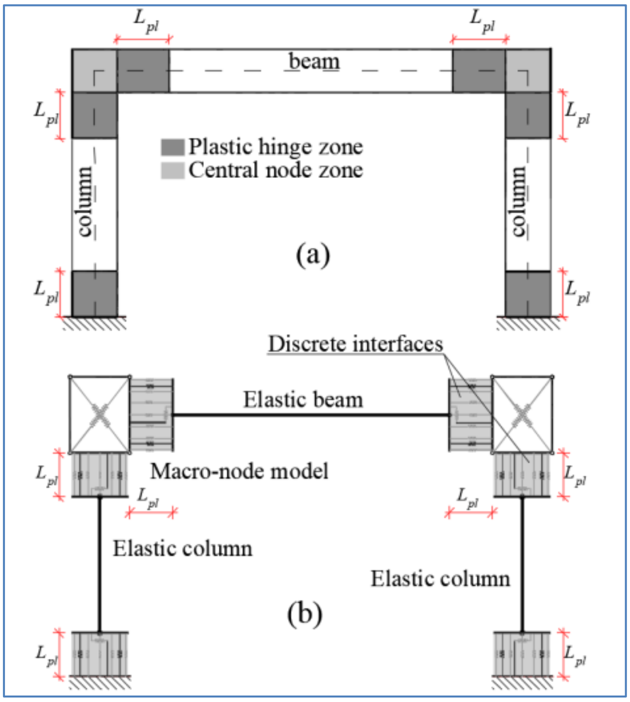

Fig-2: Schematic assembling of a frame structure by means of nonlinear macro-nodes and linear elastic beams/columns elements (Panto, 2017)

3. Verification Model

Before proceeding with the detailed study of the discrete macro node, it is necessary to first reproduce the results of the published work by Panto as we will be using his proposed macro node. For this purpose, RC frame adopted in the study by Panto (Figure 3) is remodeled as shown in below Figure 4. Nonlinear behavior of a plane three-storey three-bay frame characterized by constant cross sections 300x457mm and 300x500mm respectively for columns and beams is considered. A simple geometry is adopted with 5 m span length, 0.3m constant transversal width, 3.5m first-storey height and the others 3.2m height. A uniform vertical load of 10.0 kN/m is applied on the beams. RC frame without macro node and with macro node is modelled in SAP 2000 as shown in Figures 4 & 5. A comparison of time period of vibrations of RC frame with and without macro node is carried out between Panto study and verification model. The results are shown in Tables 1 and 2. Mode shapes of the frame with and without macro node are also compared in Figures 6 & 7.

Table-1: Verification Model without Macro Node

Time Period	Panto Study	Verification Model
T1	0.272	0.271
T2	0.086	0.089
T3	0.055	0.054

Table-2: Verification Model with Macro Node

Time Period	Panto Study	Verification Model
T1	0.239	0.239
T2	0.074	0.076
T3	0.050	0.048

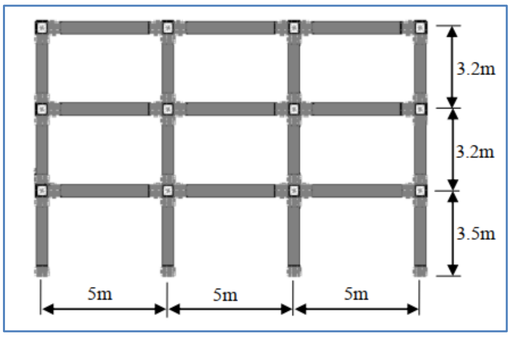

Fig-3: RC frame in Panto Study

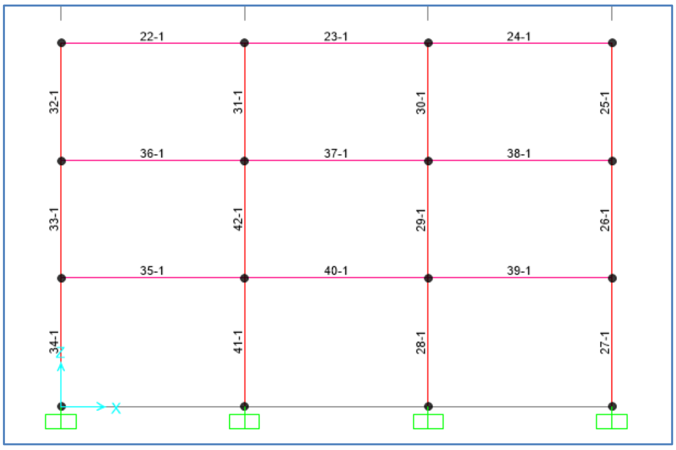

Fig-4: Verification Model without Macro Node

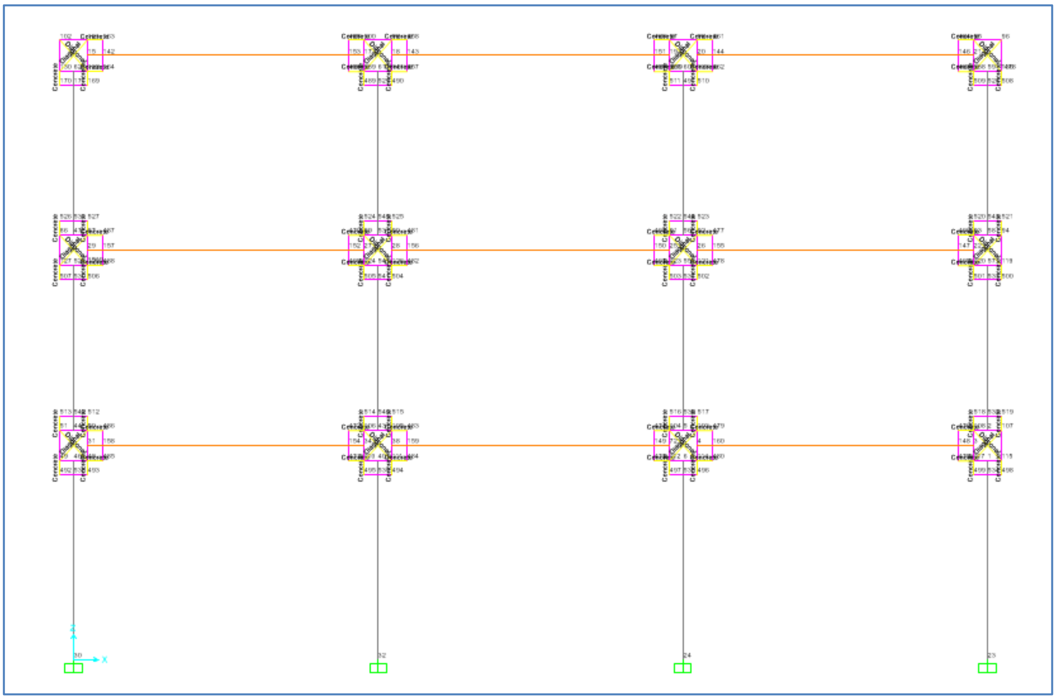

Fig-5: Verification Model with Macro Node

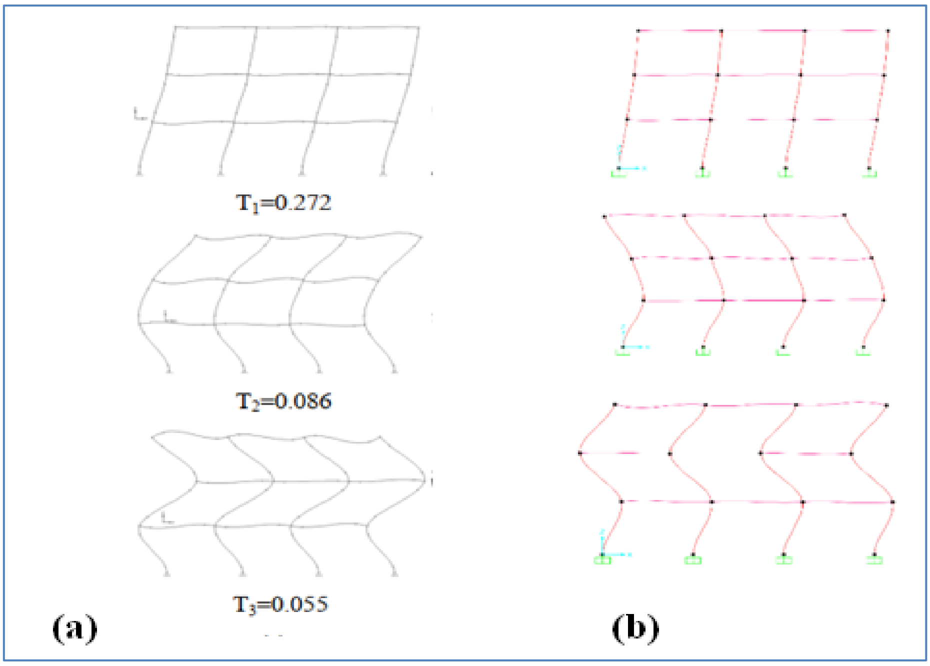

Fig-6: Comparison of Mode Shapes of RC Frame without Macro Node in Panto Study (a) and Verification Model (b)

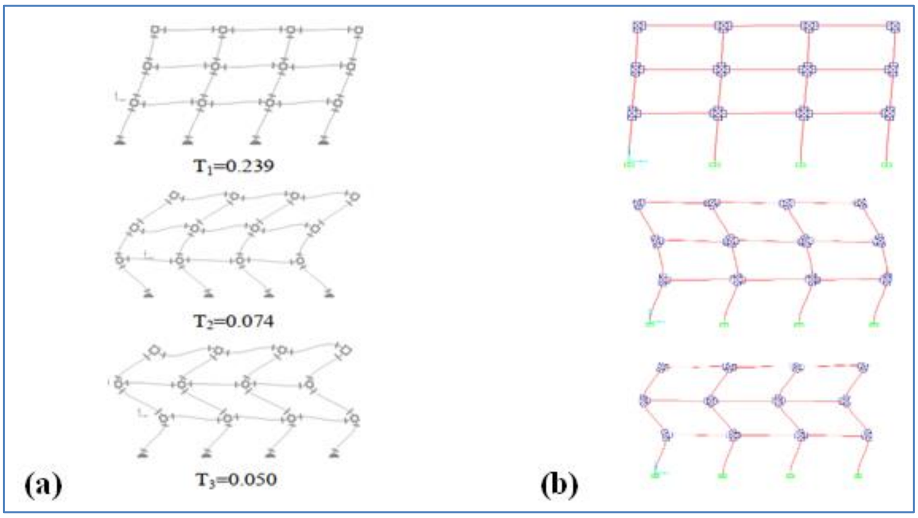

Fig-7: Comparison of Mode Shapes of RC Frame with Macro Node in Panto Study(a) and Verification Model (b)

4. Study of Beam-Column Joint Macro Node Element

In this study, the discrete macro-node (DMN) proposed by Bartolomeo Panto (….) is applied to simulate the nonlinear behavior of a plane one-story one-bay frame characterized by constant cross sections 300x450mm and 300x500mm respectively for columns and beams. A simple geometry is considered as shown in Figure 8 with 5-meter span length and 3.5-meter story height. A uniform vertical load of 10.0 kN/m is applied on the beams. Nonlinear static pushover analysis is performed to compare the pushover curve of frame with and without macro node. The rigid plate dimensions adopted in the study is 100 x 100 mm. A comparative study is also performed to assess the effect of varying thickness (5cm, 10cm, 20cm, 30cm) of rigid plate, number of nonlinear links in macro node and plastic hinge behavior in the nonlinear response of the RC frame.

4.1 DISCUSSION OF RESULTS

Significant change in pushover response was observed which had discrete macro node modelled compared to model without macro node (Figure 9). The structure modelled with macro node failed abruptly (columns failed first), whereas the structure without macro node showed gradual collapse mechanism (beams fail first then column). No significant effect on the pushover curve was observed upon varying number of links. Hence two links at the ends of the rigid plate were adopted for further study (Figure 10).

With increasing thickness of rigid plate, the base shear values of the pushover curve tend to increase (Figure 11). The results showed that smaller plate thickness value fails to provide structural stability of the joint and larger plate thickness value increased the rigidity of the joint. As the main aim of macro node element is to represent the flexibility of a beam column joint, hence 10 cm thickness was found to be appropriate in macro node element**.**

Hinges in Beams were found to be in the state "Beyond E" in model without Macro node and same beam hinges were found to be in Hinge state "B to C" or IO to LS in the model with macro node. In both models, the hinges in columns near BCJ were found to be in hinge state A to B. The hinges in columns near supports were found to be in the state "C to D" in model without Macro node and same column hinges were found to be in hinge state "Beyond E" in the model with macro node (Table 4). Overall, it can be observed that in the model with macro node, the acceptance criteria of beam hinges improved (i.e. from Collapse prevention to Immediate Occupancy) whereas that of column hinges deteriorated in terms of hinge state (C to D state to Beyond E) but acceptance criteria remains the same. i.e. Collapse prevention.

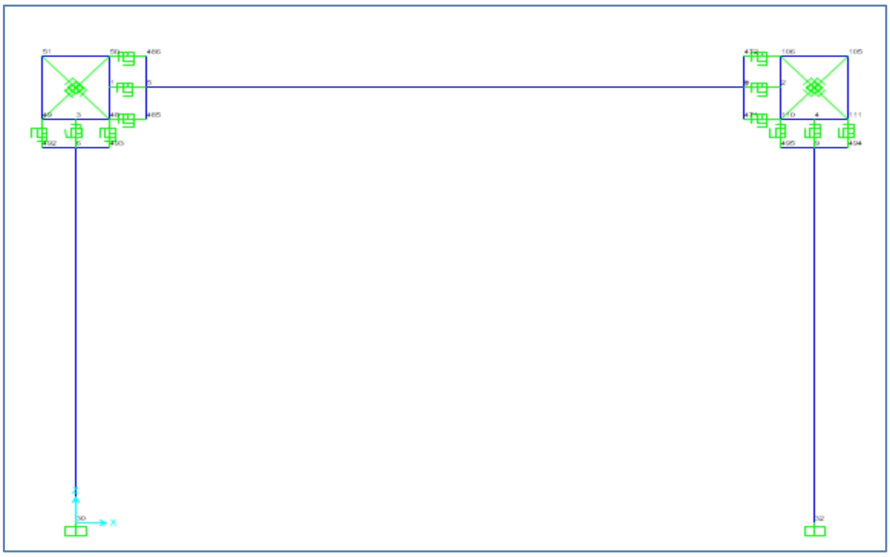

Fig-8: One bay one storey RC frame

Table-3: Pushover analysis results

	Without Macro Node		***With Macro Node***	
Step	**Displacement**	**Base Force**	**Displacement**	**Base Force**
Unitless	**cm**	**KN**	**cm**	**KN**
0	0.00	0.00	0.02	0.00
1	0.17	38.87	1.46	153.87
2	0.38	68.80	1.56	159.36
3	1.44	164.00	5.61	188.73
4	1.50	166.56	5.61	101.82
5	5.50	180.36	5.65	104.15
6	5.79	181.37	5.74	104.62
7	5.85	181.41	5.74	21.31
8	7.12	178.67	8.74	32.48
9	11.61	158.47	11.74	43.65
10	17.05	133.49	14.74	54.83
11	17.05	133.49	17.74	66.00
12	17.94	129.49	20.74	77.17
13	17.94	129.49	22.31	83.04
14	20.17	120.63	22.60	83.68
15	20.17	120.68	25.60	86.17
16	20.22	120.36	28.60	88.65
17	20.23	120.54	30.02	89.83
18	24.23	110.84		
19	28.23	101.14		
20	32.23	91.44		
21	36.23	81.74		
22	40.00	72.58		

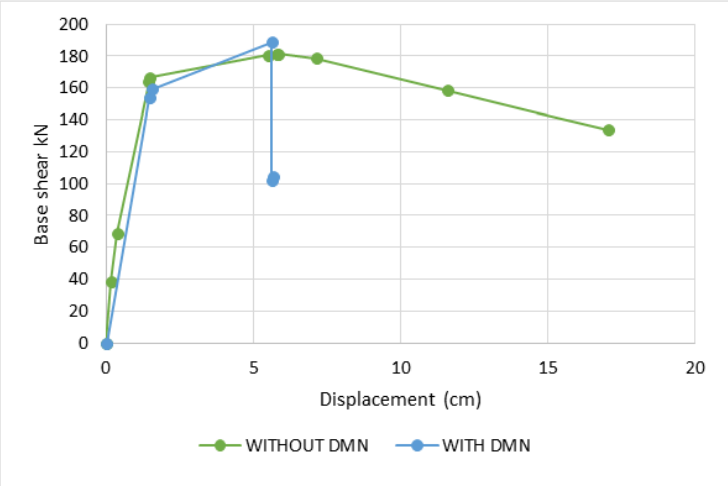

Fig-9: Pushover Curves of Model with and without Macro Node

Table-4: Plastic hinges status in models with and without macro node

	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODEL WITHOUT DMN	2	0	0	0	0	2	0	2	6
MODEL WITH DMN	2	0	2	0	0	0	0	2	6

Table-5: Pushover analysis results by varying number of links in Macro Node

2 links at ends of rigid plate	
Displacement	**Base Force**
cm	KN
0.02	0.00
1.46	153.87
1.56	159.36
5.61	188.73
5.61	101.82
5.65	104.15
5.74	104.62
5.74	21.31
8.74	32.48
11.74	43.65
14.74	54.83
17.74	66.00
20.74	77.17
22.31	83.04
22.60	83.68
25.60	86.17
28.60	88.65
30.02	89.83

11 links on rigid plate	
Displacement	**Base Force**
cm	KN
0.00	0.00
1.39	152.24
1.42	155.97
1.51	160.62
4.51	182.42
5.57	190.14
4.71	66.64

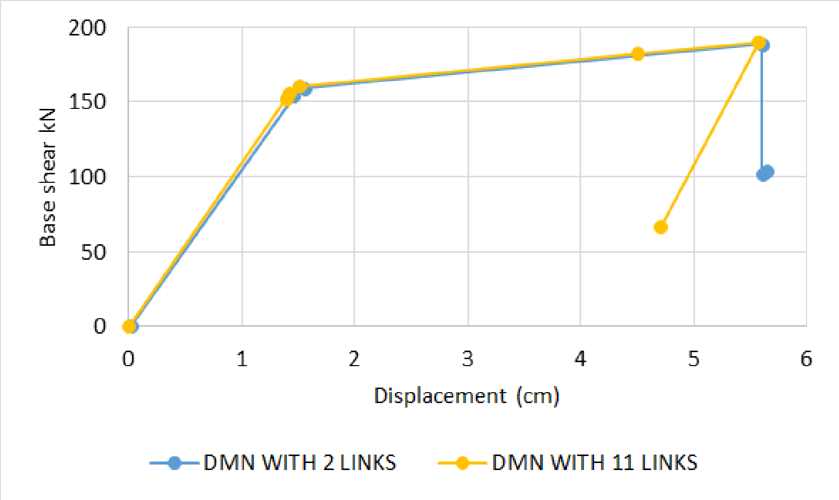

Fig-10: Pushover Curves of Model with varying links in Macro Node

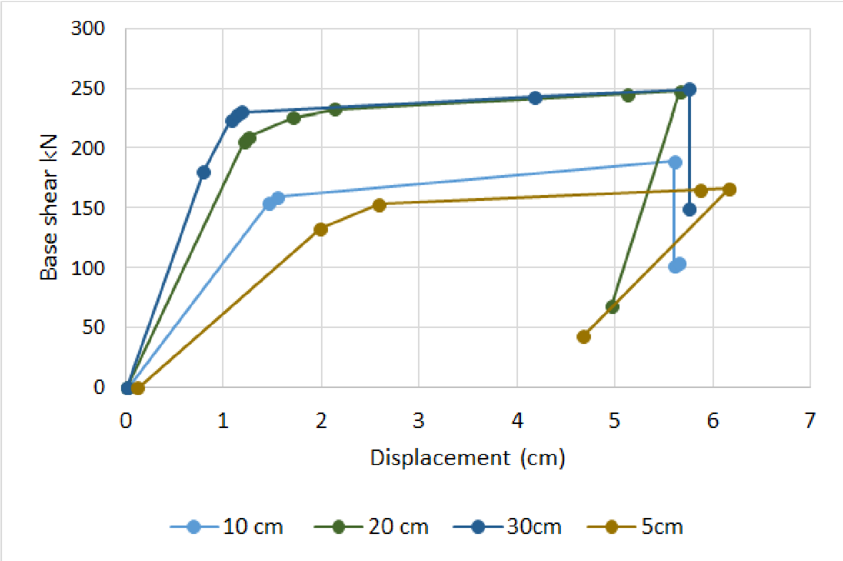

Fig-11: Pushover Curves of Model by varying thickness of rigid plate in Macro Node

5. Comparative study of Non-linear response of Existing Buildings in Jeddah city using Macro Node for Beam-Column Joint

In this study, nonlinear static pushover analysis is performed on two-dimensional RC frame of existing buildings, with and without macro node and pushover curves are compared. Modeling of buildings is done using the data of existing buildings in Jeddah city. Two buildings which come under low to mid rise buildings (2-7 storys) category are considered in this study – 3 storyed school building and 7 storyed Residential building. As-built detailing of structural members and construction details were gathered and reviewed before modeling. SAP2000 structural analysis software was used to carry out this study. The plan layout and elevation of chosen buildings are shown in below figures 1 to 4. The reinforcement and dimension details of structural members of both building frames are given in Tables 1 to 4. Two dimensional frames in XZ plane are chosen from the 3D building models to perform the pushover analysis. The chosen frames are shown in figure 5 & 6. A uniform wall load of 12 kN/m is applied on all beams in case of school building and in residential building; the load is acting on the first storey beams as shown in figures 7 & 8. Nonlinear hinges M3 and P-M2-M3 are assigned to beams and columns respectively. Comparison is done between two types of RC frames, first in which the beam column joint (BCJ) is modelled as rigid joint (commonly adopted practice so far) and second in which BCJ is modelled using the Macro node element (Figures 9, 10). Macro node element comprises of a central core and beam/column interfaces. The core and interfaces are modeled using a rigid element. These two are connected using nonlinear link elements. These interfaces account for the effect of concrete and steel. The length of the link element is equivalent to plastic hinge length in the corresponding beam or column. The length is usually adopted as half of the depth of the section. The thickness of the rigid plate element was chosen as 10 cm based on a preliminary study of the effect of varying plate thickness (5cm, 10cm, 15cm, 20cm) on nonlinear response. The results of preliminary study showed that smaller plate thickness value fails to provide structural stability of the joint and bigger plate thickness value increased the rigidity of the joint. As the main aim of macro node element is to represent the flexibility of a beam column joint, hence 10 cm thickness was found to be appropriate in macro node element. The nonlinearity of materials is defined by the hysteresis model. Kinematic hysteresis model is considered for steel as since it is based on kinematic hardening behavior which usually observed in metals. Takeda hysteresis model (Takeda *et al*., 1970) is the same as the kinematic model except it experiences a cyclic hysteretic degrading. That makes it more suitable for concrete than metals (Computers and Structures Inc., 2015).

Non liner static pushover analysis is carried out on the two-dimensional building frames of school and residential building under uniform lateral load. The building is subjected to loading till it the reaches a pre-defined displacement value or until failure whichever comes first. Pushover curve generated as a result of the analysis with base shear on the Y axis and roof displacement on X axis is evaluated and compared.

Table-6: Beams details of School Building

ID	Dimension		Longitudinal Reinforcement				Stirrups
			Continuous		Additional		
	B	H	Bottom	Top	Bottom	Top	
B2	20	70	4 Φ 18	3 Φ 18	-	-	6 Φ 8 /m
B11	30	70	6 Φ 18	4 Φ 18	-	-	6 Φ 12 /m

Table-7: Columns details of School Building

Column ID		All Floors	Ties
C2	Dim.	30 x 60	2 x 6 Φ 8 /m
	Reinf.	8 Φ 16	
C6	Dim.	30 x 80	2 x 6 Φ 8 /m
	Reinf.	12 Φ 16	
	Reinf.	20 Φ 16	

Table-8: Beams details of Residential Building

Beam ID	Dimension		Longitudinal Reinforcement				Stirrups
			Continuous		Additional		
	B	H	Bottom	Top	Bottom	Top	
HB3	90	30	6 Φ 16	6 Φ 16	4 Φ 16	4 Φ 16	2 x6 Φ 8 /m
HB4	120	30	8 Φ 16	8 Φ 16	6 Φ 16	6 Φ 16	2 x6 Φ 8 /m

Table-9: Columns details of Residential Building

Column ID		1st + 2nd	3rd + 4th	5th + 6th + 7th	Ties
C2	Dim.	20 x 80	20 x 70	20 x 60	2 x 6 Φ 8 /m
	Reinf.	14 Φ 16	12 Φ 16	10 Φ 16	
C3	Dim.	25 x 80	20 x 80	20 x 70	3 x 6 Φ 8 /m
	Reinf.	16 Φ 16	14 Φ 16	12 Φ 16	

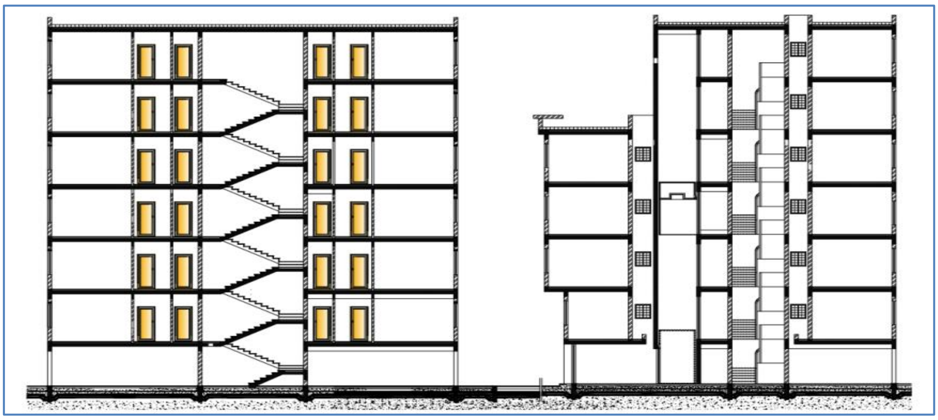

Fig-12: Elevation of Residential Building

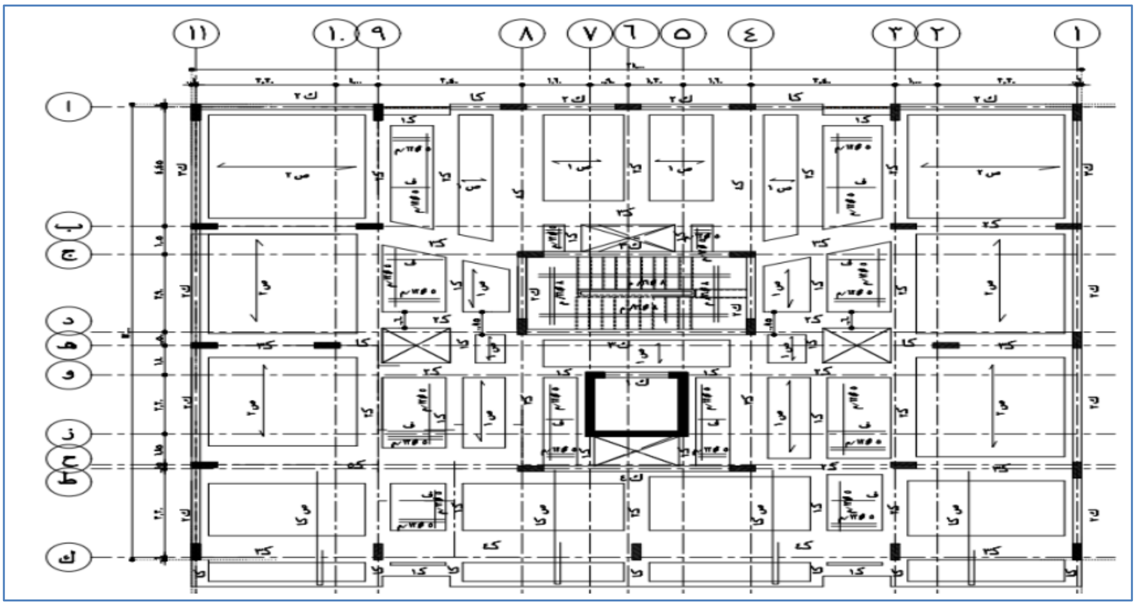

Fig-13: Layout of typical floor of Residential Building

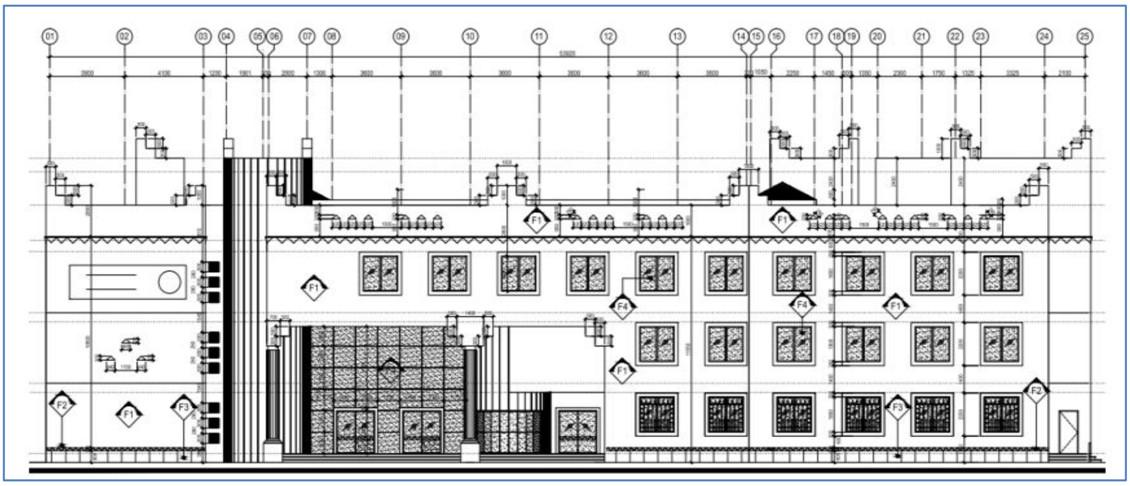

Fig-14: Elevation of School Building

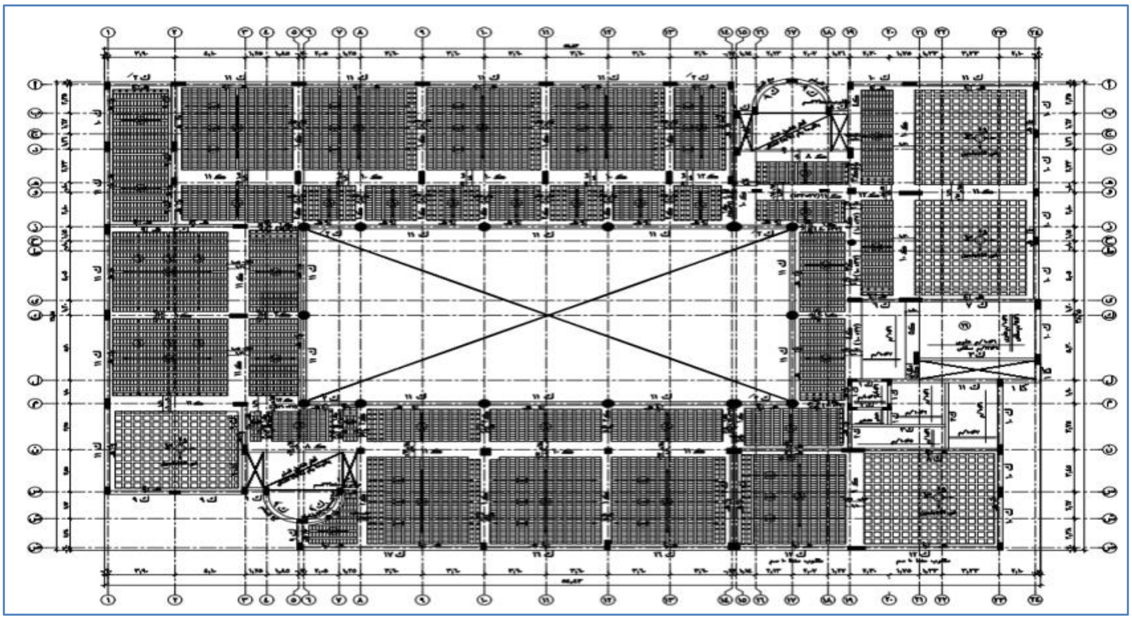

Fig-15: Floor plan of School Building

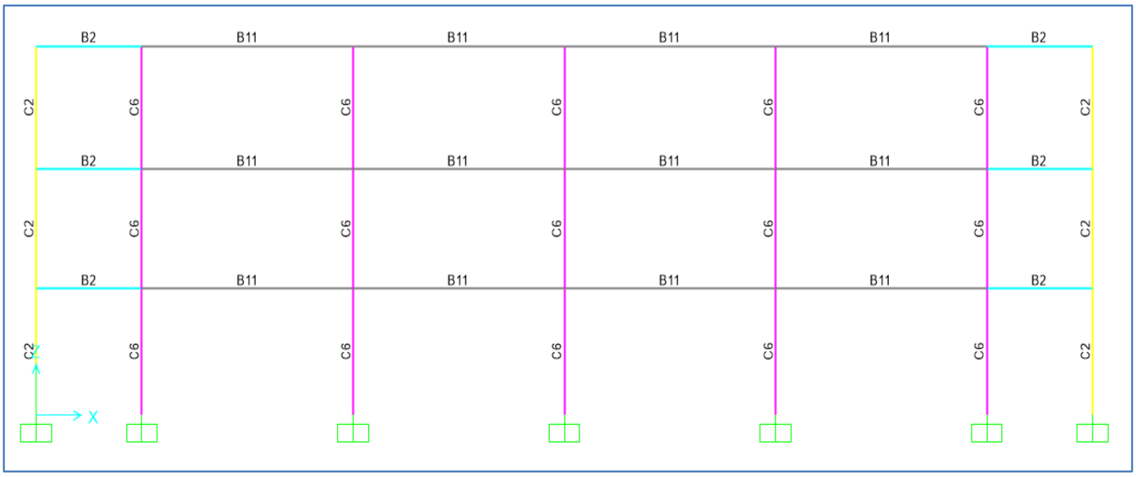

Fig-16: 2D School Building Frame

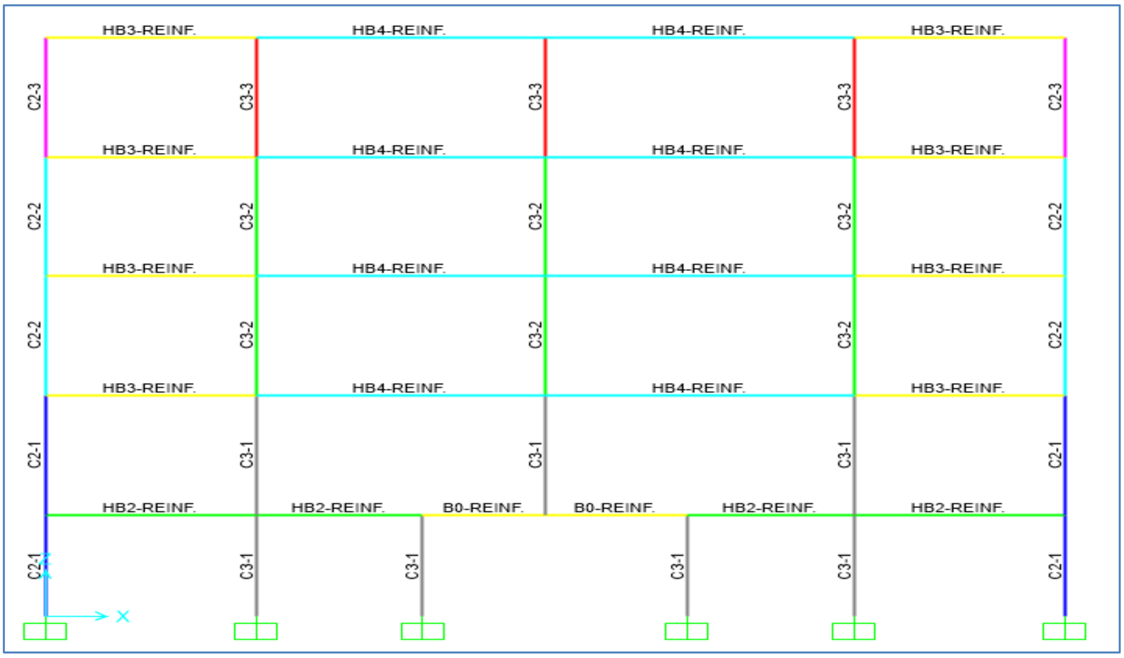

Fig-17: 2D Residential Building Frame

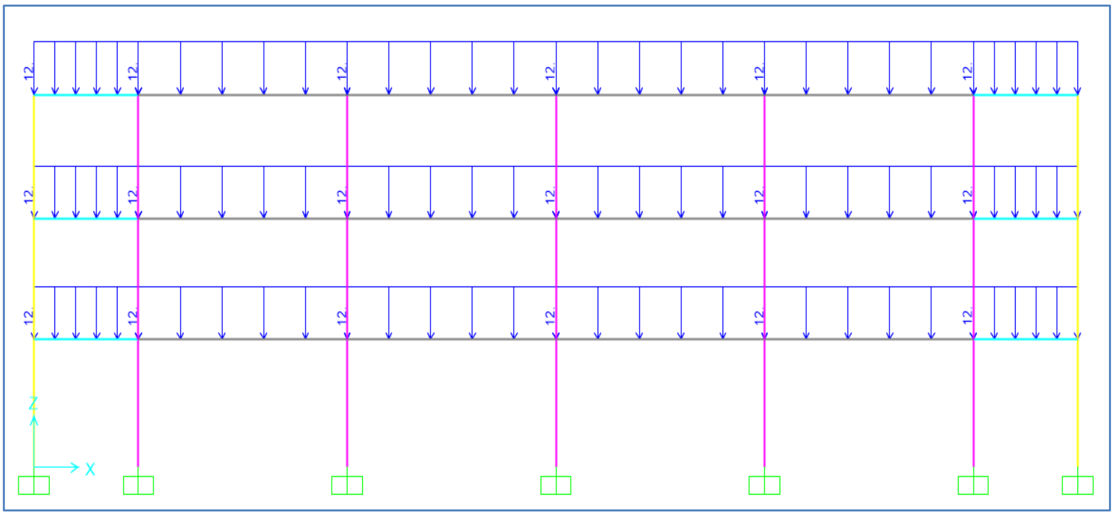

Fig-18: Loading on School Building Frame

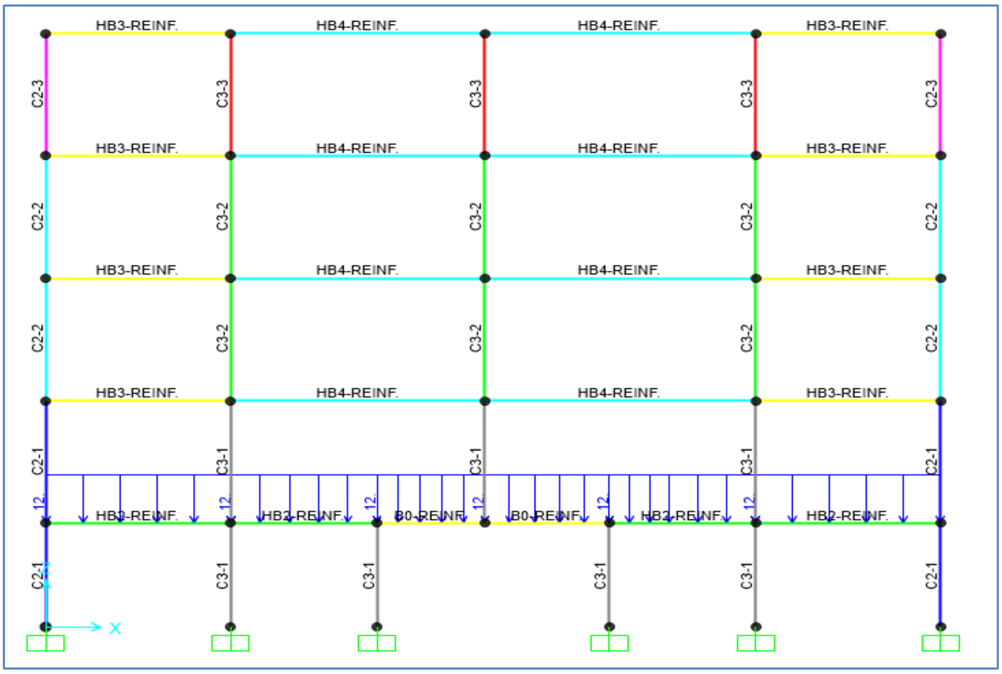

Fig-19: Loading on Residential Building frame

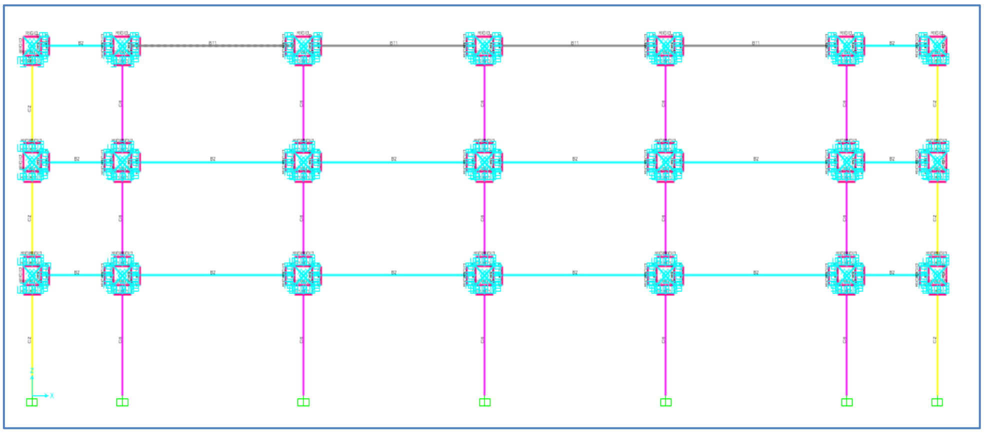

Fig-20: 2D School Building frame using Macro Node

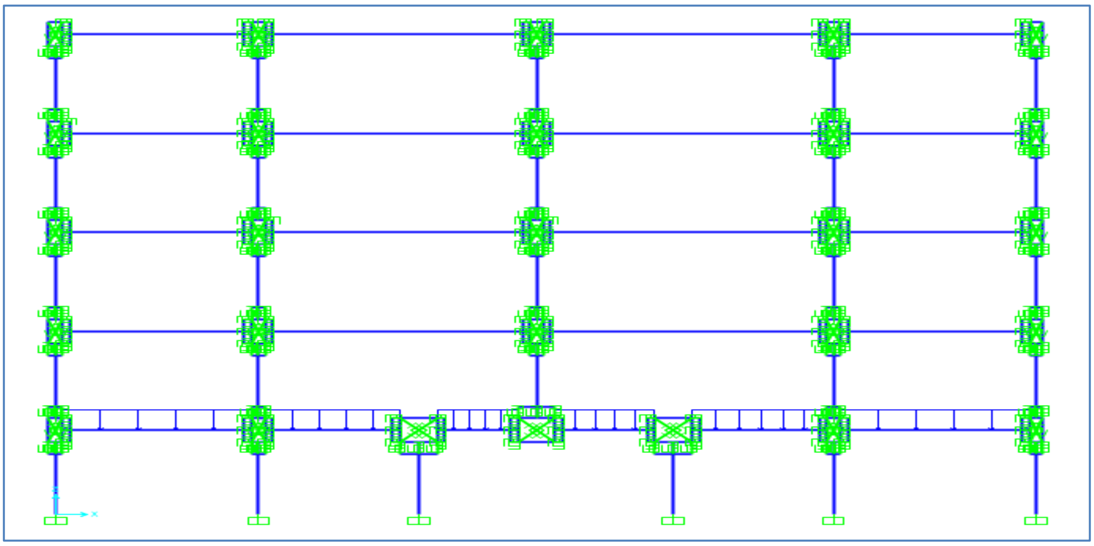

Fig-21: 2D Residential Building frame using Macro Node

6. DISCUSSION OF RESULTS

From the pushover analysis results, it can be clearly seen that both building frames in which the beam-column joints were modeled using macro node element, tends to have lower maximum base shear values 574 kN (school) 329.3 kN (residential) and higher displacement capacity under later loading when compared to maximum base shear values 1546 kN (school) and 466 kN (residential) of building frames modelled without macro node. In both school and residential building frames, the status of the hinges developed in frames without macro node element are within the Immediate occupancy (IO) performance level except for few plastic hinges in beams in residential building frame which are in Collapse prevention level. However, when the two buildings frames were modeled using macro node element, it can be seen that plastic hinges status developed in columns are beyond Collapse Prevention (CP) level highlighting overall failure of the structure.

A) School Building Frame

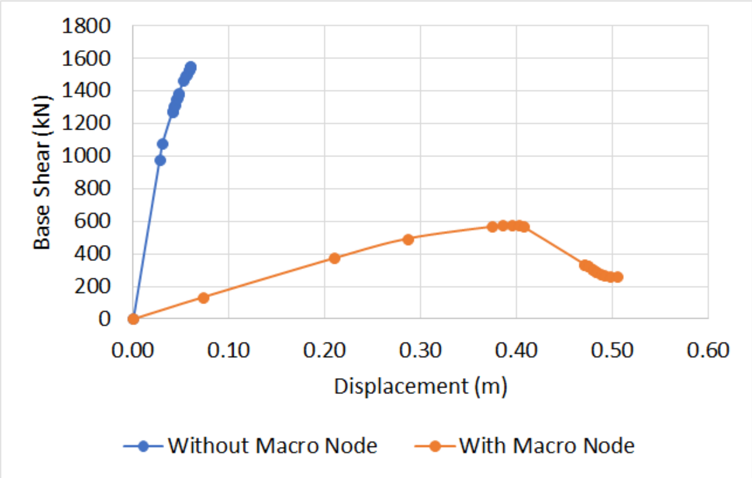

Fig-11: Pushover Curves of School building frame

Table-10: Pushover analysis results of School Building Frame

With Macro Node	
Base shear (kN)	**Displacement (m)**
0.0	0.00
134.3	0.07
373.5	0.21
493.1	0.29
566.8	0.37
572.7	0.39
574.6	0.40
572.5	0.40
565.9	0.41
335.0	0.47
335.0	0.47
324.3	0.47
324.3	0.47
302.0	0.48
302.0	0.48
291.0	0.48
291.0	0.48
272.6	0.49
265.0	0.49
260.2	0.50
258.3	0.51
259.6	0.51
300.0	0.61
363.5	0.80
376.1	0.83

Without Macro Node	
Base shear (kN)	**Displacement (m)**
0.0	0.00
978.7	0.03
1076.3	0.03
1271.0	0.04
1276.1	0.04
1307.7	0.04
1314.8	0.04
1349.8	0.05
1356.8	0.05
1378.3	0.05
1385.1	0.05
1461.8	0.05
1493.8	0.05
1495.9	0.06
1527.1	0.06
1529.2	0.06
1544.7	0.06
1546.3	0.06
1546.3	0.06

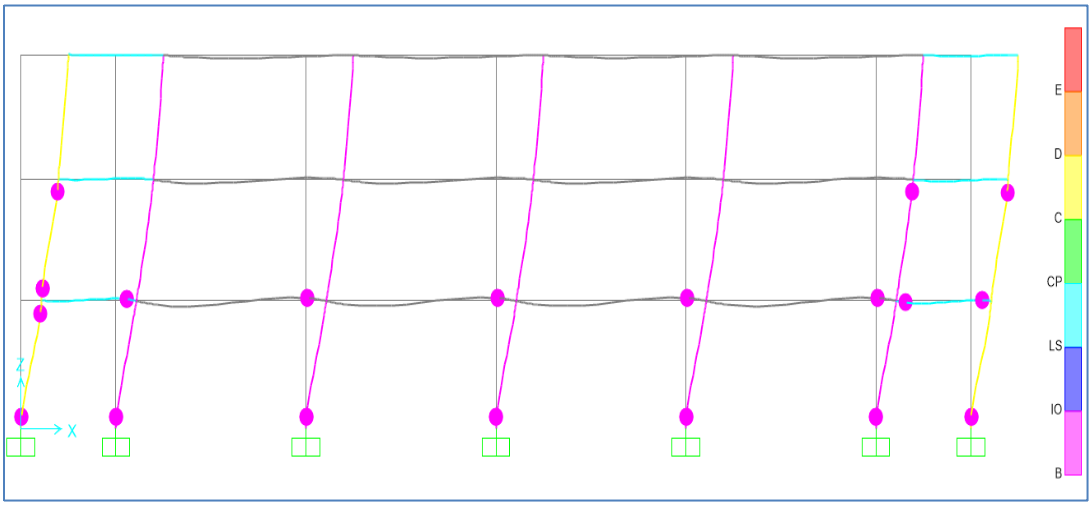

Fig-12: Hinges status in School Building frame WITHOUT Macro node

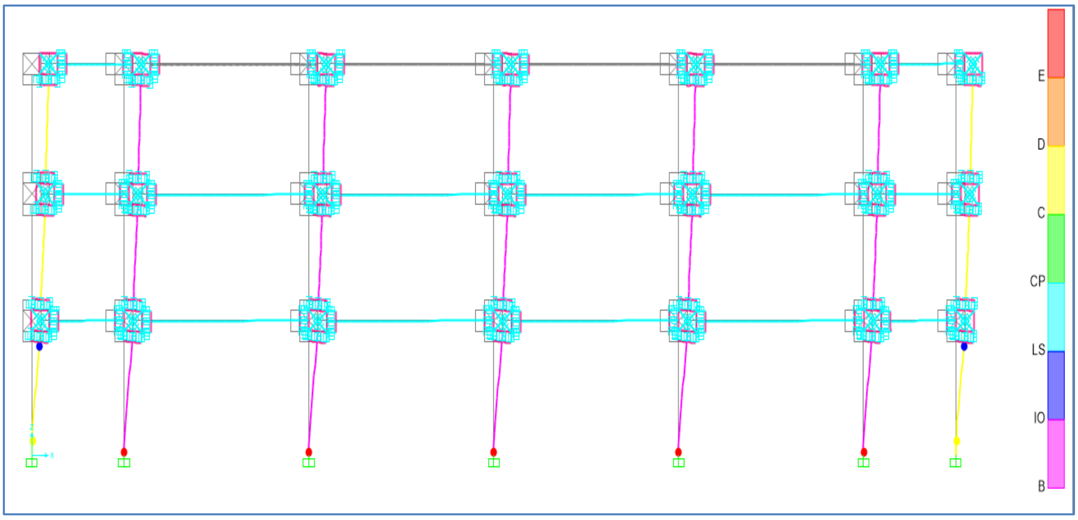

Fig-13: Hinges status in School Building frame WITH Macro node

Table-11: Pushover analysis results of Residential Building Frame

Without Macro Node	
Base shear (kN)	**Displacement (m)**
0	0.00
234	0.06
239	0.06
397	0.11
419	0.12
425	0.13
431	0.14
452	0.21
466	0.30
466	0.30
466	0.30
466	0.30
371	0.29

B) Residential Building Frame

With Macro Node	
Base shear (kN)	**Displacement (m)**
0.0	0.01
61.8	0.11
83.6	0.14
143.9	0.24
204.2	0.34
292.3	0.54
329.3	0.64

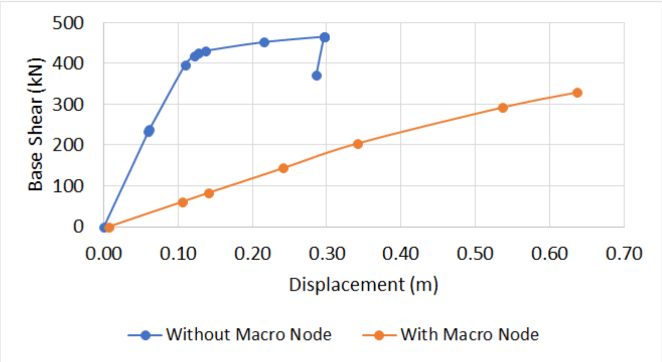

Fig-14: Pushover curves of Residential Building frame

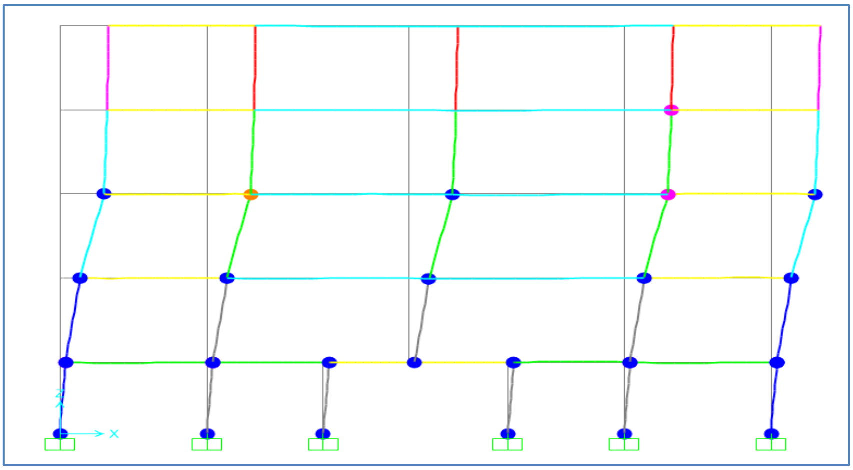

Fig-15: Hinge status in Residential Building frame without macro node

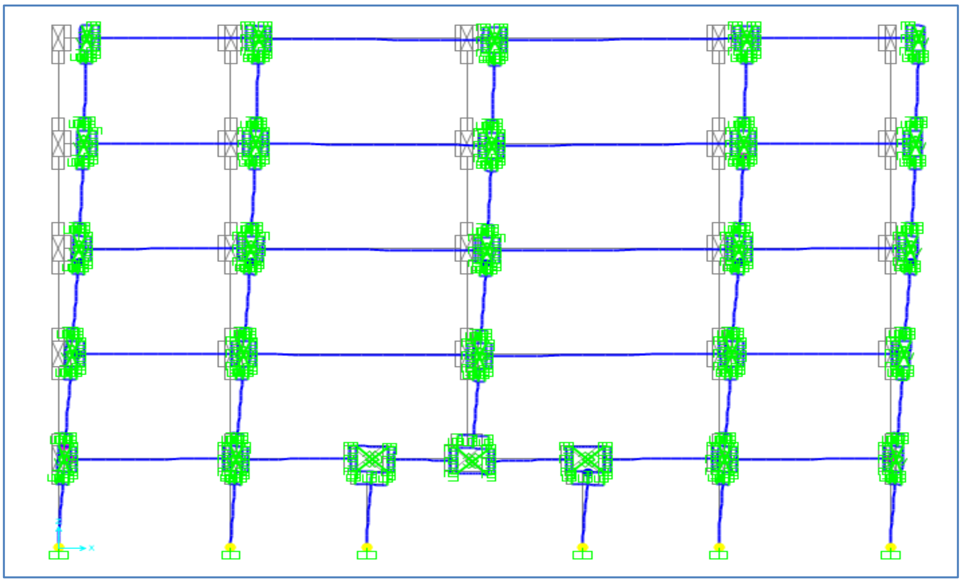

Fig-16: Hinge status in Residential Building frame with macro node

7. CONCLUSION

In this study, nonlinear static pushover analysis was performed on 2D RC frame of existing buildings in Jeddah city by using macro node element during modeling of the beam-column joint and pushover curves were compared. The two buildings (school and residential) considered in this study come under low to mid rise buildings (2-7 storeys) category. SAP2000 structural analysis software was used to model and run the pushover analysis. The beam-column joint modelling approach adopted in this study is a model with macro node element which reported for shear failure of the joint, failure due to crushing of concrete, flexural and/or shear plastic damage of the beams or columns connected and bond-slip failure. From the pushover analysis results, it can be concluded that both buildings frames in which the beam-column joints were modeled using macro node element has lower base shear values and higher displacement capacity when compared to building frames modelled without using macro node. This is because the macro node modeling of beam column joint increases the joint flexibility locally, thereby decreasing the lateral resistance capacity of the structure in overall. With respect to the status of the hinges developed in the RC frames without macro node element it was within Immediate occupancy (IO) performance level, but when the two buildings frames were modeled using macro node element, the status changed drastically to Collapse prevention (CP) performance level highlighting overall failure of the structure. It can be observed that there is substantial difference in the overall performance level category of the structure due of beam column joint modeling method. When macro node is included during modeling of the beam column joint of a structure, it leads to conservative results in comparison to modeling without using macro node. This indicates that implementing macro node element in beam-column joints during structural modeling of buildings is very crucial in order to estimate the accurate nonlinear behavior of the structure.

8. REFERENCES

1. Walker, S., Lehman, D. E., Stanton, J., Anderson, M., & Alire, D. (2004). Seismic performance of older beam-column joints. *Proc. 13th World Conf. Earthquake Engineering*, *1464*, 1464.
2. Alguhane, T. M., Khalil, A. H., Fayed, M. N., & Ismail, A. M. (2015). Seismic Assessment of Old Existing RC Buildings on Madinah with Masonry Infilled Using Ambient Vibration Measurements. 9(1), 43–51.
3. Ismaeil, M., Elhadi, K., Alashker, Y., & Yousef, I. E. (2017). Seismic Analysis and Design of a Multi-Storey Building Located in Haql City, KSA. *Soft Computing in Civil Engineering*, *1*(2), 35–51
4. Alashker, Y., Elhady, K., & Ismaeil, M. (2019). Effect of Earthquake loads on School Buildings in the Kingdom of Saudi Arabia. *Civil Engineering Journal*, *5*(1), 142. https://doi.org/10.28991/cej-2019-03091232
5. Ismaeil, M. (2019). Evaluation and Strengthening of an Eight-Storey Existing Reinforced Concrete Building in Abha City, KSA. Advancements in Civil Engineering & Technology, *2*(5), 245–262. https://doi.org/10.31031/acet.2019.02.000550
6. Hassaballa, A. E., Adam, F. M., & Ismaeil, M. A. (2017). Seismic Analysis of a Ten-Storey Reinforced Concrete Building in Jazan Area, KSA. *Open Journal of Civil Engineering*, *07*(02), 252–266. https://doi.org/10.4236/ojce.2017.72016
7. El-wafa, A. (2013). Evaluation of the Seismic Performance of RC Buildings in KSA According to the Seismic Requirements of the Saudi Building Code and Suggesting the Retrofit Systems.
8. Rahman, M. K., Ajmal, M., & Baluch, M. H. (2012). Nonlinear Static Pushover Analysis of an Eight Story RC Frame-Shear Wall Building in

Saudi Arabia. 15th World Conference on Earthquake Engineering, Lisbon Portugal, September, 1–7. https://doi.org/10.13140/RG.2.1.4641.7762

9. Suwondo, R., & Alama, S. (2020b). Seismic evaluation of reinforced concrete moment resisting frames using pushover analysis. IOP Conference Series: Earth and *Environmental Science*, *426*(1). https://doi.org/10.1088/1755-1315/426/1/012048
10. Alashker, Y., Nazar, S., & Ismaiel, M. (2015). Effects of Building Configuration on Seismic Performance of RC Buildings by Pushover Analysis. *Open Journal of Civil Engineering*, *05*(02), 203–213. https://doi.org/10.4236/ojce.2015.52020
11. Ibrahim, Y. E. (2018). Seismic risk analysis of multistory reinforced concrete structures in Saudi Arabia. Case Studies in Construction Materials, 9, 1–9. https://doi.org/10.1016/j.cscm.2018.e00192
12. Hakim, R. A., Alama, M. S. A., & Ashour, S. A. (2014). Seismic Assessment of an RC Building Using Pushover Analysis. *Engineering, Technology & Applied Science Research*, *4*(3), 631–635. https://doi.org/10.48084/etasr.428
13. Hakim, R. A., Alama, M. S., & Ashour, S. A. (2014). Seismic Assessment of RC Building According to ATC 40, FEMA 356 and FEMA 440. *Arabian Journal for Science and Engineering*, *39*(11), 7691–7699. https://doi.org/10.1007/s13369-014-1395-x
14. Hakim, R. A. (2014). Seismic Assessment of R.C.C Frame Building using Pushover Analysis. *Engineering Technology & Applied Science Research*, *4*(3), 630–634. https://doi.org/10.35940/ijrte.c4523.099320
15. Abdulelah. (2018). GIS based seismic risk assessment of buildings in Jeddah, Masters Thesis, King AbdulAziz University.
16. Alluqmani, A. E. (2020). Effectiveness of Different Techniques on the Seismic Retrofitting of Existing RC Framed Buildings in Saudi Arabia. *Saudi Journal of Civil Engineering*, *4*(7), 113–118. https://doi.org/10.36348/sjce.2020.v04i07.002
17. Lowes, L.N., Altoontash, A. (2003). Modeling reinforced- concrete beam-column joints subjected to cyclic loading. *Journal of Structural Engineering*, *129*, 1686–1697.
18. Nilanjan Mitra, L. N. L. (2004). Evaluation and Adavancement of a Reinforced Concrete Beam-Column Joint Model. *13 Th World Conference on Earthquake Engineering*, *185*.
19. Panto, B. (2017). A new discrete macro-node (DMN) for the nonlinear modelling of the reinforced concrete beam / column joints. *Pistoia Toscana*, *Capitale Italiana Cultura*, *September*.
20. SAP2000. (2005). *Integrated Finite Element Analysis and Design of Structures*. Computers and Structures (CSI),Berkeley, California, USA.
21. Anderson, J.C., & Townsend, W.H. (1977). “Models for RC frames with degrading stiffness”, ASCE V.103, ST12, pp 2361-2376
22. Fillipou, F.C., Popov, E.P., & Bertero, V.V.(1983). “Effects of bond deterioration on hysteretic behaviour of reinforced concrete joints” EERC Report No. UCB/EERC-83/19
23. Alath, S., & Kunnath, S. K.(1995). “Modeling inelastic shear deformations in RC beamcolumn joints,” Engg Mech Proc 10th Conf, ASCE, New York, 2, 822–825.
24. Altoontash, A. (2004). “Simulation and damage models for performance assessment of reinforced concrete beam-column joints,” PhD Dissertation, Department of Civil and Environment Engineering, Stanford University, Stanford, California, 2004.
25. Mander, J.B., Priestley, M.J.N., & Park, R. (1988). Theoretical Stress–Strain Model for Confined Concrete, Journal of the Structural Division ASCE 114, pp. 1804–1826.
26. Saudi Building Code SBC-301. (2007). Loads and Forces Requirements, Saudi Buildings Code National Committee.
27. Saudi Building Code SBC-304. (2007). Structural - Concrete Structures, Saudi Buildings Code National Committee
28. ATC-55. (2001). Evaluation and Improvement of Inelastic Seismic Analysis Procedures, Applied Technology Council, Redwood City, CA
29. ATC-40. (1997). Seismic evaluation and retrofit of concrete buildings- Volume1, *Applied Technology Council*: Redwood City, CA.
30. FEMA 356. (2000). Pre-standard and Commentary For The Seismic Rehabilitation of Buildings, ASCE, Reston, Virginia
31. FEMA 440. (2005). NEHRP Improvement of Nonlinear Static Seismic Analysis Procedures, Department of Homeland Security, and Federal Emergency Management Agency: Redwood City, CA.