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Seismic Performance Assessment of RCS Building By Pushover Analysis

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Abstract: *RCS moment-resisting frame systems, consisting of Reinforced Concrete (RC) columns and Steel (S) beams, take advantage of the inherent stiffness and damping, as well as low-cost of concrete, the lightweight and construction efficiency of structural steel. Past studies have shown these systems to be efficient in both design and construction stages while able to maintain sufficient strength and ductility necessary in seismic applications. Despite this past research, use of this hybrid structural system in the United States has been limited to non- or low-seismic zones. In addition, past studies have acknowledged that there is a fundamental need to test full structural systems, both analytically and experimentally, in order to (1) substantiate the knowledge that has accumulated up to this point and (2) act as a proof of concept for the composite RCS frames. This paper aim to facilitate the greater acceptance and use of composite RCS systems as a viable alternative to conventional lateral resisting systems in comparison with the ordinary RC building.*

Two structures are considered to represent low rise RCS and RC structures for study. Theses consist of two typical steel beam and RC columns frame buildings without shear walls. Three story RCS buildings is designed according to EGP Codes of practice. Design columns under provisions of Egyptian reinforced concrete structures code and beams are designed according to Egyptian steel construction code. The comparative studies for the two buildings are presented.

Keyword: *Seismic assessment, Pushover, RCS system, Moment resisting frame*

I. Introduction

Innovative applications of composite steel and concrete structures provide attractive alternatives to conventional steel or reinforced concrete systems. RCS moment-resisting frame systems, consisting of Reinforced Concrete (RC) columns and Steel (S) beams, take advantage of the inherent stiffness and damping, as well as low-cost of concrete, and the lightweight and construction efficiency of structural steel (Liang et al 2004)

RCS frame systems have shown to possess several advantages from economic and construction viewpoints (Griffis 1986) compared to either RC or steel frame systems. RC columns are approximately 10 times more cost-effective than steel columns in terms of axial strength and stiffness (Sheikh et al 1987). On the other hand, steel floor systems are significantly lighter compared to RC floor systems, leading to substantial reductions in the weight of the building, foundation costs, and inertial forces. From the construction viewpoint, RCS buildings are generally built by first erecting a steel skeleton with light columns, which for medium- or high-rise buildings could be as high as 8 to 10 stories. This steel frame then allows the simultaneous performance of several construction tasks at different floor levels, such as placing of steel deck, pouring of concrete slabs, and encasement of the light steel columns by RC columns (Griffis 1986).

In the past thirty years, RCS moment-resisting frame systems have mostly used for high-rise buildings located in regions of low seismicity. In recent years, research efforts have made to develop seismic design guidelines for RCS systems located in regions of high seismic risk (Liang et al 2004)

Several groups of researchers have developed trial designs of RCS frames based on a common theme building devised for the US-Japan program (Mehanny 2000, Bugeja 1999, Noguchi 1998). These studies apply the proposed seismic design provisions for RCS systems and then evaluate the seismic performance of resulting designs using nonlinear analyses and advanced performance assessment techniques. Traditional steel frames were also investigated in these studies to benchmark the performance of conventional frames compared to the composite RCS frames. Using a common floor plan, the building heights varied as well as the implementation of perimeter versus space frame systems. These design studies have shown that the steel beam sizes tend to be similar for the RCS and steel system and that the main differences lie in the column and connection designs. Given the additional stiffness provided by the RC columns, the RCS frames tended to be controlled more by the minimum strength requirements whereas the steel frames were restricted by lateral drift limitations. In general, these investigations have shown that the inelastic dynamic response of the RCS frames is similar to comparably designed steel moment frames.

The US-Japan program included two reduced-scale RCS moment frames – one at the Osaka Institute of Technology (Baba and Nishirmura 2000) and the second at Chiba University (Noguchi and Uchida 2004). Both are about 1/3-scale two-bay two-story RCS frames with through-beam type connections with differences only in the joint details (one had cover plates and band plates while the other had face bearing plates and band plates). The frame was designed such that the plastic strength of the beams was nearly equal to the ultimate shear strength of the joints, so as to provide information on the interaction between frame and connection response. Both test specimens were subjected to reverse cyclic loading and withstood story drift ratios in excess of 5% without significant strength or stiffness degradation, thus confirming the reliable seismic behavior of RCS framing systems.

Cordova et al 2005 design, and test a full scale 3-story composite RCS moment frame. Using the pseudo-dynamic loading technique, this specimen is subjected to a series of earthquake motions ranging in hazards from frequent to extremely rare events. Using the results of the test specimens and recommendation, trial designs of three case study buildings (3, 6, and 20-stories) are generated, analytically modeled, and subjected to a suite of earthquake ground motions at a range of hazard levels. They Investigate differences between the response of beam-column subassembly and full-scale system testing and evaluate how this affects the interpretations from these tests.

One of the efficient tool of addressing the behaviour of building under earthquake loading is the pushover analysis. Due its simplicity, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. It is widely accepted that, when push over analysis is used carefully it provide useful information that cannot be obtained by linear static or dynamic analysis procedure (mehmet inel et al (2006).

This paper aim to study the seismic performance of the RCS system for buildings in comparison with the ordinary RC buildings.

II. Pushover Analysis

Structures are expected to deform inelastically when subjected to severe earthquakes, so seismic performance evaluation of structures should be conducted considering post-elastic behavior. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behavior can not be determined directly by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behavior.

Pushover analysis is an analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially.

A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the capacity curve (Fig 1).

The pushover analysis is very useful in estimating the following characteristics of a structure. M. Mouzzoun (2013)

1- The capacity of the structure as represented by the base shear versus roof- displacement graph
2- Maximum rotation and ductility of critical members load
3- The distribution of plastic hinges at the ultimate load
4- The distribution of damage in the structure, as expressed in the from of load damage indices, at the ultimate load
5- Determination of the yield lateral resistance of the structure
6- Estimates of inter-story drifts and its distribution along the height
7- Determination of force demands on members, such as axial force demands on columns, moment demands on beam-column connections
8- To assess the structural performance of existing or retrofitted buildings.

III. Seismic Perforamnace Assessment Of Buildings

The seismic performance of buildings is measured by the state of damage under a certain level of seismic hazard. The state of damage is quantified by the drift of the roof and the displacement of the structural elements. Initially, gravity push is carried out using force control method. It is followed by lateral push with displacement control using commercial programs as SAP2000.

For carrying out displacement based pushover analysis, target displacement need to be defined. Pushover analysis gives an insight into the maximum base shear that the structure is capable of resisting. A building performance level is a combination of the performance levels of the structure and the nonstructural components. A performance level describes a limiting damage condition, which may be considered satisfactory for a given building with specific ground motion. The performance of the structure is determined by hinges formation. Various types of plastic hinges: uncoupled/coupled moment, torsion, axial force and shear hinges are available. After yielding, plastic hinges will form at different location indicating the risk of occupant as shown in the (Fig. 2 and Fig. 3). The performance point is calculated from the guideline defined in FEMA-356 and ATC-40. The lateral force is applied at the deformed state of the general loading from point A (Fig. 2). No hinges will formed before point B where structure will shows linear behavior and after that one or more hinges will start to form. Software will shows hinges with following remarkable indication:

Immediate occupancy IO: damage is relatively limited; the structure retains a significant portion of its original stiffness and most if not all of its strength.

Life safety level LS: substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Collapse prevention CP: at this level the building has experienced extreme damage, if laterally deformed beyond this point; the structure can experience instability and collapse base shear.

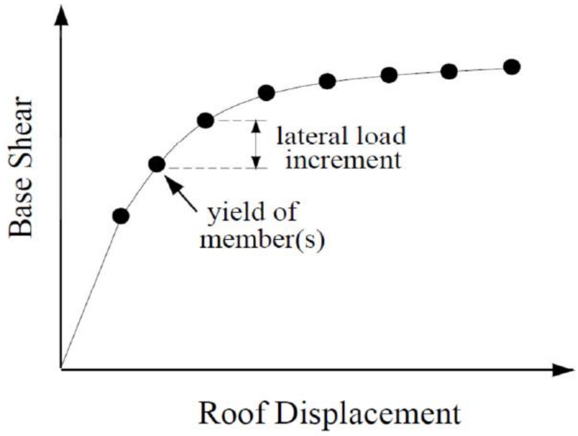

Fig.1 Expected Capacity Curve of the frame element

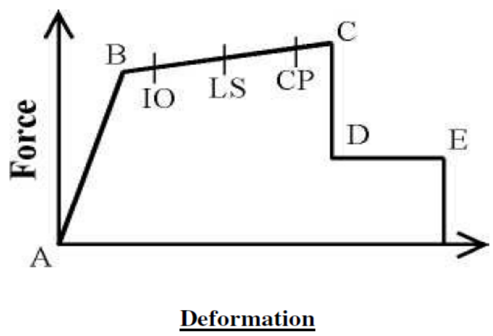

Fig.2 Risk indicator curve

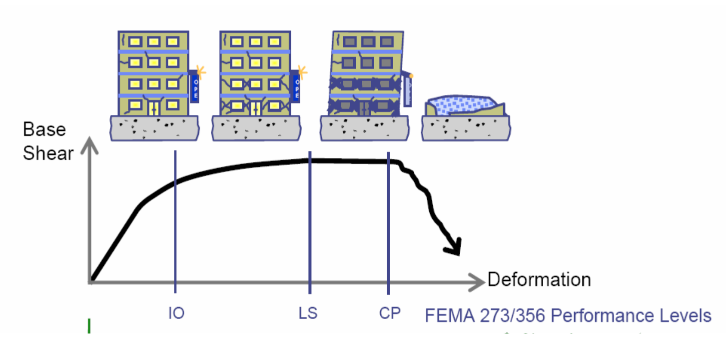

Fig.3 FEMA 273/356 Performance levels (taken from Fajfar et al. 2004)

IV. Description Of Studied Structures

Two structures without shear walls are considered to represent low-and medium rise RCS and RC structures to study. Theses consist of a typical steel beam and RC columns frame building Three story RCS buildings are designed according to EGP Codes of practice. Design columns under provisions of Egyptian reinforced concrete structures code and beams are designed according to Egyptian steel construction code.

Material properties are assumed to be 25 Mpa for the concrete compressive strength and 360 Mpa for the yield strength of reinforcement steel. For steel beams steel 52 is used with yield strength of 360 Mpa

Both building have 3 bays with 4.8 m span in both direction , story height is assumed to be 3.0 m. The interior frame represents 2-D models of these buildings The columns dimensions in this study are considered constant for each three story .

The three story building is assumed to be 9.0 m in elevation . Column dimensions are kept constant and chosen to be 40*40 cm reinforced by 8Φ 16mm as longitudinal reinforcement , Steel beams are considered as BFI 220 section

For RC building, beams are considered of section 25*60 cm reinforced by 6 Φ 16 mm as main reinforcement and 2Φ16 mm as secondary reinforcement at middle columns and with 2Φ16 mm as top and bottom reinforcement at outer columns.

V. Building Performance

The lateral load pattern in Cairo City corresponding to the Egyptian Loading Code (ECP201-2012) is adopted and applied as auto lateral load pattern in SAP 2000. The load pattern is calculated using DL+SDL+0.25LL for the EQ load case. The direction of monitoring the behavior of the building is same as the push direction. In case of columns, program defined auto PM2M3 interacting hinges are provided at both the ends according to FEMA 356, while in case of beams, M3 auto hinges are provided.

In this study, displacement-controlled pushover analyses were performed on three models for RC, RCS buildings with three and six floors. The displacement-controlled pushover analysis is basically composed of the following steps:

1- A three dimensional model that represents the overall structural behavior is created.
2- Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.
3- A predefined lateral load pattern representing EQ load pattern is then applied.
4- Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.
5- Base shear and roof displacement are recorded at first yielding.
6- The structural model is modified to account for the reduced stiffness of yielded member(s).
7- A new lateral load increment is applied to the modified structural model such that additional member(s) yield. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.
8- Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.
9- Steps 6, 7 and 8 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.
10- The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure.

Both RC. and RCS, buildings were analyzed using SAP2000 program. Base columns are assumed hinged at the foundation level. The beams and columns are modeled as nonlinear frame elements with lumped plasticity , hinges are defined according to the section properties at both ends at the columns and beams

The push over curve for RCS building are shown in Fig. 4 and for RC building in Fig 5. The push over curves with each associated response spectrum curves for different levels of shaking levels are shown in Fig 6 for RCS structures and in Fig 7 for RC structure . The hinge patterns are shown in Fig 8 for RC structure and in Fig 9 for RCS structure.

In RC building plastic hinges formation starts with beam ends then propagates to the beams of the second story. After that point intermediate base columns of lower stories, then propagates to the intermediate columns of the second story the plastic hinges performed at outer columns of the lower story and continue with yielding of interior columns in the upper stories until failure occurs.

In RCS building plastic hinges formation starts with intermediate columns of lower story , then propagates to interior columns in the upper stories and the intermediate columns of the lower story reaching collapse before the outer columns , then a failure mechanism occur as soft story of the lower story.

VI. Summary And Conclusions

A commercial nonlinear finite element computer program (SAP2000) was used to investigate the static nonlinear behavior (pushover analysis) of (RCS) structures for lateral seismic loads. Two buildings are modeled to represent low buildings. A Comparison with ordinary RC buildings are presented. the results shows that for even both structures have almost the base shear capacity, the RCS structures behave linearly till the maximum shear base capacity is reached , and soft story failure mechanism occurs.

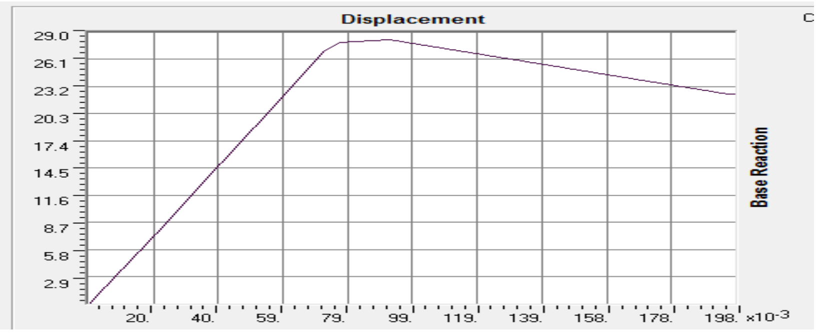

Fig. 4 displacement vs base shear for RCS structure

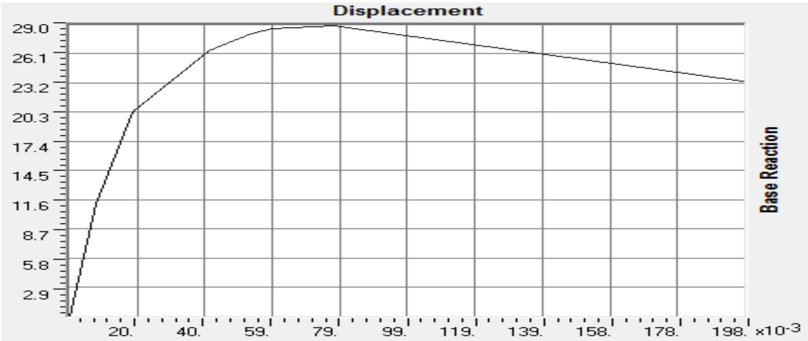

Fig. 5 displacement vs base shear for RC structure

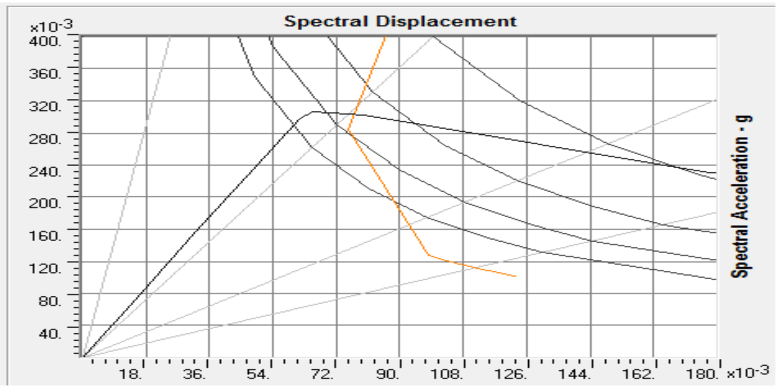

Fig. 6 pushover and demand spectrum for RCS building

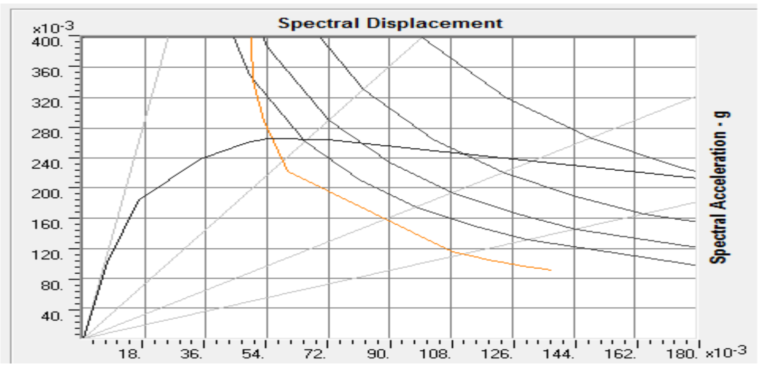

Fig.7 Pushover and demand spectrum for RC building

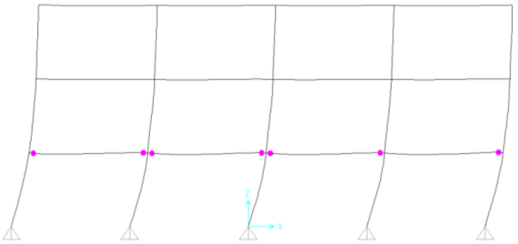

Fig8-a Plastic hinges in RC building starts at beams of lower floor

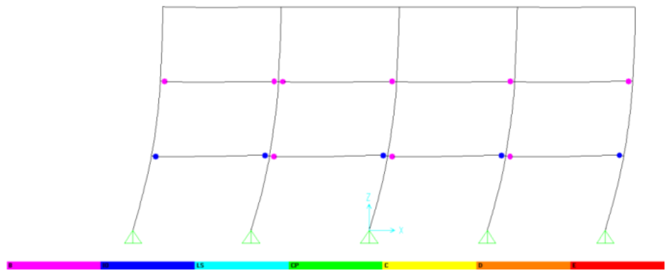

Fig8-b Plastic hinges in RC building propagates to the at beams upper story

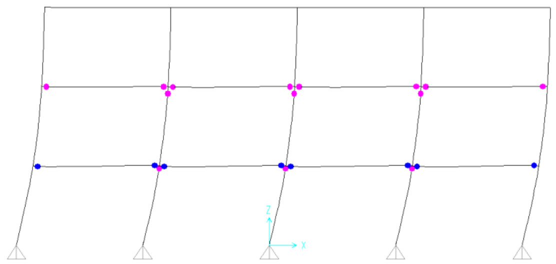

Fig8-c Plastic hinges in RC building propagates to the intermediate column

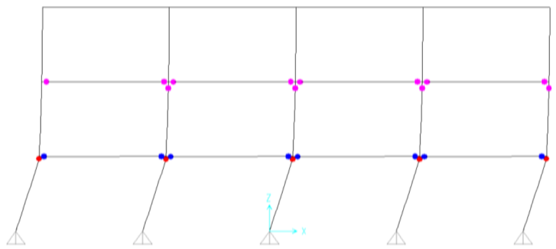

Fig8-d Plastic hinges in RC building propagates to the external columns
Fig. 8 Hinge pattern for RC building

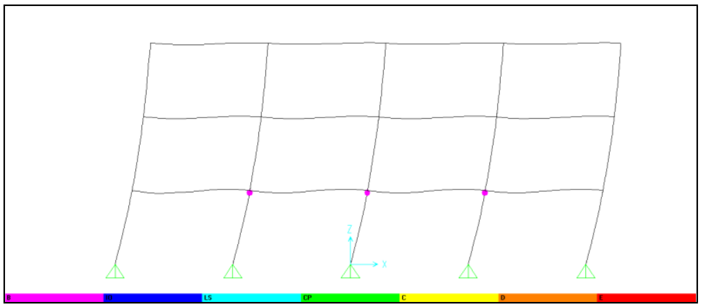

Fig9-a Plastic hinges in RCS building starts at intermediate columns of the lower story

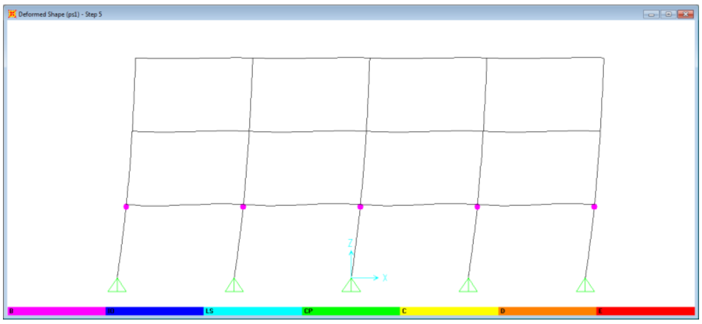

Fig9-b Plastic hinges in RCS building propgate to the outer columns of the lower story

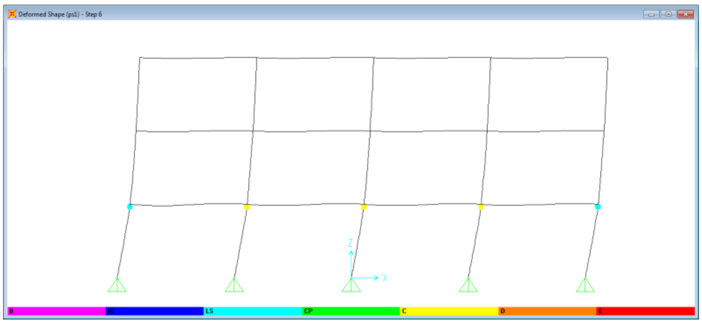

Fig9-c plastic hinges in RCS building at failure
Fig. 9 hinge pattern for RCS building

Refrences

[1]. Liang Xuemei, Gustavo J. and James K. Wight , "Seismic behavior of RCS beam-column-slab subassemblies designed following a connection deformation-based capacity design approach" , 13th World Conference on Earthquake Engineering,Vancouver, B.C., Canada, August 1-6, 2004.

[2]. Griffis, L.G. (1986). "Some Design Considerations for Composite-Frame Structures," AISC Engineering Journal, Second Quarter, pp. 59-64.

[3]. Sheikh, T.M., Yura, J.A., and Jirsa, J.O. (1987). "Moment Connections between SteelBeams and Concrete Columns," PMFSEL Report No. 87-4, University of Texas at Austin, Texas.

[4]. Mehmet Inel , Hayri Baytan Qzmen, " Effect of plastic hinge properties in nonlinear analysis of reinforced concrete building," Engineering Structures Journal (2006), pp. 1494-1502

[5]. Mehanny, S.S., Cordova, P.P., and Deierlein, G.G. (2000). Seismic Design of Composite Moment Frame Buildings – Case Studies and Code Implications, Composite Construction IV, ASCE, in press.

[6]. Bugeja, M., Bracci, J.M., and Moore, W.P. (1999). "Seismic Behavior of Composite Moment Resisting Frame Systems," Technical Report CBDC-99-01, Dept. of Civil Engrg., Texas A & M University.

[7]. Noguchi, H. and Kim, K. (1998). "Shear Strength of Beam-to-Column Connections in RCS System," Proceedings of Structural Engineers World Congress, Paper No.T177-3, Elsevier Science, Ltd.

[8]. Baba, N., Nishimura, Y. (2000). "Seismic Behavior of RC Column to S Beam Moment Frames," Proc.12WCEE.

[9]. Noguchi, H. and Uchida, K. (2004). "Finite Element Method Analysis of Hybrid Structural Frames with Reinforced concrete Columns and Steel Beams." Journal of Structural Engineering, 130(2), 328-335.

[10]. Cordova, P.P. and Deierlein, G.G. (2005), Validation of the Seismic Performance of Composite RCS Frames: Full-Scale Testing, Analytical Modelling, and Seismic Design, Report No. 155 the John A. Blume Earthquake Engineering Center, Stanford University.

[11]. M.Mouzzoun, O.Moustachi, A.Taleb, S.Jalal "Seismic performance assessment of reinforced concrete buildings using pushover analysis" IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE). ISSN: 2278-1684 Volume 5, Issue 1 (Jan. - Feb. 2013), PP 44-49

[12]. FEMA273 "Federal Emergency Management Agency", recommended Provisions for Seismic Regulations for New Buildings and Other Structures

[13]. ATC 40 "Applied Technology Council, Seismic Evaluation and Retrofit of Concrete Buildings", Volume 1 Report, , Redwood City, California, 1996

[14]. Vision 2000 committee "performance based seismic engineering of buildings", structural Engineers Association of California (SEAOC), California.

[15]. Fajfar P., Krawinkler H. (2004), 'Performance-Based Seismic Design Concepts and Implementation", -Proceedings of the International Workshop Bled, Slovenia, June 28 - July 1, 2004. College of Engineering, University of California, Berkeley.

[16]. The Egyptian Loading Code, ECP-201 (2012), p 108-158.