# PERFORMANCE BASED DESIGN OF RCS FRAMES

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**ABSTRACT:**
To facilitate the analysis, design and construction process of RCS frames (Reinforced Concrete columns and Steel beams) with eliminated embedded steel erection column in low and moderate rise buildings, this paper introduces a type of rigid beam to column connection and discusses its moment curvature behavior obtained from finite element analysis using ABAQUS software. The next step was to develop an estimation of acceptance criteria required for performance based seismic design of such frames by using nonlinear static analyses results. At the end, Ant Colony Optimization (ACO) theory was employed to accomplish the performance based design process, utilizing a MATLAB program for optimization and OpenSEES (Open System for Earthquake Engineering Simulation) software capabilities in structural analysis.

**KEYWORDS:** RCS Frames, Connection, Performance Based Design, Ant Colony Optimization

## 1. INTRODUCTION

Throughout the last century, reinforced concrete structures and steel structures have been two main categories for building constructions around the world. In the past few decades, this has called for numerous researches [1 to 5] for efficient consumption of concrete and steel which introduced RCS frames to the engineering society as the best choice for high rise building constructions. RCS frame systems typically consist of reinforced concrete columns, which usually have a small embedded I-shape for erection purposes. On the other hand, performance based design, mainly explained in FEMA (Federal Emergency Management Agency) documents including FEMA-273 [8] and optimization algorithms like ACO [9 to 11] were broadly used for accurate modeling and economical design of structures. Our goal is to use RCS frames with eliminated steel erection column for low to moderate rise buildings in zones of high seismic risk. This paper is considered as the first step in this process.

## 2. BEAM TO COLUMN CONNECTION

In the past few years many types of Beam-Column connections were investigated for RCS frames. Our aim is to propose another connection using positive aspects and preventing drawbacks of the previous models as much as possible and simultaneously reducing construction difficulties. In order to exploit this connection in practice, column reinforcement placement, formwork and concrete placement are accomplished respectively in construction process as usual. Afterward, a pre-fabricated steel part which is completely depicted in figure 1 for the main direction consisting of I-sections, cruciform shear connectors and end plates, fully welded together should be placed on the column capital while ‘recently-poured’ concrete is still fresh. Steel beams will be bolted to connection parts after the column is fully cured, using end plate connections. Column capitals can facilitate erection sequences and increase seismic performances significantly.

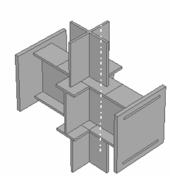

Figure 1 Connection shop built steel part

Due to one-dimensional analysis only a small part of I-sections without end plates in the perpendicular direction is shown in figure 1. Since connection rigidity can affect structural performances, a finite element model of the completed connection, shown in Figure 2, was constructed using ABAQUS software for further investigations. Steel yield-stress and concrete compressive strength were assumed 400 and 30 MPa respectively. Web height and flange width of I-sections were 300 and 150 mm respectively. 300 mm long cruciform shear connectors and 400*400*25 mm end plates were assumed. All steel parts except end plates were modeled with 15 mm thick, plates.

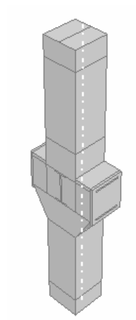

Figure 2 Completed connection model via ABAQUS for the main direction

Half of the top and bottom columns (each 1500 mm long) with hinged ends and 400*400 mm square sections made up of confined reinforced concrete with 2.4% longitudinal reinforcement were constructed while hinged boundary regions were assumed to be a virtual rigid material to prevent stress concentration and early total failure. In addition, it should be mentioned that 10% of total column axial compressive capacity has been applied to the connection, vertically. Incremental loading as counterclockwise couples, caused by compressive and tensile stresses have influenced the 300*25 mm rectangular areas on the end plates. Moment-curvature relationship of the connection as one of the most important results of the analysis is illustrated in figure 3.

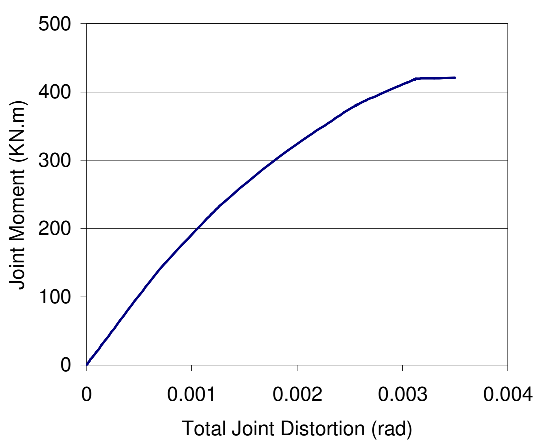


Figure 3 Moment-curvature relationship of the assumed RCS beam-column connection

Although it seems that many factors including part dimensions, column axial load, percentage of longitudinal reinforcement, material strength etc can affect the above diagram, but further investigations [5] demonstrated that connection sensitivity to parameter variations is not significant. In other words, figure 3 can conservatively be used as moment curvature relationship for all well designed RCS beam to column connections with discussed characteristics. The interesting point is that the mentioned connection can be regarded as a rigid RCS connection in finite element frame analysis.

## 3. PRELIMINARY TOOLS FOR PERFORMANCE BASED DESIGN OF RCS FRAMES

While performance based design of structures discussed in FEMA documents is mainly concerned about rehabilitation of existing buildings, its generality brought about many researches in this field for designing new buildings during the past few years. Most of the academic activities in this area were devoted to ordinary steel and reinforced concrete systems and the best tools for such investigations were laboratory tests. In this paper we are going to prepare theoretical necessities for future practical expensive tests of RCS frames; therefore, some logical assumptions were accepted.

First of all, earthquake spectrum shown in figure 4 and accepted in Iranian seismic building code was used as Basic Safety Earthquake 1 (BSE-1) or earthquake having 474 (typically known as 500) years of return period. In addition, Life Safety Performance Level (LS) for BSE-1 demands was selected as Safety Objective.

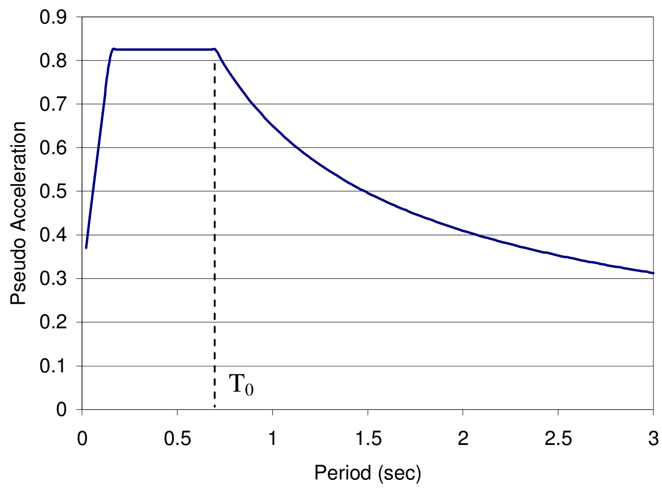


Figure 4 Assumed spectrum for Basic Safety Earthquake 1

It seems necessary to briefly describe coefficient method introduced in FEMA-273 for performance based design of structures.

### *3.1. Linear Static Procedure (LSP)*

Linear static procedure as the easiest way of structural analysis requires calculation of pseudo lateral load (V) which should be evaluated using Eqn. 3.1.

$$V = C_1 C_2 C_3 S_a W \tag{3.1}$$

This force when distributed over the height of the linearly-elastic model of the structure is intended to produce lateral displacements approximately equal to those that are expected in the real structure during the design event. Since it is anticipated that the actual structure will yield during the design event, the force given by Eqn. 3.1 may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in section 4 are developed to take this aspect into account. $C_1$ is a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. This factor can be assumed 1.5 for $T<0.1$ second and 1 for $T>T_0$ second. Linear interpolation shall be used to calculate $C_1$ for intermediate values of T, where T is the fundamental period of the building in the direction under consideration estimated by eigenvalue (dynamic) analysis of the mathematical model of the building and $T_0$ is as shown in figure 4. $C_2$ represents the effects of stiffness degradation and strength deterioration on maximum displacement response which can be assumed equal to 1.3 for $T=0.1$ second and 1.1 in case $T>T_0$. $C_3$ is a modification factor representing increased displacements due to dynamic P-$\Delta$ effects. For values of stability coefficient $\theta$ calculated by Eqn. 3.2 less than 0.1, $C_3$ may be set equal to 1. For values of $\theta$ greater than 0.1, $C_3$ shall be calculated as $1+5(\theta-0.1)/T$. The maximum value of $\theta$ for all stories in the building shall be used to calculate $C_3$. At each story, the quantity $\theta_i$ shall be calculated for each direction of response, as follows:

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \tag{3.2}$$

In the above equation $P_i$ is portion of the total weight of the structure including dead, permanent live and 25% of transient live loads acting on the columns within story level i. $V_i$ is the total calculated lateral shear force in the direction under consideration at story i due to earthquake response assuming that the structure remains elastic. $h_i$ in this equation is height of story i, which may be taken as the distance between the centerline of floor framing at each of the levels above an below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference. Finally, $\delta_i$ is lateral drift in story I in the direction under consideration at its center of rigidity, using the same units as for measuring $h_i$.

In Eqn. 3.1, $S_a$ is response spectrum acceleration at the fundamental period and damping ratio of the building in the direction under consideration. The value of $S_a$ in this paper was obtained from figure 4. W in that equation is total dead load and anticipated live load as indicated here: (a) In storage and warehouse occupancies, a minimum of 25% of floor live load. (b) The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater. (c) The applicable snow load. (d) The total weight of permanent equipment and furnishings.

The lateral load $F_x$ applied at any floor level x shall be determined from the following equations in which $C_{vx}$ is vertical distribution factor, V pseudo lateral load from Eqn. 3.1, $h_i$ and $h_x$ height from the base to floor level i and x respectively in any building with n stories.

$$F_x = C_{vx} V \tag{3.3}$$

$$C_{vx} = \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}} \tag{3.4}$$

In Eqn. 3.4, k can be set equal to 1 for T<0.5 second and 2 for T>2.5 second with permitted linear interpolation for intermediate values, $w_i$ and $w_x$ are portion of the total building weight W located on or assigned to floor level i and x respectively.

### *3.2. Nonlinear Static Procedure (NSP)*

In case of nonlinear static analysis, it is essential to perform a preliminary nonlinear analysis in order to calculate effective fundamental period $T_e$ of that structure using $T_i$ as elastic fundamental period (in seconds) calculated by elastic dynamic analysis and information obtained from a graph like figure 5 for the specified structure and Eqn. 3.5.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{3.5}$$

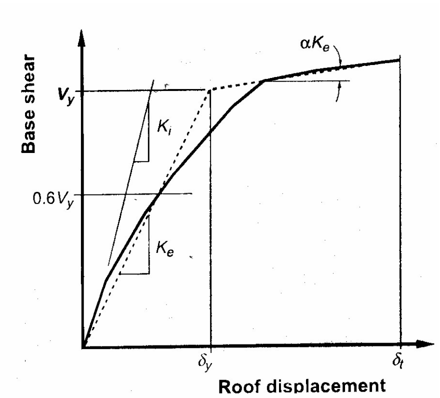


Figure 5 Calculation of effective stiffness [7]

In the above equation, $K_i$ and $K_e$ are elastic and effective lateral stiffness of the building in the direction under consideration respectively which are shown in figure 5 schematically.

Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. In NSP at least two vertical distributions of lateral load shall be considered. The first pattern, often termed the uniform pattern, shall be based on lateral forces that are proportional to the total mass at each floor level. The second pattern, termed the modal pattern in this paper, can be either a load pattern represented by values of $C_{vx}$ given in Eqn. 3.4 or a pattern proportional to spectral analysis of the building.

Target displacement $\delta_t$ at control node for a building with rigid diaphragm can be evaluated using Eqn. 3.6. Guidelines usually consider control node to be the center of mass at the roof of a building.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{3.6}$$

In the above equation, $C_0$ is a modification factor to relate spectral displacement and likely building roof displacement which can be evaluated using Table 1 with permitted linear interpolation.

Table 1 Values for modification factor $C_0$ [7]

| Number of Stories | Modification Factor |
|---|---|
| 1 | 1.0 |
| 2 | 1.2 |
| 3 | 1.3 |
| 5 | 1.4 |
| 10+ | 1.5 |

Other parameters of Eqn. 3.6 have the same expressions mentioned earlier, while in some cases their evaluations for nonlinear static analysis requires slight modifications. Ratio of elastic strength demand to calculated yield strength coefficient, R according to Eqn. 3.7 is necessary to estimate $C_1$. In this equation, $V_y$ is as shown in figure 5 and other variables were described in the previous parts.

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \tag{3.7}$$

With known R, $C_1$ should be set equal to 1 for $T_e > T_0$ and $[1.0+(R-1)T_0/T_e]/R$ for $T_e < T_0$, but need not exceed those values mentioned in section 3.1. For buildings with positive post-yield stiffness, $C_3$ in NSP shall be set equal to 1.0, while for other systems further discussion in dynamic P-Δ effects is essential.

## 4. ACCEPTANCE CRITERIA IN PERFORMANCE BASED DESIGN OF RCS FRAMES

Usually forces obtained in LSP are significantly larger than what is expected to happen during the design event mainly because of nonlinear behavior of structures; therefore, it seems essential to reduce elastic forces for efficient and economical design purposes of systems. Laboratory tests to verify analytical studies are the best tools to investigate nonlinear properties of different structural systems. Eqn. 4.1 is used to take into consideration this effect for deformation controlled actions in performance based design of structures.

$$Q_{CE} = \frac{Q_{UD}}{m} \tag{4.1}$$

$Q_{CE}$ is the expected strength of the component or element at the deformation level under consideration for deformation-controlled actions and $Q_{UD}$ is design action due to gravity and earthquake loads. In Eqn. 4.1, m is component or element demand modifier to account for expected ductility of the deformation associated with actions at selected Performance Level.

In the absence of experimental tests for described RCS frames and to have a general view of m factors for this type of structure, results of mathematical nonlinear models and elastic ones will be compared in this section. As mentioned earlier, the described method of RCS construction with its beam-column connection can be proposed for low to moderate rise buildings which seems to be most common in different countries around the world. In order to investigate the real behavior of such structures in response to earthquake loading, a few different two-dimensional RCS frames with 4 to 8 stories were modeled, consisting of elements with logical sections by engineering judgments and analyzed in accordance with FEMA documents. Both linear and nonlinear static analyses were performed for all of the models using OpenSEES software [6,7]. The outline of obtained results, accompanied by final conclusion will be presented here. Target displacement at roof level was selected as a reference point to investigate the nonlinear properties of buildings at Life Safety Performance Level.

Analysis results of a two-dimensional RCS frame including beam-to-column connection effects as demonstrated in figure 3 with 6 stories, each 3.1 m high; and 2 bays, each 5 m wide; will be reported here as a sample. Pushover or nonlinear static analysis of this frame using uniform and modal load patterns are shown in figure 6.

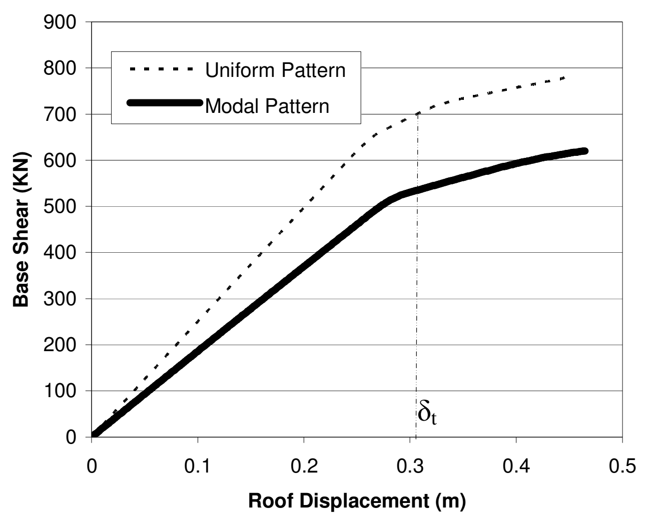


Figure 6 Pushover analysis of the sample RCS frame with two load patterns

The important trend observed in all pushover analyses of all investigated RCS frames including the sample one is that modal load patterns assumed proportional to $C_{vx}$ in this paper resulted in less load carrying capacity than the uniform load pattern; hence, nonlinear element forces obtained from modal load pattern have been selected to compare with their corresponding elastic values for further investigations.

For each of the RCS models, column flexural moment, shear and axial forces and beam flexural moments for both LSP and NSP in all elements were compared in graphs similar to the one shown in figure 7. Using curve fitting techniques, a line was fitted on each set of data, the slope of which indicates ratio of elastic forces to nonlinear actions. For all effects except column axial forces, a uniform trend like what is shown in figure 7 was observed.

In case of column axial forces it has been concluded that linear static procedures do not necessarily predict compressive and tensile forces in columns correctly. For edge columns, axial forces calculated using LSP are approximately 4 times larger than those evaluated by NSP; but for interior columns, real axial forces may be even more than what is resulted from linear static procedure. Comparison between column compressive forces for the sample frame is demonstrated in figure 8.

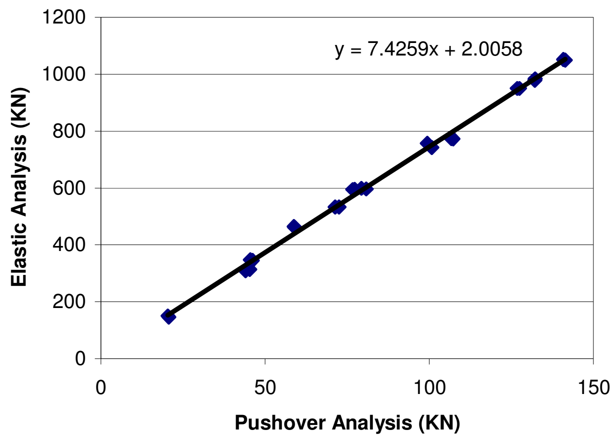


Figure 7 Comparison of column shear forces in LSP and NSP for the sample RCS frame

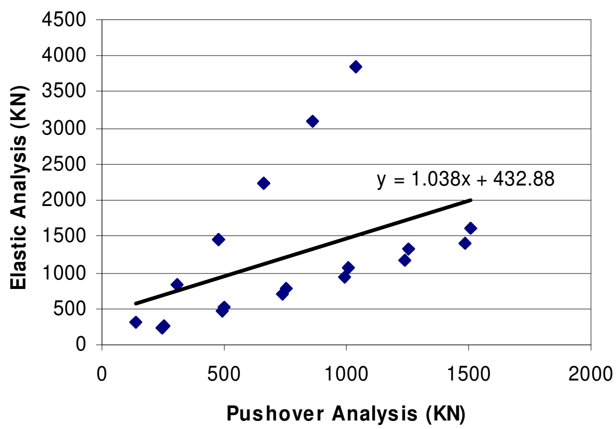


Figure 8 Comparison of column compressive axial forces in LSP and NSP for the sample RCS frame

All in all, the subscribed process of comparing equivalent actions in different models corresponding to 4 to 8 stories, has led us to the results which are tabulated in Table 2. This is solely to present a general view of m factors required in design process of RCS systems with eliminated erection steel column and proposed beam to column connections.

Table 2 Estimation of m factors for acceptance criteria of RCS frames

| Column Axial Forces | All Other Actions |
|---|---|
| 1 | 6 |

## 4. DESIGN PROCESS OF RCS FRAMES

Ant Colony Optimization algorithm which is formulated in a MATLAB program was used to follow performance based design requirements of RCS frames and simultaneously optimizing any desired frame [11]. The developed code for this purpose performs a powerful tool to investigate effects of different parameters in performance based design of RCS frames in the future. Convergence process of the mentioned program for the sample frame discussed is shown in figure 9.

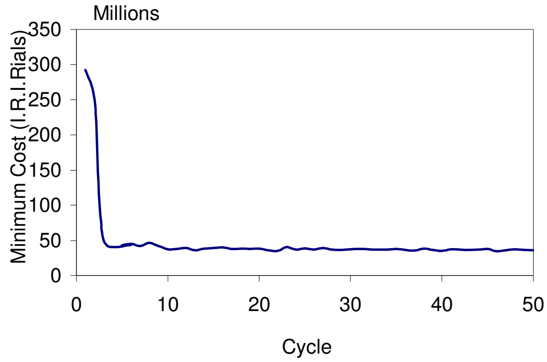


Figure 9 Convergence process of ACO Algorithm for the sample RCS frame

## 4. CONCLUSION

This paper provided a general view in performance based design of RCS frames in the absence of accurate experimental results. Due to unique and highly beneficial characteristics of RCS frames, such as material consumption efficiency, easier and faster construction process compare to steel and reinforced concrete structures etc, it seems necessary to pay more attention to laboratory tests in order to investigate their seismic performance which consequently will result in broaden the scope for developing this type of system around the world. Analytical researches on the proposed beam to column connections have proved that this connection can be assumed rigid, while figure 3 can be employed when the highest possible accuracy in analysis procedure is needed. In addition, performance based design of RCS frames with the subscribed details for Life Safety Performance Level can be accomplished following FEMA-273 requirements, Table 2 of this paper for acceptance criteria of frame element and ACO algorithm, all introducing a complete package for RCS analysis, design and construction.

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