Collapse Performance Evaluation of Reinforced Special Moment Frame Systems Considering Soil–Structure Interaction

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SUMMARY
This study investigates an application of FEMA P695 (ATC-63) methodology for assessing collapse performance of reinforced concrete (RC) structures including the effect of soil-structure interaction. The new generation of performance based design approaches consider the SSI as a direct concern in order to more realistically estimate the seismic performance of structures. In response, this study investigates the SSI effects on seismic performance from within the framework of collapse performance evaluation of a medium height (eight-story) RC special moment frame structure. Comparison with similar studies on fixed-base counterparts results in prescribing conditions for how collapse evaluation should be implemented with SSI taken into account. In this paper the effect of SSI on the collapse behavior of the RC structure is illustrated by both pushover and IDA analysis in comparison with the fixed base structure.

Keywords: Seismic collapse performance, soil-structure interaction, FEMA P695, push-over, IDA.

1. INTRODUCTION

It is well recognized that the nonlinear behavior of a soil–foundation interface due to the consequent energy dissipation during a seismic event may be utilized to reduce the force and ductility demands of a structure. Consideration of Soil-Structure Interaction (SSI), as a phenomenon affecting the dynamic behavior of structures goes back to as early as 1930's (Allotey, 2006). ATC 3-06 standard may be assumed as the first design code that offered recommendations through a fixed-base approach (Modified Fixed-Base) in which the structure's importance indicates the SSI requirements (Allotey, 2006). With the development of the new generation of design codes, the so called Performance-Based design (PBD), and the increase in their application in the design process, the need to incorporate the SSI effects has become well understood. SSI may affect the response of a structure in several ways. Namely, foundation movement can alter the period of a system with introducing flexibility; nonlinear behavior and hysteretic energy dissipation may reduce the force demand to the structure; and the foundation flexibility may alter the input ground motion (Gajan, 2007). However, the prevailing practice has been to ignore the SSI in design procedures due to a general belief in its beneficial consequences. Several approaches are available for modeling the soil foundation interaction, i.e. from Finite and Boundary Element Methods to different Macro-elements and beam-on-nonlinear Winkler foundation (BNWF). Although it is clear that the continuum approaches provide the most capabilities when simulating the SSI, but they are computationally expensive (Allotey, 2006). But the Winkler springs has been shown to give satisfactory results for the practical SSI problems.
This study describes an application of the methodology for assessing collapse performance of reinforced concrete (RC) structures including the effect of soil-structure interaction. The RC structure under study is conformed to the present sample code. The methodology is applied to assessing the collapse performance of a special moment frame (SMF) building using nonlinear static (pushover) and dynamic (response history) analyses. The methodology promoted by FEMA P695 does not specifically accounts for foundation flexibility; however, as it is meant to be practically a realistic method for seismic performance evaluation, using a more detailed model at structure's base is

justified. Soil-Structure Interaction (SSI) can alter the performance of structure totally including its dynamic characteristics, response maxima and more important, distribution of nonlinear response through structure where the accurate calculation of it is vital for the performance evaluation.

2. DETAILS OF THE RC STRUCTURE

The two-dimensional three-bay, eight-story frame is considered for this study. This archetype building has a common story height as the other medium size structures. All design requirements of ASCE/SEI 7-10 and ACI 318-08, including drift limits, minimum base shear requirements and strong-column weak-beam design for a space frame under the maximum seismic load intensity (SDC Dmax) and site class D (Stiff soil) (FEMA, 2009) are satisfied in the design. The range of permissible configurations and structural loading are listed in Table 2.1.

Table 2.1 Key Design Variables in the Design for Reinforced Concrete Moment Frame System

Key Design Variable		
Configuration	Building Height	8 story
	Bay Width	6 m
	First and upper story heights	4.5 and 4 m
Loading	Ratio of tributary areas for gravity and lateral loads	1.0 (space frame)
	Design floor loads	850 kgf/m2
	Design floor live load	250 kgf/m2

The building has a floor plan of 18m×18 m, three bays in each horizontal direction and a uniform mass distribution over height. Figure 1 show the plan of the framing system and show the elevation of the chosen frame for this study. The fundamental period of the frame is 2.36 sec.

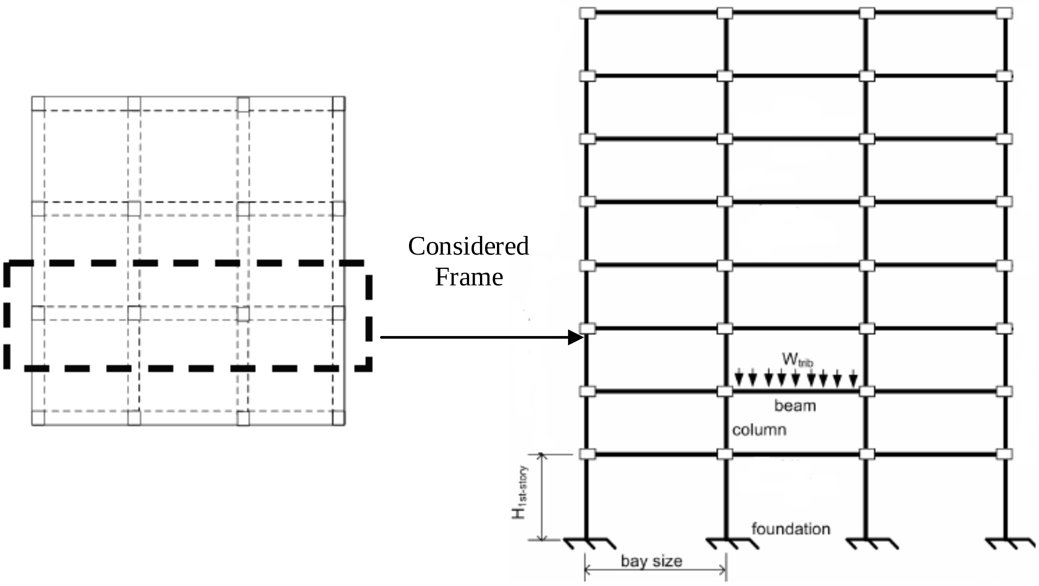

Figure 1. Archetype model for reinforced concrete moment frame system

3. NUMERICAL MODELING

3.1. Beam-Column Element Model

The numerical modeling of the system is developed with the finite element method using the Open System for Earthquake Engineering Simulation (OpenSees) software. For element-level modelling, many RC element models exist, but most of them cannot be used to simulate the structural collapse. A recent research by Ibarra et. al has resulted in a beam-column element model with concentrated inelastic rotational hinges at each end and finite size beam-column joints that employ five concentrated inelastic springs to model joint panel shear as shown in Fig. 2 (Ibarra, 2005). This model is chosen because it is capable of capturing the important modes of deterioration that results in sideway collapse of reinforced concrete frames.

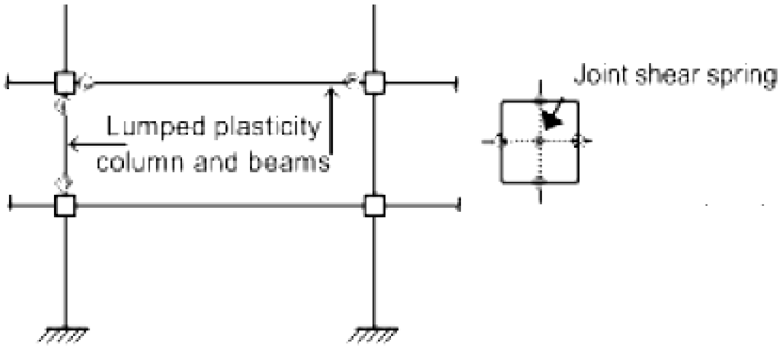

Figure 2. Schematic diagram illustrating elements of nonlinear frame model for RC frame system. (FEMA, 2009)

Figure 3 shows the tri-linear monotonic backbone curve and associated hysteretic behavior of this model. This model includes important aspects, such as the "capping point," where monotonic strength Loss begins, and the post-capping negative stiffness. These features enable modeling of the strain-softening behavior associated with concrete crushing, rebar buckling and fracture, or bond failure. In general, accurate simulation of sideway structural collapse requires modeling this post-capping behavior (Haselton, 2007). The tri-linear monotonic backbone model, was introduced into the OpenSees software by using the Clough material implemented OpenSees by Altoontash (Altoontash , 2004).

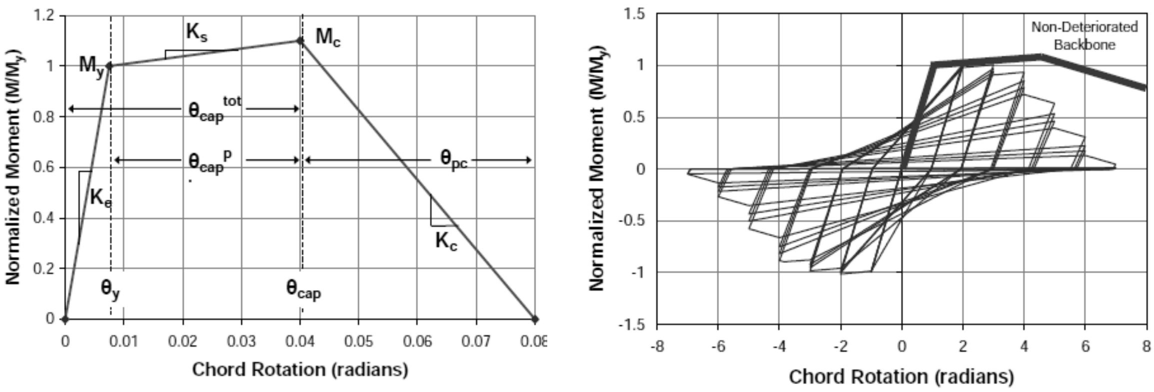

Figure 3. Monotonic and cyclic behavior of component model used to model RC beam-column elements (FEMA, 2009).

3.2. Nonlinear Soil–Structure Interaction Model

The interaction between the foundation and soil interface is modeled using the BNWF model. The beam-on-nonlinear Winkler foundation (BNWF) shallow footing model is constructed using a mesh generated of elastic beam-column elements to capture the structural footing behavior and zero-length soil elements to model the soil-footing behavior with nonlinear inelastic behavior modeled using modified versions of the Qzsimple1, PySimple1 and TzSimple1 material models implemented in OpenSees by Boulanger (Boulanger, 2000). The BNWF model has shown good capability to predict the behavior of shallow foundations including square and strip footings, static and dynamic loading and footings on sand and clay. The BNWF model has capability of simulating the uplift and rocking motions (geometrical nonlinearity) as well as the nonlinear behavior of the soil (material nonlinearity). Figure 4 show The Nonlinear backbone curve used for describing the behaviour of the QzSimple1 material (Raychowdhury, 2009).

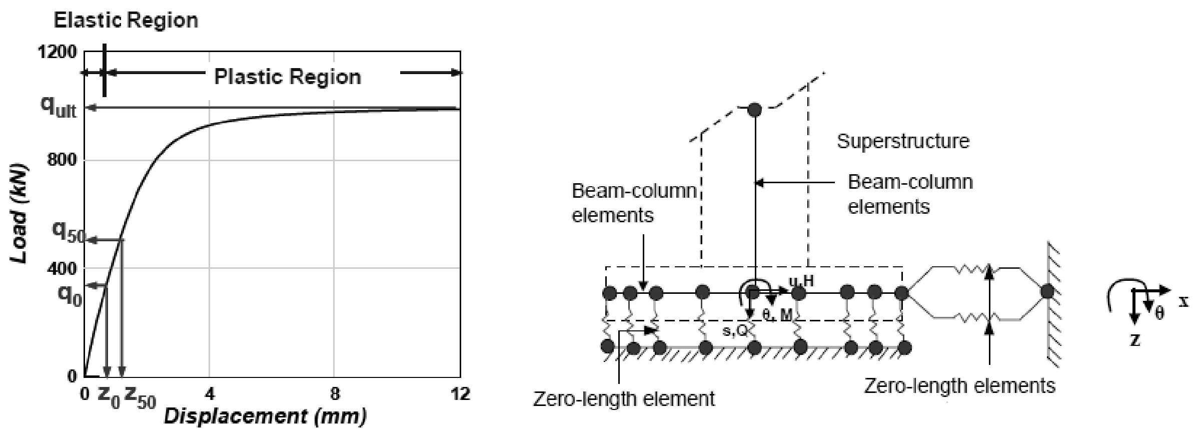

Figure 4. Numerical modeling of soil–structure interaction (Raychowdhury, 2008).

In this study ultimate bearing capacity used for the strength of the Winkler springs was calculated based on Meyerhof (1963) bearing capacity theory (Bowles, 1997) and the vertical and lateral stiffnesses were calculated using equations prescribed by design codes (FEMA-356, Chapter 4) (FEMA, 2000). It has been experimentally established that during the rocking motion, a higher stiffness would be developed in the soil medium at the compression zone, because of the rounding phenomenon to retain the stability of the structure. Thus, more stiff springs were placed at the ends of the footing, providing the rotational stiffness of the soil-footing system.

4. FOOTING AND SOIL CONDITION

The structure is assumed to be resting on two different bases including the fixed-base condition and the flexible base with SSI springs. It is necessary to mention that uncertainties play an important role in the characterization of the soil behavior. The outcome of assessment of structural (rather than geotechnical) aspects of the SSI effects on the seismic performance of an RC frame is highly depending on the properties of soil beneath the footings. Since the RC frame is designed on site class D (Stiff soil, ASCE/SEI 7-10) with shear wave velocity in the range of 180 m/s < ν_s < 360 m/s, the soil beneath the footing is considered to be sand. For the frame a strip footing with the dimensions of 22m× 1.5m×0.8m is designed while the depth of embedment of footing in soil is assumed to be zero. Soil parameters are considered as c = 0, φ = 38°, γ = 19kN/m^3 and G = 30MPa. These parameters are selected based on the available information in the literature for the sand soil (Bowles, 1997).

5. PUSHOVER ANALYSIS

Nonlinear pushover analysis is performed in order to compute the base shear–deformation behavior of the frame for different cases. In this study pushover analysis is performed following the nonlinear static procedure (NSP) of ASCE/SEI 41-06 (FEMA, 2009) and the vertical distribution of the lateral forces at each story level was in proportion to the fundamental mode shape of the frame model. Figure 5 shows the result of analysis for the above cases in form of roof drift ratio versus total base shear. The figure represents the effect of SSI on the frame in reducing the initial stiffness of the frame when on a BNWF base in comparison with the fixed-base case. Collapse drift in the pushover curve is defined by the point where the base shear decreases to 80% its maximum value.

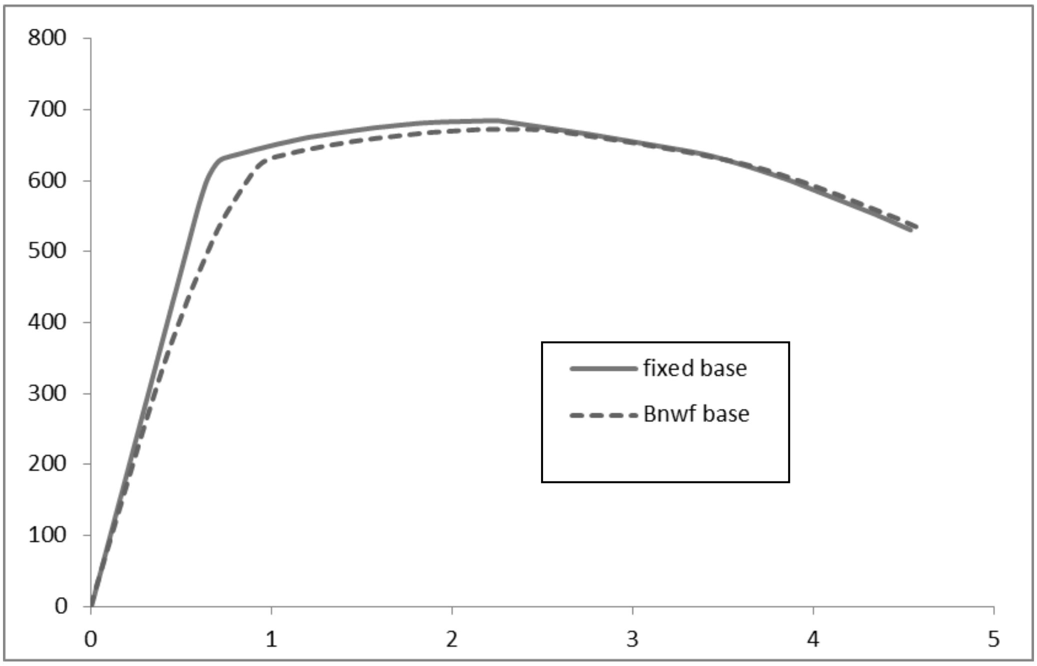

Figure 5. Pushover analysis for the frame on fixed and Bnwf bases

Table 5.1 Yield and Collapse force, Yield and Collapse drift ratio (total) from nonlinear pushover analysis.

Base condition	Yield force (KN)	Yield drift (%)	Collapse force (KN)	Collapse drift (%)
Fixed base	620	0.69	550	4.2
Bnwf base	450	0.52	530	4.6

6. IDA ANALYSIS

In this study the Incremental Dynamic Analysis (IDA) is conducted using the Capitola station ground motion recorded at a site-source distance of 15 km during the Mw 6.9 1989 Loma Prieta earthquake. The CAP000 component with PGA 0.53g was used for this analysis. This motion was scaled to match a particular level of ground motion consistent with the ground motion scaling requirements of ASCE/SEI 7-10, with the notable exception that the median value of the scaled record needs only match the MCE demand at the fundamental period, T, rather than over the range of periods required by ASCE/SEI 7-10. Figure 6 shows the IDA curves in the form of maximum story drift ratio versus spectral acceleration. The difference between the fixed base and BNWF responses represents a reduction of drift on the BNWF base in comparison to the fixed base case. However, it seems that assessing only the drift parameter in collapse performance analysis of an RC structure with IDA cannot establish the effect of SSI on the response of the RC structure clearly. Collapse Drift in the IDA curve is defined by the final point where the local tangent reaches 20% of the initial elastic slope.

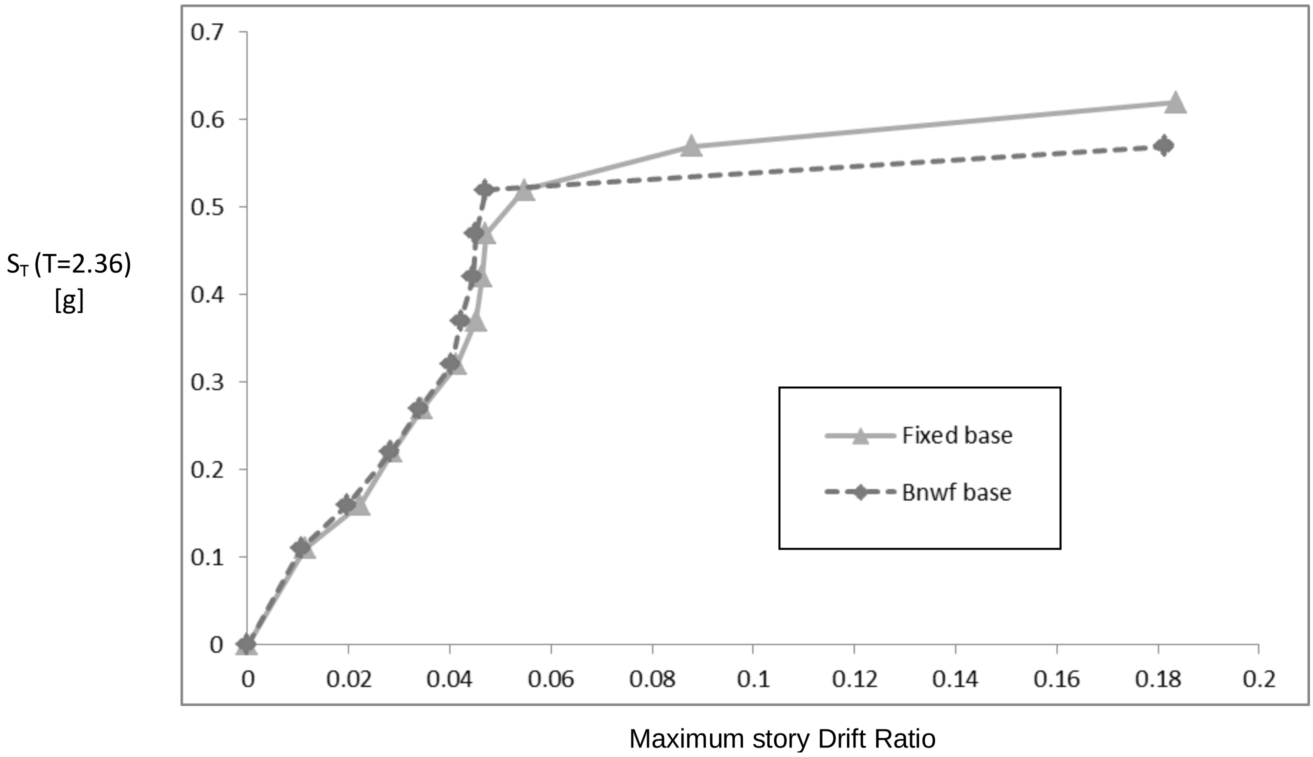

Figure 6. Incremental dynamic analysis response plot of spectral acceleration versus maximum story drift ratio for fixed and Bnwf bases.

Table 6.1 Collapse drift ratio from incremental dynamic analysis

Base condition	**Collapse story drift ratio**
Fixed base	0.058
Bnwf base	0.042

7. CONCLUSIONS

Nonlinear foundation movements and associated energy dissipation may be utilized to reduce the force and story drift of a structure, particularly in a high-intensity earthquake event, if the potential consequences such as excessive settlement are taken care of. However, these aspects of SSI are not considered in the current design practice, mostly due to the absence of reliable nonlinear SSI modeling techniques. The BNWF model has shown good capability to predict the behavior of shallow foundations. The present study focused on the effect of foundation flexibility and nonlinearity on the structural collapse in terms of story drift. A Winkler-based model (BNWF) was adopted for this purpose. The results were illustrated for fixed-base and BNWF-base models. The following specific observations obtained from the analyses show that the story drift was not a distinctive behavior when considering the effect of SSI on the collapse performance of an RC structure. Other response parameters such as the base moment, base shear and distribution of ductility demand throughout structure should also be investigated in this regard.

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