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Effects of the Soil-Foundation-Structure Interaction on the Component Demand Modifier Factor of Concrete Gravity Beams based on ASCE 41-06 Standard

A. Azarbakht[*a], E. Rajabi [b]

[a] *Assistant Professor, Department of Civil Engineering, Faculty of Engineering, Arak University, Arak, Iran*
[b] *Graduate Student, Department of Civil Engineering, Faculty of Engineering, Arak University, Arak, Iran*

Abstract

The aim of the present paper is to evaluate the influence of the Soil- Foundation- Structure Interaction (SFSI) effects on the component demand modifier factor of concrete gravity beams based on ASCE 41-06 standard. To this end, the beam on the nonlinear Winkler foundation approach is employed which is a simple and efficient method. At first, four sets of 3-, 6-, 10- and 15-storey concrete moment-resisting frames founded on soft, medium and hard soils are designed and analysed for the case of fixed-base and flexible-base assumptions. A comparison is then made between the results of the nonlinear response history analysis of each frame in the flexible-base and fixed-base conditions with the response based on the equivalent linear static approach. The findings show that the equivalent linear static approach loads for gravity beams, which are controlled by deformation actions, can lead to non-conservative predictions of seismic demand. Finally, a modified load combination is proposed to be used in the equivalent linear static approach to avoid this imperfection.

Key word: CE Database subject headings; Winkler spring; Soil-Foundation-Structure Interaction; ASCE 41-06 standard; Nonlinear seismic response history analysis; Ground motion.

1. Introduction

The seismic excitation on a firm soil, which can be implemented as an input for the structural analysis, is the same as the excitation in the free-filed motion. However, the ground motion input can be significantly modified in the case of soft soil as a result of the SFSI effect. The characteristics of the structure can also be changed due to this important effect in comparison with the fixed- base assumption. Moreover, as the most important consequence of the SFSI effect, the natural period of a given structure can extend and in turn the level of seismic input would decrease [1].

The SFSI effect is a combined phenomenon in which the structure is influenced by the dynamic behavior of the soil and the foundation and vice versa

[2]. This phenomenon is very important in designing structures mostly because of the problems with modeling and analysis as well as the mistaken belief that neglecting the SFSI is conservative.

The SFSI effect on the seismic behavior of structures was a major research topic in the last decade. A pioneer work was the analysis and design of a fifty-storey building in California with the fixed base assumption [3]. The SFSI modeling and design are more advanced now and mostly can take the soil nonlinearity into account [3]. The procedures for including the SFSI into the design calculation are described in ATC 3-06 standard [4] as a force design regulation as well as in ASCE 41-06 standard [5] as a performance-based design regulation.

✉*Corresponding Author Email: a_azarbakht@araku.ac.ir

2. Challenge of the Current Study

The design force for deformation-controlled members, e.g. gravity beams, in ASCE 41-06 standard is divided by the m factor as described in Equation (1) with considering the knowledge factor equal to unity:

$$mQ_{CE} \geq Q_G + Q_E \qquad (1)$$

Where m is the component demand modifier factor, Q_G and Q_E is the actions due to design gravity and design earthquake loads, respectively, and Q_{CE} is the expected strength for a given element. Equation (1) can be rewritten as $Q_{CE} \geq (Q_G + Q_E)/m$ which means that the combination of gravity and earthquake loads is divided by the m factor. The different applications of m factor in Equation (1) can change the results in the case of considering the SFSI effects and in the case of the elements carrying significant amounts of gravity loads, e.g. gravity beams. Therefore, in this paper the effects of m factor on the beam internal moments are investigated with giving consideration to the SFSI effects. To do so, four sets of 3-, 6-, 10- and 15-storey concrete moment resisting frames are designed for three different soil conditions and fixed and flexible based conditions. The beams internal moments are also compared between the linear static procedure and the nonlinear response history analysis. Finally, a revised load combination is proposed in order to solve the inherent problem in Equation (1).

3. Selection and Modeling of the Structures

A set of 2-D concrete moment resisting frames with shear walls containing 3-, 6-, 10- and 15-storey frames are designed based on FEMA 450 [6] static linear guidelines on hard, medium and soft soil conditions [7] introduced through site classes B, C and D (Figure 1). The shear walls are considered in the design procedure since the SFSI effects are significant in the case of stiff structures. The storey height and the bay length are 600 and 350 cm, respectively. The shear wall thickness is 25 cm for the 3- and 6-storey frames and equals to 30 cm for the 10- and 15-storey frames.

With identified category and shear wave velocity for the site classes, satisfactory values were estimated to represent their design parameters according to several well-known geotechnical references [8-15]. The selected values are presented in Table 1.

Soil Type	E (MPa)	G (MPa)	γ kN/m³	D_r %	μ
B	15000	6000	24	-	0.25
C	200	74.1	21	85	0.35
D	65	24	19	75	0.25

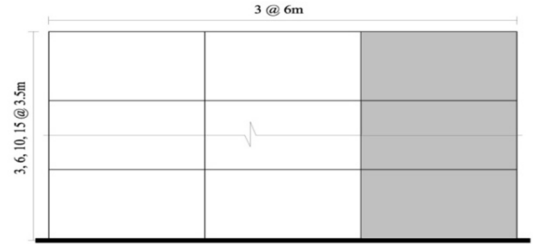Table 1. Selected characteristics of site classes B, C and D.

Figure 1. The schematic elevation of the studied frames [7].

Geotechnical design parameters in Table 1 include E modulus of elasticity, G shear modulus, γ specific weight, D_r relative density and μ Poisson's ratio of the soil. It should be pointed out that uncertainties play an important role in the characterization of the soil behavior. However, by considering the structural (rather than the geotechnical) aspects of the current study, the assumed values in Table 1 could be reasonably accepted for the common practice.

The gravity loads were selected based on the values typically employed in engineering practices. Therefore, 3-, 6-, 10- and 15-storey frames have masses equal to 307, 640, 927 and 1511 tons, respectively. Further, equivalent lateral design forces were determined based on FEMA450 guidelines i.e. the design spectrum for each site class was derived and the corresponding design base shears were calculated as reported in Table 2. Both gravity and seismic loads were then imposed on the frames according to the additive and counteractive load combinations of ASCE7-05 standard [16] (the reader is referred to Section 12.4.2.3 of ASCE7-05 for further details). The given frames were designed as special frames based on FEMA450 guidelines. According to these guidelines, 15-storey frames should be designed with regard to dual lateral resisting systems. The thickness of the shear wall was chosen to be 0.25 m in the 3-/6-storey frames and 0.30 m in the 10-/15-storey cases. The geometric and material properties of the designed 3-, 6-, 10- and 15-storey frames are presented in Tables 3 and 4. The strip footings with 2.0-4.6 m in width, 19.6-20.0 m in length and 1.0 m in height (which is fixed) were designed for all the frames. The fundamental periods of all the frames are presented in Table 5. Subsequently, the complete numerical models of the CSW frames were constructed through an appropriate assembly of nonlinear shear wall elements and nonlinear beam-column elements.

Table 2 . Calculated design base shears (Ton)

Lateral Resisting System	Number of Stories	Site Class		
		B	C	D
Shear Wall Frame	10	71.4	89.0	101.0
	6	99.2	106.9	106.9
	3	51.3	51.3	51.3
Dual System (Shear Wall + Moment Resisting Frame)	15	56.8	73.9	85.2

Table 3 . Geometric properties of the designed 3-, 6-, 10- and 15-story frames

Number of Story	Level	Column Width (cm)	Column Height (cm)	Beam Width (cm)	Beam Height (cm)
3	1, 2, 3	45	45	45	45
6	1, 2, 3, 4, 5, 6	45	45	45	45
10	1, 2, 3	55	55	55	40
	4, 5, 6, 7	45	45	45	40
	8, 9, 10	45	45	45	40
15	1, 2, 3, 4	70	70	70	50
	5, 6, 7, 8	60	60	60	50
	9, 10, 11, 12	45	45	45	40
	13, 14, 15	45	45	45	40

Table 4 . Material properties of the designed 3-, 6-, 10- and 15-story frames

Specified Concrete Compression Strength, f (kg/c)	Modulus of Elasticity, E	Yield Stress,	Shear Modulus, G
250	2.388 e+5	4000	99500

Table 5 . The fundamental fixed-base period of the frames (Seconds).

Number of Stories	Elastic Model	Nonlinear Model
3	0.13	0.16
6	0.43	0.48
10	0.92	1.02
15	1.52	1.70

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4. Frames Modeling

The seismic nonlinear behavior of frame elements plays the most important role in the global behavior of the considered frames. The Open Sees framework [17] has been used for the purpose of response history analysis. In the present paper, the nonlinear beam element with concentrated hinges is employed for the beam modeling. Beams with concentrated plastic hinges and columns of fiber section are employed to simulate the nonlinear flexural behavior of the moment frames. The beam With Hinges element is chosen for the beams. Thus, a pre-determined length at both ends was allocated to the plastic hinges and an elastic material was assigned to the mid-span. As the nonlinear behavior was assumed to be focused in the hinges, expansion of the nonlinearity to the elastic region was less likely to happen [18]. Therefore, the coefficient of cracking was set to be 0.5 for the elastic segment of the beams. The nonlinear behavior of the plastic hinges was defined in accordance with Haselton et al. [19] who proposed essential relationships in their study based on the calibration of numerous test results in the form of the tri-linear backbone curve suggested by Ibarra [20,21]. An important feature of the model is that softening due to concrete crushing, reinforcement buckling and yielding and bond slip can be considered in the negative stiffness region, namely the post cap behavior [19].

The tri-linear Ibarra model, as mentioned above, was employed in the Open Sees platform using the Clough material proposed by Altoontash [22]. Then Uniaxial sections with pre-defined $M-\theta$ according to the Clough material were assigned to the plastic hinges. It should be noted that all parameters calculated to form the Ibarra model were in terms of rotations. Thus, in order to make them applicable to a beam With Hinge element, the simple equation $\phi = \theta/L$ (ϕ curvature, θ rotation and L plastic hinge length) was used to transform rotations into curvatures. This is an advantage of the selected beam element [23]. The plastic hinge's length was set to equal the beam's height for all the cases.

Columns were modeled by means of the fiber method with the capability of developing distributed plasticity along the element's length. This choice was made mostly due to the fact that the flexural behavior in the columns is highly dependent on the interaction of their axial and bending forces. However, the aforementioned approach for beams was incapable of considering variable axial forces during the analysis. As a result, the fiber sections were assigned to the nonlinear Beam Column elements. Each element was also divided into four sub-elements in a story level to provide more robustness.

5. Shear Wall Modeling

Recently, 'Flexure-Shear Interaction Displacement-Based Beam-Column' element has been developed in the Open Sees platform based on the concept of formerly used Multiple Vertical-Line-Element Model (MVLEM). In this new element, previous multiple vertical columns are defined as fibers of a section. The interaction between the flexural and shear behaviors is provided by assigning a biaxial response to the fibers incorporating a membrane material model. The flexure-shear interaction displacement-based beam-column element was thus selected to simulate the shear wall element in the Open Sees platform because of its inclusive features. In particular, with its application, the numerical models showed a proper agreement with the characteristics of the designed frames. More information about the element can be found in Orakcal et al. [24].

The definition of the boundary elements was also provided in the model. Hence, the resulting shear wall element would take the form of a single column. However, attaching this column to the moment frame was quite problematic. To cope with this problem, the mid-panel of the shear wall was constructed with the flexure-shear interaction displacement-based beam-column element while the boundary elements were modeled as columns of the main frame. To enhance the robustness, each element was divided into four sub-elements in a story level. End nodes located at the same elevation of the boundary elements and the mid-panel column element were then joined by means of rigid beams. This provided an integrated simulation of the whole shear wall system. Clearly, the system could benefit from the strong features of both the flexure-shear interaction model and the fiber section.

6. Soil -Footing Interface Modeling

In this study, the Beam on Nonlinear Winkler Foundation (BNWF), which is capable of simulating the uplift and rocking motions (geometrical nonlinearity) as well as the nonlinear behavior of the soil (material nonlinearity), was employed to model the soil-footing interface. The BNWF allows for likely changes in the soil springs' stiffness and spacing along the foundation length too. In this study, the BNWF numerical model was constructed by assigning nonlinear Beam Column and zero Length elements to the strip footing and the soil springs, respectively. It is worth mentioning that the beam at the base of the shear wall was set to be rigid because of the high flexural stiffness that the shear wall added to the footing's rigidity. In addition, the footing was constrained against sliding [25-27]. In order to define the Winkler springs, first their properties were determined according to different site classes and the corresponding footing dimensions. Second, Qzsimple1 material (in the Open Sees) was chosen to represent the soil behavior based on the computed parameters. Moreover, the Gazetas concentrated stiffness [28] was employed to define the stiffness of the soil springs. Therefore, the distributed stiffness of the Winkler foundation was estimated based on the continuum approaches. Initially, the total vertical and rotational stiffness of the footing-soil systems were found according to the relations proposed by Gazetas as shown in Table 6. A specific distribution of Winkler springs with varying stiffness was later selected for each system to produce the same total vertical and rotational concentrated stiffness.

Table 6 . The soil-footing elastic vertical/rotational Gazetas stiffness

		Number of Stories			
	Site Class	3	6	10	15
Vertical Stiffness Intensity $K_z/A \ \left(MN/m^3\right)$	B	5419	3708	2882	2574
	C	77	53	41	37
	D	25	17	13	12
Rotational Stiffness Intensity $K_{\theta y}/I_y \ \left(MN.m/m^4\right)$	B	7697	6545	5744	5432
	C	110	93	82	77
	D	36	30	27	25

Table 7 . The foundation bearing capacity based on Meyerhof's relationship.

		Number of Stories			
	Site Class	3	6	10	15
Foundation Bearing Capacity $q_{ult} \ (kPa)$	B	49304	61931	75183	83381
	C	14526	17629	20967	22989
	D	5291	6227	7267	7886

It has been experimentally established that during the rocking motion, higher stiffness would develop in the soil medium at the compression zones. The so-called rounding phenomenon happens to retain the stability of the structure [29]. Accordingly, in this study more stiff springs were placed at the ends of the footing strip to supply the rotational stiffness of the soil-footing system. The end lengths were determined based on [29]. Finally, a contribution of vertical springs of particular stiffness, located in the middle and end zones of the footing strip, was chosen based on [29] to obtain the total rotational stiffness. Likewise, the strength of the Winkler springs was calculated based on the bearing capacity of the foundations. Among several equations available to determine the bearing capacity, the Terzaghi's relationship (1943) is widely employed in the engineering problems [9]. However, a more rigorous form of the Terzaghi's relationship, proposed by Meyerhof (1963) [9], was selected to estimate the foundation bearing capacities in this study as shown in Table 7.

7. Selection of the Ground Motion Records

The input ground motion selection can significantly change the nonlinear response of structures. The procedure for record selection suggested by Ghafory Ashtiany et al. [30] has been employed in this research. The main philosophy of the record selection based on [30] is to choose a few strong ground motion records in order to get approximately the same result as a large set of records. Using the mentioned procedure can decrease the computational time significantly. In this study, the records selected based on the natural period of each frame are appropriate for estimating the unbiased median response of structures. The selected records are presented in Table 8 and in the corresponding spectrum in Figure 2. The gravity loads are applied first and then the ground motion record is used. In this regard, a sample of the nonlinear behavior of the 15-storey frame on the D soil type is illustrated in Figure 3.
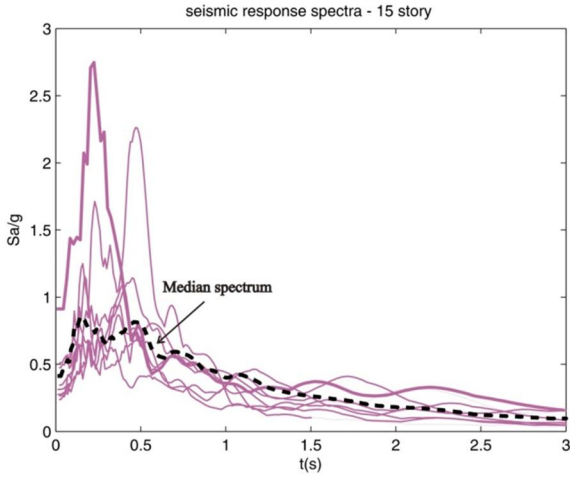
Figure 2. The seismic response spectra and the corresponding mean spectrum for analysing the 15-storey frame.

8. Comparison of the Results

As mentioned before, the component modifier factor (m) can alternatively increase the expected strength or decrease the input load resulting from the gravity and earthquake forces as described in Equation (1). As long as the fixed based assumption of the linear analysis is used, the two mentioned alternatives result in the same output. However, in the case of flexible based assumption, it is expected that the m factor influences the earthquake force

but does not affect the gravity force. Hence, Equation (1) is rewritten as Equation (2) in order to restrain the m factor effect only to reduce the earthquake force:

$$Q_{CE} \geq Q_G + \frac{Q_E}{m} \tag{2}$$

As the performance level of the designed frame in the immediate occupancy (IO), the m factor equals 3 and 1.25 in the case of the internal moment and the internal shear actions, respectively. The results of the four different analyses are compared in Figures 4, 5, 6, 7. In the figures, LC1 and LC2 respectively denote the case of linear static forces based on Equation (1) and (2), and LC3-fixed base and LC3-flexible base represent the case of nonlinear response history results for the fixed-based and flexible-based assumptions. It is clear that the LC3-flexible base can be treated as the benchmark for comparison. In this regard, the results of all the cases are depicted in Figures 4 to 7 for both base conditions of different beams. The results of the LC1 are quite different from the benchmark ('LC3-flexible base') as seen in Figures 4 to 7; this confirms that contrary to expectations, the m factor is reducing the gravity loads. On the other hand, the results of the proposed LC2 are closer to the benchmark so that they show more robustness in comparison with the LC1. The results of the other frames confirm this conclusion.
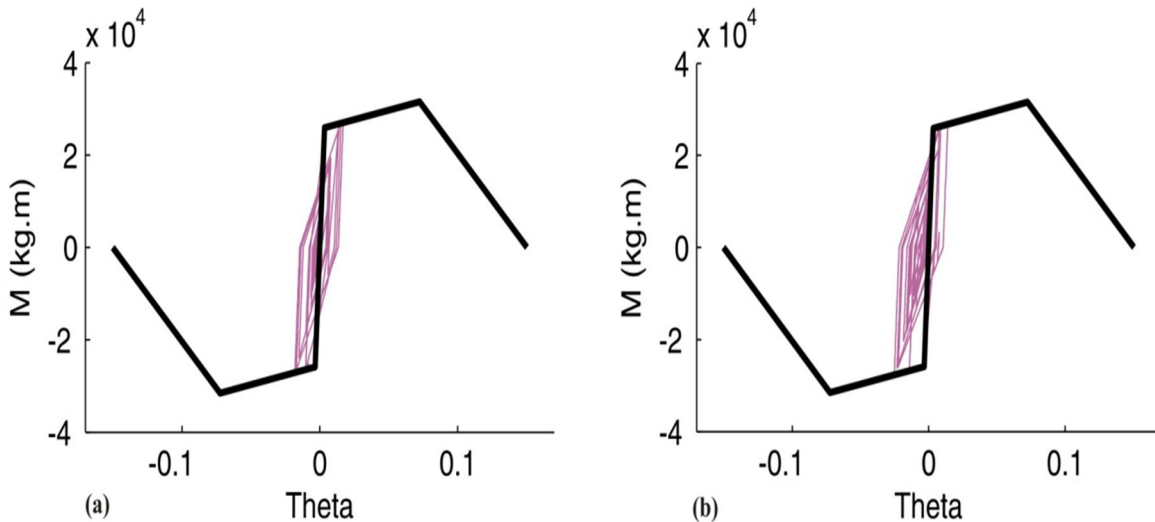

Figure 3. Moment-rotation curve, (a) fixed base, (b) flexible base (for the 15-storey frame on the D soil type).
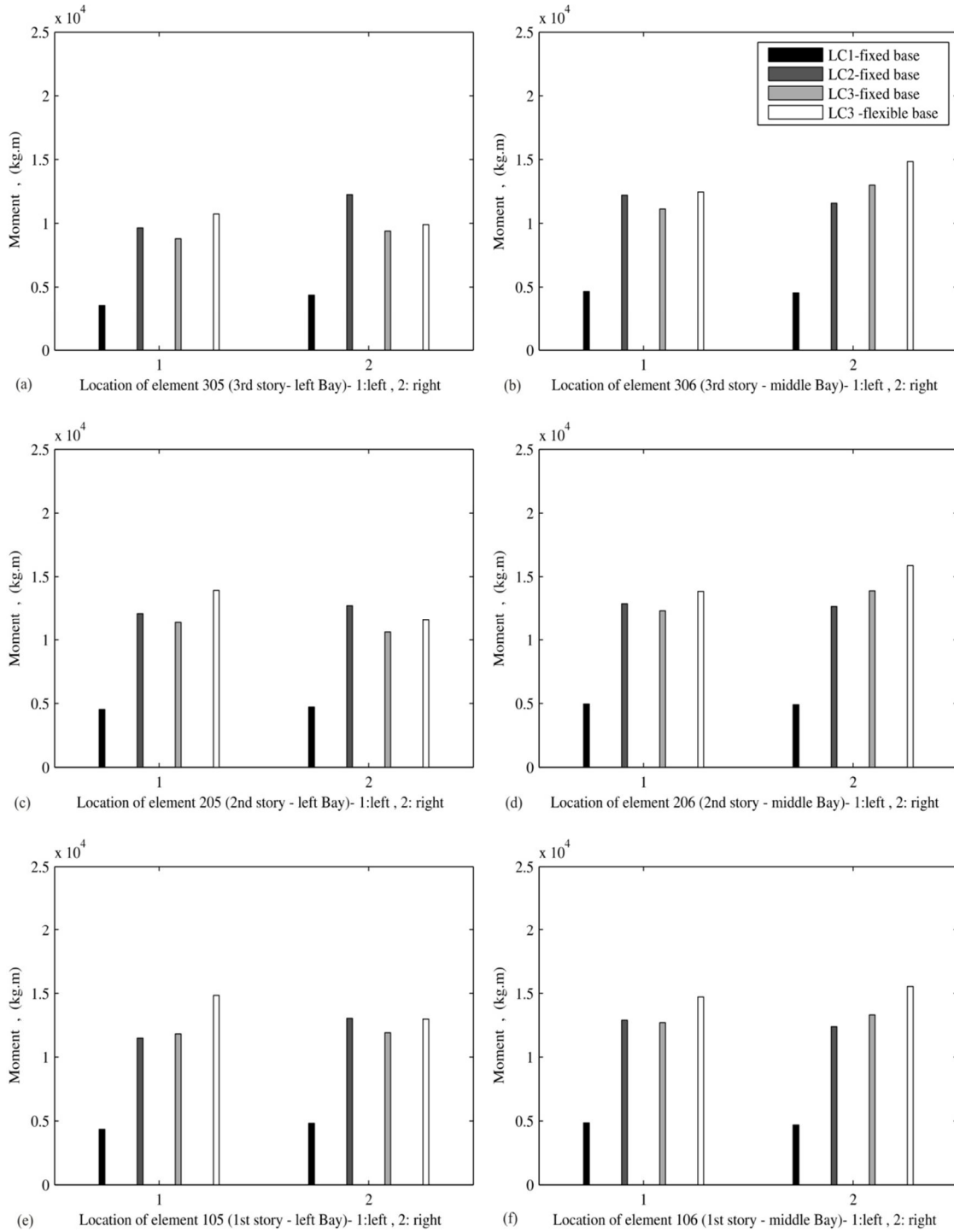
Figure 4. Comparisons between the internal moments of the 3-storey frame located on the B soil type.

(a) The beam element located in the 3rd storey – left bay, (b) The beam element located in the 3rd storey – middle bay,
(c) The beam element located in the 2nd storey – left bay, (d) The beam element located in the 2nd storey – middle bay,
(e) The beam element located in the 1st storey – left bay, (f) The beam element located in the 1st storey – middle bay.
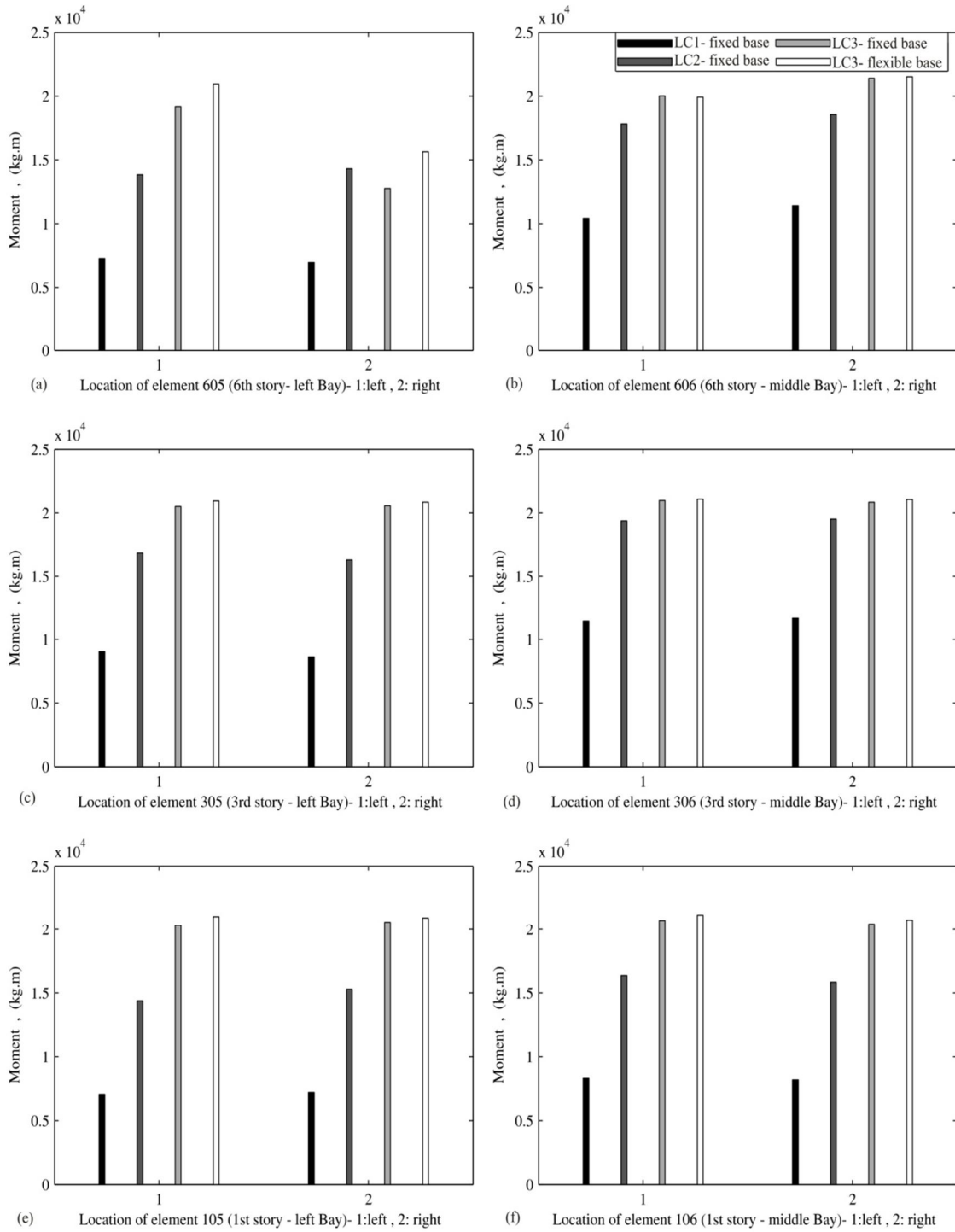
Figure 5. Comparisons between the internal moments of the 6-storey frame located on the C soil type.

(a) The beam element located in the 6th storey – left bay, (b) The beam element located in the 6th storey – middle bay,
(c) The beam element located in the 3rd storey – left bay, (d) The beam element located in the 3rd storey – middle bay,
(e) The beam element located in the 1st storey – left bay, (f) The beam element located in the 1st storey – middle bay.
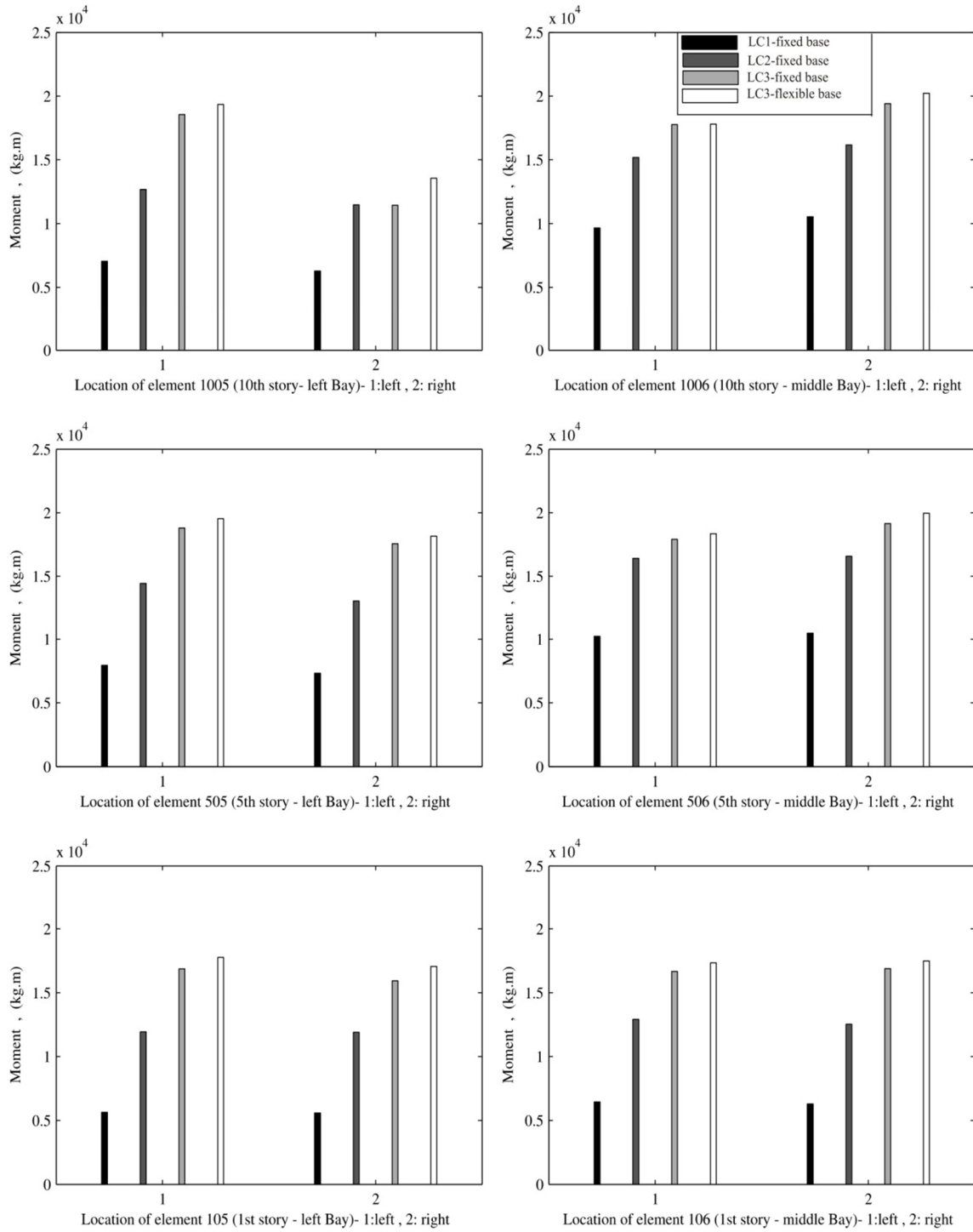
Figure 6. Comparisons between the internal moments of the 10-storey frame located on the C soil type.

(a) The beam element located in the 10th storey – left bay, (b) The beam element located in the 10th storey – middle bay,
(c) The beam element located in the 5th storey – left bay, (d) The beam element located in the 5th storey – middle bay,
(e) The beam element located in the 1st storey – left bay, (f) The beam element located in the 1st storey – middle bay.
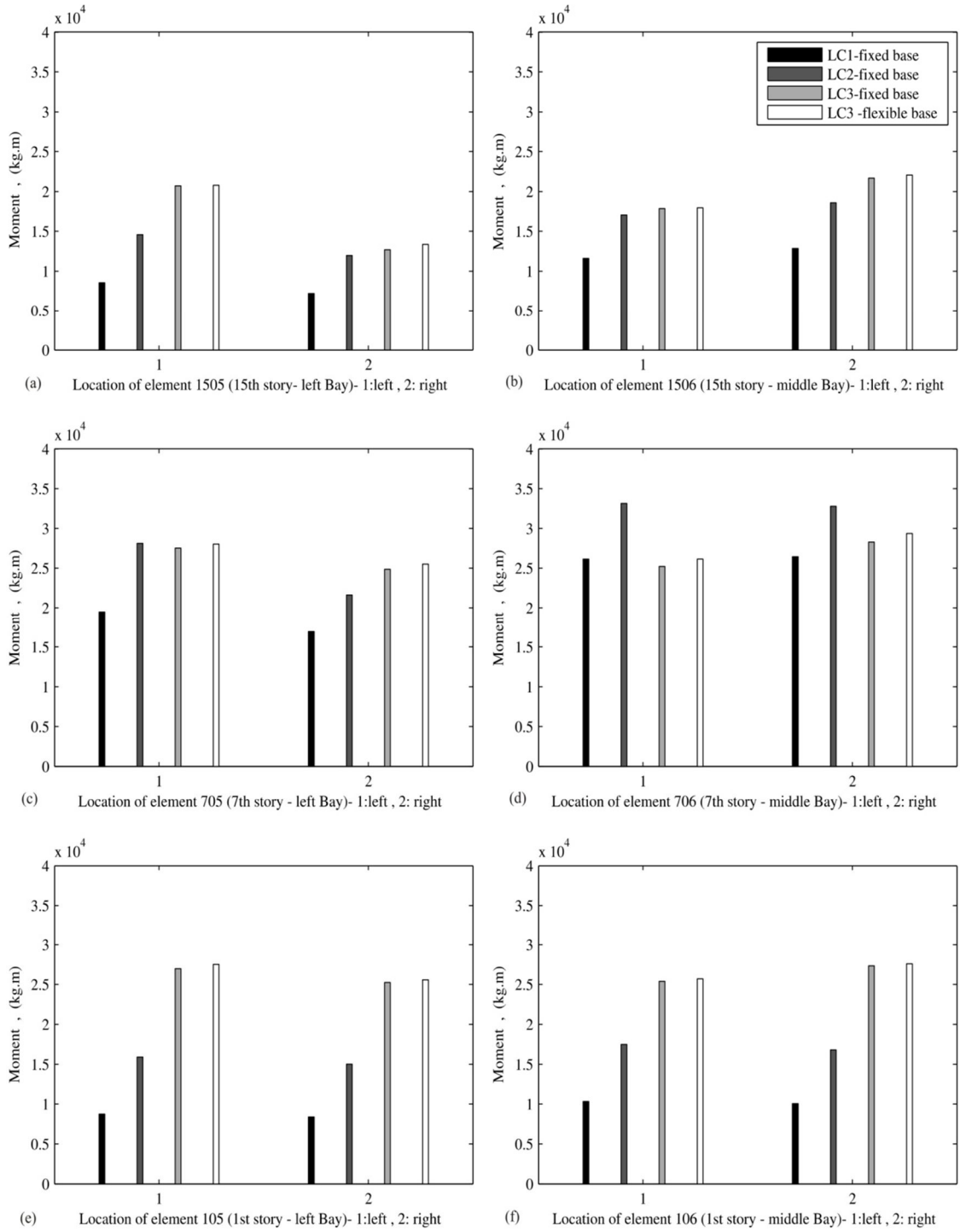
Figure 7. Comparisons between the internal moments of the 15-storey frame located on the C soil type.

(a) The beam element located in the 15th storey – left bay, (b) The beam element located in the 15th storey – middle bay,
(c) The beam element located in the 7th storey – left bay, (d) The beam element located in the 7th storey – middle bay,
(e) The beam element located in the 1st storey – left bay, (f) The beam element located in the 1st storey – middle bay.

9. Conclusion

The effect of the component modifier factor on the internal moment distribution of gravity beams is investigated in this paper. Four different sets of 3-, 6-, 10- and 15-storey concrete moment-resisting frames founded on soft, medium and hard soils were designed and analysed for the case of fixed-base and flexible-base assumptions. A comparison was then made between the results of the nonlinear response history analysis of each frame in the flexible-base and fixed-base conditions with the response based on the equivalent linear static approach obtained from ASCE 41-06 standard. The results show that the equivalent linear static approach load combinations for the beams that are controlled by deformation actions can lead to the non-conservative predictions of the seismic demand. At last, a modified load combination was proposed to address this problem.

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