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Progressive Collapse Analysis of Reinforced Concrete Buildings Including Soil-Structure-Interaction

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Abstract: **The study of Progressive collapse continues to be an active area of research in structural engineering due to the repeated events caused by accidents or act of terrorism. Design guidelines have been developed to mitigate the risk of progressive collapse including direct and indirect design methods. This paper investigates the effect of soil-structure interaction on the capacity of reinforced concrete buildings to resist progressive collapse. The alternate path method was used by removing external columns and applying the UFC 4-023-03 (2009) design guidelines. For soil-structure-interaction, the simplified uncoupled spring model suggested by UFC 273 was adopted to represent the effect of foundation and underneath soil. Three and Seven story building examples designed based on the most common practice of design were studied with edge column removed at ground-, mid-, and roof-floor as per guidelines. Results indicate that soft soil may slightly enhance the behavior of buildings with few floors (the three-story building example) and for more floors building (the seven-story building example), fixed base conditions leads to better behavior. The effect of changing the base conditions is observed to be less for upper story column removal.**

Keywords: **Buildings, Nonlinear, Progressive Collapse, Soil-Structure Interaction, Static**

1. INTRODUCTION

The problem of progressive collapse in structural engineering gained its interest at 1968, after the Ronan Point apartment building partial collapse. More and more attention was directed to the problem after the disproportionate collapse of the Alfred P.Murrah Federal Building (Oklahoma City, 1995), the damage of Al-Khobar building (Al-Khobar, 1996) and the catastrophic collapse of the twin World Trade Center towers (New York, 2001) [1,2,3,4]. The FEMA (2002) [5] report on the collapse of the World Trade Center (WTC) towers concluded a call for studies to determine "whether there are feasible design and construction features that would permit such buildings to arrest or limit a collapse, once it began." [6]. Progressive collapse is defined as a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause [2]. Several sources of progressive collapse were reported with their probability of occurrence as; gas explosions: (2E-05 /yr per dwelling), bomb explosions: (2E-06 /yr per dwelling), vehicular collisions: (6E-04 /yr per building), fully developed fires: (5E-08/sq.m/ yr per building) [7]. Since abnormal loads are extremely rare events that can occur during the lifetime of a building, it is more appropriate to mitigate the risk for progressive collapse than to especially design them to resist for these loads. The probability of failure due to progressive collapse can be assessed as [7]

$$P(F) = \Sigma P[F|DHi]\, P[D|Hi]\, P[Hi] \quad (1)$$

Where P(F) represents the probability of structural collapse, P[Hi] is the probability of hazard Hi, P[D|Hi] is the probability of local damage, given that Hi occurs, and P[F|DHi] is the probability of collapse, given that hazard and local damage both occur. The third term (P[Hi]) in the above equation can be controlled by event control reducing or preventing the occurrence of the hazard (e.g., imposing a minimum stand-off distance through placement of physical barriers and similar devices, or preventing access to certain building zones) [8]. The first and second terms of the equation represent the structural response to progressive collapse scenario which is typically complex and nonlinear for which a number of analysis and design approaches can be followed [9]. Indirect (implicit) and direct (explicit) methods are derived for design and evaluation of buildings for progressive collapse [10]. Indirect approaches include applying prescriptive design rules (minimum requirements on strength, continuity, ductility), providing resistance to progressive collapse; however progressive collapse behavior is not addressed explicitly [2,8]. In other words, indirect design approach addresses the problem by identifying and incorporating into the building system characteristics that enhance robustness, without special consideration of loads or events that could trigger disproportionate collapse.

Direct Design approaches include explicit consideration of resistance to progressive collapse during the design process. These include: 1) the Alternate Path (AP) method, which requires that the structure be capable of bridging over a missing structural element, with the resulting extent of damage being localized, 2) the Specific Local Resistance (SLR) method, which requires that the building, or parts of the building, provide sufficient strength to resist a specific load or threat, and 3) the tie force method aims to enhance continuity, ductility, and structural redundancy by specifying minimum tensile forces that must be used to tie the structure together [10]. Regulations related to the consideration of progressive collapse were issued as a guide in design and evaluation of structures for more safe design. Although ASCE 41-06[11] addresses seismic loads, which are horizontal and transient, it can be applied to progressive collapse design, where the loads are vertical and permanent. UFC 2009[10] proposed material properties and acceptance criteria that are more suitable for progressive collapse analysis and supersedes the corresponding values at the ASCE 41-06[11]. Following such guidelines has become relatively common especially that the costs of applying the progressive collapse prevention strategies was reported to be not more than 1.4 percent of the total structure costs for sample studied cases [9].

Literature includes also the application of different design methods and analysis types in addition to the suggestions for increasing the resistance of buildings to progressive collapse. The majority of studies concerning progressive collapse applies the evaluation is the alternate path method using linear static, nonlinear static and dynamic analysis procedures. Nonlinear dynamic analysis has the advantage that dynamic effects are inherently incorporated in the analysis as opposed to a priori assumed dynamic amplification factor (DAF) applied in case of static analysis [1]. Nonlinear dynamic analysis was used for external column removal of regular structure [3] and to validate the mitigation system proposed [12]. It was also applied to investigate the role of material and design method in the progressive collapse of RC structures [13] and to verify a failure experiment [1]. The simplicity of inelastic static pushover analysis compared to inelastic dynamic analysis has led to its application in many investigations. The applicability of nonlinear static analysis was validated compared to dynamic push-over especially for low rise and short period structures [14]. It was also used for external column removal of regular structure [3] and for assessing how can seismic design affect the progressive collapse resistance. The potential for progressive collapse of structures is assessed using the linear static analysis procedure as a simplified alternate method [15,16].

The tie (TF) force method was reported to be inadequate in increasing the progressive collapse resistance through a numerical study on two reinforced concrete (RC) frame structures [17]. The TF method fails to consider such important factors as load redistribution in three dimensions, dynamic effect, and internal force correction; thus an improved TF method was proposed [17]. Methods and approaches other than the member removal were also suggested for blast loads based on non-zero initial conditions [18]. Investigations of the Dynamic Amplification Factors (DAFs) for structures subjected to sudden support loss were also carried out experimentally [19] using small-scale test and analytically for an inelastic SDOF model subjected to step loadings. The mitigation of progressive collapse risk was proposed using steel cables embedded or attached to columns and hanging the cables at the top of building by hat braced frame [12]. Horizontal segmentation and vertical segmentation of high rise buildings with shock absorbing zones was proposed to eliminate the transfer of collapse between segments in addition to the proper design of primary system, secondary system and building envelope [6]. Seismically designed buildings were reported to have inherent ability to resist progressive collapse compared to buildings designed only for vertical loads [15]. Columns are found to be always safe and beams only need additional reinforcement.

In this paper, the effects of considering soil-structure-interaction on the progressive collapse resistance of buildings are investigated. Soil-structure-interaction is incorporated using the uncoupled linear spring model suggested by FEMA 273 into the buildings model. The alternate path approach is applied to three-, and seven-story building examples designed according to the most common building design practice. The removal of edge column is applied at different stories as per the UFC 2009 as it is the most probable and critical scenario. Linear static and nonlinear static methods are used and three cases of foundation soil representing fixed base, hard soil and soft soil are investigated. Comparison of results such as the shear forces, bending moments, point displacements, and the formation of plastic hinges are compared for the studied cases.

2. MATHEMATICAL MODEL AND METHODOLOGY

2.1. Alternate Path Method:

The alternate path method is directly related to redundancy; which is defined as the availability of multiple load-carrying components or multiple load paths which can bear additional loads in the event of a failure. If one or more components fail, the remaining structure is able to redistribute the loads and thus prevent a failure of the entire system [2]. A column is removed from locations specified by regulations to represent inner, edge, and corner columns. For each plan location defined for element removal, the AP analyses is suggested to be applied for the cases of; (1) First story above grade, (2) Story directly below roof, (3) Story at mid-height, and (4) Story above the location of a column splice or change in column size. For all the three recommended analysis types (LS, NS, and ND), the building is structurally adequate if none of the elements, components, or connections exceeds the acceptance criteria listed in the guidelines [10]. The main load combination used for progressive collapse analysis using the alternate path method is defined as:

$$\Omega(1.2\,D + 0.5\,L) \qquad (2)$$

where

Ω is the load increase factor considered here as 2 for those bays immediately adjacent to the removed element and at all floors above the removed element and 1 for other positions.

D, L are the dead and live loads of floors

2.1.1. Linear Static Analysis:

In linear static analysis procedure, analysis is carried out assuming linear elastic behavior and the values of member forces (Shear and moment) are compared with the section capacities as instructed in regulations. As shear failure is categorized as brittle failure leading to catastrophic sudden failure [12], Shear capacity check is performed on the linear static analysis to avoid such failure and certify ductile behavior in form of plastic hinge formation. Shear capacity of concrete section can be estimated as per the ACI 318-08 [20] as:

$$V_n = V_c + V_s \qquad (3)$$

Where V_n is the nominal shear strength of the section, V_c is the nominal shear strength provided by concrete, and V_s is the nominal shear strength provided by reinforcement defined as:

$$V_c = \frac{\sqrt{f_c^-}}{6} b_w\, d \qquad (4)$$

$$V_s = \frac{A_v\, f_{yt}\, d}{s} \qquad (5)$$

$$V_n \le \frac{2\sqrt{f_c^-}}{3} b_w\, d \qquad (6)$$

Where f_c^- is the characteristic concrete strength, b_w, d are the breadth and depth of beam web, f_{yt} is the yield strength of transverse reinforcement, A_v, s are the area and spacing of transverse reinforcement. Moment capacity is calculated here as:

$$M_n = A_s\ f_y\left(d - \frac{a}{2}\right) + A_{\bar{s}}\ f_y\left(\frac{a}{2} - \bar{d}\right) \qquad (7)$$

Where A_s, $A_{\bar{s}}$ are the area of tension and compression reinforcement, f_y is the yield strength of reinforcement $d, \bar{d}$ are the depth of tension and compression reinforcement, a is the compression zone depth obtained from section equilibrium. The acceptance criteria for linear static analysis according to the UFC [10] can be defined as:

$$\emptyset\, m\, Q_{CE} \ge Q_{UD} \qquad (8)$$

Where Q_{ud} is the shear or Moment from Linear Static model, m is the component or element demand modifier (m-factor) as defined in [10], Φ is the strength reduction factor from the design code [20], and Q_{CE} is the expected strength of the element as described above

2.1.2. Nonlinear Static Analysis:

In the nonlinear static analysis, loads are applied using a load history that starts at zero and is increased to the final load values. Modeling parameters and acceptance criteria of reinforced concrete are applied according to the UFC, 2009 [10]. Plastic hinges are allowed to form along the members. These hinges are based on maximum moment values calculated using the UFC parameters. However, only flexural moments can cause a plastic hinge to form in beam members, and only the axial-moment interaction (PMM) can cause a plastic hinge to form in a column. Hinges are allowed to occur at the ends of each member and at the mid-span of the flexural members. Moment-rotation curves for beam hinges are extracted from UFC's table 4.1 as shown in Fig.1. Values of a, b and c as .05, .06 and 0.2, respectively with the immediate Occupancy and Life Safety adopted at 0.0125 and 0.025, respectively. The acceptance criteria stated that the rotation of formed plastic hinge does not exceed 0.05 (collapse prevention state). As reported before, nonlinear static analysis is considered conservative in the estimation of total number of plastic hinges and shear force in beams [3]. The building model was performed using the ETABS software at which the plastic hinge properties were incorporated and nonlinear push-over analysis was carried out.

2.2. Soil-Structure Interaction Model:

Uncoupled spring model is applied for the building soil interaction as recommended by the FEMA 273, NEHRP [21] and shown in Fig.2. Linear elastic supports are assumed to represent the building footings for which the spring coefficients are calculated as follows

$$K_x = K_y = \frac{8\,G\,R}{2-\upsilon} \qquad (9)$$

$$K_z = \frac{4\,G\,R}{1-\upsilon} \qquad (10)$$

$$K_{xx} = K_{yy} = \frac{8\,G\,R^3}{3(1-\upsilon)} \qquad (11)$$

$$K_{zz} = \frac{16\,G\,R^3}{3} \qquad (12)$$

Where

K_x, K_y and K_z are the spring coefficients in translation along the x, y, and z directions.

K_{xx}, K_{yy} and K_{zz} are the spring coefficients in rotation about the x, y, and z directions.

G is the foundation soil shear modulus

υ is the Poisson ratio of foundation soil

R is the equivalent radius of rectangular footing

The equivalent radius of rectangular / square footing and the correction of coefficients due to footing shape are calculated based on the recommendations of FEMA 273 [21] while embedment depth is ignored. Two soil types are used in the study to represent soft and hard soils for which the shear moduli are estimated using the shear wave velocity of the top 30 m taken as 100 and 200 m/sec for the soft and hard soils, respectively [22]

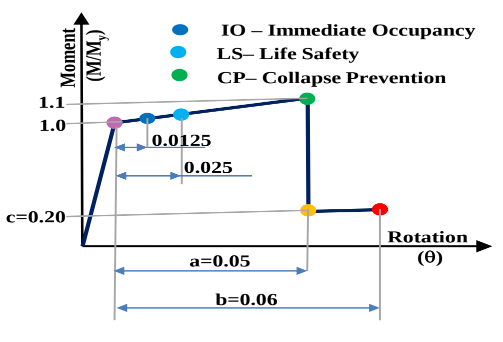

Fig.1. Moment curvature properties of plastic hinge

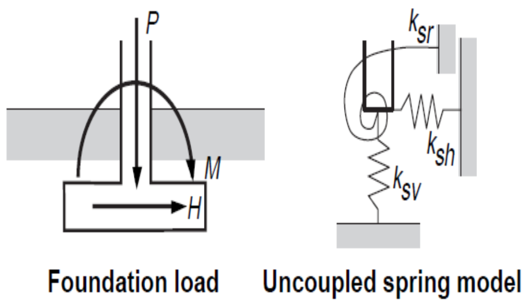

Fig.2. Uncoupled Spring Model [21]

2.3. Building Examples:

Two building examples were selected with three- and Seven-story reinforced concrete buildings. The structural system of building examples is composed of concrete slabs with drop beams rested on concrete columns. For the seven-story building, to core wall was added for lateral load resistance while the three-story building was designed without seismic load consideration as common for buildings constructed before the later update of building codes. All studied cases are founded on isolated reinforced concrete footings. The plan of the example buildings are shown in Fig.3 and the dimensions of the structural elements are listed in Table 1. Story height is considered as four meters and live loads as 2 kN/m^2 as typical residential building. C30 concrete having 24 MPa characteristic strength and reinforcement with yield strength 415 MPa are used

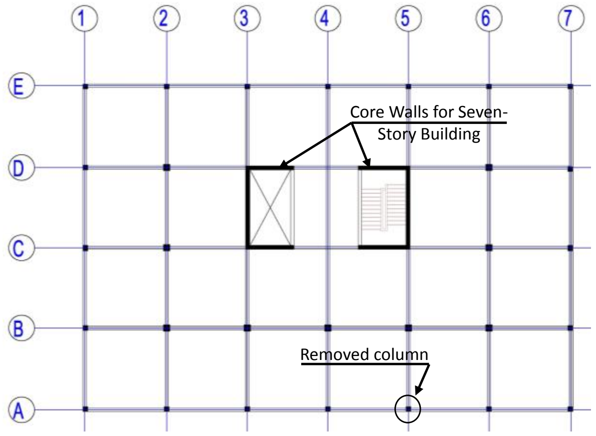

Fig.3. Plan of example buildings

Table.1 Information of Building Examples

Member	Three-Story Building		Seven-Story Building	
	Dimensions	Reinforcement	Dimensions	Reinforcement
Interior Column	400 × 400	12 ϕ 16	400 × 800	20 ϕ 20
Edge & Corner Column	300 × 300	8 ϕ 20	400 × 400	12 ϕ 16
Beam	200 × 500	4 ϕ 16 Top & Bott.	200 × 500	4 ϕ 20 Top & Bott.
Core Wall	NA#	NA#	250 mm thk.	ϕ 16 @150 mm
Internal Footing	3500 × 3500##	NA###	4500 × 4500##	NA###
Edge Footing	2400 × 2400##	NA###	3000 × 3000##	NA###
External Footing	1700 × 1700##	NA###	2000 × 2000##	NA###

Not applied to this example - ## Thickness of footing has no effect - ### has no effect in the model

For the examples considered, exterior column is omitted at different levels according to design guidelines. This selection was made as exterior bays are most vulnerable to damage, particularly for buildings that are close to public streets. They are also less capable of redistributing loads in the event of member loss, because full two way load distribution is not possible.

3. RESULTS AND DISCUSSIONS

3.1. Results of the Linear Static Analysis:

The linear static analysis procedure is applied to the pre-defined examples and its results are discussed in this section. For each example, the edge column marked in the plan at Fig. 3 is omitted at the ground floor, the middle floor and below the roof floor as specified. Table 2 shows the maximum beam shear force ratios for different examples and cases normalized with respect to the shear capacity of the beam section ($\emptyset\ Q_{CE}$). For each example, the maximum beam shear ratios and the floor at which it takes place are listed for the design case with all columns exist and for the cases of ground, middle and roof column omitted. The same values are shown for the case of buildings supported on fixed base, hard soil, and soft soil.

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As can be observed from the table, for the three-story building, the maximum beam shear is in the range of 1.33 to 1.45which is within the acceptance criteria. The maximum beam shear for all types of soil is located at the second floor for the first and second floor column omitted and in the roof floor for the case of removing the last floor column. For the same column removal location, the maximum beam shear ratio is slightly decreased as the foundation soil stiffness decreased such that values observed are 1.4553, 1.4510, and 1.4273 for the cases of fixed base, hard soil, and soft soil, respectively, in case of the ground floor column removal. For the cases of seven-story building, similar results are observed for the case of fixed base and hard soil with slightly increased beam shear ratios. For the case of soft soil, the maximum beam shear ratio recorded higher values reaching 2.2 for the case of ground floor column removal. For the case of the seven-story building on soft soil, the maximum beam shear is observed to be located on the ground floor, i.e. tie beams; that means that they are directly affected by the differential settlement between the omitted column and the column adjacent to it as shown in Fig.4. Fig.4 shows the deformed shape of the seven-story building in case of the ground floor column removal for different cases of foundation soil. Column removal affects the deflection for beams above significantly compared to the original design. As also observed from the figure, tie beam deformation is very significant in case of soft soil and has less effect in case of hard soil, no effect in case of fixed base. This is attributed to the increase of reaction of columns adjacent to the removed column and the cancelation of the removed column reaction leading to high difference in foundation reaction and consequently higher values of differential settlement based on the soil elasticity.

Fig.5 illustrates the effect of soil type on the deflection values at the column removal point for the three- and seven-story buildings. As shown, the deflection at that point has a very small value in design as there is a column below the point and as the column is removed, the deflection values increase too much. This increase is observed to be more for elastic soil and increase with decreasing the soil stiffness. The peak deflection in case of soft soil is approximately 1.53, and 1.88 times that in case of fixed base for the three-, and seven-story buildings, respectively. The results for design, fixed base, and hard soil is similar between the three- and seven-story cases, while relatively different in case of soft soil for which the seven story building undergoes more deflection.

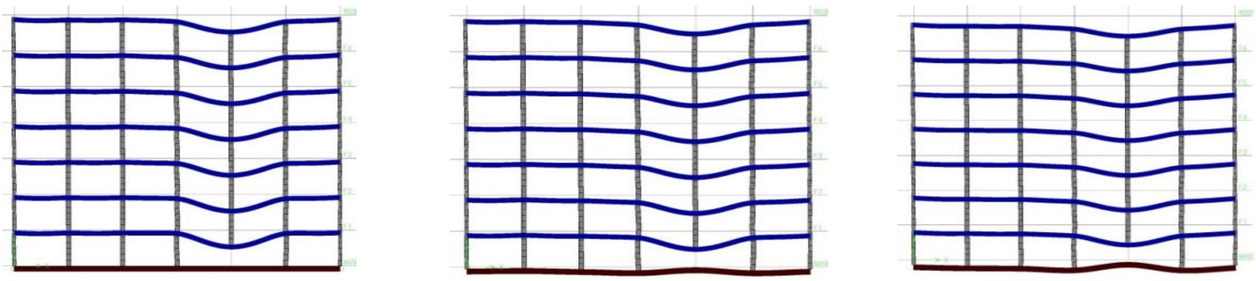
Table.2 Maximum Beam Shear Force Ratios of the Building Examples, Linear Static Analysis

Building	Case	Column Omitted					
		Ground		Middle		Roof	
Three-Story	Fixed Base	1.4553	Floor 2	1.4312	Floor 2	1.3573	Roof
	Hard Soil	1.4510	Floor 2	1.4249	Floor 2	1.3518	Roof
	Soft Soil	1.4273	Floor 2	1.3980	Floor 2	1.3331	Roof
Seven-Story	Fixed Base	1.5595	Floor 2	1.5160	Floor 5	1.4055	Roof
	Hard Soil	1.5466	Floor 2	1.5030	Floor 5	1.3984	Roof
	Soft Soil	2.2043	Ground	2.1396	Ground	2.1072	Ground

Table 3 shows the maximum beam moment ratios for different examples and cases normalized with respect to the moment capacity of the beam section ($\emptyset\ Q_{CE}$). For the three-story building, omitting the ground floor column results in maximum moment in the second floor beams with moment ratios between 2.43 and 2.34 which decrease with increasing the soil flexibility. Less moment ratios are observed for omitting the first (middle) floor with the maximum being at the second floor. In case of roof floor removal, the moment ratio decrease also and the maximum moment is located at the roof floor. This is attributed to the increased load transmitted to the other columns leading to high foundation settlement and consequently higher moment ratios. Many differences are observed as the number of story increases. The first observation is that in case of soft soil, the maximum moment for the seven-story building takes place at the ground level caused by foundation differential settlement. the values of maximum moment ratios are also increased for soft soil case such that they reach 3.0253, 2.8224, and 2.7706 for the removal of the ground, mid (fourth), and roof floor, respectively.

Table.3 Maximum Bending Moment Ratios of the Building Examples, Linear Static Analysis

Building	Case	Column Omitted					
		Ground		Middle		Roof	
Three-Story	Fixed Base	2.4283	Floor 2	2.3918	Floor 2	2.1453	Roof
	Hard Soil	2.4138	Floor 2	2.3682	Floor 2	2.1178	Roof
	Soft Soil	2.3379	Floor 2	2.2715	Floor 2	2.0231	Roof
Seven-Story	Fixed Base	1.5657	Floor 2	1.4712	Floor 5	1.3499	Roof
	Hard Soil	1.5391	Floor 2	1.4476	Floor 5	1.3263	Roof
	Soft Soil	3.0253	Ground	2.8224	Ground	2.7706	Ground



a. Fixed Base **b. Hard Soil** **c. Soft Soil**

Fig.4 Deformed Shape of The Seven Story Building

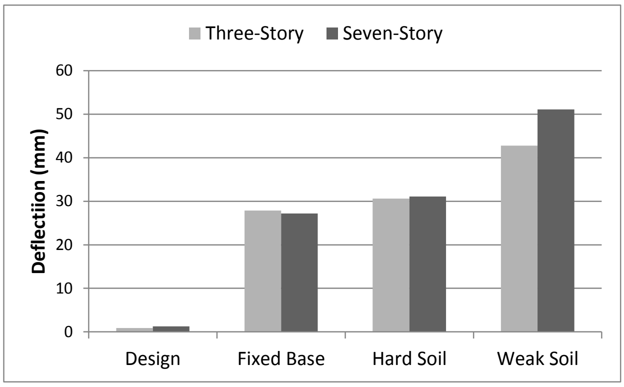


Fig.5. Deflection at point of column removal – Linear Static Analysis

3.2. Results of the Nonlinear Static Analysis:

To illustrate the effect of column removal on bending moments, the bending moment diagrams for the first load step of the nonlinear analysis are shown in Fig.6 for the case of fixed base. The first load step is selected as at this stage, no plastic hinges are formed and no redistribution of moments takes place. The huge increase of positive moments at the missed column location in the elevation view and negative moments at beam ends in the cross section can be easily observed. As a result of the increased moment values, the formation of plastic hinges is expected to take place. The distribution of plastic hinges at the last load step for the case of fixed base is shown in Fig.7 which shows the deformed shapes with plastic hinges in two directions through the removed column. As shown, plastic hinges are formed in both directions indicating the increase of beam moment resulting from bridging the loads above the missed column. All hinges are in the life safety to collapse prevention (CP-LS) range except one critical hinge exceeding the collapse prevention state. This critical hinge is located at the second floor in the location of maximum negative moment through the transverse section passing through the missing column.

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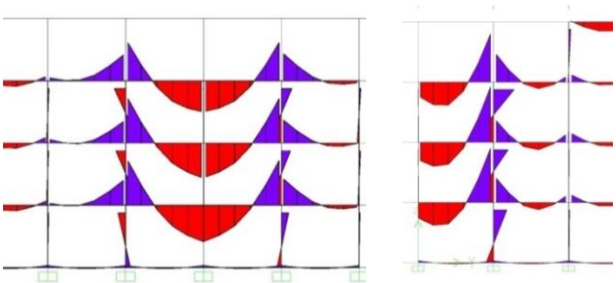
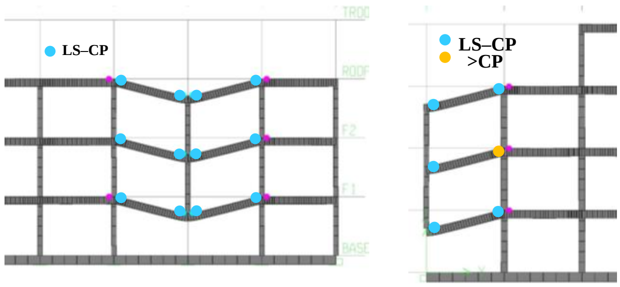
Shown in Fig.8 is the load deflection relationship for the three-story building with the ground floor edge column omitted for different cases of foundation conditions. The total load ratio is defined here as the total vertical load of the progressive collapse nonlinear static case at each step normalized with respect to the corresponding total load of the linear static case defined by Eq.1. Nonlinear static displacement at the point of column omission is normalized also with respect to the static linear deflection at the same point. The bi-linear behavior of the building is clear from the plot such that for all cases, the relation is approximately linear until specific point after which the slope decreases up to the analysis termination due to numerical divergence. The slope is almost the same for the cases of fixed base and hard soil while for soft soil, the slope is slightly decreased which can be attributed to the increased deflections due to footing settlement in case of soft soil. The nonlinear static analysis converges to different load levels and to different displacement values for different base conditions. Fixed base case and hard soil cases converge up to almost the same value of 83% of the total progressive collapse load case with nonlinear deflection approximately 5.5 times the linear static deflections. The case of building on soft soil cannot resist beyond 68% of the same load case and developed nonlinear deflection about 3.85 times the linear static analysis. This indicates that stiffer base conditions lead to more resistance to progressive collapse and more ductile behavior in case of ground floor removal. The numbers of plastic hinges formed at the three-story example building during the last load step of the nonlinear static analysis are listed in Table 4 for all cases of support conditions with the edge ground column removed. To satisfy the acceptance criteria according to regulations, no plastic hinges to exceed the collapse prevention point. In case of ground floor removal, the total number of plastic hinges is the same for all base conditions. The fixed base and hard soil cases produced plastic hinges beyond the collapse prevention stage while the case of soft soil fails before any plastic hinge exceeds the collapse prevention stage.



a. Longitudinal Section **b. Cross Section**

Fig.6. Three-Story Building Moment for Ground-Floor Column Removal, Fixed Base



a. Longitudinal Section **b. Cross Section**

Fig.7. Plastic Hinge Formation in the Three-Story Building for Edge Ground-Floor Column Removal, Fixed Base

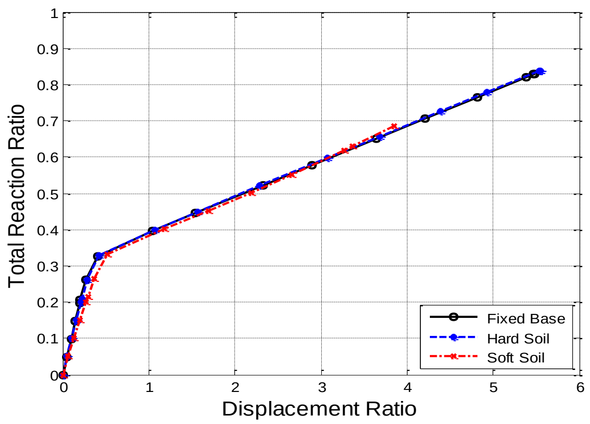
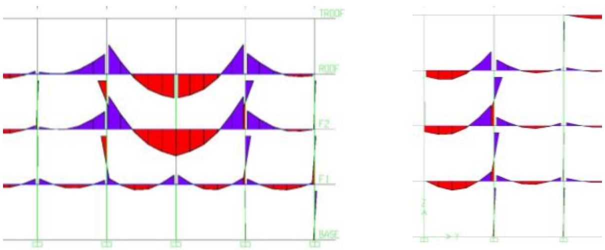


Fig.8 Total Load–Deflection relation for Edge Ground-Floor Column Removal of the Three-Story Building

Table.4 Distribution of Plastic Hinges during the Last Load Step for Edge Ground-Floor Column Removal of the Three-Story Building

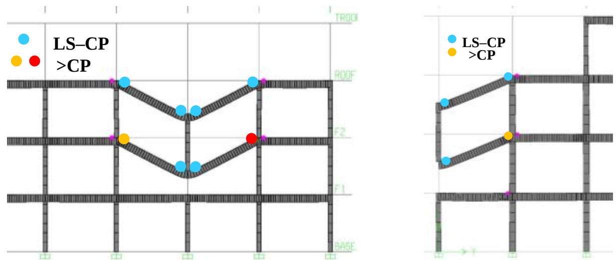
	IO : LS	LS : CP	> CP
Fixed Base	---	17	1
Hard Soil	---	15	3
Soft Soil	---	18	---

For the omission of mid-floor (first-floor) column, the same results are shown and discussed. The bending moment diagrams for the first time step of the fixed base case shown in Fig.9 illustrate that the effect of removing the column affects only floors above such column in both directions. The second and roof floor beams are subjected to large increase of positive moment at the point of column removal while the cantilever action enlarges the negative at the other direction in floors above the first. These amplified moments result in the formation of plastic hinges shown in Fig.10. Two plastic hinges exceed the acceptance criteria (CP) at location of extreme positive moments in the longitudinal direction and one plastic hinge exceeds such criteria at the location of extreme negative moment along the cross section.



a. Longitudinal Section b. Cross Section

Fig.9 Three-Story Building Moment for Mid-Floor Column Removal, Fixed Base



a. Longitudinal Section b. Cross Section

Fig.10 Plastic Hinge Formation in the Three-Story Building for Edge Mid-Floor Column Removal, Fixed Base

The total load-deflection ratio relationship for the three-story building with the mid-floor edge column omitted for different cases of foundation conditions is shown in Fig.11. Similar behavior is observed for all soil conditions especially for the first part of the plot during the early load steps with slightly more slope for hard soil and fixed base conditions which is logic according to the increased base stiffness of these cases. For the second part of the plot and for later load steps, the behavior becomes different such that the case of soft soil shows higher slope and reaches higher level of total load ratio. It reaches about 96% total load ratio at displacement ratio of 5.75 while the cases of fixed base and hard soil reach the same value of total load ratio of 84% at displacement ratio 5.6. These results can be attributed to the existence of one story below the removed column capable of distributing the load on the base in good manner and the flexible base works as elastic support absorbing such load differences. This indicates that stiffer base conditions may not be better in case of upper floor removal for such low rise construction. The numbers of plastic hinges formed at the three-story example building during the last load step of the nonlinear static analysis are listed in Table 5 for all cases of support conditions with the edge mid-floor column removed. The number of plastic hinges verifies the above-mentioned behavior such that less plastic hinges exceed the acceptance criteria in case of soft soil conditions. This certifies that the case of flexible bases is closer to satisfy the acceptance criteria with more load resistance.

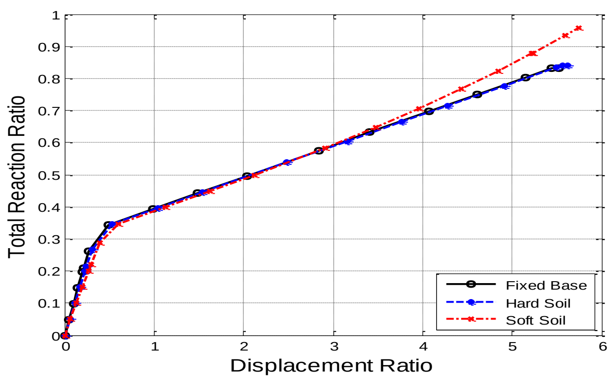


Fig.11Total Load – Deflection relation for Edge Mid-Floor Column Removal of the Three-Story Building

Table.5 Distribution of Plastic Hinge during the Last Load Step for Edge Mid-Floor Columns Removal of the Three stories Building

	IO : LS	LS : CP	> CP
Fixed Base	1	10	3
Hard Soil	---	9	3
Soft Soil	---	11	1

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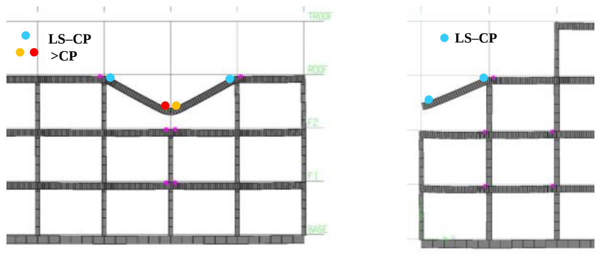
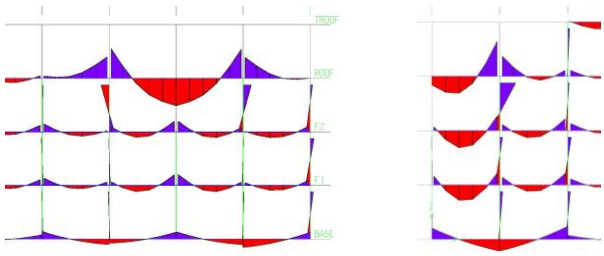
Similar results can be observed for the omission of uppermost-floor (Second floor) column. Fig.12 shows the bending moment diagrams for the first time step of the soft soil base case as sample diagram. The effect of removing the column affects the roof floor which is supported by that column in both directions. All members below the level of the removed column are not affected except slight increase of ground beam moments in the spans besides the removed column. Plastic hinges are also limited to the roof floor as shown in Fig.13 such that two plastic hinges exceed the acceptance criteria (CP) at location of extreme positive moments in the longitudinal direction.



a. Longitudinal Section **b. Cross Section**

Fig.13 Plastic Hinge Formation in the Three-Story Building for Edge Roof-Floor Column Removal, Soft Soil Conditions



a. Longitudinal Section **b. Cross Section**

Fig.12 Three-Story Building Moment for Roof-Floor Column Removal, Hard Soil Conditions

The load-deflection relationship for the three-story building with the uppermost-floor edge column omitted for different cases of foundation conditions is shown in Fig.14. The response of building to the removal of the roof-floor column is almost the same as that for the removal of mid-floor column discussed above. The building with fixed base and hard soil condition fails at total load ratio of 83% and deflection ratio of 5.7 while the building in case of soft soil conditions survived to 96% total load ratio and deformed up to 5.9 times the elastic displacement. This can be attributed to the flexible nature of elastic support leading to more ductile behavior and the little effect of column removal on the lower part of the building. The numbers of plastic hinges formed at the three-story example building during the last load step of the nonlinear static analysis are listed in Table 6 for all cases of support conditions with the edge roof-floor column removed. The number of plastic hinges verifies the above-mentioned behavior such that for more flexible base, less plastic hinges exceed the acceptance criteria are developed. This indicates that the flexible base buildings are can resist better the collapse of roof floor column.

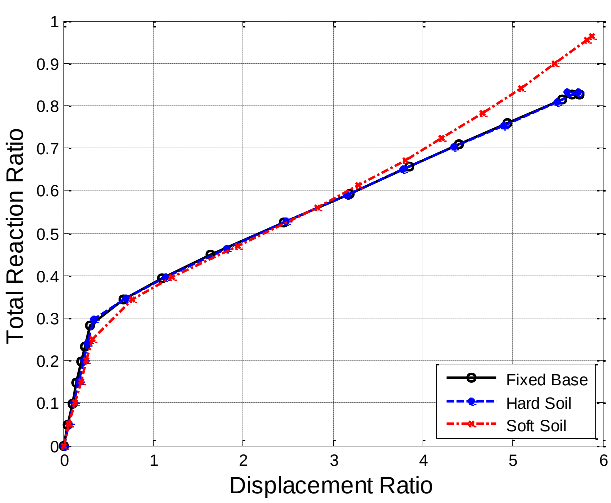


Fig.14 Total Load – Deflection relation for Edge Roof-Floor Column Removal of the Three-Story Building

Table.6 Distribution of Plastic Hinge during the Last Load Step for Edge Roof-Floor Column Removal of the Three-Story Building

	IO : LS	LS : CP	> CP
Fixed Base	---	1	5
Hard Soil	---	3	3
Soft Soil	---	4	2

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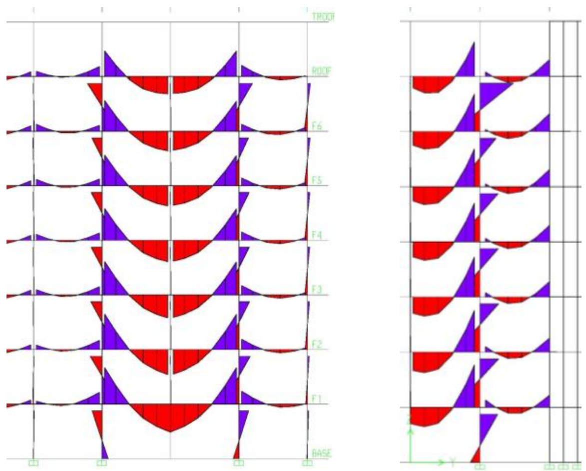
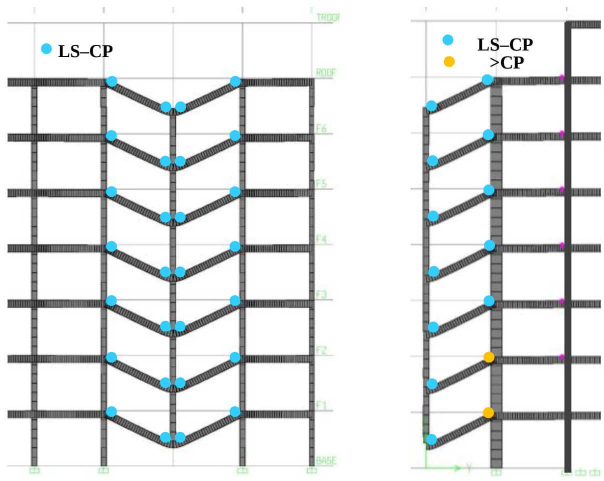
The same investigations were carried out for the seven-story example representing relatively higher building. As shown in Fig.15, removing the ground floor edge column leads to the increase of moment values for all beams connected to the removed column and above the ground floor level. Positive moments at the location of removed column and negative moments due to cantilever action are more for lower floors and decrease for higher floors with the peak values at the first floor beams. Plastic hinges indicate the same as shown in Fig.16 such that all plastic hinges formed at the last load step are in the life safety to collapse prevention (LS-CP) except tow hinges exceed the CP stage. These two hinges are located at the position of maximum negative (cantilever) moment at the first and second floors.



a. Longitudinal Section **b. Cross Section**

Fig.15. Seven-Story Building Moment for Ground-Floor Column Removal, Fixed Base



a. Longitudinal Section **b. Cross Section**

Fig.16. Plastic Hinge Formation in the seven-Story Building for Edge Ground-Floor Column Removal, Fixed Base

The load deflection relationships for the seven-story building with the ground floor edge column omitted for different cases of foundation conditions are shown in Fig.17. The effect of base conditions on the building behavior is more clear here as the number of stories are more and the consequently, more loads are transmitted to foundation. The building with fixed base conditions survived to more loads and developed more deflections and the increase of foundation flexibility leads to failure in earlier stages. Building with fixed base, hard soil base, and soft soil base fails at 79%, 64%, and 33%, respectively, of the total progressive collapse load combination. The deflection ratios observed are 5.03, 3.6, and 0.59 for the cases of fixed base, hard soil, and soft soil, respectively, indicating dramatic decrease of ductility for more flexible base conditions. The numbers of plastic hinges formed at the seven-story example building during the last load step of the nonlinear static analysis and listed in Table 7 indicate the same. The brittle failure in case of soft soil conditions happened early before the formation of any plastic hinge while many plastic hinges are formed in cases of fixed base and hard soil conditions.

For removing the mid-floor (third-floor) edge column, the increase of moments can be observed in floors above that level as shown in Fig.18. The highest values of these moments are located at the beams of the floor directly above the removed column (fourth-floor) and are less for upper floors. As the figure shows the bending moment in case of soft soil conditions, the ground beam moments are shown with relative increase of moments at bays connected to the removed column. Removing the mid-floor column for the seven-story building in case of soft soil conditions leads to the formation of too many plastic hinges below the immediate occupancy (IO) state as shown in Fig.19. Only one plastic hinge exceeded the collapse prevention (CP) state after which the failure happened located at the position of maximum negative moment directly above column removal.

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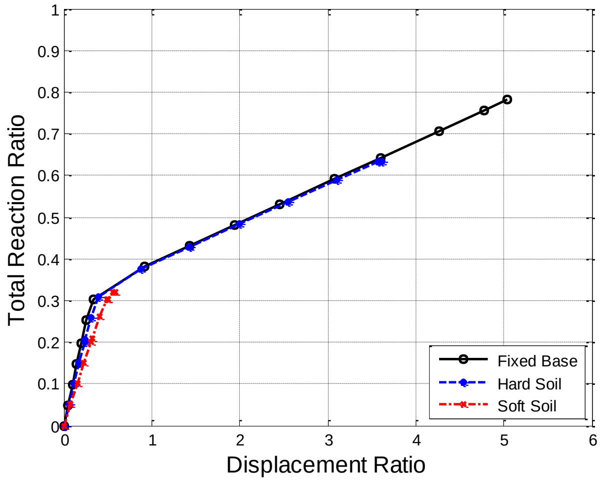
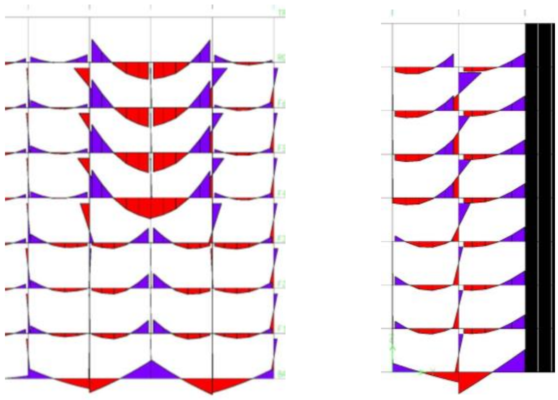


Fig.17 Total Load – Deflection relation for Edge Ground-Floor Column Removal of the Seven-Story Building

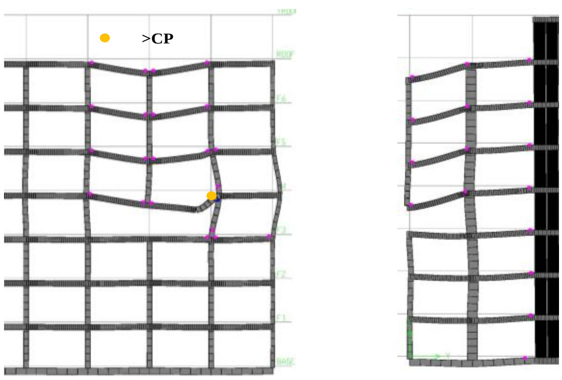
Table.7 Distribution of Plastic Hinges during the Last Load Step for Edge Ground-Floor Column Removal of the Seven-Story Building

	IO : LS	LS : CP	> CP
Fixed Base	---	40	2
Hard Soil	--	43	1
Soft Soil	---	---	---



a. Longitudinal Section **b. Cross Section**

Fig.18. Seven-Story Building Moment for Mid-Floor Column Removal, Soft Soil Conditions



a. Longitudinal Section **b. Cross Section**

Fig.19. Plastic Hinge Formation in the seven-Story Building for Edge Mid-Floor Column Removal, Soft Soil Conditions

Fig.20 shows relatively different behavior of the total load-deflection curve for case of mid-floor column removal of the seven-story building example. The cases of fixed base and hard soil conditions are similar in load-deflection behavior such that they survived to 79% of the total load and developed a deflection ratio of 5.1. The case of soft soil conditions is different such that the slope (stiffness) is less than the previously-mentioned cases and the building failed at load ratio of 33% with deflection ratio of 1.2 which also indicates brittle behavior for this case. The total number of plastic hinges for the cases of fixed base and hard soil conditions are the same (24 hinges) as listed in Table 8. The only difference is that in case of fixed base conditions, 4 of them exceed the collapse prevention (CP) while none of them exceeds CP in case of hard soil. In case of soft soil conditions, only one plastic hinge beyond the CP state was developed before failure.

Figs.21, 22, 23, and Table 9 illustrate the same for removing the roof-floor edge column of the seven-story example building. As shown in Fig.21, the roof floor beams are affected by the removal of roof floor column while no effect is observed for lower floors. The development of plastic hinge follows the increase of bending moment and takes place only in the roof floor as shown in Fig.22. Critical plastic hinges (>CP) are formed in the negative moment location while other hinges (LS-CP) are formed in the positive moment location at beams connected to the removed column.

Roof-floor column removal of the seven-story building example shows similar behavior for load-deflection relation as shown in Fig.23 which is also similar to their response to removing mid-floor column. Ductile behavior is observed for the fixed base and hard soil conditions up to about 80% of the total load and deflection ratio equals 5.5 before failure. The case of soft soil conditions is changed slightly in load and considerably in deflection such that failure takes place at load ratio of 34% with deflection ratio of 3.4. Little number of plastic hinges was observed in case of roof floor column removal. This can be due to the limited area affected by such removal (only 3 beams) which leads to sudden local failure.

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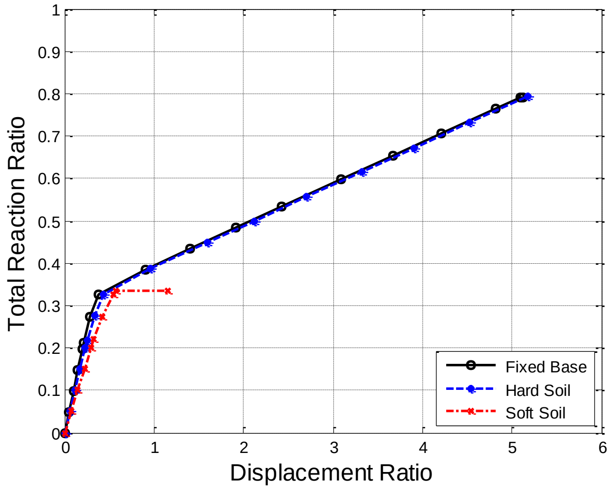
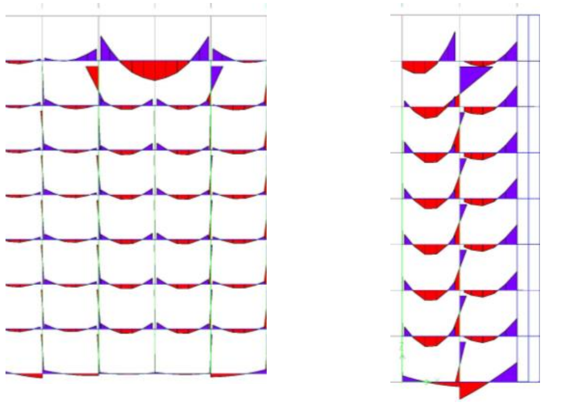


Fig.20 Total Load – Deflection relation for Edge Mid-Floor Column Removal of the Seven-Story Building

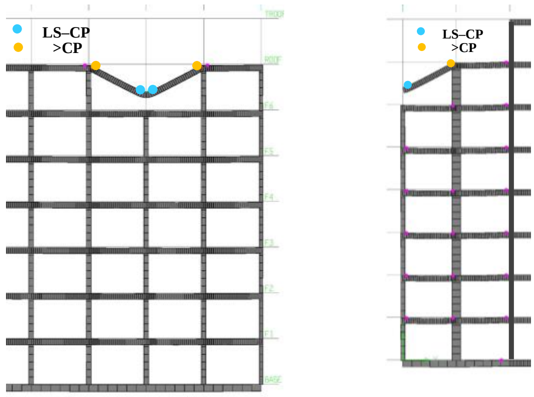
Table 8 Distribution of Plastic Hinge during the Last Load Step for Edge Mid-Floor Column Removal of the Seven-Story Building

	IO : LS	LS : CP	> CP
Fixed Base	---	20	4
Hard Soil	---	24	---
Soft Soil	---	---	1



a. Longitudinal Section **b. Cross Section**

Fig.21. Seven-Story Building Moment for Roof-Floor Column Removal, Hard Soil Conditions



a. Longitudinal Section **b. Cross Section**

Fig.22 Plastic Hinge Formation in the seven-Story Building for Edge Roof-Floor Column Removal, Hard Soil Conditions

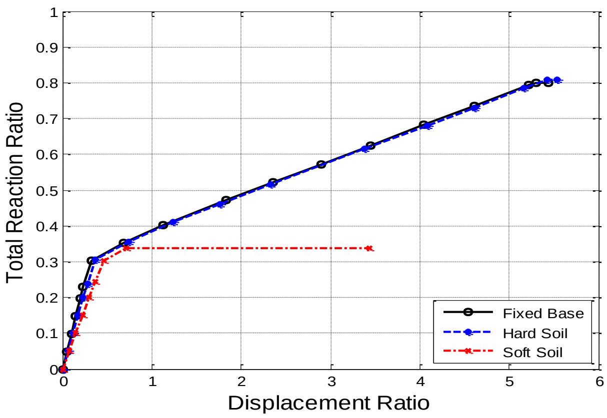


Fig.23 Total Load – Deflection relation for Edge Roof-Floor Column Removal of the Seven-Story Building

Table.9 Distribution of Plastic Hinge during the Last Load Step for Edge Roof-Floor Column Removal of the Seven-Story Building

	IO : LS	LS : CP	> CP
Fixed Base	---	3	3
Hard Soil	---	3	3
Soft Soil	---	3	1

4. CONCLUSIONS

The effect of soil-structure interaction on the capacity of reinforced concrete buildings to resist progressive collapse was investigated. The alternate path method as recommended by UFC-2009 was applied to three- and seven-story building examples with edge column removed at ground-, mid-, and roof-floor as per guidelines. Linear and nonlinear static analysis procedures were applied for which the simplified uncoupled linear spring model proposed by UFC 273 was customized for soil-structure-interaction. The main conclusions of the study are as follows:

- For the three-story building, maximum beam shear and moment resulted from linear static analysis are more for fixed base and decrease for the cases of hard soil and soft soil for all cases of column removal. This indicates flexible behavior and more resistance of progressive collapse in case of flexible base.
- On the other hands, the linear static behavior of the seven-story building differs such that , for all cases of column removal, ,beam shear and moment for the case of hard soil conditions is less than that for the fixed base conditions. The case of soft soil conditions produces relatively high values of shear and moment compared to the other two cases. This indicates that for more floors, relatively flexible foundation conditions are better but extremely flexible conditions lead to reducing the resistance to progressive collapse.
- In case of nonlinear static analysis, for ground floor removal, fixed base conditions show better behavior in terms of ductility and plastic hinge formation for the three- and seven-story buildings.
- For upper floors removal (mid or roof), soft soil conditions slightly enhance the behavior for the three-story building while significantly produce not recommended brittle failure in case of more floors (seven-story building)
- The effect of changing base conditions decreases as the removed columns become on upper floors for all cases.

REFERENCES

[1] Kokot S., Anthoine A., Negro P. and Solomos G, Static and Dynamic Analysis Of A Reinforced Concrete Flat Slab Frame Building For Progressive Collapse, JRC 62663, European Commission, Joint Research Centre, 2010.

[2] Kokot S, Literature Survey on Current Methodologies of Assessment of Building Robustness and Avoidance of Progressive Collapse, JRC 55989, European Commission, Joint Research Centre, 2009.

[3] Khazaee A, Nonlinear Static and Dynamic Analysis for Assessment of Progressive Collapse Potential in intermediate RC frame structures, I.J. of Advances in Engineering Sciences 2013; Vol.3, Issue 4.

[4] Bing-bing Tu, Dong Zhao, Judgment of Key Components during Progressive Collapse. EJGE, 19; PP: 195-204, 2014

[5] FEMA, World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations, FEMA 403, 2002.

[6] Starossek U, Avoiding disproportionate collapse of tall buildings. Structural Engg. International, 3:238–245, 2008.

[7] Ellingwood B. R, Load and Resistance Factor Criteria for Progressive Collapse Design. Proceedings of the Workshop on Prevention of Progressive Collapse, Multi-hazard Mitigation, Council of the National Institute of Building Sciences, Rosemont,2002,

[8] Ellingwood B. R et al, Best Practices for Reducing the Potential for Progressive Collapse in Buildings, National Institute of Standards and Technology, U.S. Department of Commerce, 2007.

[9] Stevens D., Martin E, Williamson E, McKay A. and Marchand, RECENT DEVELOPMENTS IN PROGRESSIVE COLLAPSE DESIGN, Protection Engineering Consultants, San Antonio, Texas, 2013

[10] UNIFIED FACILITIES CRITERIA (UFC), Design Of Buildings To Resist Progressive Collapse, Department of Defense (DOD), 2009.

[11] American Society of Civil Engineers (ASCE), Seismic Rehabilitation of Existing Buildings, ASCE 41-06, Reston, Virginia, 2006.

[12] Alrudaini T. M. S, A New Mitigation Scheme to Resist Progressive Collapse of Reinforced Concrete Buildings. Doctor Thesis, University of Wollongong, 2011

[13] Iribarren, B. S, Progressive Collapse Simulation of Reinforced Concrete Structures: Influence of Design and Material Parameters and Investigation of the Strain Rate Effects. Ph.D. Thesis, Université Libre de Bruxelles, 2011.

[14] Mwafy A. M., Elnashai A.S, Static pushover versus dynamic collapse analysis of RC buildings, Engineering Structures,23, PP: 407–424, 2001

[15] Rakshith K G, Radhakrishna, Progressive Collapse Analysis Of Reinforced Concrete Framed Structure, IJRET: I.J. of Research in Engg. and Technology, PP:32-40, 2013

[16] Gururaja B., and Sridhar R, Progressive Collapse Potential of Irregular Concrete Building. IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), 67; pp: 320-34, 2014.

[17] Li Y., Lu X., Guan H. and Ye L, An improved tie force method for progressive collapse resistance design of reinforced concrete frame structures. Engineering Structures, 33, PP: 2931–2942, 2011.

[18] Shi Y. Li Z. Hao H, A new method for progressive collapse analysis of RC frames under blast loading, Engineering Structures, 32, PP:1691_1703, 2010.

[19] Tsai M. You Z, Experimental evaluation of inelastic dynamic amplification factors for progressive collapse analysis under sudden support loss, Mechanics Research Communications, 40,PP: 56– 62, 2012.

[20] ACI Committee, Building code requirements for structural concrete (ACI 318-08) and commentary, American Concrete Institute, 2008.

[21] BUILDING SEISMIC SAFETY COUNCIL, NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS (FEMA Publication 273), FEDERAL EMERGENCY MANAGEMENT AGENCY, 1997.

[22] Wair B.R, DeJong J.T, Shantz T, Guidelines for Estimation of Shear Wave Velocity Profiles, PEER Report 2012/08, Pacific Earthquake Engineering Research Center, 2012.