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RESEARCH

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Evaluation of the Progressive Collapse of the Reinforced Concrete Frames Considering the Soil–Structure Interaction: Parametric Study Based on the Sensitivity Index

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Abstract

In this essay, to investigate the progressive collapse of the reinforced concrete (RC) frames, a nonlinear static push-down analysis was performed with column removal scenarios from the first story. At first, a numerical model was simulated and verified with the experimental model in SeismoStruct software without soil–structure interaction (SSI). Afterward, the foundation, soil, and the RC frame were modeled simultaneously in FLAC software and verified with the numerical model of the SeismoStruct software. Furthermore, the effect of SSI was studied on the progressive collapse of RC frames based on the sensitivity index (SI). The sensitivity index is defined as the ratio of the residual capacity under gravity loading of the structure by removing the column to the value of the undamaged structure. The results showed that by considering SSI, the sensitivity index decreases. Then, a parametric study of the framed structures (thickness of the foundation) and substructures (soil density, soil types, soil layers, and the soil saturation conditions) was performed to evaluate the progressive collapse-resisting capacity based on the sensitivity index. The results showed that by considering SSI, with an increase in the soil density and decrease in the groundwater level, the conditions would be better for preventing progressive collapse. It was also shown that rock and silty sands (SM), compared to other studied soil types, and SM and silty sands—silty clay with low plasticity—silty sands (SM-CL/ML-SM), compared to other studied soil layers, are better for preventing progressive collapse.

Keywords: progressive collapse, soil–structure interaction, substructure, sensitivity index

1 Introduction

All types of buildings can be exposed to severe events caused by, for example, earthquakes, tsunamis, explosions, hurricanes, terrorist attacks or human error. Such events usually cause local damage to the structures. The most serious local damage of a structure occurs when one or several vertical load-bearing components, for example columns or walls fail. Progressive collapse is defined as the spread of an initial local failure from one element to another, which eventually leads to the collapse of the entire structure or a large part of it (Cormie et al., 2009; kheyroddin et al., 2019; Kiakojouri et al., 2021; Starossek, 2017). The importance and necessity of investigating progressive collapse were first recognized after the collapse of the Ronan Point Apartment Tower in England in 1968. Additionally, the collapses of the Skyline Towers Building in Virginia (1973), the Murrah Federal Building in 1995, and the World Trade Center collapse in 2001 are the worldwide known examples of progressive collapses (Farahani et al., 2018; Yi et al., 2021). Nowadays, due to the lack of land in large cities and population growth, high-rise buildings have received a lot of attention and

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their use is inevitable. Since progressive collapse of high-rise buildings is more dangerous to residents and adjacent buildings than low-rise buildings, the progressive collapse of high-rise structures is of great interest.

The nonlinear static analysis is adopted to investigate the structural performance of the reinforced concrete (RC) frames against progressive collapse. The advantage of this procedure is its ability to account for nonlinear effects. It also has the ability to determine elastic and failure limits of the structure. Patel and Shah (2017) expressed that the nonlinear static analysis is found to be the most efficient method for progressive collapse assessment of the reinforced concrete structure with consideration of soil effect. Powell (2005) compared the linear static and nonlinear static analyses and concluded that basically the nonlinear procedure should be used. The phenomenon of progressive collapse is nonlinear in nature. Therefore, it is more reasonable to carry out nonlinear analyses with nonlinear modeling of each element. Marjanishvili (2004) showed that nonlinear static analysis may lead to more demand for ductility. The nonlinear dynamic analysis method is not suitable because of the complexity of the analysis and the extensive computation time. Therefore, nonlinear static analysis is still very relevant in studying the collapse behavior of a structure.

New guidelines have been developed to prevent the progressive collapse of structures such as those published by the General Service Administration (GSA) () and the U.S. Department of Defense (DoD) ().

Progressive collapse of the RC frames has been investigated by many researchers (Adam et al., 2018; Kiakojouri et al., 2020; Russell et al., 2019). An experimental study was performed to investigate the progressive collapse resistance of RC frames with and without slabs under the corner column removal scenario (Lim et al., 2017). Progressive collapse of RC frames, their dynamic response, and the collapse mechanism were studied by Xiao et al. (2015). Kai et al. (2019) experimentally investigated the progressive collapse behavior of the prestressed RC frames with middle column removal scenario. Alshaikh et al. (2020) reviewed several studies on the progressive collapse of RC structures focusing on experimental studies on various types of structures, such as beam–slab and beam–column assemblies, large-scale buildings, and planar frame structures.

In seismic investigations, it is very important to study the extent and ability to respond and the ductility of the lateral system of the structure against earthquakes. In the structural analysis, the effects of soi–structure interaction (SSI) are usually ignored and the seismic response of the structure is evaluated as a structure with a solid foundation (Azimi & Molaei Yeznabad, 2020). In the occurrence of earthquakes, deformations occur in the substructure, which is entered from the foundation, and the dynamic response of the structure is also affected by changes in the behavior of the substructure. Considering the interaction of soil and structure makes the analysis a time-consuming process, but using it in conjunction with the conventional method leads to real results. Soil–structure interaction may have a significant influence on the seismic response of the buildings. By considering SSI, the evaluated forces in the structures' members differ from the usual method of analysis (Khatibinia et al., 2013).

Due to the semi-infinite and inhomogeneous nature of the soil environment, its modeling is more complex than the modeling the structure. Choosing a suitable modeling technique and a precise computational method is a challenging and important issue in SSI analysis. Modeling the soil environment with the appropriate modeling method makes the seismic response resulting from the dynamic analysis of soil–structure realistic.

Generally, there are two main methods for evaluating the SSI: first, the direct approach, and second, the substructure approach (Behnamfar & Banizadeh, 2016; Cavalieri et al., 2020; Mourlas et al., 2020). In the direct approach, foundation, soil, and structure are modeled and considered as a single system (Far & Flint, 2017); whereas, in the substructure approach, SSI is evaluated as two separate systems, and coupling of the subdomains can be obtained by the impedance functions. The use of the substructure method makes it possible to consider the complex soil–structure system as more manageable parts that can be easily examined. This method is more computationally efficient (Tavakoli et al., 2019). In contrast, the use of this method requires the assumption of the linear behavior of soil and structure. This is while in reality, the behavior of the subsoil is nonlinear. Hence dynamic analyses may not be easily accessible by this method (Kutanis & Elmas, 2001).

Since the superposition assumptions are not required, it is possible to perform accurate nonlinear analysis in the direct method because of its acceptability and adaptability to deal with material properties and complex geometries (Far, 2019). As a result, the direct method is more accurate for the dynamic analysis of soil–structure systems. Fathi et al. (2020) assessed the SSI on seismic behavior of a masonry building. Güllü and Karabekmez (2017) investigated the effect of near-fault and far-fault earthquakes on the buildings through three-dimensional (3D) dynamic soil–structure interaction. Karapetrou et al. (2015) studied the seismic vulnerability of high-rise RC structures by considering SSI. The effect of pile foundation type and its size was investigated on the performance of the structures in soft soil sites during the earthquake considering SSI (Nguyen et al., 2017).

In this study, at first, a numerical model was simulated and verified with the experimental Li model (Li et al.,

2016) in SeismoStruct software. Then, it was developed and a 20-story RC frame without SSI was modeled in SeismoStruct software. To investigate the progressive collapse of RC frames, the nonlinear static pushdown analyses were conducted with structural elements removed from the buildings in the first story.

Although several researches have been performed to investigate the effects of soil–structure interaction on seismic response of the structures, there are only a few studies that examine the effects of SSI on the vulnerability of the structures to progressive collapse following the column removal. Furthermore, the effects of soil condition on progressive collapse of RC structures have not been investigated in detail. In normal structural designs, the effect of soil–structure interaction is usually ignored due to the more complex calculations and longer computation time. However, the effect of SSI, especially in soft soils, has a significant effect on the design of beams, columns, and other structural components and ignoring it leads to the incorrect design. Therefore, in evaluating the progressive collapse of structures, the results are not realistic without considering SSI, especially in soft soils.

In this work, the two-dimensional (2D) fast Lagrangian analysis of continua (FLAC) finite-difference software, FLAC2D V7.0, was utilized to evaluate the effect of SSI on the progressive collapse of RC frames, which is based on the Finite Difference Method (FDM). Here, the direct approach was utilized to investigate the SSI. The foundation, soil, and framed structure were modeled simultaneously in FLAC software. The simulated frame in SeismoStruct software was also simulated and verified in FLAC software. Then, a gradual loading was defined in FLAC software. Furthermore, a parametric study of the framed structures (thickness of the foundation) and substructures (soil density, soil types, soil layers, and the soil saturation conditions) was performed on the progressive collapse of the frames. The vulnerability of the frames against progressive collapse was assessed based on the sensitivity index (SI), following the (middle, corner, and edge) column removal.

Table 1 The geometric characterization of the test specimens.

The space of the columns from center-to-center (S)	1.70 m
Inter-story heights of the first story	1.35 m
Inter-story heights of the second story	1.10 m

2 Modeling and Analysis

2.1 Reference Specimen

The experimental specimen performed by Li et al. (2016) was used as a reference to confirm the validity of the inelastic macro-model specified in this study and make it usable for the analysis of the nonlinear static progressive collapse of the RC frames. The reinforcement layout and the main features of the frame are shown in Fig. 1.

Herein, a 1/3 scaled, 2-story, planar frame with 4-bay examined by Li et al. (2016) were utilized. The details of the test specimens are mentioned in Table 1 and Table 2.

2.2 Numerical Modeling of the Frames

At first, the described reference specimen was simulated in SeismoStruct software. Then, the numerical model was

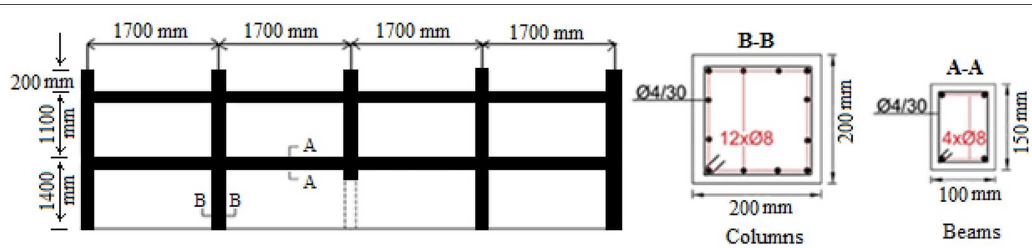


Fig. 1 The geometry and reinforcement layout of the reference model examined by Li et al. (2016).

Table 2 The specifications of beams and columns of the test specimens.

	Beam	Column
Cross-sections	150 × 100 mm^2 (height to width)	200 × 200 mm^2
The transverse reinforcement	2-leg Φ4	4-legΦ 4
distance of stirrups	30 mm at both end zones	33 mm at the base of the ground-floor columns and 50 mm everywhere else
The longitudinal rebars	4Φ8	12 Φ8

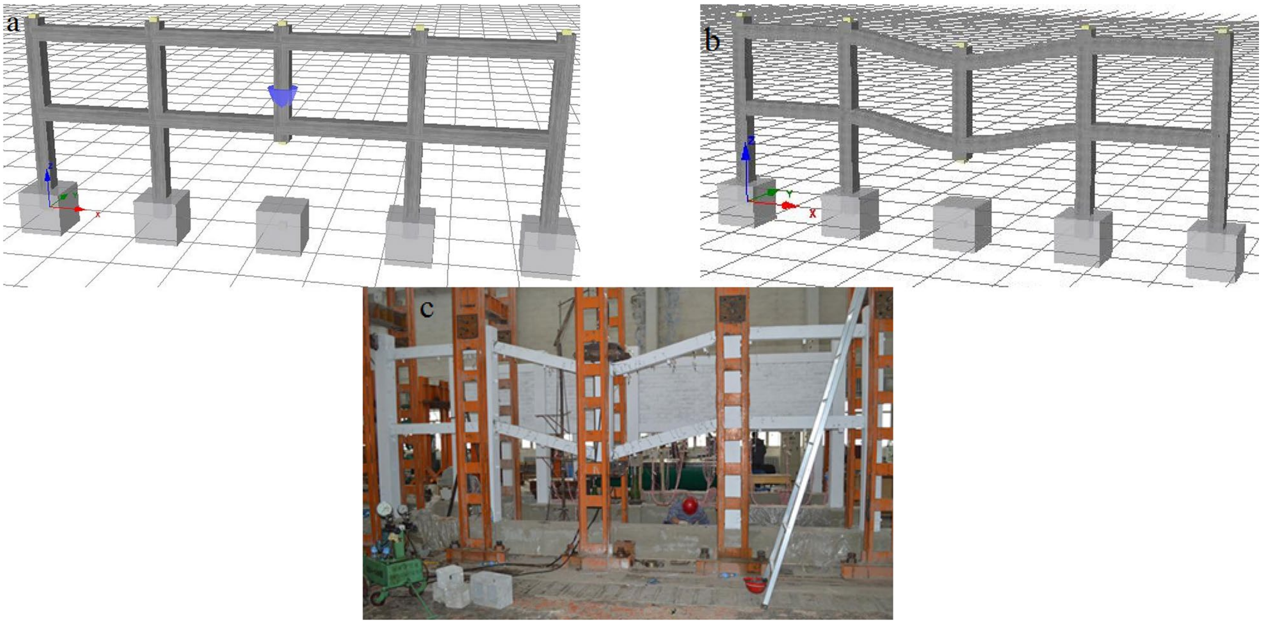


Fig. 2 **a** Undeformed and **b** deformed shapes of the case-study frame and **c** deformed shape of the Li studied specimen (Li et al., 2016).

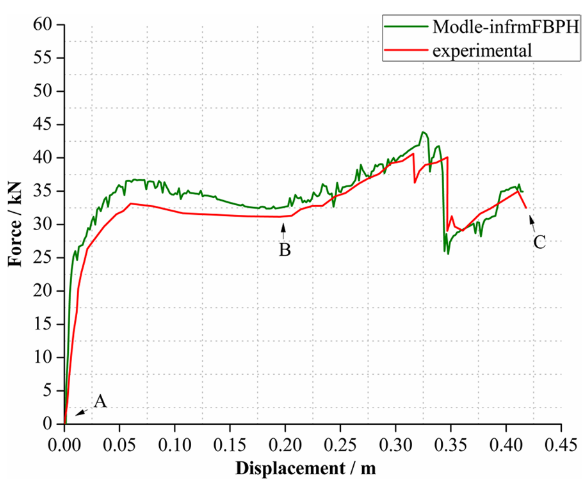


Fig. 3 Force–displacement curves of the experimental (Li et al., 2016) and numerical models of the frames.

verified. In this analysis, the inelastic force-based plastic-hinge frame elements with 16.67% plastic hinge length and 400 section fibers were utilized. Modeling of both columns and beams was performed as the single inelastic force-based plastic-hinge frame elements (infrmFBPH).

To simulate the inelastic behavior of concrete, the uniaxial uniform confinement model, suggested by Mander et al. (1988) was utilized. The following stress–strain relationships, implemented according to Martinez-Rueda and Elnashai (1997), were used:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r}, \tag{1}$$

f_c is the longitudinal compressive concrete stress,

$$f'_{cc} = k f'_{co}, \tag{2}$$

Table 3 The specifications of the important points in the experimental and numerical models.

Points	Vertical displacement (mm)					Force (kN)				
	A	B	C	First rebar fracture	Second rebar fracture	A	B	C	First rebar fracture	Second rebar fracture
Experimental model	0	195	418	316	347	0	31.14	32.49	40.63	40.08
Numerical model	0	186	415	324	339	0	32.38	34.93	43.86	41.76
Discrepancy (%)	0	4.6	0.7	2.47	2.3	0	3.8	6.98	7.4	4

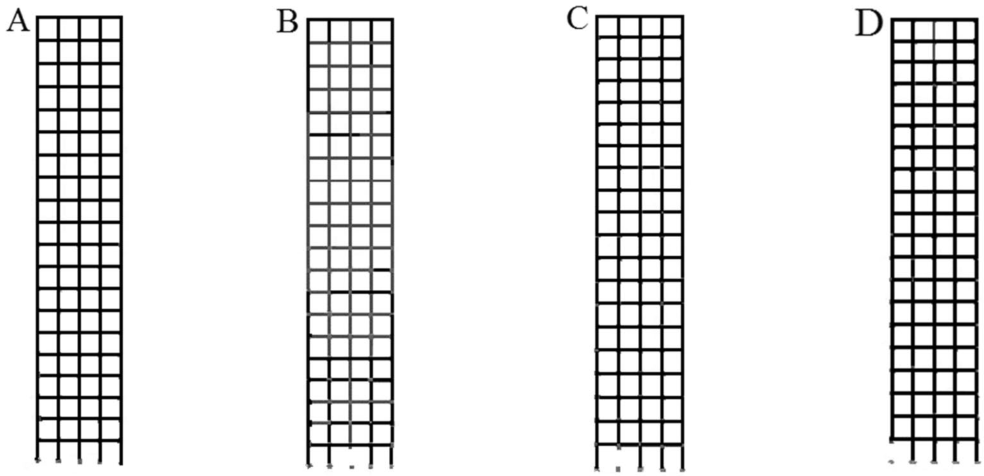
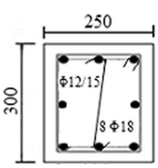
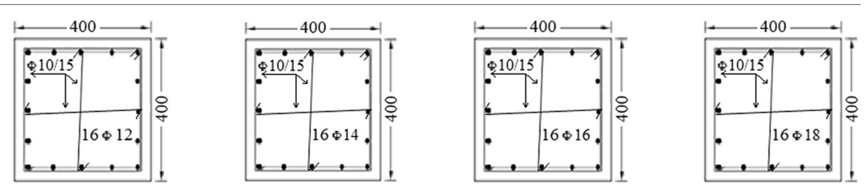


Fig. 4 The numerical models of the 20-story frames. **A** without column removal, and with **B** middle, **C** corner, and **D** edge column removal in SeismoStruct software.

Table 4 Dimensional specification and the reinforcement Layout of the 20-story frames.

Beams			**Columns**		
Size (depth × width) (mm²)	**Longitudinal reinforcement**	**Transverse reinforcement**	**Size (depth × width) (mm²)**	**Longitudinal reinforcement**	**Transverse reinforcement**
300 × 250	8 Φ 18	Φ12/150 mm	400 × 400	16 Φ 12 16 Φ 14 16 Φ 16 16 Φ 18	Φ10/150 mm
Beam		**Columns**			
					

$$x = \varepsilon_c/\varepsilon_{cc}, \tag{3}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right], \tag{4}$$

$$r = \frac{E_c}{E_c - E_{sec}}, \tag{5}$$

$$E_c = 5000\sqrt{f'_{co}}, \tag{6}$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}. \tag{7}$$

f'_{cc} is the compressive strength (peak stress) of confined concrete, f'_{co} is the unconfined concrete strength, k is the confinement factor, ε_c is the longitudinal

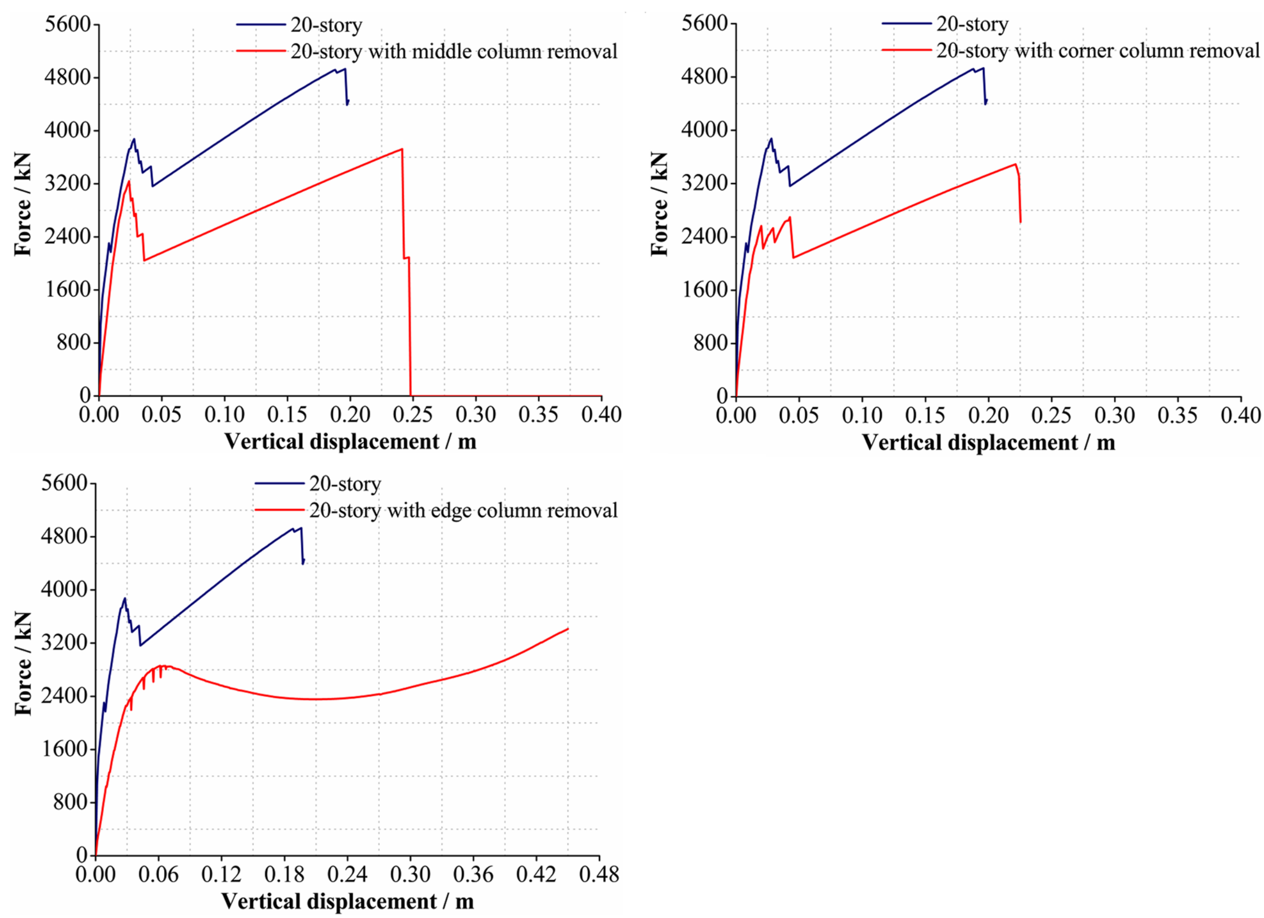


Fig. 5 Force–displacement curves of the 20-story frames with and without column removal in different locations (middle, corner, and edge).

compressive concrete strain, ε_{cc} is the strain at maximum concrete stress f'_{cc}, ε_{co} is the strain at unconfined stress f_{co}, E_c is the initial modulus of elasticity of concrete and E_{sec} is the secant modulus of elasticity of concrete at peak stress.

The finite element (FE) analysis was done by static method, using a hybrid solution procedure between the classic Newton–Raphson and the modified Newton–Raphson approaches in SeismoStruct software. The threshold for the convergence criterion based on the assumed displacement/rotation was set equal to 10^{-3}. The imposed displacement rate was at least 0.1 mm per step. All the model nodes were restrained against out-of-plane displacements as well as rotations around the in-plane horizontal axis and the vertical axis of the frame. Geometrical nonlinearities due to large displacements/rotations and P-Delta effects were included according to a total corotational formulation. The following procedure was utilized for removing a column in the software: at first in Pre-Processor menu, in the Element Connectivity section, the desired column was selected and removed. Then, in the Applied Loads menu, the node above the removed column was selected for incremental load. Afterwards, in the Loading Phases menu, target displacement and steps were determined.

The undeformed and deformed shapes of the case-study frame and the deformed shape of the Li studied specimen are shown in Fig. 2.

3 Verification of Numerical Model

The force versus vertical displacement curves of the experimental model and the numerical model obtained by pushdown analysis are exhibited in Fig. 3.

The collapse resistance of the structures is mainly related to the beam mechanism and the catenary action (CA) mechanism. Three points are shown in the force–displacement curves of the models. A is the initial point. B refers to the transition point between the beam and the catenary mechanisms. Stage AB is related to the beam mechanism which is attributed to the flexural capacities and the compressive arch action (CAA) of the beams.

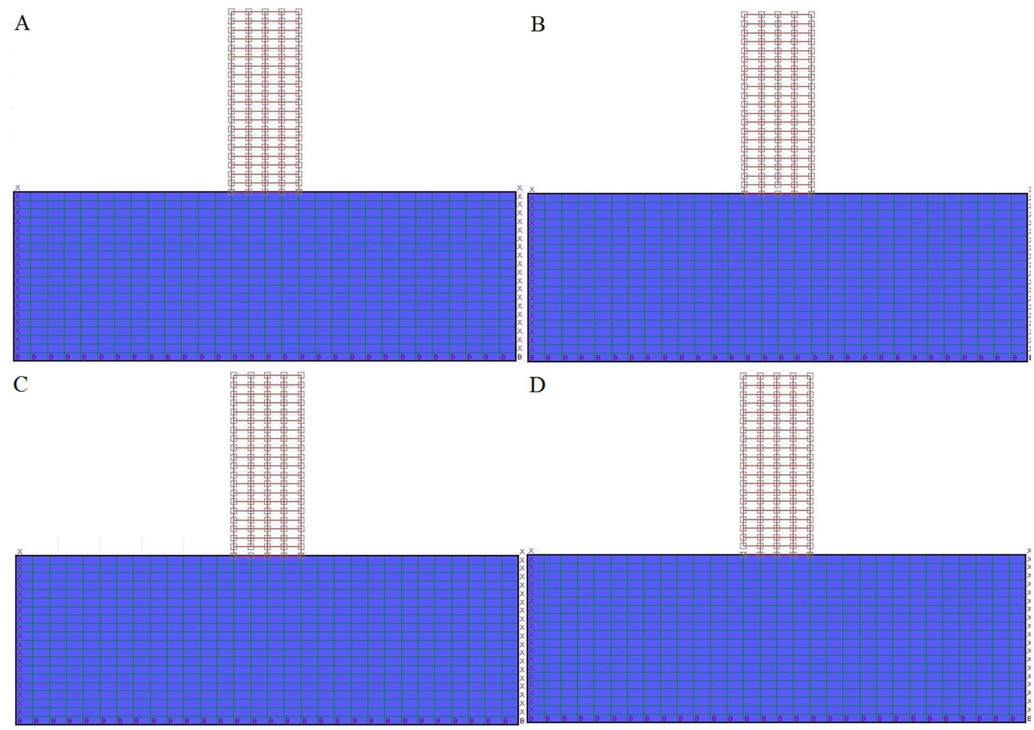


Fig. 6 The numerical models of the 20-story frames. **A** without column removal, and with **B** middle, **C** corner, and **D** edge column removal in the FLAC software.

After B point, with increasing the vertical displacement, the load capacity increases due to the catenary action mechanism. This continues until the first rebar failure. Point C refers to the maximum vertical displacement. Stage BC is related to the catenary action. The specifications of the first and second rebar fractures and the points A, B, C in the experimental and numerical models are presented in Table 3. As shown in Fig. 3 and Table 3, there is a good agreement between the results of the experimental model and the numerical infrmFBPH model.

It is worth noting that the numerical curve noises in Fig. 3 are due to the convergence error of the Newton–Raphson approximation solution method in the software. The Newton–Raphson method is a technique for solving equations numerically based on the simple idea of linear approximation. As the solution steps increase, the convergence error and the resulting noises decrease, but the solution steps become longer.

4 Development of the Numerical Model

After simulating the numerical model according to the experimental specimen (Li et al., 2016), the two-dimensional model was developed. A 20-story frame with 3.2 m story height and 3 m length of all spans was modeled in SeismoStruct software with and without column removal (Fig. 4). The characteristic yield strength of the longitudinal and transverse reinforcing rebars was 234 MPa, and the mean compressive strength of concrete was 41.3 MPa. The dead and live loads used to carry out the earthquake-resistant design of the structures in accordance with European seismic provisions (European Committee for Standardization, 2004a, 2004b) were 4 kN/m^2 per each. The ground type was considered to be a B-type one. The peak ground acceleration at bedrock was taken as 0.24 g, for life safety limit state. The structures were designed to meet criteria for structures relating to the so-called medium ductility class and a behavior factor of 5.85 was considered. The dimensional specifications and the reinforcement layout of beams and columns of the frames are presented in Table 4.

To assess the progressive collapse-resisting capacity of RC frames, nonlinear static pushdown analyses were performed with structural elements removed from the middle, corner and, edge of the buildings in the first story.

According to the DoD regulations, loading was done in the structures. Since the loading condition after sudden removal of a column is completely dynamic, using amplification factor in the nonlinear static analysis method, its dynamic nature is approximated with combination load that is proposed in DoD. Here, the

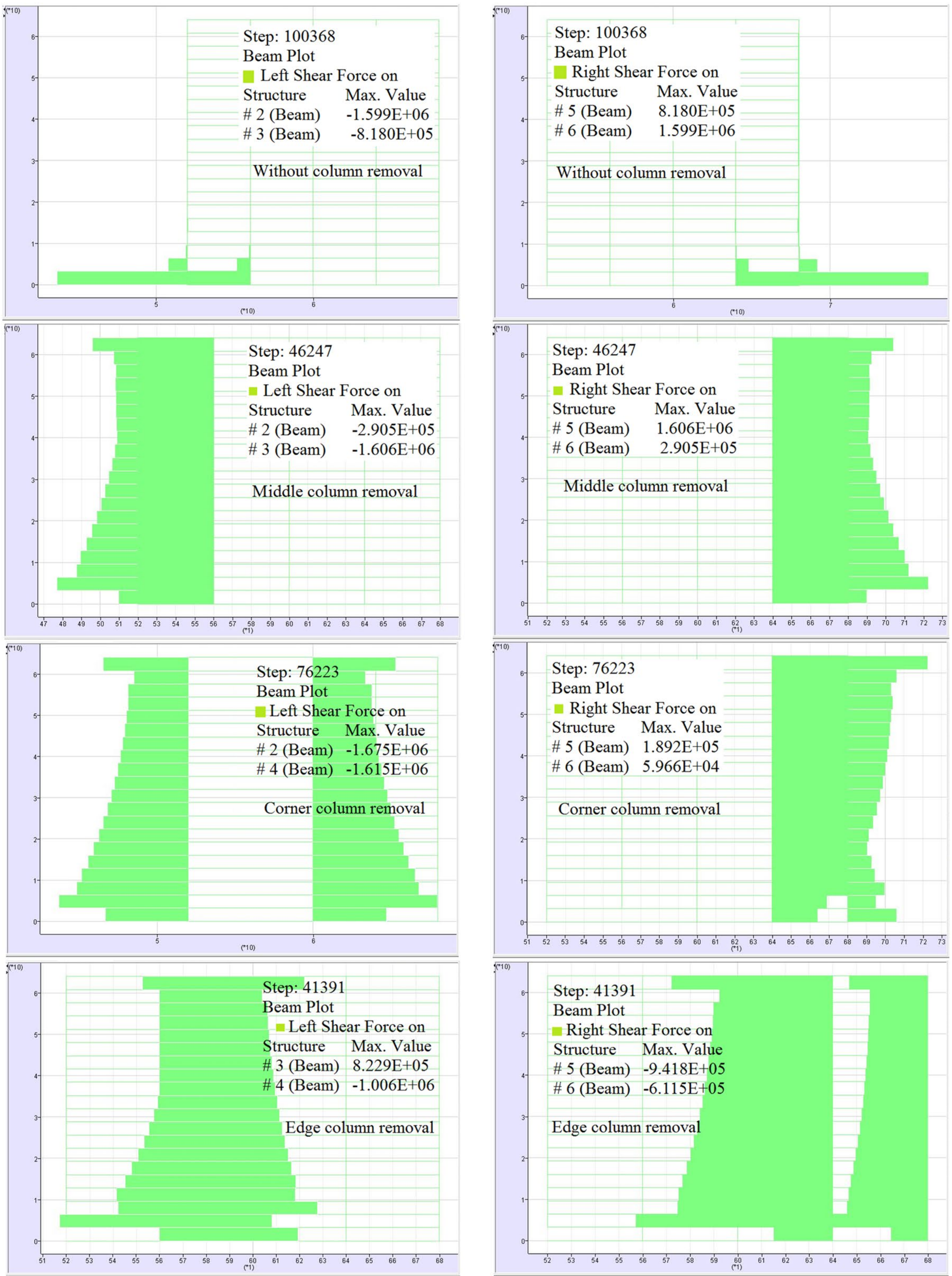


Fig. 7 Total base shear of the RC frames with and without column removal in various positions.

Table 5 Redistribution of the imposed loads and Y-displacement in the 20-story RC frames.

Conditions	Software			Discrepancy (%)
Middle column	SeismoStruct	y-disp$_0$ (m)	0.0846	
		y-disp$_{damage}$ (m)	0.1207	
		λ_0 (kN)	4932.427	
		λ_{dmage} (kN)	3724.434	
		SI	0.245	
	FLAC	y-disp$_0$ (m)	0.08286	2.05
		y-disp$_{damage}$ (m)	0.1165	3.48
		λ_0 (kN)	4834	1.99
		λ_{dmage} (kN)	3793	1.81
		SI	0.2153	
Corner column	SeismoStruct	y-disp$_0$ (m)	0.0846	
		y-disp$_{damage}$ (m)	0.1056	
		λ_0 (kN)	4932.427	
		λ_{dmage} (kN)	3491.132	
		SI	0.292	
	FLAC	y-disp$_0$ (m)	0.08286	2.05
		y-disp$_{damage}$ (m)	0.1106	4.52
		λ_0 (kN)	4834	1.99
		λ_{dmage} (kN)	3538.86	1.35
		SI	0.2679	
Edge column	SeismoStruct	y-disp$_0$ (m)	0.0846	
		y-disp$_{damage}$ (m)	0.2249	
		λ_0 (kN)	4932.427	
		λ_{dmage} (kN)	3413.043	
		SI	0.308	
	FLAC	y-disp$_0$ (m)	0.08286	2.05
		y-disp$_{damage}$ (m)	0.220	2.18
		λ_0 (kN)	4834	1.99
		λ_{dmage} (kN)	3382.2	0.90
		SI	0.3003	

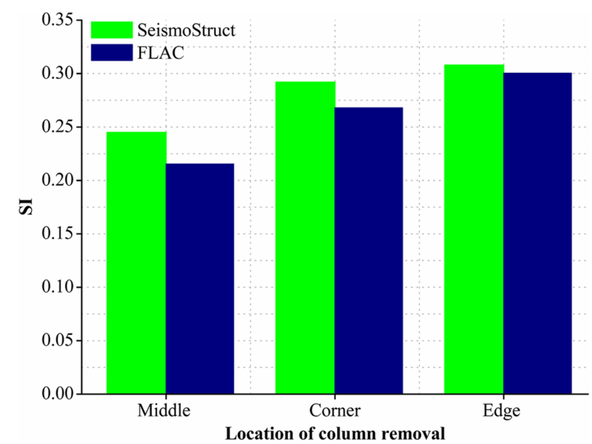


Fig. 8 The plot of the sensitivity index versus different locations of column removal in FLAC and SeismoStruct software.

Table 6 Redistribution of the imposed loads in the 20-story RC frames with different thicknesses of foundation in FLAC software.

Conditions	Thickness of foundation (cm)	λ_0 (kN)	λ_{damage} (kN)	SI
Middle column	180	4834	3793	0.2153
	190	4613	3564.6	0.2273
	200	4381	3338.1	0.2381
	210	4231.2	3179.6	0.2485
Corner column	180	4834	3538.86	0.2679
	190	4613	3337.33	0.2765
	200	4381	3148.2	0.2814
	210	4231.2	2995.28	0.2921
Edge column	180	4834	3382.2	0.3003
	190	4613	3192.5	0.3079
	200	4381	2988.1	0.3179
	210	4231.2	2838.2	0.3292

dynamic amplification factor of 2 in load combination proposed by DoD guideline is used. Loads are applied to the beams adjacent to the removed columns in nonlinear static analysis method as illustrated in Eq. 8:

$$Load = 2[1.2D + 0.5L]. \tag{8}$$

For dynamic analysis, the DoD guidelines do not recommend using the dynamic amplification factor. According to the DoD guideline, in linear and nonlinear dynamic analysis methods, the load applied to the beams connected to the removed column is illustrated in Eq. 9:

$$Load = [1.2D + 0.5L]. \tag{9}$$

D and L are floor dead load and Live load, respectively.

The force–displacement curves of the 20-story RC frames with and without column removal in different locations (middle, corner, and edge) are shown in Fig. 5.

In this study, the progressive collapse resistance of the frames under threat-independent column loss scenarios was evaluated numerically. For this purpose, the sensitivity index was utilized, which is expressed as follows (Farahani et al., 2018; Jiang et al., 2020):

$$\mathrm{SI} = (\lambda_0 - \lambda_{damage})/\lambda_0. \tag{10}$$

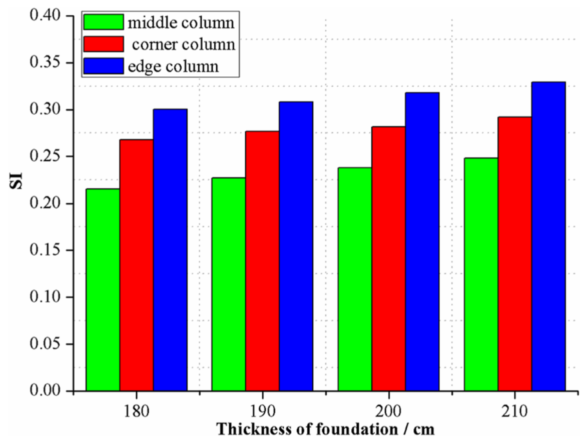


Fig. 9 The plot of the sensitivity index versus the thickness of foundation.

λ_0 is the collapse load factor in the original state, in other words, λ_0 is the capacity ratio of the intact structure, defined as the ratio of the ultimate loading capacity to the applied load.

λ_{damage} is the collapse load factor of a structure after removal of a member. It is the capacity ratio of the damaged structure due to the failure of the member.

If the vertical load-carrying capacity is not affected by the column removal, the corresponding sensitivity index is very small (SI $\approx$ 0). It means that the relevant column has no effect on the carrying capacity of the entire structure and that column is less important in terms of load-carrying capacity storage. Conversely, when a column with a large sensitivity index (SI $\approx$ 1) is removed, that part of the structure or the whole structure collapses immediately. Therefore, such a column is considered a key element (Ito et al., 2005). To investigate the effect of SSI on the progressive collapse of the RC frames following the (middle, corner, and edge) column removal, the soil, foundation, and framed structures were modeled in FLAC software (Fig. 6).

The soil–structure model consisted of beam structural elements to model the structural components, and 2D plane-strain grid composed of quadrilateral elements to model the soil medium. Beam structural elements are two-nodded, straight, finite elements with six degrees of freedom per node containing three translational and three rotational components. Rigid boundary condition was assigned to the bedrock and lateral boundaries of the soil medium were assumed to be quiet (viscous) boundaries. The Mohr–Coulomb model was adopted as the constitutive model to simulate the nonlinear behavior of the soil medium.

The horizontal distance between soil boundaries was assumed to be six times the structure width. The bedrock depth was considered to be 60 m. The strip RC foundation was 2 m wide, 20 m long and 1.80 m deep. Since this is a plane-strain problem, the strip foundation width was considered to calculate the moment of inertia of the concrete element only.

As the 20-story RC frames and the substructure soil were modeled in FLAC software, the total base shears of the frames in the cases with and without column removal in various locations (middle, corner, and edge) are shown in Fig. 7.

The results of the numerical models in the SeismoStruct, and FLAC software are represented in Table 5. As shown, there is a good agreement between the mentioned models. Comparing the data in Table 5 as well as the values of the sensitivity index in SeismoStruct and FLAC (Fig. 8) showed that in both software, the values of the sensitivity index in the case of removing the edge column are more than the corner column and in removing the corner column are more than the middle column. It is also shown that in all cases, the sensitivity index values in FLAC are less than SeismoStruct software. That is, by considering the soil–structure interaction, the sensitivity index decreases. In fact, by modeling soil, foundations, and framed structures in FLAC software, shear forces and flexural anchors in the stories of the framed structures are reduced. This leads to the smaller dimensions for the design of structural elements.

5 Parametric Studies on Progressive Collapse of RC Frames by Considering SSI

5.1 The Effect of Thickness of the Foundation

To study the effect of thickness of foundation on the progressive collapse of RC frames following the (middle, corner, and edge) column removal, a 20-story RC frame with different foundation thicknesses, along with the substructure was modeled in FLAC software. The vulnerability of the frames to progressive collapse was determined based on sensitivity index. At first, the thickness of foundation was determined to be at least 180 cm based on the conventional analyses to meet the needs of the structure without considering the progressive collapse and soil–structure interaction. Then, to investigate the effect of foundation thickness on progressive collapse of RC frames, different thicknesses of 190, 200 and 210 cm were also examined. According to the DoD regulations, loading was done in the structures. Finally, the sensitivity index was determined for each foundation thickness and the results were compared with each other. λ_0 and λ_{damage} values were determined, considering the SSI, for

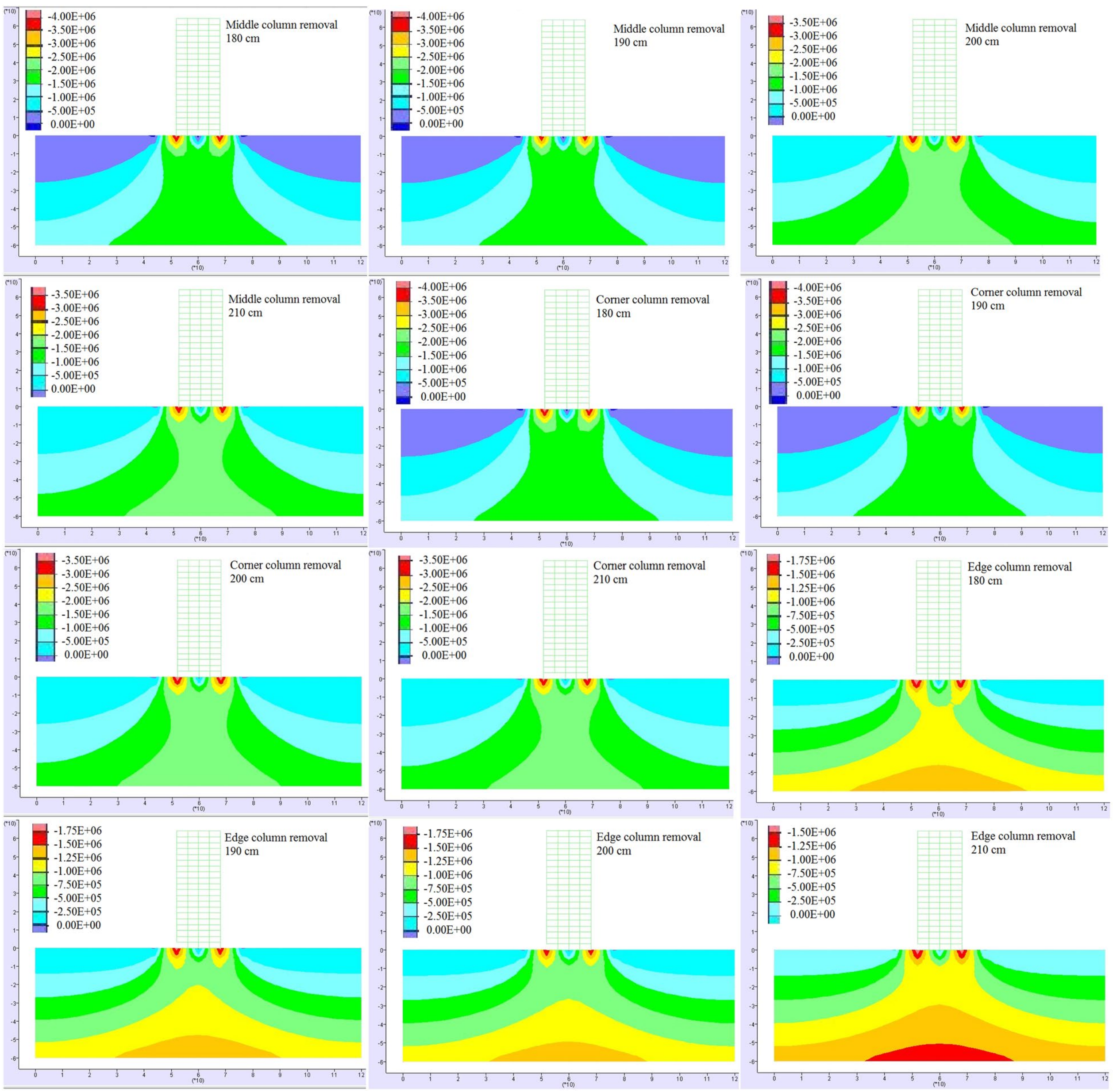


Fig. 10 SYY contours of the frames with column removal with different thicknesses of foundation.

progressive collapse following the column removal for each foundation thickness. The sensitivity index for the frames with different thicknesses of the foundation is shown in Table 6. As shown in Table 6 and Fig. 9, with decreasing the thickness of foundation in the studied range (180–210 cm), the sensitivity index decreases. Therefore, at the thickness of 180 cm, the conditions are better to prevent progressive collapse.

In fact, it is observed that with increasing the thickness of foundation and thus increasing its weight, the amount of anchor increases. Also, increasing the thickness of foundation increases the maximum subsidence, the average subsidence, and the flexural anchorage of the foundation. Its only positive effect is the reduction of non-uniform subsidence of the foundation.

The changes of SYY (SYY is the force per unit area acting in the Y direction on a plane perpendicular to the Y axis) contours of the frames with column removal with different thicknesses of the foundation are shown in Fig. 10. It is observed that in the case of (middle,

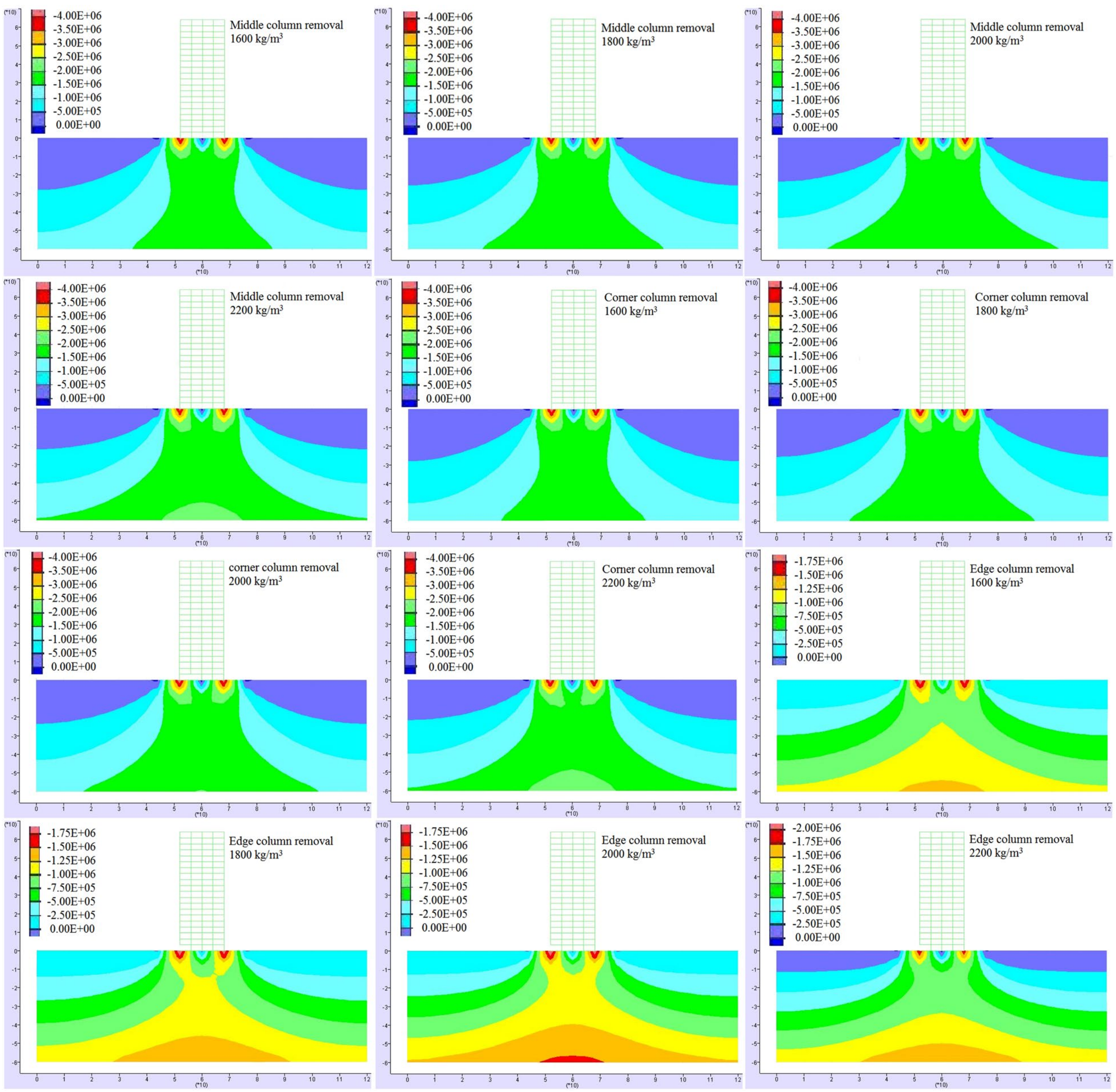


Fig. 11 SYY contours of the frames with column removal with different soil densities.

corner, and edge) column removal, the stress in the substructure of the adjacent columns of the removed one increase and shows the redistribution of force after column removal.

5.2 The Effect of Soil Density

To investigate the effect of soil density on the progressive collapse of the frames following the (middle, corner, and edge) column removal, a 20-story RC frame with different soil densities of 1600, 1800, 2000, and 2200 kg/m^3 was modeled in FLAC software. According to the DoD regulations, loading was performed in the frames. The sensitivity index was obtained for each soil density and the results were compared with each other. The changes of SYY contours of the frames with (middle, corner, and edge) column removal with different soil densities are exhibited in Fig. 11. As observed, following the column removal, the stress in the substructure of the adjacent columns of the removed one increases, and the forces are redistributed after column removal.

Table 7 Redistribution of the imposed loads in the 20-story RC frames with different densities of substructure soil in FLAC software.

Conditions	Soil density (kg/m^3)	λ_0 (kN)	λ_{damage} (kN)	SI
Middle column	1600	4836.2	3792.8	0.2157
	1800	4834	3793	0.2153
	2000	4819	3791.9	0.2131
	2200	4809	3791.7	0.2115
Corner column	1600	4836.2	3538.85	0.2683
	1800	4834	3538.86	0.2679
	2000	4819	3550.76	0.2632
	2200	4809	3550.76	0.2616
Edge column	1600	4836.2	3382.2	0.3006
	1800	4834	3382.2	0.3003
	2000	4819	3380.8	0.2984
	2200	4809	3393.3	0.2944

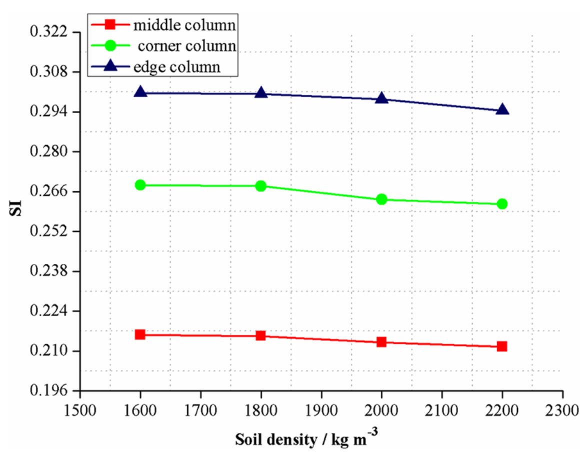


Fig. 12 The plot of the sensitivity index versus the substructure soil density.

As shown in Table 7 and Fig. 12, the sensitivity index decreases with increasing the substructure soil density (1600–2200 kg/m^3). Consequently, the situation would be better for preventing progressive collapse. Therefore, for soils with lower density, their density can be increased with intentional compaction. Soil compaction is the removal of pore spaces within soil structures and drainage channels between soil structures. This prevents the penetration of roots and the movement of air and water in the soil. Soil compaction occurs intentionally or unintentionally, of which the intentional soil compaction is desirable.

5.3 The Effect of Soil Type

The effect of different soil types was investigated on the progressive collapse of a 20-story RC frame following the (middle, corner, and edge) column removal. Different soil types of rock, silty sands (SM), silty clay with low plasticity (CL-ML), well-graded gravel (GW), and clay–clayey silt (clay-MC) were selected for substructure. The 20-story RC framed structure and its substructure were modeled in FLAC software. According to the DoD regulations, loading was done in the frames. The sensitivity index was determined for each soil type and the results were compared with each other. The characteristics of the substructure soils are mentioned in Table 8.

The SYY contours of the frames with (middle, corner, and edge) column removal with different soil types are shown in Fig. 13. It is observed that in the case of column removal, the stress in the substructure of the adjacent columns of the removed one increase and shows the redistribution of force after column removal.

The sensitivity index for the framed structures with different types of substructure soil including rock, SM, CL-ML, GW, and clay-MC is represented in Table 9. As shown in Fig. 14, by changing the type of substructure soil from clay-MC to rock, the sensitivity index decreases. Therefore, rock and SM, compared to other soil types are better for preventing progressive collapse.

Table 8 The characteristics of the different soil types.

Soil type	Dry density (kg/m^3)	Bulk modulus (MPa)	Shear modulus (MPa)	Friction angle (degree)	Cohesion (Pa)	Dilation angle (degree)	Tensile strength (Pa)
Rock	2700	555.50003	416.6	33	10,000	0.0	1000
SM	1980	30.30	25.60	32	0.0	0.0	0.0
CL-ML	1880	16.670001	10.17	30	0.0	0.0	0.0
GW	1700	78.43	53.20	38	0.0	0.0	0.0
Clay-MC	1600	11.1111	3.703700	25	5000	0.0	0.0

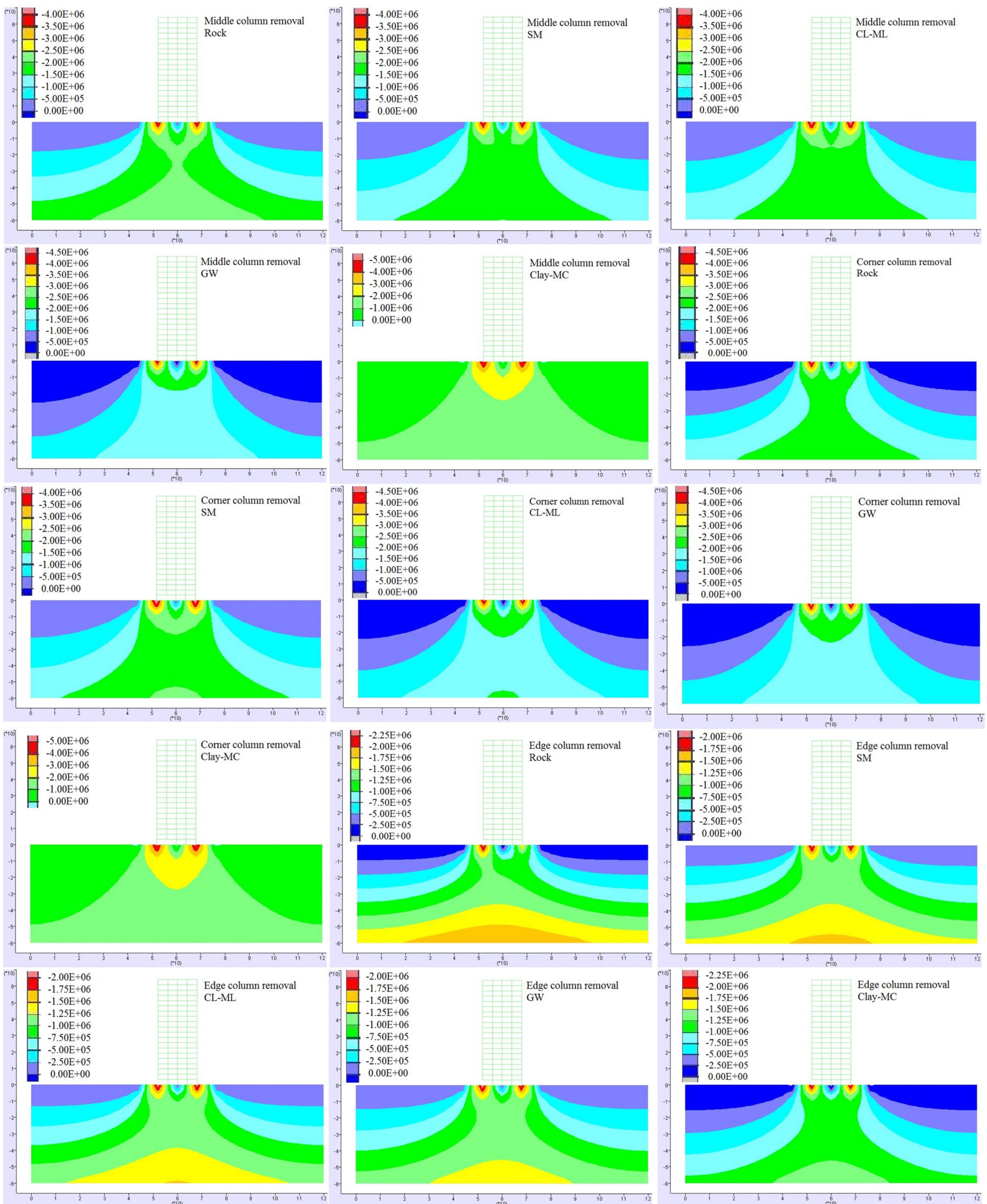


Fig. 13 SYY contours of the frames with column removal with different soil types.

Table 9 Redistribution of the imposed loads in the 20-story RC frames with different soil types.

Conditions	Soil type	λ_0 (kN)	λ_{damage} (kN)	SI
Middle column	Rock	4839.6	3929.5	0.1881
	SM	5052.6	4056.1	0.1972
	CL-ML	5553.6	4189.4	0.2456
	GW	5839.4	4321.3	0.2600
	Clay-MC	6992	5004	0.2843
Corner column	Rock	4839.6	3837.83	0.2070
	SM	5052.6	3966.2	0.2150
	CL-ML	5553.6	4058.37	0.2692
	GW	5839.4	4167.73	0.2863
	Clay-MC	6992	4721.73	0.3247
Edge column	Rock	4839.6	3786.8	0.2175
	SM	5052.6	3925.8	0.2230
	CL-ML	5553.6	3922.5	0.2937
	GW	5839.4	3925.2	0.3278
	Clay-MC	6992	4499.6	0.3565

5.4 The Effect of Substructure Soil Layers

Since in the study of different soil types, it was shown that SM has good conditions against progressive collapse, so to investigate the effect of different substructure soil layers on the progressive collapse of the framed structures, SM was selected as the base component of all layers. Different soil layers were examined as follows:

Layer 1: SM, Layer 2: SM-CL/ML (very hard clay)-SM, Layer 3: SM-CL/ML (very hard clay)-CL/ML (very hard clay), Layer 4: SM-CL/ML (hard clay)-SM, Layer 5: SM-CL/ML (hard clay)-CL/ML (hard clay), Layer 6: SM-very soft clay-SM.

The framed structures with different substructure soil layers with and without (middle, corner, and edge) column removal were modeled in FLAC software. According to the DoD regulations, loading was performed in the frames. The sensitivity index was obtained for each soil layer and the results were compared with each other. The characteristics of the different substructure soil layers and their cross-section features are mentioned in Tables 10 and 11, respectively. Modeling of the frames

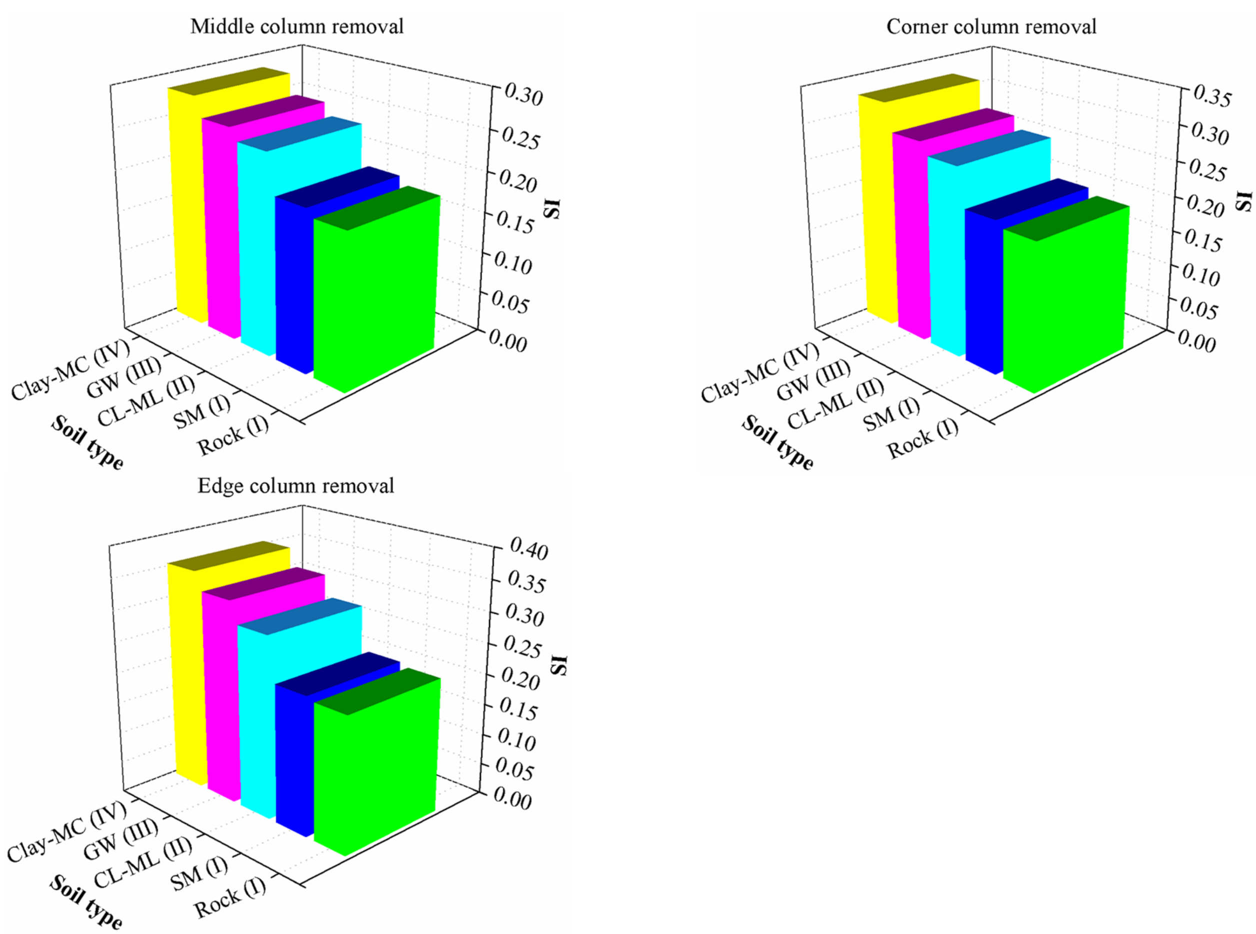


Fig. 14 The plot of the sensitivity index versus the type of substructure soil.

Table 10 The characteristics of the different substructure soil layers.

Soil type	Dry density (kg/m^3)	Bulk modulus (MPa)	Shear modulus (MPa)	Friction angle (degree)	Cohesion (Pa)	Dilation angle (degree)	Tensile strength (Pa)
SM	1980	30.30	25.60	32	0.0	0.0	0.0
CL-ML (very hard clay)	1910	33.330002	20.25	30	0.0	0.0	0.0
CL-ML (hard clay)	1880	16.670001	10.17	30	0.0	0.0	0.0
Very soft clay	582	1.6699999	0.172	0.0	5000	0.0	0.0

Table 11 The cross-sectional features of the different substructure soil layers.

Depth (m)	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
0–10	SM	SM	SM	SM	SM	SM
10–15	SM	CL/ML (very hard clay)	CL/ML (very hard clay)	CL/ML (hard clay)	CL/ML (hard clay)	Very soft clay
15–60	SM	SM	CL/ML (very hard clay)	SM	CL/ML (hard clay)	SM

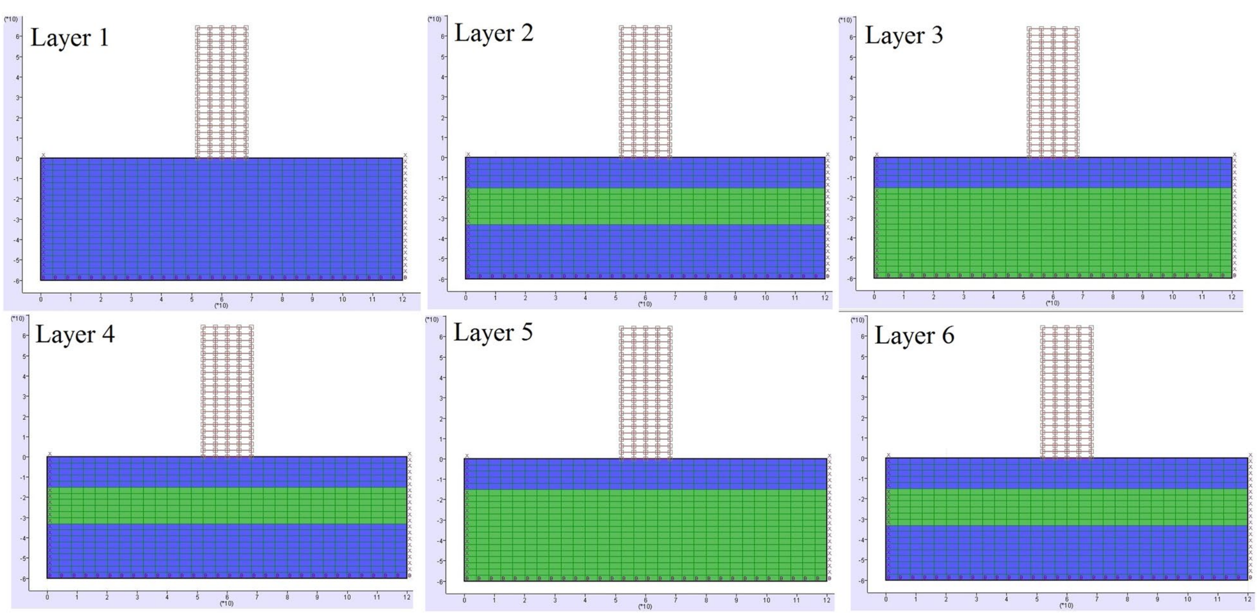


Fig. 15 Modeling of the 20-story RC frames with different soil layers in FLAC software.

without column removal with different soil layers is shown in Fig. 15.

The changes of SYY contours of the frames with (middle, corner, and edge) column removal with different soil layers are shown in Fig. 16. It is observed that after the column removal, the force is redistributed and the stress in the substructure of the columns adjacent to the removed column increases. The sensitivity index for the frames with different soil layers from layer 1 to 6 is mentioned in Table 12. As shown in Fig. 17, by changing the soil layer from layer 6 to layer 1, the sensitivity index decreases. Therefore, layers 1 and 2, compared to other soil layers are better for preventing progressive collapse.

5.5 The Effect of Soil Saturation Condition

To investigate the effect of the saturation conditions of the substructure soil on the progressive collapse of the framed structures, the 20-story RC frames with and

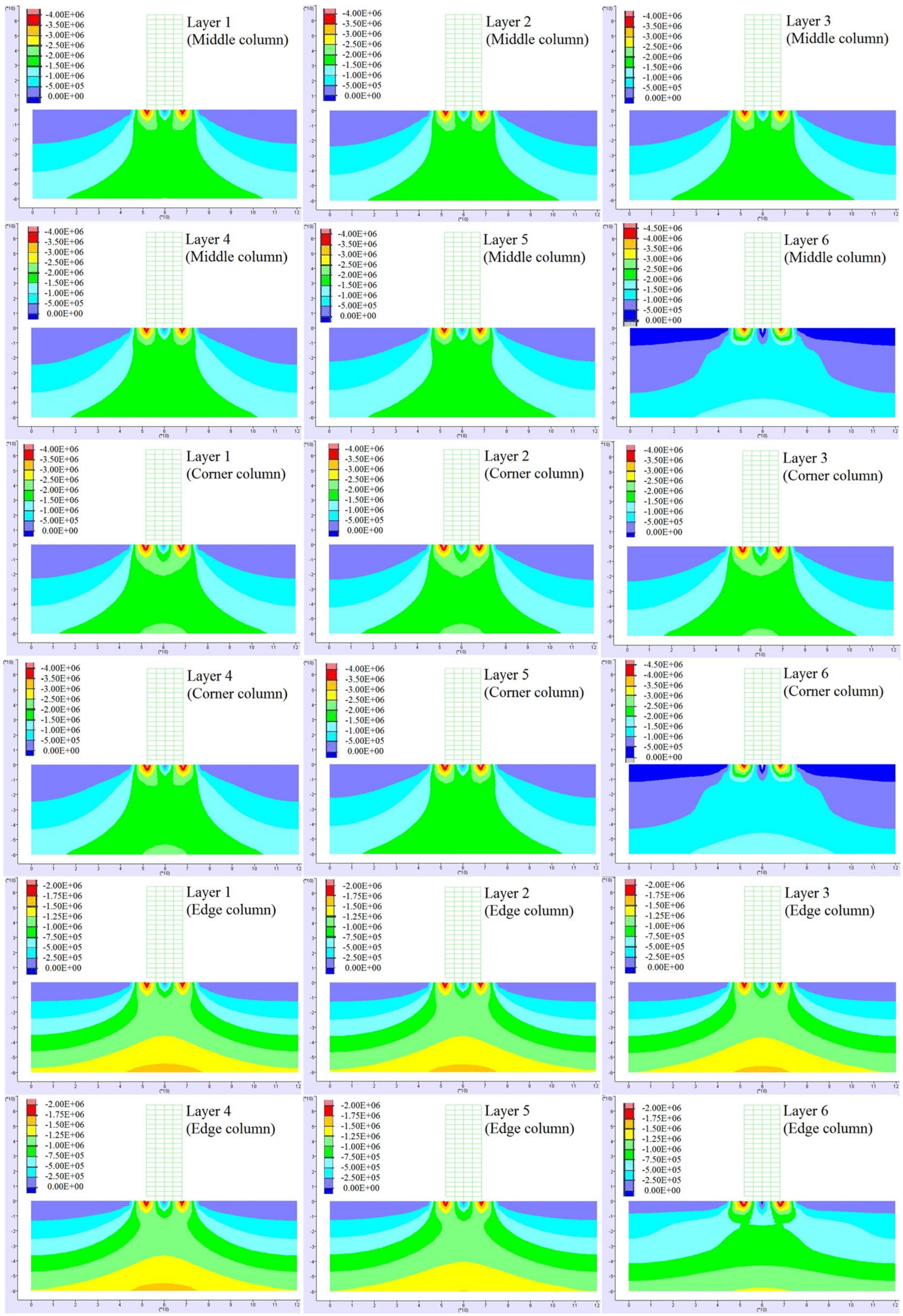


Fig. 16 SYY contours of the frames with column removal with different soil layers.

Table 12 Redistribution of imposed loads in the frames with different soil layers in FLAC software.

Conditions	Soil layers	λ_0 (kN)	λ_{damage} (kN)	SI
Middle column	SM	5052.6	4056.1	0.1972
	SM-CL/ML (very hard clay)-SM	5153	4055.5	0.2130
	SM-CL/ML (very hard clay)-CL/ML (very hard clay)	5272.8	4056.1	0.2308
	SM-CL/ML (hard clay)-SM	5633.4	4054.1	0.2803
	SM-CL/ML (hard clay)-CL/ML (hard clay)	5734.2	4049.8	0.2937
	SM-very soft clay-SM	6972	4750.2	0.3187
Corner column	SM	5052.6	3966.2	0.2150
	SM-CL/ML (very hard clay)-SM	5153	3966.78	0.2302
	SM-CL/ML (very hard clay)-CL/ML (very hard clay)	5272.8	3967.3	0.2476
	SM-CL/ML (hard clay)-SM	5633.4	3972.35	0.2949
	SM-CL/ML (hard clay)-CL/ML (hard clay)	5734.2	3977.5	0.3064
	SM-very soft clay-SM	6972	4737.4	0.3205
Edge column	SM	5052.6	3925.8	0.2230
	SM-CL/ML (very hard clay)-SM	5153	3926	0.2381
	SM-CL/ML (very hard clay)-CL/ML (very hard clay)	5272.8	3926.6	0.2553
	SM-CL/ML (hard clay)-SM	5633.4	3927.8	0.3028
	SM-CL/ML (hard clay)-CL/ML (hard clay)	5734.2	3928.6	0.3149
	SM-very soft clay-SM	6972	4534.3	0.3496

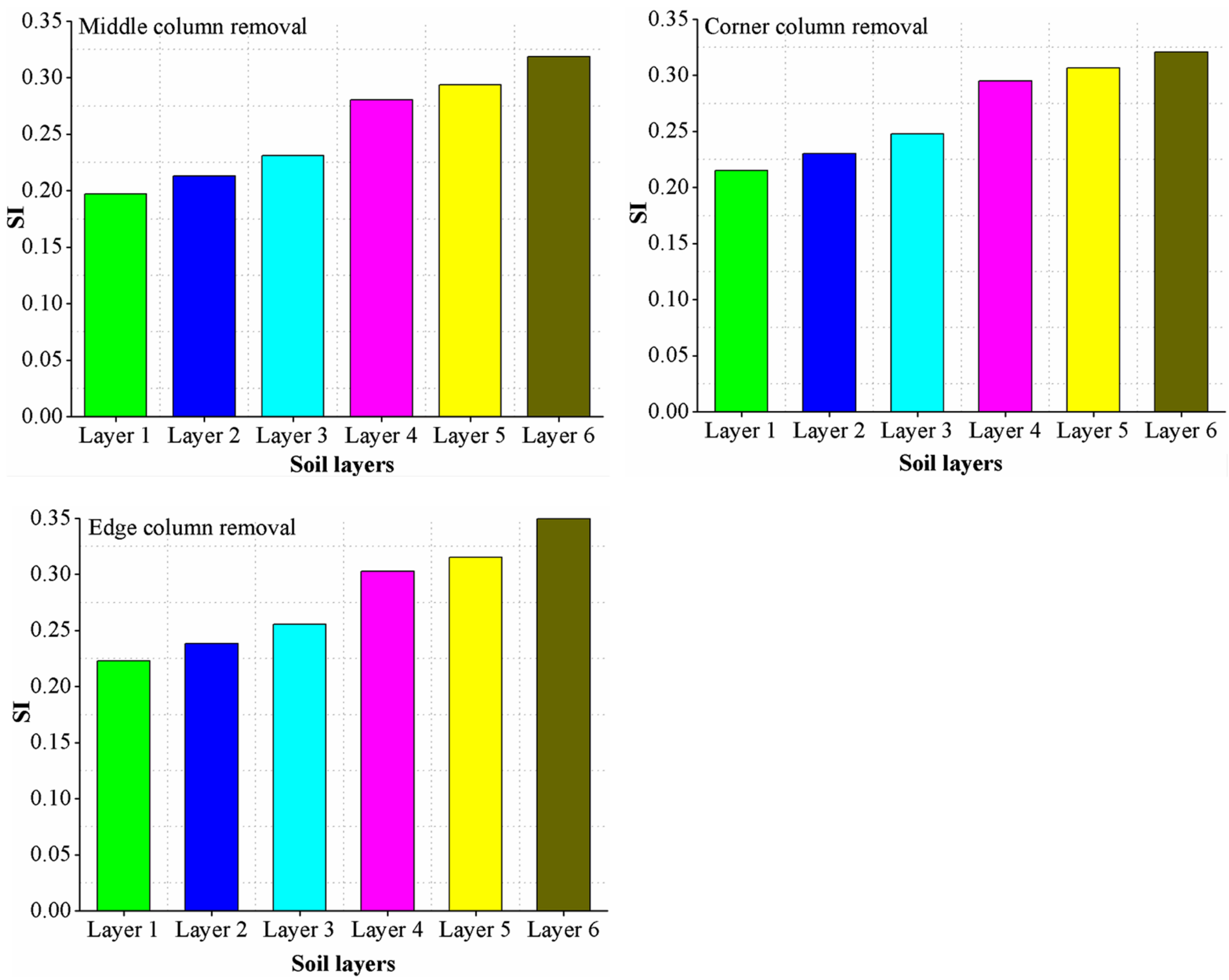


Fig. 17 The plot of sensitivity index versus soil layers.

without (middle, corner, and edge) column removal with different levels of groundwater were modeled in FLAC software. According to the DoD regulations, loading was done in the frames. The sensitivity index was determined for each level of groundwater and the results were compared with each other. The substructure soil layer was SM-CL/ML (very hard clay)-SM. The changes of SYY contours of the frames with (middle, corner, and edge) column removal with different levels of groundwater are shown in Fig. 18.

It is observed that in the case of column removal, the stress in the substructure of the adjacent columns of the removed one increase and shows the redistribution of force after column removal.

The sensitivity index for the framed structures with different levels of groundwater in the substructure soil is mentioned in Table 13. As the groundwater level of the substructure soil decreases, the sensitivity index decreases (Fig. 19). Therefore, by lowering the groundwater level from full saturation to − 40 m of groundwater and lower, the condition would be better to prevent progressive collapse.

As the soil layer (SM-CL/ML-SM) saturates, due to the change in the stress amplitude, the movement of soil particles and eventually the phenomenon of sudden subsidence occurs. Soil saturation conditions affect the parameters of shear strength and the rate of sudden soil subsidence. In the saturated state, the strength parameters (shear strength, friction angle, and shear modulus) are reduced compared to the dry state.

Due to the potential threats of high groundwater levels in the subsoil, which include reduced bearing capacity of the subsoil, demolition of the foundation, and damage of the structure, the dry condition of the subsoil is ideal to prevent the progressive collapse. Therefore, it is recommended to drain the soil before constructing the

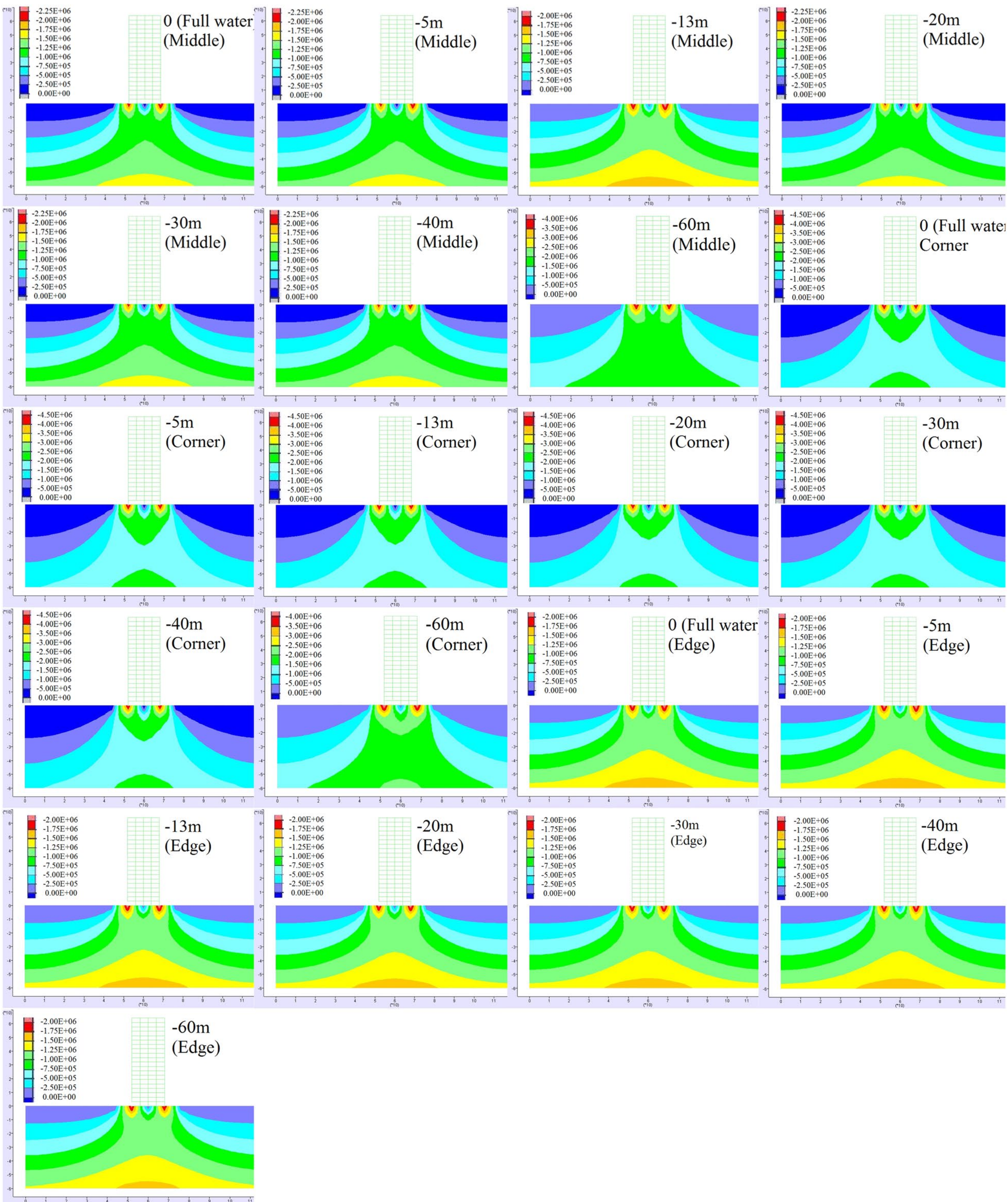


Fig. 18 SYY contours of the frames with column removal with different levels of groundwater.

Table 13 Redistribution of the imposed loads in the 20-story RC framed structures with different levels of groundwater in the substructure soil/

Conditions	Levels of groundwater (m)	λ_0 (kN)	λ_{damage} (kN)	SI
Middle column	0 (full water)	5770.5	4139.7	0.2826
	− 5	5616.7	4125	0.2656
	− 13	5462.2	4051.8	0.2582
	− 20	5260.2	3959.6	0.2473
	− 30	5235.3	4052	0.2260
	− 40	5174.2	4052	0.2169
	− 60	5153	4055.5	0.2130
Corner column	0 (full water)	5770.5	3997.67	0.3072
	− 5	5616.7	4053.31	0.2783
	− 13	5462.2	4029.16	0.2624
	− 20	5260.2	3943.14	0.2504
	− 30	5235.3	3943.14	0.2468
	− 40	5174.2	3954.61	0.2357
	− 60	5153	3966.78	0.2302
Edge column	0 (full water)	5770.5	3947.5	0.3159
	− 5	5616.7	3983.9	0.2907
	− 13	5462.2	3947.5	0.2773
	− 20	5260.2	3864.2	0.2654
	− 30	5235.3	3919.7	0.2513
	− 40	5174.2	3932.7	0.2399
	− 60	5153	3926	0.2381

structure in areas where the groundwater saturation level is high.

6 Conclusions

In this study, a numerical model was simulated and verified with the experimental Li model in SeismoStruct software. Then, it was developed and a 20-story RC frame without SSI was modeled in SeismoStruct software. Afterward, the simulated frame in SeismoStruct software was also simulated and verified in FLAC software. The effect of SSI on the progressive collapse of RC frames was investigated. Then, a parametric study of the framed structures and substructures was performed on the progressive collapse. The vulnerability of the frames against progressive collapse was assessed based on the sensitivity index, following the column removal. Based on the studies, the following results are obtained:

– The numerical model was simulated in SeismoStruct software and verified with the Li experimental model. The experimental and numerical load capacities exhibited a discrepancy of about 7.4% and 4% at the first and second rebar fractures, respectively; whereas, the mismatch in terms of downward displacement in the first rebar fracture was 2.47% and in the second one was about 2.3%. There was a good agreement between the results of the experimental and numerical models.
– The results of pushdown analysis on the progressive collapse of the RC frames showed that the edge column is more critical than the corner one and the corner column is more critical than the middle one. All of the following investigations are similar in this result.
– Comparing the data of modeling the 20-story RC frames in SeismoStruct, and FLAC software indicated that by considering the soil–structure interaction, the sensitivity index decreases. In fact, by considering the effect of SSI, the shear forces and flexural anchors in the stories of the framed structure are reduced. This leads to smaller dimensions for the design of structural elements.
– The effect of different foundation thicknesses (180–210 cm) on the progressive collapse of the frames was investigated by considering SSI. The results showed that increasing the thickness of the foundation increases the maximum subsidence, the average subsidence, and the flexural anchorage of the foundation. Its only positive effect is the reduction of non-uniform subsidence of the foundation. By comparing the values of the sensitivity index in the studied foundation thicknesses, the results showed that in the foundation thickness of 180 cm, the sensitivity index is lower than the other thicknesses. Therefore, the condition is better to prevent progressive collapse.
– The effect of different soil densities (1600–2200 kg/m^3) on the progressive collapse of the frames was investigated. The results showed that with increasing the soil density, the sensitivity index decreases. Consequently, the situation would be better for preventing progressive collapse.
– The effect of different soil types on the progressive collapse of the framed structures was studied. Different soils of rock, SM, CL-ML, GW, and clay-MC were selected for substructure. The results showed that by changing the type of substructure soil from clay-MC to rock, the sensitivity index decreases. Therefore, the use of rock and SM, compared to other soil types is better for preventing progressive collapse.
– The effect of different soil layers (layer 1 to layer 6) on the progressive collapse of the frames was investigated. The results showed that by changing the soil layers from layer 6 to layer 1, the sensitivity index decreases. Therefore, layer 1 (SM) and layer 2 (SM-

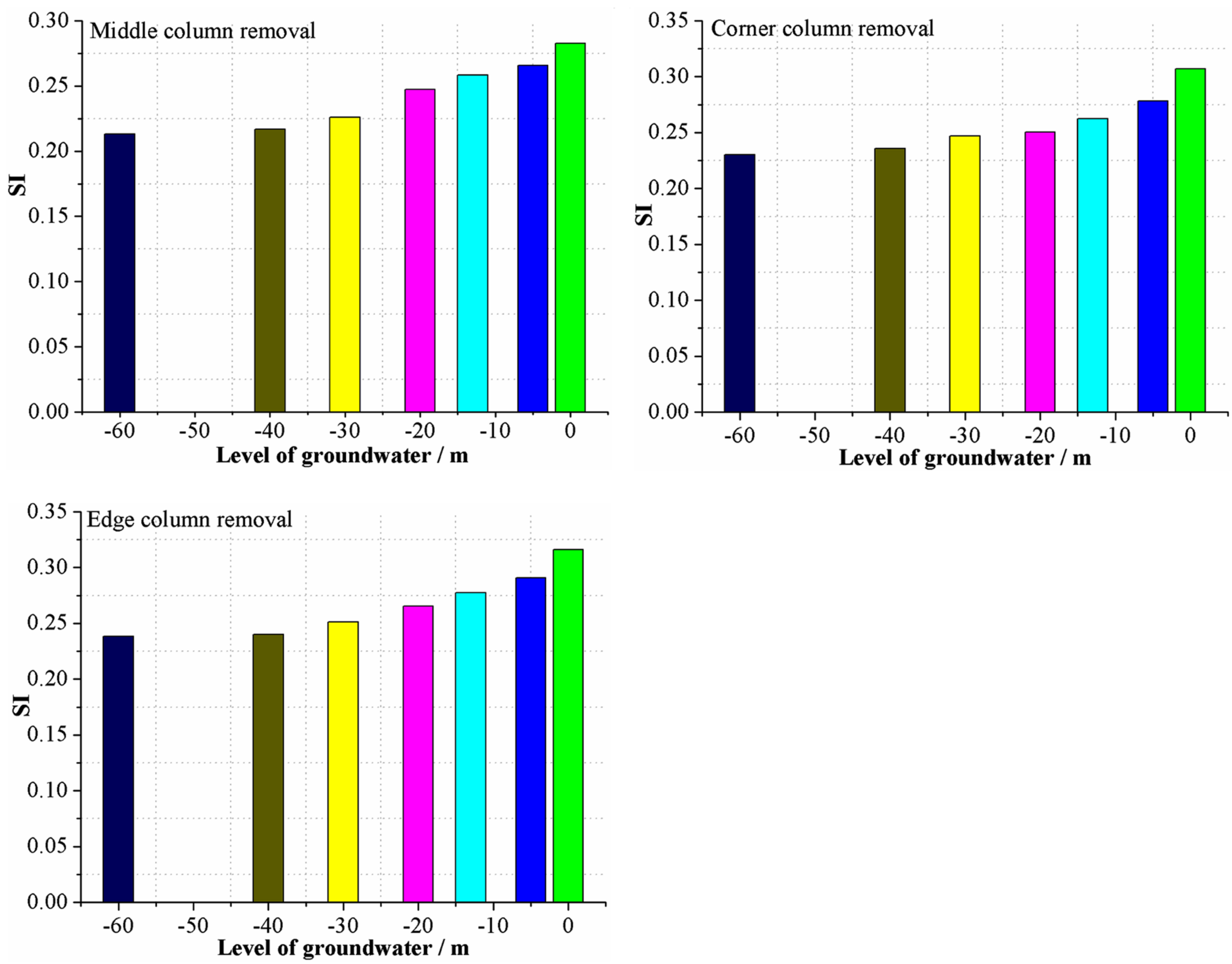


Fig. 19 The plots of the sensitivity index versus the groundwater levels in the substructure soil.

CL/ML (very hard clay)-SM), compared to other soil layers are better for preventing progressive collapse.

– The effects of the soil saturation conditions on the progressive collapse of the framed structures were studied. The results showed that as the groundwater level of the substructure soil decreases, the sensitivity index decreases. Therefore, by lowering the groundwater level from full saturation to − 40 m of groundwater and lower, the condition would be better to prevent progressive collapse. Due to the potential threats of high groundwater levels in the subsoil, which include reduced bearing capacity of the subsoil, demolition of the foundation, and damage of the structure, the dry condition of the subsoil is ideal to prevent the progressive collapse. Therefore, it is recommended to drain the soil before constructing the structure in areas where the groundwater saturation level is high.

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Author contributions
AK provided the basic idea and critically revised this article. SAEK performed the literature search, data analysis, and draft writing. AM contributed to the further literature search and revising. All authors read and approved the final manuscript.

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Declarations

Competing interests
The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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