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Progressive Collapse Analysis of Steel Structures on Elastic Foundation: Part-1. Linear Static Procedures

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Abstract

Progressive collapse analysis, a new challenging area of research in Civil Engineering, has being applied to the strategically important structures in many developed countries. This type of analysis is especially of great importance for societies that have been struggling with the terrorism. However, the progressive collapse analysis has not yet gain enough attention in developing countries. Studies on progressive collapse analysis in the literature have especially increased after 2000s and until today. Experimental and analytical studies on the subject have been obtaining usually by assuming a rigid base underneath the building in consideration. However, many studies show that the structural behavior and so the internal forces of the frame are effected considerably when elastic subsoil is assumed underneath the building. Contrary to the studies performed in the literature with rigid base assumption, in this study which is consists of two parts, the soil-structure interaction is also taken into account in the progressive collapse analysis of steel frames. In this first part, a progressive collapse analysis of a 10-storey steel structure is carried out using linear static procedures in the UFC 4-023-03 guide of the United States Department of Defense. Three different column removal scenarios were applied to the building using the Alternate Way method via SAP2000 software. The cases of building resting on a rigid base and elastic foundation were examined comparatively. The Modified Vlasov elastic subsoil model has been included in the analysis of SAP2000 software with an interface coded in the MATLAB. The modulus elasticity of the subsoil (E_s) was considered as loose sand, dense sand and gravel, and hard clay, respectively. The results are presented in the tables. It is concluded that the soil-structure interaction effect significantly changes the progressive collapse behavior of the steel frame depending on the column removal scenarios.

Keywords: *Progressive collapse, Steel structures, Linear static procedures, Modified Vlasov model, OAPI.*

1 Introduction

The concept of progressive collapse is first used in the literature due to the chain collapse mechanism that occurred in an apartment building in London in 1968. In general terms, this concept is defined as the failure of a load-bearing system to balance after the disappearance of a structural member in a building and as local or total collapse of the building. Researchers have been working on this topic since a report published on collapse of the apartment of Ronan Point in 1970. Apart from design loads, it has been observed that especially in strategically important structures, extremely high loads that are likely to be exposed during their long service life should also

be considered. Today, the concept of progressive collapse is still up to date and attracts the attention of researchers.

Mohamed and Khattab (2019) studied progressive collapse resistance of a steel framed structure based on the Alternate Path method using linear static, non-linear static and non-linear dynamic analysis methods. Lee et al. (2018) evaluated the progressive collapse resistance performance of steel moment frames of which designed with different connection details such as welded unreinforced flange-bolted web (WUF-B) and reduced beam section (RBS) connections. Yousefi et al. (2014) investigated the vulnerability of steel moment resistant frames in building structures seismically designed according to the regulations of Iranian National Building Codes for design and construction of steel structures, using the guidelines of UFC regulations for alternative path method. They showed that the building structures do not meet the criteria stated in UFC. Hence, they have proposed some modifications on the INBC codes to meet the UFC limits. Rezvani et al. (2015) investigated the effect of span length on the structural safety levels against progressive collapse of seismically designed steel moment resisting frames. Mahmoudi et al. (2015) proposed a new displacement-based method for nonlinear static progressive collapse analysis. Rezazadeh et al. (2019) evaluated the resistance of sample steel moment frames with knee bracings using nonlinear static alternate path method proposed by UFC guideline. Kheyroddin et al. (2019) developed a modified sensitivity analysis (MSA) to figure out the location of the most critical column called the key element for progressive collapse analysis. In almost all the studies about progressive collapse until today, it has been seen that the structure is supported by a rigid base and subsoil effects are neglected. However, it is known that the subsoil effect is important for the behavior of the building. In this study, a 10-storey steel building has been studied by taking the subsoil effects into consideration. Subsoil effects are modelled by using Modified Vlasov model, the accuracy and reliability of which has been previously demonstrated in the literature. The analyses are performed via UFC 4-023-03 (2016) guideline used by the US Defense Department for military structures built in countries such as America, Afghanistan, Iraq and Syria. The Alternative Way method in the guideline was chosen and the linear static procedure (LSP) was used for the progressive collapse analysis.

2 Modified Vlasov Model

One of the oldest and most well-known model used in soil-structure interaction problems is the Winkler model. However, in this model, since the subsoil is modeled as a completely discontinuous stratum with the separate springs, the interaction within the subsoil medium and the effect of the soil outside the building foundation are neglected. Over time, subsoil models to consider the continuity in the soil stratum have been developed. In most of these models, in addition to the subgrade reaction modulus (k) used in the Winkler model, the shear parameter ($2t$) to consider the interaction between the springs is defined. But, the difficulty arises here in revealing the relationship between soil parameters (k and $2t$) and soil properties. These parameters are considered as a function of soil properties only in most of the models and taken from tables according to soil type from various books. A third parameter is proposed in some other models called three-parameter models. The Modified Vlasov subsoil model being one of these methods is used in this study (Fig 1). All three soil parameters are being calculated as a function of both the soil properties and the geometry of the structure along with type and distribution of the loading. (Vallabhan et al. (1988); Vallabhan et al (1991)).

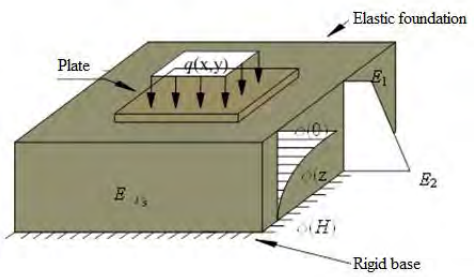


Figure 1. A plate resting on elastic foundation

The subsoil effect in the model is expressed as follows, depending on the vertical displacements (w) of the plate.

$$q_z = -2t\nabla^2 + kw \tag{1}$$

In this expression, the subgrade reaction modulus k, also called the first parameter of the subsoil, and the soil shear parameter $2t$, known as the second parameter of the subsoil, are calculated as follows.

$$k = \int_0^H \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)} \left(\frac{\partial \varphi(z)}{\partial z} \right)^2 dz \tag{2}$$

$$2t = \int_0^H G_s \phi(z)^2 dz \tag{3}$$

In the above expressions, H is the depth of the soil, ν_s is the Poisson's ratio of the soil and G_s is the shear modulus of the soil. When the $2t$ parameter is equal to zero in Eq 1, the reaction of the subsoil for the plate resting on the Winkler type elastic foundation is obtained. The function $\phi(z)$ showing the variation of the vertical displacement of the subsoil underneath the structure and the nondimensional soil surface parameter γ are given by the following equations.

$$\phi(z) = \frac{Sinh\gamma(1-\frac{z}{h})}{Sinh\gamma} \tag{4}$$

$$\left(\frac{\gamma}{H}\right)^2 = \frac{(1-2\nu_s)}{2(1-\nu_s)} \frac{\int_{-\infty}^{+\infty}\int_{-\infty}^{+\infty}(\nabla w)^2 dxdy}{\int_{-\infty}^{+\infty}\int_{-\infty}^{+\infty} w^2 dxdy} \tag{5}$$

The function $\phi(z)$ is equal to 1 at $z = 0$ and zero at $z = H$. In other words, the vertical displacement of the subsoil is zero at depth H while it is equal to the vertical displacement of the plate at the soil-structure interface. (Vallabhan and Daloglu (1999); Turhan (1992); Ozgan(2007)).

Hamarat (2012) and Hamarat et al. (2012) used the nonlinear / layered shell element and orthotropic material behavior in his study to consider the interaction between springs. By using OAPI functions with an interface coded in Excel, he has enabled the use of the Modified Vlasov model in SAP2000. In this study, via an interface developed in MATLAB following the approach proposed by Hamarat (2012), SAP2000 and MATLAB programs are used simultaneously through OAPI functions.

3 Linear Static Procedures for Progressive Collapse Analysis

The use of the Linear Static Procedures in the Progressive Collapse Analysis is limited to structures that meet some requirements given the UFC 4-023-03 (2016) manual. If there are no structural irregularities as defined in the manual, a linear static procedure may be performed and it is not necessary to calculate the DCRs. The progressive collapse resistance of the structures can be determined by using the coefficient calculated by proportioning the internal forces in the beams connected to the removed column to the expected (average) strength of the same beam section.

$$m \geq Q_{UD} / (\phi Q_{CE}) \tag{6}$$

where Q_{UD} is internal forces, from Linear Static model, m is a limit value (m-factor), ϕ is strength reduction factor from the appropriate material specific code and Q_{CE} is expected strength of the component or element. In the method, the progressive collapse resistance of the structure is decided by considering whether the ratio calculated above exceeds the limit value. If the calculated value is greater than the limit value, it is understood that the structural member is insufficient, and if it is small, it provides the gradual collapse resistance. Before m is calculated, the model is analyzed under vertical loads by applying the load combination given below.

$$G = 1.2D + (0.5L \text{ or } 0.2S) \tag{7}$$

Here G is vertical design load, D is dead load, L is live load and S is show load. Using the data obtained from the analysis, the m-factor value and the load increase factor (Ω_{LD}) are taken from the Table 4-2 and Table 3-4 in the UFC 4-023-03 (2016) guideline respectively. Then, the vertical design loads (G_{LD}) to be applied depending on the column removal scenario are calculated from the following equation.

$$G_{LD} = \Omega_{LD}\left[1.2D + (0.5L \text{ or } 0.2S)\right] \tag{8}$$

The calculated loads are assigned to the model according to the figure given below, and progressive collapse analysis is performed for each scenario UFC 4-023-03 (2016).

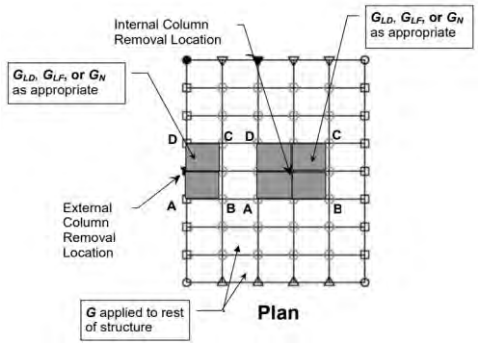


Figure 2. Loads and load locations

4 Numerical Example

A 10-storey steel building taken as a numerical example studied by Tansel (2010) previously (Fig.3). It is a residential building with the dimensions of 21 m × 13 m and 3.30 m story height.



Figure 3. Formwork plan of the building

The grade of the concrete and steel are C25 and S355 respectively. Subsoil depth is taken as 35 m. Poisson ratio and elasticity modulus of the subsoil are 0.30 and 18000 kN/m^2 for loose sand, 90000 kN/m^2 for dense sand and gravel, 180000 kN/m^2 for hard clay respectively. The column cross-sections are HE500B at the first 4 floors, HE450B at the 4th and 5th floors and HE400B at the last 4 floors. Primary beams are IPE330, secondary beams are IPE270. The connection of primary beams to the weak axes of columns are hinged. Primary beams are connected to the strong axes of the columns rigidly. Dead load and live load for roof slab and other slabs are 5.00 kN/m^2, 1.50 kN/m^2, 4.50 kN/m^2 and 2.00 kN/m^2 respectively. Slab loads are converted to line loads and assigned as external load on the secondary beams. Also; self-weight of secondary beam is taken as 0.361 kN/m for IPE270 and self-weight of primary beam is 0.491 kN/m for IPE330. Besides 7.0 kN/m weigth is added for the external walls on primary beams. Then the building modeled in SAP2000 is analyzed with and without soil structure interaction effect.

According to UFC 4-023-03, acceptance criteria for linear procedures of structural steel components should be taken from Table 9-4 in ASCE 41-13 (Table 1). Various values of *m*-factor are presented in the table depending on the behavior of the beam. In the example, only the relevant part of the table (Beams-Flexure) is shown, since the bending behavior is effective. The suitable value of *m*-factor is decided according to the flange and web slenderness of the element. For bending elements, the CP performance point according to the guideline shows the limit value. Accordingly, the value of *m*-factor was calculated as 8.00 for the example in consideration.

Table 1. Acceptance criteria for linear procedures of structural steel components (ASCE 41-13, Table 9-4).

Component/Action	m-Factors for Linear Procedures*				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Beams—Flexure					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_y}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_y}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_y}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_y}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				

If steel frame connections are to be considered in the calculations, the *m*-factor should also be determined for the connections. Then, the smaller of these values should be used as the *m*-factor. For connections, acceptance criteria for linear static modeling should be taken from the Table 5-1 of the UFC 4-023-03 guide (Table 2).

Table 2. Acceptance criteria for linear static modelling of steel frame connections (UFC 4-023-03, Table 5-1).

Connection Type	Linear Acceptance Criteria *m*-factors	
	Primary[1]	Secondary[1]
Fully Restrained Moment Connections		
Improved WUF with Bolted Web	2.3 – 0.021d	4.9 – 0.048d
Reduced Beam Section (RBS)	4.9 – 0.025d	6.5 – 0.025d
WUF	4.3 – 0.083d	4.3 -0.048d
SidePlate©	6.7 – 0.039d[2]	11.1 – 0.062d
Partially Restrained Moment Connections (Relatively Stiff)		
Double Split Tee		
a. Shear in Bolt	4	6
b. Tension in Bolt	1.5	4
c. Tension in Tee	1.5	4
d. Flexure in Tee	5	7
Partially Restrained Simple Connections (Flexible)		
Double Angles		
a. Shear in Bolt	5.8 – 0.107d_{bg}[3]	8.7 – 0.161d_{bg}
b. Tension in Bolt	1.5	4
c. Flexure in Angles	8.9 – 0.193d_{bg}	13.0 – 0.290d_{bg}
Simple Shear Tab	5.8 – 0.107d_{bg}	8.7 – 0.161d_{bg}

[1] Refer to Section 3-2.4 for determination of Primary and Secondary classification
[2] d = depth of beam, inch
[3] d_{bg} = depth of bolt group, inch

In the example, the beams are connected with hinged (Shear in Bolt) in the weak axis of the column and with rigidly (improved WUF with bolted web) in the strong axis of the column. *m*-factor values for fully restrained moment connection and partially restrained simple connection are calculated as 2.03 and 4.96 respectively. According to the guideline, the smallest of the *m*-factors calculated for steel components and connections should be used. Therefore, the *m*-factor was determined as 2.03. After the column removal scenarios, the *m* values calculated for the beams should not be more than 2.03. Otherwise, it means that the progressive collapse resistance of the building is insufficient.

The load increase factor is calculated as 2.927 using Table 3-4 in UFC 4-023-03 (Table 3). Using Eq 7 and 8, increased gravity load combination (G_{LD}) for those bays immediately adjacent to the removed element and for all floors above the removed element and gravity load (G) for other floors were calculated as follows. G_{LD}=19.099 kN/m^2 and G=6.525 kN/m^2 for roof slabs and G_{LD}=19.008 kN/m^2 and G=6.400 kN/m^2 for other slabs.

Table 3. Load increase factors for linear static analysis (UFC 4-023-03, Table 3-4).

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	0.9 m_{LIF} + 1.1	2.0
Reinforced Concrete	Framed[A]	1.2 m_{LIF} + 0.80	2.0
	Load-bearing Wall	2.0 m_{LIF}	2.0
Masonry	Load-bearing Wall	2.0 m_{LIF}	2.0
Wood	Load-bearing Wall	2.0 m_{LIF}	2.0
Cold-formed Steel	Load-bearing Wall	2.0 m_{LIF}	2.0

Progressive collapse analysis was performed for each column removal scenario and beam maximum moments were taken from SAP2000. In addition, the expected (average) moment strength for IPE 330 was calculated as 371.61 kNm by drawing the beam section in the Section Designer of the SAP2000. These values are used in Eq 6, and the value of *m* for each beam were calculated.

4.1 B4 Column removal scenario

The B4 column has been removed as the first scenario. The system was solved again by loading the G and G_{LD} to the floors according to Fig 2. The *m* ratios given in Eq 6 were calculated by using the beam moments obtained from the analysis. The *m* ratios obtained by the linear static analysis method are given in Table 4.

Table 4. Changes of m ratio with subsoil kind for B4 column removal (m_{factor}=**2.03**).

Elasticity modulus of subsoil (kN/m^2)									
Beams in the x-z plane					Beams in the y-z plane				
	Without subsoil	180000	90000	18000		Without subsoil	180000	90000	18000
K117	1.52	1.50	1.52	1.74	**K126**	0.58	0.52	0.49	0.39
K217	1.46	1.44	1.48	1.68	**K226**	0.54	0.49	0.45	0.33
K317	1.40	1.38	1.43	1.61	**K326**	0.51	0.46	0.43	0.36
K417	1.34	1.32	1.39	1.56	**K426**	0.48	0.43	0.40	0.32
K517	1.28	1.26	1.34	1.52	**K526**	0.45	0.40	0.37	0.29
K617	1.23	1.21	1.31	1.48	**K626**	0.42	0.38	0.35	0.27
K717	1.19	1.16	1.27	1.44	**K726**	0.40	0.35	0.32	0.24
K817	1.16	1.14	1.26	1.43	**K826**	0.38	0.33	0.30	0.22
K917	1.16	1.13	1.26	1.43	**K926**	0.36	0.31	0.28	0.20
K1017	1.06	1.04	1.13	1.26	**K1026**	0.31	0.27	0.25	0.17

The results for column B4 removal scenario indicate that, compared to those obtained without subsoil, the *m* values of the beams (K117-K1017) located in the x-z plane decrease in the range of 1-3% for E_s=180000 kN/m^2. In the other hand, it is observed increases in the range of 1-9% for E_s=90000 kN/m^2 and 14-23% for E_s=18000 kN/m^2 respectively. In the y-z plane, on the contrary, *m* values decreased as the soil become more flexible. This reduction occurred in the range of 9-45%. The subsoil effect on the results is clearly seen in both planes. Since all *m* values are less than the *m*-factor value, the column B4 removal scenario does not pose a progressive collapse risk in the building.

4.2 D2 Column removal scenario

The column D2 has been removed as the second scenario. The system was analyzed again by loading the G and G_{LD} to the floors according to Fig 2. The *m* ratios given in Eq 6 were calculated by using the beam moments obtained from the analysis. The *m* ratios obtained by the linear static analysis method are given in Table 5.

Table 5. Changes of m ratio with subsoil kind for D2 column removal (m_{factor}=**2.03**).

Elasticity modulus of subsoil (kN/m^2)									
Beams in the x-z plane					Beams in the y-z plane				
	Without subsoil	**180000**	**90000**	**18000**		**Without subsoil**	**180000**	**90000**	**18000**
K108	0.48	0.42	0.43	0.47	**K131**	3.68	3.54	3.35	3.31
K208	0.50	0.44	0.44	0.49	**K231**	3.78	3.64	3.63	3.59
K308	0.51	0.45	0.45	0.50	**K331**	3.66	3.53	3.52	3.53
K408	0.52	0.46	0.47	0.51	**K431**	3.58	3.45	3.44	3.41
K508	0.53	0.48	0.48	0.52	**K531**	3.50	3.37	3.37	3.39
K608	0.54	0.48	0.49	0.52	**K631**	3.43	3.31	3.30	3.32
K708	0.55	0.49	0.49	0.53	**K731**	3.38	3.26	3.26	3.27
K808	0.56	0.50	0.50	0.54	**K831**	3.35	3.24	3.24	3.28
K908	0.56	0.58	0.58	0.55	**K931**	3.42	3.30	3.30	3.33
K1008	0.70	0.63	0.63	0.67	**K1031**	3.13	3.02	3.01	3.04

In the column D2 removal scenario, *m* values obtained by considering soil-structure interaction or by neglecting subsoil effects are close the each other. When the subsoil effects are considered or soil type is changed, the *m* values did not change as much as in the first column removal scenario. The subsoil effects caused the *m* values to decrease by approximately 2-12%. The *m* ratios of all beams in the y-z plane in the building exceeded the *m*-factor value in all cases, and progressive collapse resistance was not achieved in the building.

4.3 F4 Column removal scenario

The column F4 has been removed as the last scenario. The system was analyzed again by loading the G and G_{LD} to the floors according to Fig 2. The m ratios given in Eq 6 were calculated by using the beam moments obtained from the results. The m ratios obtained by the linear static analysis method are given in Table 6.

Table 6. Changes of m ratio with subsoil kind for F4 column removal (m_{factor}=**2.03**).

Elasticity modulus of subsoil (kN/m^2)									
Beams in the x direction					Beams in the y direction				
	Without subsoil	**180000**	**90000**	**18000**		**Without subsoil**	**180000**	**90000**	**18000**
K120	1.53	1.58	1.61	1.73	**K130**	0.73	0.78	0.81	0.95
K220	1.47	1.52	1.56	1.70	**K230**	0.73	0.79	0.84	1.04
K320	1.41	1.46	1.50	1.65	**K330**	0.71	0.74	0.77	0.90
K420	1.35	1.40	1.44	1.58	**K430**	0.68	0.72	0.74	0.88
K520	1.30	1.34	1.38	1.51	**K530**	0.66	0.69	0.72	0.85
K620	1.25	1.30	1.33	1.45	**K630**	0.63	0.67	0.69	0.82
K720	1.21	1.25	1.28	1.39	**K730**	0.61	0.64	0.66	0.78
K820	1.18	1.22	1.25	1.35	**K830**	0.59	0.61	0.64	0.75
K920	1.16	1.19	1.22	1.32	**K930**	0.57	0.60	0.62	0.73
K1020	1.05	1.08	1.10	1.20	**K1030**	0.50	0.52	0.54	0.63

The results indicate that when soil-structure interaction is considered, the *m* ratios increase by approximately 3 to 42% compared to those obtained with the acceptance of rigid base. However, it is seen that the results obtained for the dense sand and gravel and hard clay are very close to each other. As the subsoil softens, its effect on the results increases. For this scenario, it is seen that all beams have *m* ratios less than *m*-factor, so there is no need to re-design the building against progressive collapse.

5 Conclusions

It is known that local or total collapses occur in buildings due to unexpected load effects such as gas explosions and terrorist attacks. The concept of progressive collapse of which first used in literature after the gas explosion that occurred in the Ronan Point apartment in London in 1968 and the subsequent partial collapse, had an

increasing interest especially with the terrorist attack on the World Trade Center on September 11, 2001. Nowadays, in developed countries, the progressive collapse risk analyses began to be performed for especially strategically important structures.

A 10-storey steel building has been studied for progressive collapse analysis using linear static procedures for three different columns removal scenarios. In one of the column removal scenarios, it has been observed that the building cannot provide the progressive collapse resistance. It is difficult to reach a general conclusion for each column removal scenario about the effect of soil-structure interaction on the risk of progressive collapse for the building analyzed using the LSP method in the study. However, it can be said that the subsoil effect significantly changes the progressive collapse behavior of the building depending on the column removal scenario.

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