

EFFECTS OF VERTICAL SEISMIC ACTIONS ON THE RESPONSES OF SINGLE-STOREY INDUSTRIAL STEEL BUILDING FRAMES

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Abstract

Single-storey industrial steel frames with cranes are considered as being vertically irregular in structural configuration and load distribution under strong earthquake excitations. In this paper, various frames with their spans of 20, 26, 32 and 38 m and locations built in Hanoi and Son La regions were designed to resist dead, roof live, crane and wind loads. The equivalent horizontal and vertical static earthquake loads applied on the frames were determined. Next, by using linear elastic analyses of structures, the effects of vertical seismic actions on the responses of the frames were evaluated in terms of the ratios K_1 and K_2 at the bottom and top of the columns corresponding to different combinations of dead loads and static earthquake loads, as denoted by CE1, CE2 and CE3. The effects of seismic actions compared with those of wind actions were also evaluated in terms of the ratios K_3 and K_4 . As a result, the effects of vertical seismic actions were significant and increased with the span lengths of the frames. In addition, by using nonlinear inelastic analyses of structures, the levels of the static earthquake loads were determined corresponding to the first yielding and maximum resistances of the frames.

Keywords: single-storey industrial buildings; steel frames; span lengths; irregularity; vertical seismic actions; earthquake levels; wind loads.

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1. Introduction

It has been recognized that the procedure for earthquake-resistant design of a building structure consists of two analysis stages [1–4]. In the first stage, the analysis method for no-damage requirement of structure under equivalent static earthquake loads is used for design of the structural members, so-called linear elastic analysis of structure. The earthquake load used at this design stage needs to be significantly reduced in comparison to that corresponding to maximum design earthquakes when a completely linear elastic behavior of the structure is assumed. This reduction in load is represented in general by the use of a strength reduction factor (e.g., the structural behavior factor specified in EC8 [1]). Thus, the equivalent static earthquake load is considered as an elastic design threshold in order to determine the design internal forces in the structural members. This load corresponds to frequent earthquakes that can occur during the building life of 50 years, which can be assumed to have a mean return period of 95 years or 41-percent probability of exceedance in 50 years [1]. Under the equivalent static earthquake load, the structure is considered to be undamaged and the material works within an elastic limit.

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Next, in the second stage, the analysis method for damage limitation requirement is used for prevention of local and global collapses of the structure under maximum design earthquakes, so-called nonlinear structural analysis of structure. This corresponds to rare earthquakes that may occur once during the 50-year use of the building, which is often assumed to have the mean return period of 475 years or 10-percent probability of exceedance in 50 years of using the building [1]. In this case, the earthquake excitation transmitted to the building is represented in term of ground acceleration motions and the inelastic behaviors of structural materials are resulted in term of plastic hinges characterized by the maximum ductility factors [5–7].

In this study, single-storey industrial steel frame structures are considered with their characteristics of large column heights, long beam spans, sloping roof beams and traveling crane loads applied on column cantilevers. It can be said that these frame structures are categorized as being vertically irregular in structural configuration and load distribution [8–13]. In addition, the vertical vibration of the roof beams will increase the bending moments occurred at both ends of the columns and beams and consequently increase the load-bearing capacity requirements.

As specified in EC8, the value of the behavior factor is often reduced by 20% for design of irregular structures. This means that the corresponding equivalent horizontal static earthquake loads used at the first analysis stage are increased by 20% in comparison to those used for regular structures. The increase in load corresponds to the probability of a greater earthquake occurrence with the mean return period of 116 years or 35-percent probability of exceedance in 50 years of using the building, rather than 95 years or 41-percent probability as mentioned above. However, this specification may be conservative for single-storey industrial steel structures as vertically irregular ones. In addition, other issues need to be studied including the evaluation of structural irregularities and structural behavior factors used for determining the equivalent static earthquake loads, which is out of scope of this paper. Also the effect of vertical vibration can not be considered in studies based on analyses of single-degree-of-freedom systems [5, 14].

For the evaluation of the effect of vertical vibration on the response of single-storey industrial steel structures with cranes under earthquakes, various frames were considered with the spans of 20, 26, 32 and 38 m and they were assumed to be built in Hanoi and Son La regions, in which the former location has strong earthquakes and strong winds while the later one has very strong earthquakes but small winds. These frames were designed in accordance with Vietnamese standards [4, 15, 16] and EC8 to ensure the structures with adequate capacities against dead load, roof live loads, wind forces and crane loads. Thus, a total of eight frames considered with different span lengths and construction regions were examined in this study. Next, the effect of vertical earthquake excitation was evaluated by using linear elastic static structural analysis under the equivalent static earthquake loads applied in horizontal and vertical directions. In addition, nonlinear inelastic static analyses of structures were used to evaluate the inelastic responses of the frames. The results show the effect of vertical vibration on the structural responses, which depends on the frame span lengths and seismic locations.

2. Design of single-storey industrial steel building frames

2.1. Description of analytical frames

Consider typical single-storey industrial steel building frames with their single spans of 20, 26, 32 and 38 m in length; frame bays of 6.5 m; and roof beam slopes of 10 degrees. Longitudinal struts were located at 3.7 m from the footing level to support the columns in out of the frame plane. Fig. 1 shows the configuration of analytical frames considered, in which the lengths $L_1 = 3, 4, 5, 6$ m and

$L_2 = 4, 5, 6, 7$ m correspond to the frame spans of 20, 26, 32, 38 m, respectively. The sky doors had their heights of 2 m. The buildings were assumed to be built in Hanoi and Son La regions. There were eight analytical frames considered as shown in Table 1.

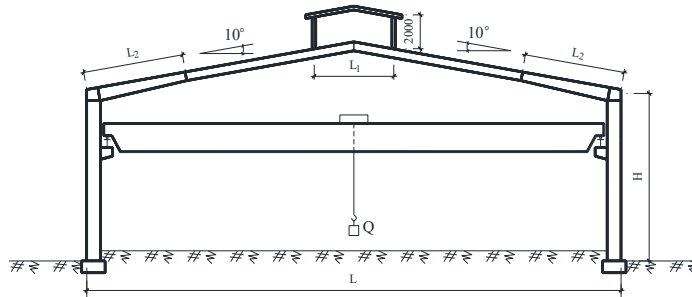


Figure 1. Configuration of single-storey industrial steel building frames

Table 1. Analytical frames

No.	Frames	Span lengths (m)	Crane capacities (kN)	Locations
1	H-20-100	20	100	Hanoi
2	H-26-100	26	100	Hanoi
3	H-32-100	32	100	Hanoi
4	H-38-100	38	100	Hanoi
5	S-20-200	20	200	Son La
6	S-26-200	26	200	Son La
7	S-32-200	32	200	Son La
8	S-38-200	38	200	Son La

2.2. Loads used for design of frames

a. Dead load

The characteristic dead loads applied on the frames consist of the self weight of the roof cladding system of 0.25 kN/m^2 (including the profile sheeting, insulation layer, purlins, roof braces), which is assumed to be uniformly distributed on the roof plane; and the self weight of the peripheral wall system of 0.18 kN/m^2 (including the profile sheeting, skirts, column braces) to be uniformly distributed on the wall plane. In addition, the self weight of a single crane runway girder with the span of 6.5 m, including the crane rail fastened on the girder, was 17.67 kN and applied on the column bracket. The self weight of the structural frame members (columns and beams) was automatically generated in the analysis program. The safety factor of dead load is taken as 1.1.

b. Roof live load

The characteristic live loads applied on the building roofs were taken as 0.3 kN/m^2 assumed to be uniformly distributed with respect to the building ground plan [15]. For determination of critical forces, there are three possible cases of live loads assumed acting on the half-left, half-right and full spans of the frames. The safety factor of live load is taken as 1.3.

c. Wind load

The characteristic wind loads acting on the frames were determined according to TCVN 2737:1995 [15], in which the characteristic wind pressures were taken as 0.95 and 0.55 kN/m² for Hanoi and Son La regions, respectively. These pressures correspond to the mean velocities of wind of 40 and 30 m/s, respectively. The topography type C was used for these areas. The safety factor of wind load is 1.2.

d. Crane load

The maximum lifting loads that each crane can carry were taken as 100 and 200 kN for the frames built in Hanoi and Son La regions, respectively. All the cranes were assumed to operate with medium frequencies of use. There were two traveling cranes operating together in each frame span. The safety factor of crane load is 1.1. As a result, Table 2 shows the maximum and minimum vertical forces, $D_{\max}$ and $D_{\min}$, caused from the two cranes acting on the frames through the column cantilevers; the maximum horizontal forces, $T_{\max}$, transferred to the columns at the level of top of the crane runway girders; and the self weight of two crane bridges, W_{cb} .

Table 2. Vertical and horizontal forces from cranes (kN)

Frames	$D_{\max}$	$D_{\min}$	$T_{\max}$	W_{cb}
H-20-100	171.48	42.74	7.12	44.02
H-26-100	185.66	63.44	6.52	67.47
H-32-100	198.50	88.78	5.93	95.32
H-38-100	208.54	106.78	5.49	114.86
S-20-200	318.34	67.58	13.99	66.84
S-26-200	322.62	83.30	12.94	88.34
S-32-200	321.05	108.44	11.53	115.99
S-38-200	325.08	124.28	10.80	134.30

2.3. Design dimensions of beam and column sections

Table 3 shows the cross-section dimensions of beams and columns derived from the design of the frames in accordance with the Vietnamese standards [15, 16]. These dimensions were checked to

Table 3. Design cross-sections of columns and beams (mm)

Frames	Column flanges	Column webs	Beam flanges	Beam webs	
				At ends	At middles
H-20-100	300 × 10	550 × 10	300 × 10	480 × 8	300 × 8
H-26-100	300 × 10	650 × 8	300 × 10	650 × 8	400 × 8
H-32-100	300 × 10	680 × 10	300 × 10	600 × 8	430 × 8
H-38-100	300 × 12	730 × 10	300 × 10	650 × 8	480 × 8
S-20-200	300 × 10	550 × 10	300 × 10	500 × 8	350 × 8
S-26-200	300 × 10	660 × 8	300 × 10	580 × 8	380 × 8
S-32-200	300 × 10	700 × 10	300 × 10	620 × 8	430 × 8
S-38-200	300 × 12	730 × 10	300 × 10	670 × 8	500 × 8

be sufficient to ensure the frames to withstand the most critical combination cases of internal forces possibly induced from the dead, roof live, wind and crane loads.

The dimensions of the beam and column sections were designed to satisfy the column buckling conditions in both directions in and out of the frame plane as well as the bending resistance conditions of the roof beams [16]. As a result, the member sections of the frames are often controlled by the lateral displacement limit at the top of the columns in accordance with the serviceability limit state.

In the check, the maximum lateral displacement at the top of the column was controlled to be within the range of about 5% less than the allowable displacement of $1/300H$ where H is the height of the column. The maximum deflections of the roof beams were much smaller than the allowable deflection of $1/250L$ where L is the span of the frame.

3. Determination of earthquake loads acting on frames

3.1. Equivalent static earthquake loads

a. Seismic weights participating for frame responses

For simplicity, the seismic weights participating for the frame responses were assumed to be concentrated at fourteen locations as shown in Fig. 2. The total seismic weight included the self weight of the roof cladding system (roof dead load), the self weight of the crane system (including crane bridges, crane runway girders, rails, connection details) and the maximum lifting load arbitrarily assumed to be taken as ten percents. It is noted that under this assumption, the seismic weights contributed from the cases of using the maximum lifting loads of 100 and 200 kN were, respectively, about 2 and 4% of the total one as mentioned in [13]. The live load on the roof was not considered to calculate the seismic weights of the frames because the probability of occurrence of the maximum design earthquake during the roof repair work is very rare and it can be ignored in this case.

The first natural vibration periods of the structures in horizontal and vertical directions were obtained by using the program SAP as shown in Table 4. As a result, the natural vibration periods of the single-storey industrial steel frames considered in this study were quite small, ranging from $T_{1x} = 0.57$ to 0.63 sec in the horizontal direction and $T_{1y} = 0.3$ to 0.54 sec in the vertical direction.

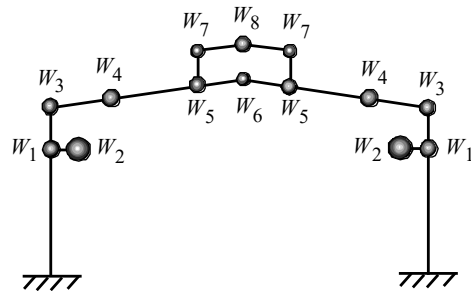


Figure 2. Seismic weights concentrated on the frames

Table 4. Total seismic weights and natural vibration periods

Frames	W (kN)	T_{1x} (sec)	T_{1y} (sec)	Frames	W (kN)	T_{1x} (sec)	T_{1y} (sec)
H-20-100	227.42	0.57363	0.30097	S-20-200	286.85	0.61549	0.29898
H-26-100	289.47	0.57014	0.35808	S-26-200	343.91	0.60411	0.37149
H-32-100	365.90	0.61120	0.49047	S-32-200	418.32	0.62630	0.48963
H-38-100	421.11	0.60067	0.53499	S-38-200	470.44	0.61867	0.52866

Fig. 3 shows the relationships of the total seismic forces in horizontal and vertical directions and the span lengths of the frames, as denoted by V and P , respectively. In Fig. 3, it is observed that the horizontal forces V increased with the span lengths whereas the vertical forces P tended to be independent of the lengths. This is because the first vibration periods in horizontal direction were all less than the spectral period of 0.8 sec corresponding to the ground type D considered in this study whereas those in vertical direction were larger than the spectral period of 0.15 sec (Table 4).

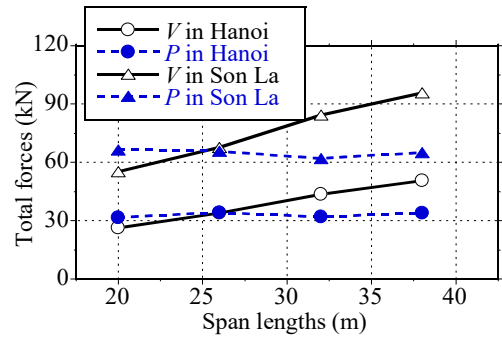


Figure 3. Total seismic forces of the frames in horizontal and vertical directions

b. Equivalent horizontal static earthquake loads

The horizontal acceleration spectrum of type 1 was used, in which the reference ground accelerations are $a_{gR} = 0.1097g$ and $0.1893g$ corresponding to the frames built in Hanoi and Son La regions, respectively; the importance factor was unity and the soil factor of ground type D was 1.35 [1, 4]. For single-storey industrial steel frame structures considered as being vertically irregular in elevation and weight distribution, the behavior factor used to determine the equivalent horizontal static earthquake loads was taken as 3 [1, 9]. The equivalent horizontal static earthquake loads were applied at the concentrated weight locations of the frames and their values were determined in accordance with design standards [1, 16] as shown in Tables 5 to 8. The horizontal forces F_i were applied mostly at the locations 1 and 2 (at the cantilever levels) with the values ranging from 64.09 to 72.22% of the total horizontal forces.

Table 5. Equivalent horizontal and vertical static earthquake loads for frames H-20-100 and S-20-200

Locations	H_i (m)	H-20-100			S-20-200		
		W_i (kN)	F_i (kN)	P_i (kN)	W_i (kN)	F_i (kN)	P_i (kN)
8	13.40	3.74	0.527	3.908	3.74	0.937	8.019
7	13.03	2.39	0.348	2.519	2.39	0.615	5.163
6	11.06	1.03	0.171	1.131	1.03	0.297	2.314
5	10.82	6.37	1.078	6.708	6.36	1.859	13.729
4	10.01	10.65	1.853	4.441	10.65	3.189	9.867
3	9.35	4.80	0.797	0.013	4.80	1.380	0.027
2	6.65	56.70	6.189	-0.495	79.52	15.392	-0.840
1	6.65	23.39	2.552	0.043	23.39	4.525	0.094

c. Equivalent vertical static earthquake loads

The vertical acceleration spectrum of type 1 was used, in which the design ground accelerations were $a_{vg} = 0.9a_{gR} = 0.09873g$ and $0.17037g$ corresponding to Hanoi and Son La regions, respectively; and the soil factor was unity [1, 16]. The equivalent vertical static earthquake loads were applied at the concentrated weight locations of the frames and their values were determined in accordance with [1, 16] as shown in Tables 5 to 8.

Table 6. Equivalent horizontal and vertical static earthquake loads for frames H-26-100 and S-26-200

Locations	H_i (m)	H-26-100			S-26-200		
		W_i (kN)	F_i (kN)	P_i (kN)	W_i (kN)	F_i (kN)	P_i (kN)
8	14.03	5.05	0.716	4.557	4.07	1.125	7.615
7	13.02	3.06	0.447	2.774	2.47	0.703	4.637
6	11.72	1.52	0.252	1.446	1.56	0.493	3.088
5	10.51	8.68	1.475	7.864	8.62	2.794	16.175
4	8.78	13.92	2.446	4.372	13.86	4.634	9.138
3	9.35	6.00	0.983	0.013	6.02	1.876	0.024
2	6.70	79.90	8.683	-0.919	101.11	18.612	-2.557
1	6.70	23.30	2.531	0.035	24.18	4.450	0.061

Table 7. Equivalent horizontal and vertical static earthquake loads for frames H-32-100 and S-32-200

Locations	H_i (m)	H-32-100			S-32-200		
		W_i (kN)	F_i (kN)	P_i (kN)	W_i (kN)	F_i (kN)	P_i (kN)
8	14.02	6.31	0.949	4.354	6.31	1.653	8.539
7	13.70	3.82	0.591	2.648	3.82	1.030	5.194
6	11.67	1.93	0.341	1.406	1.93	0.596	2.760
5	11.35	10.89	1.971	7.546	10.89	3.446	14.803
4	10.34	17.26	3.222	3.976	17.26	5.635	7.673
3	9.50	7.34	1.284	0.007	7.34	2.244	0.014
2	6.48	108.08	11.438	-1.035	128.75	23.893	-2.407
1	6.48	25.60	2.707	0.016	25.60	4.747	0.032

Table 8. Equivalent horizontal and vertical static earthquake loads for frames H-38-100 and S-38-200

Locations	H_i (m)	H-38-100			S-38-200		
		W_i (kN)	F_i (kN)	P_i (kN)	W_i (kN)	F_i (kN)	P_i (kN)
8	15.16	7.45	1.050	4.790	7.45	1.844	9.227
7	14.63	4.39	0.655	2.835	4.39	1.147	5.459
6	12.81	2.53	0.435	1.727	2.53	0.756	3.325
5	12.29	13.06	2.356	8.432	13.06	4.082	16.235
4	10.70	20.42	3.860	4.040	20.42	6.677	7.943
3	9.54	8.62	1.479	0.007	8.62	2.569	0.013
2	6.65	127.29	13.442	-1.546	146.73	27.174	-3.377
1	6.65	26.42	2.788	0.014	26.42	4.890	0.027

The vertical forces P_i were largely applied on the roof beams due to large deflections induced while those applied on the columns were almost zero. It is noted that the vertical forces applied at the location 2 (at the cantilever end) corresponding to the first vibration mode of the frame in the vertical direction have inverse signs in order to increase the beam deflections.

4. Effects of vertical seismic actions on frame responses and their comparisons with wind effects

4.1. Using linear elastic structural analyses

In the first stage, linear elastic analyses of the frames were conducted under various design static loads. Table 9 shows the obtained results of bending moments induced at the bottom and top of the columns under the static earthquake loads acting in the horizontal and vertical directions. It is noted that in these frames, the moments at the top of the columns are corresponding to those at the beam ends connected to the columns.

Table 9. Moments at the bottom and top of columns under equivalent horizontal and vertical static earthquake loads (kNm)

Frames	Under equivalent horizontal static earthquake loads			Under equivalent vertical static earthquake loads		
	At bottom of column	At top of column	Ratios	At bottom of column	At top of column	Ratios
H-20-100	77.58	21.86	3.55	45.19	62.96	0.72
H-26-100	101.19	28.49	3.55	67.41	86.66	0.78
H-32-100	134.36	30.49	4.41	83.81	101.14	0.83
H-38-100	161.68	32.93	4.91	122.03	121.71	1.00
S-20-200	155.44	48.52	3.20	87.53	127.10	0.69
S-26-200	203.71	48.80	4.17	134.35	167.01	0.80
S-32-200	257.49	55.99	4.60	166.53	199.26	0.84
S-38-200	291.40	62.95	4.63	229.06	235.22	0.97

In Table 9, for the cases under the equivalent horizontal static earthquake loads, the obtained moments at the bottom of the columns were much larger than those at the top of the columns, ranging from 3.2 to 4.91 times, depending on the span lengths and seismic regions. In contrast, for the cases under the equivalent vertical static earthquake loads, the obtained moments at the bottom of the columns were smaller than those at the top of the columns, ranging from 0.72 to 1.0 times. It is indicated that in all cases, as shown in Table 9, the ratios of the moments at the bottom of the columns to those at the top increased with the span lengths, by about 1.5 times for the frames with the lengths of 20 to 38 m.

For comparison of the effects of wind and earthquake loads on the frame responses, we considered the basic combinations of internal forces which consist of dead loads combined with earthquake loads or wind forces as denoted by CE1, CE2 and CE3 in Table 10 and CW1 and CW2 in Table 11. For example, the combination CE2 in Table 10 represents the internal forces induced by 1.0 time the dead

Table 10. Internal force combinations related to dead and earthquake loads

No.	Loads	CE1	CE2	CE3
1	Dead loads	1.0	1.0	1.0
2	Equivalent horizontal static earthquake loads	1.0	1.0	0.3
3	Equivalent vertical static earthquake loads	0.0	0.3	1.0

Table 11. Internal force combinations related to dead and wind forces

No.	Loads	CW1	CW2
1	Dead loads	1.0	1.0
2	Transverse wind forces	1.0	0.0
3	Longitudinal wind forces	0.0	1.0

loads, 1.0 time the equivalent horizontal static earthquake loads and 0.3 times the equivalent vertical static earthquake loads.

It is noted that the combining value depends on both the value and the sign of internal forces. Consider in the case of horizontal static earthquake loads acting from the left, both the values and signs of the moments at the bottom of the left and right columns were the same. On the other hand, in the case of dead loads, the values of the moments at the bottom of the left and right columns were the same, but they were different in signs. Therefore, the combining value of the moment was larger at the bottom of the left column than that of the right column. In addition, consider in the case of transverse wind forces acting from the left, the moment value at the bottom of the left column was larger than that of the right column although they had the same signs. When combined with the moments caused by dead loads, the combining value of the moment at the bottom of the left column was reduced because of different signs and that of the right column was increased because of the same signs.

The effects of vertical seismic actions on internal forces in the frames were represented in term of the ratios $K_1 = M_{CE2}/M_{CE1}$ and $K_2 = M_{CE3}/M_{CE1}$ in which the moments MCE1, MCE2 and MCE3 are obtained from the combinations of CE1, CE2 and CE3, respectively. Table 12 shows the obtained values of the ratios K_1 and K_2 , in which the values of the ratio K_1 were larger than those of the ratio K_2 at the bottom of the columns, but less than at the top of the columns for all frames. This indicates that the maximum combining moments at the bottom and top of the columns were obtained from the combinations CE2 and CE3, respectively. As a result, the values of the ratio K_1 at the bottom of the columns were from 1.09 to 1.14 and those of the ratio K_2 at the top of the columns were from 1.34 to 1.79. These values were all greater than unity which means that the effects of vertical seismic actions on the internal forces in the frames were significant, particularly at the top of the columns and for the frames in the Son La region with having very strong earthquakes but small winds.

Table 12. The obtained ratios K_1 , K_2 , K_3 and K_4 at the bottom and top of columns

Frames	Effects of vertical seismic actions				Comparisons of seismic and wind actions			
	K1		K2		K3		K4	
	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	Top
H-20-100	1.10	1.19	0.93	1.48	0.96	2.39	0.81	2.98
H-26-100	1.10	1.17	0.98	1.43	1.20	2.66	1.07	3.26
H-32-100	1.09	1.13	0.96	1.35	1.36	2.78	1.21	3.32
H-38-100	1.09	1.13	1.02	1.34	1.50	3.04	1.40	3.63
S-20-200	1.13	1.31	0.90	1.75	2.15	2.57	1.71	3.44
S-26-200	1.14	1.30	0.97	1.79	2.27	2.49	1.94	3.44
S-32-200	1.12	1.24	0.97	1.63	2.31	2.30	1.99	3.04
S-38-200	1.13	1.22	1.05	1.60	2.25	2.27	2.08	2.97

As previously presented in Table 9, the moments at the top of the columns under the equivalent horizontal static earthquake loads were much smaller than those at the bottom of the columns. It is recalled that the columns of analytical frames had their uniform cross-sections over the heights. Therefore, the effects of vertical seismic actions on the inelastic responses of the frames can be seen at the bottom of the columns, which will be presented at the next section. In addition, the moments at the roof beam ends of the frames were similar to those of at the top of the columns. This shows that the effects of vertical seismic actions can be resulted in development of plastic hinges at the beam ends, rather than at the top of the columns.

Next, the comparisons of the effects of seismic and wind actions were represented in term of the ratios $K_3 = M_{CE2}/M_{CW}$ and $K_4 = M_{CE3}/M_{CW}$ in which the moments $M_{CW} = \max\{M_{CW1}; M_{CW2}\}$, M_{CW1} and M_{CW2} are obtained from the combinations of CW1 and CW2, respectively. As shown in Table 12, the values of the ratio K_3 were larger than those of the ratio K_4 at the bottom of the columns, but less than at the top of the columns for all frames, which was similar to the ratios K_1 and K_2 as previously discussed. As a result, the values of the ratio K_3 at the bottom of the columns were from 0.96 to 2.31 and those of the ratio K_4 at the top of the columns were from 2.97 to 3.63. The results indicate that the effects of seismic actions on the column moments were much larger than those of wind forces. The ratios K_3 and K_4 also tended to increase in the cases of analytical frames in the Son La region.

4.2. Using nonlinear inelastic static analyses

In the second design stage, nonlinear inelastic static (pushover) analyses of structures using plastic hinge beam-column elements [17–19] were conducted to evaluate the inelastic responses of the frames under the combinations of dead loads and equivalent static earthquake loads as previously denoted by CE1, CE2 and CE3. In the analysis, the dead loads were firstly applied and then the static earthquake loads were incrementally applied with a step-by-step increase in load. The second-order effect was included in the structural analysis by using the stability functions [20] and inelastic behaviors were considered by using the refined plastic hinge model [21]. Beam and column members were modeled by using flexural yield surfaces represented by the parabolic functions at both the member ends [22]. The effect of lateral-torsional buckling of columns was directly considered. The effect of local buckling was neglected.

Table 13. Level of equivalent static earthquake loads at the first yielding and maximum resistance of the frames obtained from pushover analyses (%)

Frames	At the first yielding			At the maximum resistance		
	CE1	CE2	CE3	CE1	CE2	CE3
H-20-100	183.0	155.0	143.5	404.7	347.0	255.3
H-26-100	171.0	141.8	172.7	786.8	664.6	507.1
H-32-100	110.5	93.4	70.0	611.8	510.7	348.5
H-38-100	87.0	70.9	43.3	594.3	485.2	309.0
S-20-200	114.0	96.8	96.0	415.9	358.7	256.2
S-26-200	88.0	72.9	76.0	380.4	320.2	235.0
S-32-200	60.0	50.4	41.3	333.0	277.2	186.3
S-38-200	47.5	38.4	27.9	323.8	264.0	170.4

Table 13 shows the levels of equivalent static earthquake loads in percentage at which the frames began behaving in a nonlinear inelasticity by pushover analyses corresponding to the combinations of CE1, CE2 and CE3. It is noted that the obtained percentages at the first yield development were larger than 100%, indicating that the frames behaved in a linear elasticity under the static earthquake loads. As a result, all the analytical frames built in the Son La region except the case of S-20-200 under the combination of SE1 behaved in a nonlinear inelasticity under the static earthquake loads. In addition, we increased the static earthquake loads up to the level at which the maximum resistances of the frames were obtained as being from 170.4 to 786.8%, depending on the frame spans and seismic locations (Table 13). These obtained results were corresponding to those using linear elastic analyses of structures as previously discussed. Fig. 4 illustrates the results of yielding points and maximum resistance obtained from pushover analyses of the frame S-26-200 corresponding to the combination CE1.

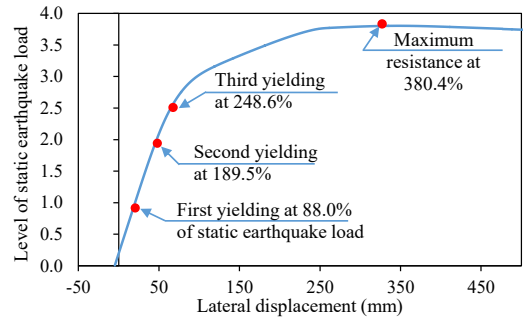


Figure 4. Results obtained from pushover analysis of the frame S-26-200 corresponding to the combination CE1

5. Conclusions

In this paper, the effects of seismic actions and their comparisons with the wind effects on the responses of single-storey industrial steel frames with cranes were valued. The analytical frames had the spans of 20, 26, 32 and 38 m and they were built in Hanoi and Son La regions. The evaluation was conducted corresponding to the design stages using linear elastic analyses and nonlinear inelastic analyses of structures under various combinations related to static earthquake loads and wind forces. From this study, the following can be concluded:

- The determination of equivalent horizontal and vertical static earthquake loads acting on single-storey industrial steel frames with cranes was presented by assuming various locations of seismic weights concentrated on the frames.

- By using linear elastic analyses of structures, the ratios of the moments at the bottom of columns to those at the top of columns were from 3.2 to 4.91 for the frames under horizontal static earthquake loads and from 0.72 to 1.0 for the frames under vertical static earthquake loads (Table 9). These ratios were increased with the span lengths, by about 1.5 times for the frames with the span lengths increasing from 20 to 38 m.

- The effects of vertical seismic actions on the responses of the frames were evaluated in term of the ratios K_1 and K_2 , with the obtained values of $K_1 = 1.09$ to 1.14 at the bottom and $K_2 = 1.34$ to 1.79 at the top of the columns, respectively. In addition, the effects of seismic actions compared to those of wind actions were evaluated in term of the ratios K_3 and K_4 , with the obtained values of $K_3 = 0.96$ to 2.31 at the bottom and $K_4 = 2.97$ to 3.63 at the top of the columns. The ratios K_3 and K_4 tended to increase with the seismic ground levels.

- Nonlinear inelastic analyses of the frames under the combinations of CE1, CE2 and CE3 were conducted and as a result the levels of the static earthquake loads were determined corresponding to the first yielding and maximum resistances of the frames.

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