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Soil Specification Effects on the Displ. Amplification Factor of Industrial Steel Structures with Cranes

Armen Assatourians[1*], Sohrab Fallahi[2]

1.Earthquake Engineering Research Consultant, Yerevan Project Co., Yerevan, Armenia
2.Senior Structural and Earthquake Engineer, ESS Consulting Engineers Co.,Tehran,Iran

Abstract

According to construction of industrial Steel Structures with Cranes in developing countries, the importance of seismic behaviour of such kind of structures are evident. In this research, the overall structural behaviour is briefly described, then 3 models of existing industrial steel structures with cranes are selected. All models are assumed to have bolted connections, for which the damping ratio for all of them is taken into account 10% of critical damping. Then, accelerograms recorded on soil types 1, 2 & 3 , is selected according to 4th ver. of Iranian 2800 seismic code and scaled to S_a=0.25g & S_a=0.35g spectral acceleration levels. Nonlinear Time History analyses are completed for all designed finite element models, using above mentioned scaled records. Modal Pushover analyses are carried out to illustrate the yielding base shear force and the demanded base shear force for the first plastic hinge to form. Finally, the results of Ductility Factors, μ, overstrength Reduction Factors, R_s, and the Displacement Amplification Factors,C_d, obtained from various records for finite element models, due to spectral accelerations and based on 3 soil categories are summarized and discussed.

Key words: Nonlinear Time History Analysis, Ductility Factor, Industrial Steel Structures with Cranes, Displacement Amplification Factor

Introduction

Previous experiences of earthquakes illustrate that many types of structures behave nonlinearly during a severe earthquake. So a huge amount of input energy is mainly dissipated through the form of damping and hysteresis [1]. According to this, the structures are usually designed for much lower lateral forces than those demanded by aseismic design codes in elastic range. The aseismic behaviour analysis and accurate design of structures for severe earthquakes are mainly carried out using Nonlinear Time history Analysis method (NTHA). Using the NTHA method for analysis of somehow simple structures in consulting engineer's offices is not appropriate enough, due to the complexity and time taking behaviour of the method. So according to simplicity and popularity of structural linear analysis techniques, they are mainly proposed in most aseismic design codes using the reduced lateral forces meanwhile. The seismic linear force for structural design purposes is achieved from linear earthquake spectra [3]. The computed lateral force from the spectra is decreased by the means of a reduction factor or modification factor, R, and a displacement amplification factor, C_d, according to ductility, damping, overstrength and so on [4][6]. This research is carried out to compute the Displacement Amplification Factor of Industrial Structures containing

Overhead Cranes, realizing that neither in 2800 seismic code of Iran nor other countries, no C_d factor is specified for this type of structures.

Displacement Amplification Factor Theoretical Basis

Both structural and non structural collapses during severe earthquakes, usually occur due to lateral displacements. So the determination of Lateral Displacement Demand in performance based design method is of much importance. According to the reduced lateral forces (discussed in the previous section), the lateral displacements computed through a linear analysis, should be increased in order to estimate the real displacements during a severe earthquake. The seismic design codes of various countries propose a Displacement Amplification Factor (C_d), for this purpose. This factor is described due to equation (1):

$$\Delta_{max}=\Delta_w \cdot C_d \tag{1}$$

In equation 1 , Δ_{max} is the maximum inelastic displacement, Δ_w is the maximum linear displacement and C_d is the displacement amplification factor. In Figure 2 the real behaviour of the structure is replaced by a bilinear elasto - plastic model. The Displacement Amplification factor is computed based on equation 2 (Uang and Maarouf, 1994).

$$C_d=\mu \cdot R_s \tag{2}$$

In equation 2, μ is the Ductility factor and is described by the use of equation 3.

$$\mu=\Delta_{max} / \Delta_y \tag{3}$$

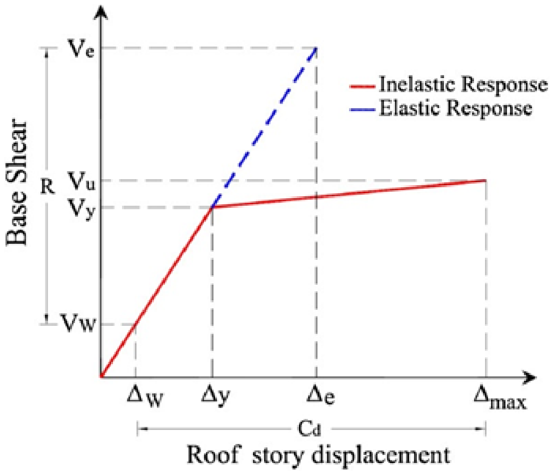


Figure 1: General seismic response of structures

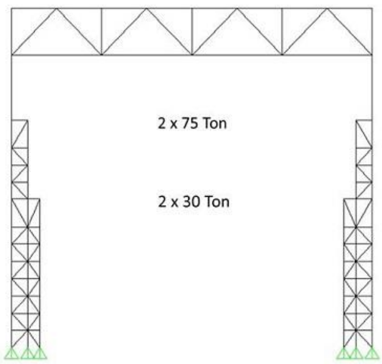
The Overstrength factor R_s, is an important factor which could not be obtained easily. Analytical and tentative methods should be used to obtain R_s. The role of R_s factor is much more important in the case of intensive earthquakes and its value is based on material properties, lateral load bearing system, geometry of the structure and the structural details. So it could be seen that this value is particular for each structure. Practical method to find R_s is based on a static push-over analysis and additional corrections should be applied to obtain the real value of R_s:

$$R_s=R_{s0} \,.\, F_1 \,.\, F_2 \,.....\, F_n \tag{4}$$

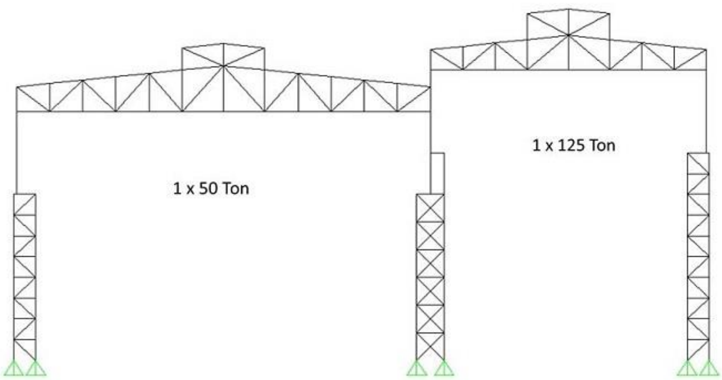
In equation (4), F_1 indicates the difference between nominal and real statically yielding strength and for steel structures F_1 is equal with 1.05. F_2 is another factor which indicates the increasing rate for yielding stress due to strain effect during an earthquake and is equal to 1.10. The remained factors could be computed due to trustable information; otherwise it should be estimated equal with 1.0.

Finite Element Computational Models

In current research, three 2D industrial structures with several spans and heights, containing various crane capacities are presented as Figure 2. For Dead and Live loadings of the models, ASCE 7-10 code is used. Crane loadings and related load combinations is completed using AISE code [2]. The AISE code indicates that for computing the seismic loads caused by the cranes, all cranes should be taken into account in parked position, in the worst case. Column sections, are considered of steel IPB sections for analysis and design purpose. All computational models are then analysed and designed based on 2800 Iranian seismic code, according to AISC 360-10 code. Sap2000 ver.18.2.0 software is used for analysis and design purposes [7]. The final sections for structural elements are obtained considering intermediate moment resisting frame (IMF) coordinates. Finally, the Modal Push-over analysis is performed to determine the Yielding Base Shear force (V_y) and Yielding Displacement (Δ_y) and the base shear force required for the first plastic hinge to form in each model.



(a)



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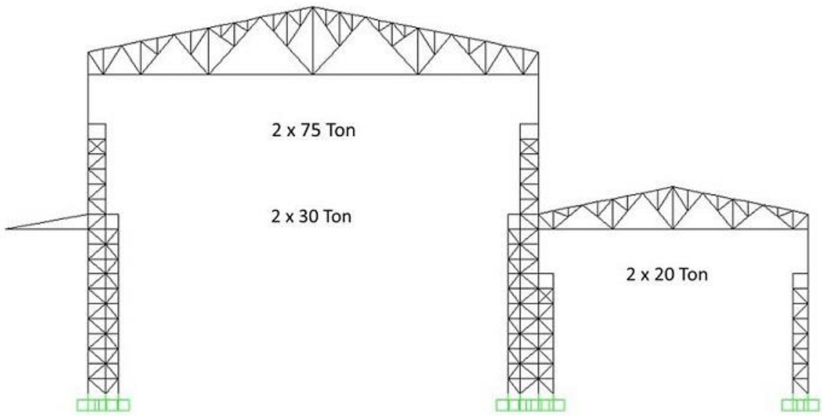


(c)

Figure 2: Proposed finite element models

Nonlinear Time History Analyses

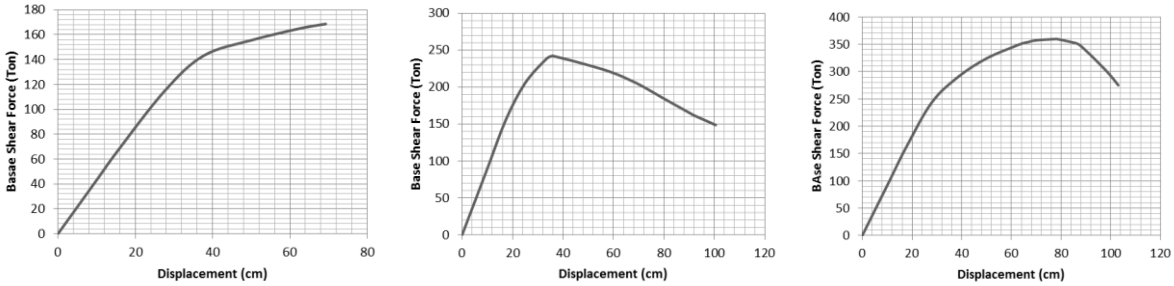
In order to perform the time history analyses, 10 accelerograms of earthquakes recorded on each soil type according to 2800 Iranian code were selected [5]. Then each record was scaled to spectral accelerations of S_a=0.25g and S_a=0.35g, according to related response spectrums in 2800 Iranian code. Selected records are of the earthquakes listed in Table 1. Then the scaled records were applied to the computational model due to the soil type and spectral acceleration for which the selected model was analysed and designed. For linear Time history analyses, the "Modal Extension Method of earthquake forces" technique was used. Nonlinear Time history analyses were completed using Newmark – β method. For nonlinear analyses, the Rayleigh damping was used, determining damping ratio equal to 0.10, according to bolted connections. By using the analysis results, over strength factor R_{so}, corrected over strength factor R_s, and ductility factor μ are calculated.

Table 1: Characteristics of used Earthquake Records

Event	Year	Mag.	Mechanism
Ducze-Turkey	1999	7.14	Strike-Slip
Chi-Chi-Taiwan	1999	7.62	Reverse-Oblique
Irpinia-Italy	1980	6.9	Normal
Tabas-Iran	1978	7.35	Reverse
Denali-Alaska	2002	7.9	Strike-Slip
Loma Prieta	1989	6.93	Reverse-Oblique
Kocaeli-Turkey	1999	7.51	Strike-Slip
Northridge	1994	6.69	Reverse
San Fernando	1971	6.61	Reverse
Landers	1992	7.28	Strike-Slip
Hector Mine	1999	7.13	Strike-Slip
Cape Mendocino	1992	7.01	Reverse

Analysis Results

After completing the required analyses, Capacity Curves for models are prepared as Figure 3. Required information for computation of C_d factors of models are also classified, based on 3 needed soil categories and equations (1) ~ (4) from the nonlinear time history analyses. Final results of computation process are illustrated in Tables 2 ~ 6.



Model “a” Model “b” Model “c”

Figure 3: Capacity Spectrums for proposed models

Table 2: Yielding Displacement D_y and Overstrength Reduction Factors R_s

	Model “a”	Model “b”	Model “c”
D_y (cm)	19.2	17.3	17.6
R_s	1.47	1.17	1.17

Table 3: Maximum Inelastic Displacements, Δ_{max}

	Rec.	Model “a”		Model “b”		Model “c”	
		.25g	.35g	.25g	.35g	.25g	.35g
Soil Type 1	1	18.37	25.71	14.25	19.95	15.48	21.67
	2	11.58	16.21	8.98	12.57	9.16	12.82
	3	28.46	39.84	15.81	22.13	18.70	26.18
	4	19.06	26.69	10.39	14.55	12.49	17.49
	5	17.56	24.59	12.47	17.46	15.22	21.31
	6	28.46	39.85	18.67	26.14	20.25	28.35
	7	21.60	30.25	10.31	14.43	12.20	17.08
	8	29.76	41.67	8.08	11.32	9.42	13.18
	9	16.20	22.68	6.82	9.55	8.45	11.82
	10	28.76	40.26	19.32	27.05	21.41	29.97
Soil Type 2	1	12.58	17.61	6.85	9.60	8.39	11.75
	2	21.50	30.10	10.38	14.53	12.22	17.10
	3	20.08	28.11	8.57	12.00	10.14	14.20
	4	16.18	22.66	11.10	15.54	13.34	18.67
	5	26.14	36.59	10.46	14.64	12.51	17.52
	6	27.87	39.01	11.47	16.06	12.58	17.61
	7	17.90	25.06	7.54	10.55	9.33	13.06
	8	8.62	12.07	6.67	9.34	7.72	10.80
	9	14.21	19.89	7.46	10.44	8.98	12.57
	10	21.12	29.57	9.97	13.95	11.74	16.43
Soil Type 3	1	25.21	35.29	12.17	17.04	14.32	20.05
	2	22.16	31.02	9.47	13.26	9.72	13.60
	3	22.02	30.82	13.95	19.53	16.58	23.21
	4	27.48	38.47	9.40	13.16	10.78	15.09
	5	23.69	33.17	12.69	17.76	14.63	20.49
	6	29.59	41.43	20.80	29.13	24.43	34.20
	7	28.36	39.70	13.91	19.47	15.29	21.41
	8	22.52	31.53	11.49	16.09	12.46	17.45
	9	14.70	20.58	10.50	14.70	11.82	16.55
	10	24.75	34.64	10.17	14.24	11.03	15.45

Table 4: Ductility Factors, $\mu = \Delta_{max} / \Delta_y$

	Rec.	Model "a"		Model "b"		Model "c"	
		.25g	.35g	.25g	.35g	.25g	.35g
Soil Type 1	1	1.00	1.34	1.00	1.15	1.00	1.23
	2	1.00	1.00	1.00	1.00	1.00	1.00
	3	1.48	2.08	1.00	1.28	1.06	1.49
	4	1.00	1.39	1.00	1.00	1.00	1.00
	5	1.00	1.28	1.00	1.01	1.00	1.21
	6	1.48	2.08	1.08	1.51	1.15	1.61
	7	1.13	1.58	1.00	1.00	1.00	1.00
	8	1.55	2.17	1.00	1.00	1.00	1.00
	9	1.00	1.18	1.00	1.00	1.00	1.00
	10	1.50	2.10	1.12	1.56	1.22	1.70
Soil Type 2	1	1.00	1.00	1.00	1.00	1.00	1.00
	2	1.12	1.57	1.00	1.00	1.00	1.00
	3	1.05	1.46	1.00	1.00	1.00	1.00
	4	1.00	1.18	1.00	1.00	1.00	1.06
	5	1.36	1.91	1.00	1.00	1.00	1.00
	6	1.45	2.03	1.00	1.00	1.00	1.00
	7	1.00	1.31	1.00	1.00	1.00	1.00
	8	1.00	1.00	1.00	1.00	1.00	1.00
	9	1.00	1.04	1.00	1.00	1.00	1.00
	10	1.10	1.54	1.00	1.00	1.00	1.00
Soil Type 3	1	1.31	1.84	1.00	1.00	1.00	1.14
	2	1.15	1.62	1.00	1.00	1.00	1.00
	3	1.15	1.61	1.00	1.13	1.00	1.32
	4	1.43	2.00	1.00	1.00	1.00	1.00
	5	1.23	1.73	1.00	1.01	1.00	1.16
	6	1.54	2.16	1.20	1.68	1.39	1.94
	7	1.48	2.07	1.00	1.13	1.00	1.22
	8	1.17	1.64	1.00	1.00	1.00	1.00
	9	1.00	1.07	1.00	1.00	1.00	1.00
	10	1.29	1.80	1.00	1.00	1.00	1.00

Table 5: Mean values of Ductility Factors, μ

Soil Type	Model "a"		Model "b"		Model "c"	
	.25g	.35g	.25g	.35g	.25g	.35g
1	1.21	1.62	1.02	1.26	1.04	1.22
2	1.11	1.40	1.00	1.00	1.00	1.01
3	1.28	1.75	1.02	1.10	1.04	1.18

Table 6: Mean values of Displacement Amplification Factor $C_d = \mu \, . \, R_s$

Soil Type	Model "a"		Model "b"		Model "c"	
	.25g	.35g	.25g	.35g	.25g	.35g
1	1.78	2.38	1.19	1.47	1.22	1.43
2	1.63	2.06	1.17	1.17	1.17	1.18
3	1.88	2.57	1.19	1.29	1.22	1.38

The mean values of $\mathbf{C_d}$ factors computed in Table 6, is summarized as follows:

Rock: $C_d = 1.58$, Dense Soil $C_d = 1.40$, Loose Soil $C_d = 1.59$

Conclusion

According to the final results, it could be observed that the displacement amplification factor of “Industrial Steel Structures with Overhead Cranes” is not so sensitive to soil categories. When degrading the soil category from 1st to 2nd grade (Rock to Dense soil), the displ. amplification factor decreases about 11.4% . The main reason of this effect could be explained according to the stiffness and the fundamental vibration period of the structures, which is located far from the fundamental period of the soil. When degrading soil category from 2nd to 3rd, the C_d factor is increased to the same extent of the 1st soil category. This could be explained by the frequency content of the records used for nonlinear analyses. In this case, the frequency content of the accelerograms used for the 3rd soil category are somehow similar to that of the 1st soil category or the frequencies of vibration modes of the models, coincide the frequency content of soil types 1 & 3 to the similar extent , which cause the displacement amplification factor to get increased to the same extent of the 1st soil category. If needed, additional PSDF (Power Spectral Density Function) analysis could be carried out to illustrate the details.

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