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Seismic soil structure interaction analysis and the codes of practice

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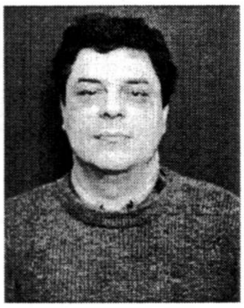

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ABSTRACT: This paper presents the main aspects that must be addressed in a SSI analysis and some of the advanced analysis techniques. Recommendations for considering the SSI effect into the seismic codes, based on simplified SSI methods are discussed. It is recommended that the already existing experience in SSI analysis, developed by the nuclear industry to be reflected into general seismic building codes.

1. Introduction

Due to the nuclear industry, the Soil Structure Interaction (SSI) phenomenon was beginning to be understood around 1970 and was considered to have significant effects on the dynamic response of the structure. Today it is known that SSI effects may govern the seismic structure response in case of relatively rigid buildings and soft soil conditions.

An important amount of research effort have been spent in this field during the 1975 - 1982 period. The result of this effort was the development of various analysis techniques and tools so called "state of the art of the industry". For the nuclear industry, these techniques became standard procedures and they were included into codes and regulations, like ASCE 4-86, US Standard Review Plan, etc. so there is a lot of experience concerning the SSI analysis techniques.

In Chapter 2 are briefly presented aspects related to the hazard level of the seismic design force, as they are reflected into building codes. Some of the basic features of the SSI problems using a very simple model, are presented in Chapter 3. Chapter 4 presents an example analyzed as follows :

- ignoring SSI effects,
- using advanced SSI methods (3D complex frequency response),
- using simplified SSI methods.

2. Hazard levels and soil structure interaction provisions in building codes

The item focuses on probabilistic definition of the key factors involved in the assessment of seismic design force according to Eurocode 8, ASCE 7 and ASCE 4 codes. The difficulty of establishing the overall reliability level of seismic design force is due to the imperfect probabilistic definition of the partial factors involved, Table 1:

$$F_b = a_g\, S\, \beta(T)\, \eta\, \frac{1}{q} W = S_e(T)\, \frac{1}{q} W = S_d(T)\, W$$

where:

F_b is the seismic base shear
a_g - (effective) peak ground acceleration at a site
S - soil factor
$\beta(T)$ - normalized acceleration response spectrum for 5% damping
η - damping correction factor for elastic response
q - behavior factor (response modification factor) to reduce the base shear from elastic level to the first yielding (ultimate strength level, not allowable stress level)
$S_e(T)$ - elastic response spectrum
$S_d(T)$ - design response spectrum
W - gravity load.

Peak (or effective peak) ground acceleration hazard induced by:		Soil factor[4]	Probability of non-exceedance of response spectra	
Source magnitude	Attenuation law[5]		Soil-dependent normalized elastic response spectra	Response modification factor [6]
(T = 50 yr.) 0.5 prob. of exceedance in 50 yr.	Mean	Mean	0.5[2]	0.5
(T=475 yr.) 0.1 prob. of exceedance in 50 yr. [1]	Mean plus one standard deviation	Mean plus one standard deviation	0.9[3]	0.9

Table 1. Hazard levels of the factors involved in the assessment of seismic design force

Note. Mean and mean plus one standard deviation values may be roughly considered respectively equal to 0.5 (median) and to 0.85 fractile of the distribution.

[1] ASCE 7-93 and Eurocode 8

[2] ASCE 4-95 draft and Eurocode 8

[3] ASCE 4-86

[4] ASCE 7-95 draft

[5] [6] Probability-based definition is missing in building codes

The peak acceleration value at a site corresponding to a specified return period is generally defined in codes by a single value, even any recorded earthquake and corresponding attenuation analysis prove that a site must be characterized at least by two values: (I) the mean and (ii) mean plus one standard deviation value. The soil factors (recently introduced by the ASCE 7-95) have different hazard levels : (i) mean value for the constant spectral acceleration branch of the response spectrum and (ii) mean plus one standard deviation value for the constant velocity range of the response spectrum.

The normalized elastic response spectrum is defined as : (i) a median spectrum in Eurocode 8 and in the draft of ASCE 4-95 code, but as (ii) a mean plus one standard deviation spectrum in ASCE 4-86 code.

The calibration of the safety level of seismic design force explicitly requires a clear probabilistic definition of the all partial factors involved in the assessment of the force. Even the hazard level induced by the source magnitude to the peak (or effective peak) ground acceleration and the hazard level of the normalized acceleration elastic response spectra are usually indicated, however, the probabilistic background of the response modification factor (due to the inelastic behavior) is always missing. Generally this factor is the product of two factors:

$$q = q_{\mu} \, q_{ov}$$

where:

q_{ov} is the over strength factor

q_{μ} is factor to reduce the base shear from elastic level to the collapse level.

The $1/q_\mu$ factor can be defined either as (I) the median factor or as (ii) a factor having a specified probability of exceedance. Moreover, the values of q_μ are clearly dependent on the spectral content of the seismic input. For wide frequency band motions it is generally independent on the structure period but for narrow frequency band motions having a clear predominant period it is a function of the ratio of the structure to the soil predominant periods.

The two-earthquake methodology used in the aseismic design of the nuclear power plants (NPP), buildings and other structures designated as essential facilities claims to assess the two-hazard levels of the seismic design force from various combinations of individual hazard levels of the factors it depends. The hazard level of each of these partial factors involved in the assessment of seismic design force must be compatible to the hazard level of the remaining factors in the product.
Last but not least, the partial safety factors used by Eurocode 1 and ASCE 7 within the ultimate state design are as follows:

$$G_k + \gamma_I A_{ed} + (0.3 \div 0.8)\, Q_k \qquad \text{(EC 1)}$$

$$1.2D + E + (0.5 \div 1.0)\, L + 0.2\, S \qquad \text{(ASCE 7)}$$

where G or D indicates the dead load, A_{ed} or E - the earthquake load, Q or L - the live load and S - snow load. The subscript k denotes the characteristic values. The importance factor γ_I in EC1 depends on the building category: from 0.8 - minor importance up to 1.4 - vital importance for civil protection.

Eurocode 1, Part 5, Chapter 6 specifies that soil-structure interaction should be considered in the case of: structures with massive or deep seated foundation, slender tall structures and structures supported on very soft soil. For these cases natural periods, damping, mode shapes, etc. will differ from those of the fixed base structures.

To account for interaction effects (when the effects are on the safe side) for regular buildings, the draft ASCE 7-95 code reduce the seismic base shear V as follows:

$$V^* = V - \Delta V$$

$$\Delta V = \left[C_s - C_s^* \left(\frac{0.05}{\beta}\right)^{0.4} \right] W < 0.3V$$

$$\beta^* = \beta_0 + 0.05(T^*/T)^3)$$

where : C_s and C_s are the overall seismic coefficients determined without and with SSI effect,
T^*, $T^*>T$, - the natural periods of flexible supported building and rigid supported building,
β^*, β - the damping coefficient with and without SSI effect,
W - the effective gravity load.

3. Soil Structure Interaction

To illustrate the SSI effect a simple model consisting of a single mass M, lumped at a height h above the base and structure stiffness K, will be used, Fig.1. For the case of a horizontal excitation the equation of motion for the mass point is:

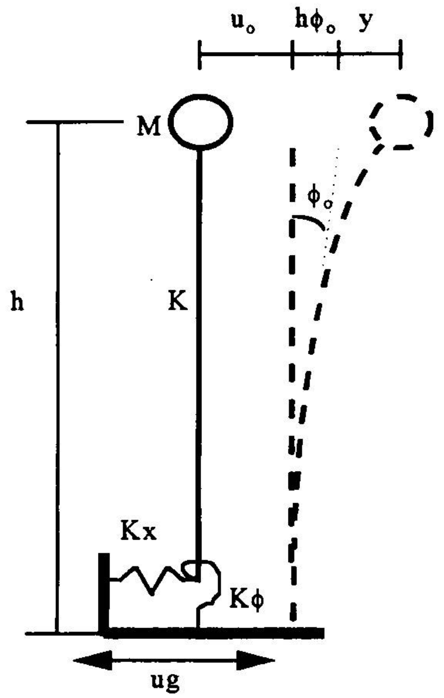

Figure 1. Simple Model

$$Mu + Ky = 0 \quad (1)$$

$$u = u_G + u_0 + y + h\Phi_0$$

$$Ky = K_x u_0$$

$$Khy = K_\Phi \Phi_0$$

where K_x is the horizontal spring representing the foundation translation stiffness, K_Φ is the corresponding rocking spring, u is the absolute displacement of mass, y is the structural deformation and u_0 and Φ_0 are the deformation of the foundation springs, and u_G is the ground displacement in the free field. Equation (1) can be written as:

$$M\,(1 + K / K_x + Kh^2 / K_\Phi)\, y + Ky = -M\, u_G \quad (2)$$

The natural frequency of the structure on a rigid base (without SSI) is :

$$\omega_o = (K / M)^{1/2} \quad (3)$$

Taking into account the flexibility of the foundation, the frequency becomes:

$$\omega = \frac{\omega_o}{(1 + K / K_x + Kh^2 / K_\Phi)^{1/2}} \quad (4)$$

Assuming the structure internal damping D_{st} of hysteretic type which is frequency independent and the soil internal material damping D_s, also hysteretic and dashpots C_x, C_Φ associates with the foundation sprongs K_x and K_Φ (to reproduce the loss of energy by radiation), then the effective damping D of the system at its natural frequency ω is given approximately by [8]:

$$D = D_{st}\left(\frac{\omega}{\omega_0}\right)^2 + D_s\left[1 - \left(\frac{\omega}{\omega_0}\right)^2\right] + D_s\left(\frac{\omega}{\omega_0}\right)^2\left[\frac{K}{K_x}\,\frac{\omega C_x}{2K_x} + \frac{Kh}{K_\Phi}\,\frac{\omega\, C_\Phi}{2K_\Phi}\right] \quad (5)$$

As it could be expected, the flexibility of the soil results in a decrease of the natural frequency, indicating that the system is more flexible.

The magnitude of this change is a function of relative stiffness of the structure with respect to soil, as indicated by terms K/K_x and Kh^2/K_Φ. Equation (5) shows the soil contribution to the effective damping of the soil-structure system. The amount of increase depends mainly on the magnitude of the last term, representing the radiation damping. From the analysis of this simple dynamic system, it can be seen that the main effects of soil structure interaction are:

- a decrease of the natural frequency of the system, depending on the relative stiffness of structure with respect to the soil;
- a change in the effective damping of the system; the main factor contributing to the increase in damping is the lose of energy by radiation of waves from the foundation;
- the appearance of the rotational component of motion at the base.

In order to estimate the magnitude of interaction effects it is necessary to know the values of terms K_x, C_x, K_Φ, C_Φ, K_z and C_z, which represent the dynamic stiffness of the foundation. These values are function of soil material, foundation shape, embedment depth and also are frequency dependent. A comprehensive review of the SSI methods was done by Roesset [8].

4. Example

The following example illustrates the principal SSI problems that should be addressed. The dynamic structure model is presented in Figure 2. In Tables 2a and 2b are presented the structure inertial and stiffness characteristics.

The SSI analysis has been performed using two parallel methods:

a) advanced method - using complex frequency domain analysis
b) simplified method - using modal analysis with a spring base model.

The seismic excitation was defined at free field level base from seismic hazard analysis. The maximum peak ground acceleration is 0.195g.

Elevation		Shear center		A	S_{ahx}	A_{ahy}	I_x	I_y	I_t
from	to	X(m)	Y(m)	(m^2)	(m^2)	(m^2)	(m^4)	(m^4)	(m^4)
10.0	13.2	14.07	12.93	232.3	158.3	169.9	9580	18410	28020
13.2	22.2	13.53	9.40	86.6	53.6	37.45	4911	10033	13323
22.2	28.2	15.73	5.46	121.9	67.1	66.4	4773	10231	12582
28.2	31.2	16.31	3.55	111.3	54.0	51.3	5808	6545	9754
31.2	36.0	15.90	8.38	137.2	86.7	92.7	5463	6851	9800
36.0	43.7	14.25	10.65	0.41	0.0	0.0	1.1	1.8	2.9
43.7	46.0	14.25	10.65	0.41	0.0	0.0	1.1	1.8	2.9

Table 2.a Stiffness properties

Elev.	Mass center		e_x	e_y	M_{vert}	M_x	M_y
(m)	X(m)	Y(m)	(m)	(m)	tones	tones	tones
10.0	14.51	11.04	0.04	1.89	3536	3311	3311
22.2	12.90	9.96	2.83	4.50	2833	2833	2833
31.2	15.06	10.30	1.23	1.51	2173	2398	2398
36.0	14.26	10.36	0.01	0.29	1981	1885	1912
48.0	14.25	10.65	0.00	0.00	138	138	138

Table 2.b Inertial properties

The seismic waves produce shear and volume strain deformation in soil material. The non-linear effect produced by the seismic waves in the soil material is called the primary nonlinearity. The dynamic foundation stiffness taking into account the soil profile layout, soil dynamic properties, primary non-linearity, foundation characteristics (shape, embedment, etc.) was computed using SUPELM computer code [7]. The dynamic foundation stiffness includes also the damping: material damping and radiation damping.

The soil profile is presented in Table 3. The dynamic soil properties are based on site measurements of shear wave velocity and lab tests. The Seed & Idriss curves G-γ and D-γ, representing the variation of the dynamic shear modulus G versus shear strain deformation γ and material damping D_s versus shear strain γ respectively corresponding to send material were used in analysis .

Layer	Height [m]	Unit weight. [t/m^3]	V_s [m/s]	G [t/m^2]	Damping %	Poisson
1 Sand+ Gravel	1.5	1.8	196.4	6943.1	2.7	0.40
2 Sand+ Clay	4.0	1.75	156.2	4269.7	11.0	0.43
3 Sand+ Cl+Grav	7.5	1.80	203.0	7417.6	12.5	0.42
4 Sand+Gravel	6.0	1.85	287.0	15238.3	10.0	0.38
5 Sand	5.0	1.90	338.8	21235.3	9.0	0.38
6 Sand	10.0	1.95	478.5	44647.6	9.0	0.36
7 Sand	100.0	2.0	565.0	63845.0	7.0	0.35

Table 3 Iterated soil properties profile

The next important problem is to determine the seismic motion corresponding to the foundation level. This step is called kinematic interaction. The result of the kinematic interaction is the modified free field motion corresponding to the foundation level. This step was performed using KININT program [7].

The last problem was to determine the soil-structure dynamic response. The structure response has been solved using advanced complex frequency analysis model EKSSI [7], simplified spring base model and without SSI effect - i.e. fixed base structure. Based on complex frequency dependent foundation stiffness matrix, equivalent soil springs constants have been calculated to be used in simplified method.

Comparison between the floor response spectra computed at elevation 36.0 Figure 3, shows a good agreement between advanced and simplified method. Comparison between fixed base structure and spring base structure are presented in terms of maximum displacements, accelerations and base shear forces in Tables 5 and Table 6

Elevation	Maximum Displacements			Maximum Acceleration		
(m)	X(cm)	Y(cm)	Z(cm)	X(g)	Y(g)	Z(g)
10.0	1.14	1.01	0.16	0.132	0.143	0.08
17.2	1.52	1.20	0.25	0.166	0.172	0.09
22.2	1.89	1.33	0.28	0.212	0.198	0.10
31.2	2.10	1.57	0.35	0.268	0.251	0.12
36.0	2.00	1.70	0.30	0.273	0.277	0.11
48.0	2.20	2.20	0.30	0.802	0.963	0.11

Table 5a. Seismic response (with SSI effect)

Shear (X)	Shear (Y)	Vertical (Z)	Overturning Moment		
			M_x	M_y	M_t
kN	kN	kN	kNm	kNm	kNm
39110.0	39540.0	23800.0	692100.0	473100.0	475200.0

Table 5b. Global force at foundation level (with SSI effect)

Elevation	Maximum Displacements			Maximum Acceleration		
(m)	X(cm)	Y(cm)	Z(cm)	X(g)	Y(g)	Z(g)
10.0	0.0	0.0	0.0	0.0	0.0	0.0
17.2	0.033	0.030	0.002	0.190	0.211	0.03
22.2	0.058	0.048	0.005	0.230	0.270	0.07
31.2	0.110	0.090	0.007	0.380	0.450	0.11
36.0	0.130	0.100	0.010	0.450	0.530	0.12
48.0	0.760	0.550	0.018	2.500	1.270	0.41

Table 6a. Seismic response (without SSI effect)

Shear (X)	Shear (Y)	Vertical (Z)	Overturning Moment		
			M_x	M_y	M_t
kN	kN	kN	kNm	kNm	kNm
40680.0	45840.0	11560.0	852000.0	757000.0	370000.0

Table 6b. Global force at foundation level (without SSI effect)

The analysis of these results shows:

- the soil-structure system frequencies are 2.14 Hz and 2.45 Hz for horizontal translation and 4.52 and 5.48 for rocking;
- the soil-structure system mode shapes correspond to rigid body translation and rocking;
- the fix base structure first modes are 7.10 Hz. and 8.35 Hz;
- the SSI effect increases the damping of the soil-structure system and decreases the seismic force and structure elastic deformation;
- simplified SSI method using spring base model can produce good results if the spring constants are properly calibrated [3], [5];

- the SSI effect consists in the reduction of natural frequencies, rigid body displacement response, and in the increase of system damping, reduction of global seismic base force and changes in the distribution of seismic forces (see accelerations)
- for higher frequency (over 3.0 Hz) the simplified method produces conservative results due to the fact that the soil stiffness and damping characteristics were considered frequency independent.

5. Conclusions

In the calculation of seismic design force using building codes, the hazard level of each partial factors involved must be consistent.
Design requirements concerning SSI effect, developed by nuclear industry, started to penetrate in a simplified form the general seismic building codes - ASCE 7-95 and EC1.
Without proper analysis, SSI is hardly predictable; the effects could be on both sides: favorable and adverse to the structure.
The SSI experience accumulated in the nuclear industry design should be used in establishing simplified design requirements applicable for regular buildings.
Further studies and numerical test are beneficial for comparison between the simplified and advanced SSI methods.

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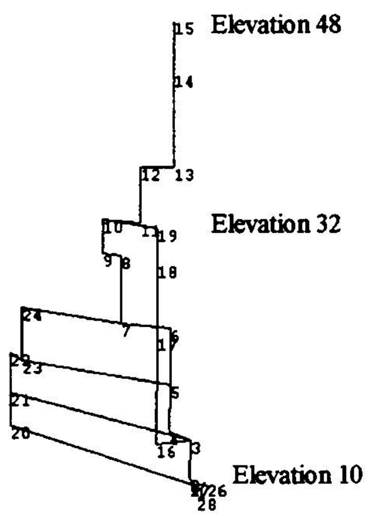
FIGURE 2 Dynamic model

Mode	Fix base Model Freq, Hz	Spring base model Freq, Hz
1	7.10	2.14
2	8.35	2.45
3	10.89	4.52
4	11.48	5.48
5	24.46	7.16
6	25.04	8.24
7	29.60	9.51
8	32.05	10.70
9	-	27.55
10	-	30.10



Translation Spring Damping = 0.30
Rocking spring Damping = 0.15

Global Foundation Stiffness

DOF	Value t/m
Horizontal Translation	6.85E+05
Vertical Translation	3.72E+06
Rocking	4.48E+08
Torsion	1.62E+08

FIGURE 3. Floor Response Spectrum, Elevation 36, Damp.= 0.02
Comparison between advanced method EKSSI and simplified method (modal analysis - spring base model)