Simplified methods for evaluating the uplift of buildings with raft foundation

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SUMMARY:

The uplift of the raft foundation of reinforced concrete buildings, and the consequent partial loss of contact between soil and foundation, is a possible effect of an earthquake. Actually, reinforced concrete buildings with raft foundation are commonly used in nuclear power plants. In this context, some simplified approaches for uplift evaluation have been proposed in the past and are still used for many practical applications. In particular, the so-called energy equivalence method is studied here. The classical form of this method can be applied to rectangular foundations with earthquake direction parallel to one of the rectangle sides and to circular foundations. A generalization of classical equations to the case of generic foundation shape and generic earthquake direction is proposed in this paper. Then, uplift assessments from energy equivalence method, pushover and non-linear time history analyses are compared, with reference to the specific situation of a nuclear power plant building.

Keywords: Foundation uplift, energy equivalence method, Betbeder's method, Pushover

1. INTRODUCTION

The uplift of the foundation of a building and the consequent partial loss of contact between soil and foundation is a possible effect of an earthquake. In the past years, several authors analyzed the problem of the accurate uplift evaluation, e.g. (Tseng and Liou, 1981, Kobori et al., 1984, Yim and Chopra, 1984, Betbeder, 2003). Actually, most of technical papers about this subject concern the uplift of reinforced concrete buildings with a foundation raft, commonly used in nuclear power plants. The analysis proposed here also regards this special type of foundation. The most direct method of uplift assessment consists in modelling the coupled soil-structure behaviour and performing time history analyses. However, this approach is computationally expensive and it cannot be used during design phases, when complex and accurate calculations are impossible, since the final configuration of the building is not known. For this reason, some simplified methods have been proposed, e.g. (Tseng and Liou, 1981, Betbeder, 2003) and are still used for some practical applications. Simple formulas are also used in codes and standards, see e.g. Harden et al. (2006). A first often accepted simplification regards the modelling of soil by a set of springs of "Winkler" type characterized by an elastic or elasto-perfectly plastic behaviour in compression and with zero strength in tension. In some cases, additional "shear springs" were also considered (Betbeder, 2003). The second simplification concerns the seismic action: instead of considering non-linear time history analyses giving the uplift at any instant for a given accelerogram, linear and/or static analyses are done, aiming to estimate the *maximum* uplift throughout the seismic event. The use of pushover analysis proposed by Harden et al. (2006) for the uplift estimation is an example of non-linear static method (the non linearity is only due to the soil-foundation unilateral contact), see Section 3. The energy equivalence method (Tseng and Liou, 1981) (Betbeder, 2003), also called "Betbeder's method" in the French technical literature, is characterized by an initial linear analysis (with soil springs also working in tension and where uplift cannot occur), which can be either transient, static or based on the modal spectrum method, followed by a semi-analytical estimation, based on the results of the linear analysis, of the non-linear behaviour due to uplift. In detail, this method is based on the assumptions of very stiff foundation (the base raft rotates but it remains plane) and of equivalence of the rocking moment work of the linear model (without uplift) and the rocking moment work of the non-linear model (with uplift). The classical equations of the energy equivalence method (Tseng and Liou, 1981, Betbeder, 2003) deal with two situations: (i) rectangular foundations with earthquake direction supposed parallel to one of the rectangle sides and (ii) circular foundations. A generalization of classical equations to the case of generic foundation shape and generic earthquake direction is proposed in Section 4. In Section 5, uplift assessments from (generalized) energy equivalence method, pushover and non-linear time history analyses are compared, with reference to the specific case of a nuclear power plant building.

2. UPLIFT EVALUATION BY TIME HYSTORY ANALYSIS

Non-linear time-history analysis of a coupled soil-structure finite element model is the most accurate approach to estimate the uplift of a building with raft foundation. Several levels of accuracy can be chosen in the modelling of the soil behaviour and of the soil-structure interface. In the simplest case, the soil is represented by a set of Winkler unilateral springs. This modelling approach is used in this paper for non-linear time-history analyses. Actually, we recall that the assumption of Winkler unilateral springs is also at the basis of some simplified methods of uplift estimation, e.g. (Yim and Chopra, 1984) (Betbeder, 2003) (Harden et al., 2006). Results obtained by non-linear time-history analysis are used as reference to assess the accuracy of uplift estimations coming from the simplified approaches described in the next two sections: pushover analysis and energy equivalence method.

3. UPLIFT EVALUATION BY PUSHOVER ANALYSIS

The pushover method consists in determining the performance point resulting from the intersection of a representative curve of the seismic action and a representative curve of the resistance capacity of the structure. This method is more accurate for structures with a fundamental mode characterized by a high participating mass (FEMA 273, 1997). Multi-modal variants (Chopra and Goel, 2004) are not considered here. In most applications, the pushover method is used to study the influence of the structural non-linearity on the seismic response of a given building. In this article, the structure (i.e. the building) is rather supposed linear and non-linearity is only due to the soil-foundation unilateral contact. A similar analysis was proposed by Harden et al. (2006).

A particular format called ADRS "Acceleration-Displacement Response Spectrum" is used to represent the response spectrum. The ADRS is a parametric plot of the form $\left(S_d(f), S_a(f)\right)$, where f is the generic frequency, $S_d(f)$ is the displacement response spectrum and $S_a(f) = (2\pi f)^2 S_d(f)$ is the pseudo-acceleration spectrum. The capacity curve is obtained by applying a quasi-static increasing lateral load parallel to a given direction to the finite element model: at each load level, the base shear V and the roof displacement δ are evaluated and the following formulas give the capacity curve in the ADRS format:

$$S_a = \frac{V}{M_{eff}} \qquad \text{and} \qquad S_d = \frac{\delta}{F\phi} \tag{3.1}$$

where M_{eff} is the effective mass of the fundamental mode, F is the participation factor of the fundamental mode for the chosen earthquake direction and Φ is the roof modal displacement related to the fundamental mode. Eqns. 3.1 hold when the lateral load is proportional to the product of the mass matrix and the mode shape vector of the fundamental mode. In the alternative case of a load proportional to the nodal masses, the acceleration S_a is obtained by dividing the base shear V by the total structural mass M.

Once the coordinates S_d and S_a of the performance point are found, the rocking moment has to be determined in order to evaluate the percentage of the uplifted part of the base raft. This can be done either by directly reading finite element results or by using the analytical expressions derived hereinafter. Let $\underline{\underline{M}}$ be the mass matrix of the structure; $\underline{Z}$ the vector such that the elements in the direction of the application of the seismic load are the vertical distances of the model nodes to the center of gravity of the raft, while all the other elements are null; P the weight of the building; M_o the rocking moment under the action of the building weight; $\underline{F}$ the vector of the lateral nodal forces; and *b* the half-length of the raft in the direction of application of the lateral load. Since the lateral load is supposed to be proportional to the product of the mass matrix and the mode shape vector of the fundamental mode, $\underline{F}$ can be written as follows:

$$\underline{F} = \alpha \underline{\underline{M}}\, \underline{D}_f \tag{3.2}$$

where α is a proportionality coefficient and $\underline{D}_f$ is the mode shape vector of the fundamental mode. The base shear V is obtained by the following formula:

$$V = \underline{F}^T \underline{\Delta} \tag{3.3}$$

In Eqn. 3.3, the elements of the vector $\underline{\Delta}$ are either equal to 1 or 0: 1 if the element corresponds to the direction of the application of the load and 0 otherwise. The total rocking moment M_t at the base of the building reads:

$$M_t = \underline{F}^T \underline{Z} + M_o \tag{3.4}$$

Let d represent the vertical distance between the base raft and the barycenter of the horizontal loads $\underline{F}$. Then, the distance d reads:

$$d = \frac{\underline{F}^T \underline{Z}}{\underline{F}^T \underline{\Delta}} = \frac{\underline{D}_f^T \underline{\underline{M}}\, \underline{Z}}{\underline{D}_f^T \underline{\underline{M}}\, \underline{\Delta}} \tag{3.5}$$

As a result, the total moment M_t becomes equal to:

$$M_t = Vd + M_o \tag{3.6}$$

Let M* represent the non-dimensional moment given by the following formula:

$$M^* = \frac{M_t}{Pb} = \frac{Vd + M_o}{Pb} \tag{3.7}$$

Moreover, let us name S* the uplift percentage, equal to the ratio between the uplifted surface and the total foundation surface. In Betbeder (2003), one finds the relationships that allow calculating S* as a function of M* in the case of a very stiff rectangular raft:

$$S^* = 0 \quad \text{if} \quad M^* \leq \frac{1}{3} \qquad \text{and} \qquad S^* = \frac{1}{2}\left(3M^* - 1\right) \quad \text{if} \quad M^* > \frac{1}{3} \tag{3.8}$$

4. UPLIFT EVALUATION BY THE ENERGY EQUIVALENCE METHOD

4.1. Classical energy equivalence method for rectangular rafts

The energy equivalence method is based on the assumption of equivalence of the rocking moment work of the linear model (with soil modelled by linear springs working in tension-compression) and the rocking moment work of the non-linear model (where the uplift is accounted for by soil-springs with unilateral behaviour), see e.g. Tseng and Liou (1981) and Betbeder (2003). The principle of this method is illustrated in Fig. 4.1. The graph shows the curves that represent the relationships between the reduced moment M* (defined by Eqn. 3.7) and the reduced rotation of the raft θ*: the blue curve corresponds to the linear model without uplift and the pink curve corresponds to the non-linear model with uplift. The reduced rotation θ* is equal to the ratio between the actual base raft rotation and the limit rotation at the beginning of the uplift phase. Classical equations concern circular or rectangular foundations, with earthquake parallel to one of the rectangle sides. Moreover, the raft is supposed very stiff.

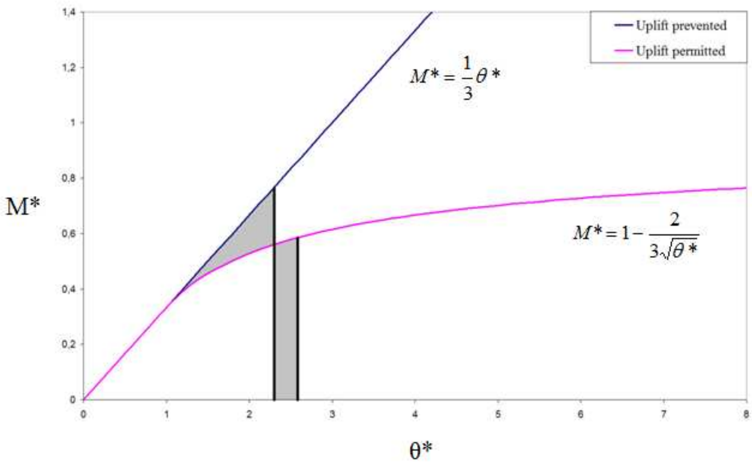


Figure 4.1. Illustration of the energy equivalence method

Let us suppose that the rocking moment M_l coming from a linear seismic (time history or modal spectrum) analysis is available. The application of the energy equivalence method consists in computing M_l* using Eqn. 3.7 and looking for the point on the pink curve in Fig. 4.1 which enables to have the same surfaces under this curve and under the blue one. In other words, this point on the pink curve is such that the grey-coloured surfaces are identical. For a very stiff rectangular raft, the uplift percentage S*, function of the linear reduced moment M_l*, reads (Betbeder, 2003):

$$S^* = 1 - \frac{3}{2 + \sqrt{\frac{1}{2}\left(27\left(M_l^*\right)^2 - 1\right)}} \tag{4.1}$$

The corresponding non-linear reduced moment M_n* becomes equal to:

$$M_n^* = 1 - \frac{1}{1 + \sqrt{\frac{1}{8}\left(27\left(M_l^*\right)^2 - 1\right)}} \tag{4.2}$$

4.2. Proposal of an extended energy equivalence method

In this Section, the energy equivalence method is extended to the case of generic earthquake direction and generic shape of the base raft, which is still supposed to be very stiff. Under these assumptions, the stress applied by the foundation on the soil in the vertical direction reads:

$$\sigma(x, y) = a + bx + cy \tag{4.3}$$

where x and y are the coordinates of a generic soil-foundation contact point with respect to a given reference; a, b and c are constants depending on the efforts applied to the soil (N: total vertical force applied on the soil, M_x: total rocking moment around the X direction and M_y: total rocking moment around the Y direction) and on the geometry of the base raft area in contact with soil. When the whole foundation surface is in contact with the soil, the stress is given by the following formula:

$$\sigma(x,y)=\frac{N}{S}+\frac{M_yI_x+M_xI_{xy}}{I_xI_y-I_{xy}^2}x-\frac{M_xI_y+M_yI_{xy}}{I_xI_y-I_{xy}^2}y \tag{4.4}$$

where S is the foundation surface, I_x (I_y) is the raft moment of inertia with respect to the X(Y) axis and I_{xy} is the product of inertia.

The work of the rocking moment W can be written as the sum of the energy of deformation of the soil W_D and of the work of the vertical force W_N:

$$W = W_D + W_N \tag{4.5}$$

with

$$W_D=\frac{1}{2nk}\left[\iint_{A_c}(\sigma(x,y))^2\,ds-\frac{N^2}{S}\right] \quad \text{and} \quad W_N=\frac{N}{nk}\left(\frac{N}{S}-a\right) \tag{4.6}$$

where n is the surface density of vertical springs (m^{-2}), k is the stiffness of a single vertical spring (N/m) and A_c is the base raft area in contact with soil. When the base raft is completely in contact with the soil, W reads:

$$W=\frac{1}{2nk}\left(\frac{M_x^2I_y+M_y^2I_x+2M_xM_yI_{xy}}{I_xI_y-I_{xy}^2}\right) \tag{4.7}$$

For the points located on the curve separating the uplifted part of the raft from the part in contact with the soil, the stress is equal to zero. Thus, the equation for this curve is:

$$a+bx+cy=0 \tag{4.8}$$

For given efforts (N, M_x, M_y) coming from the linear model, the applied efforts $\left(\tilde{N},\tilde{M}_x,\tilde{M}_y\right)$ for the nonlinear model have to be determined. A first equation comes from the equivalence of vertical forces:

$$N=\iint_{A_c}\sigma(x,y)\,ds=\tilde{N} \tag{4.9}$$

Then, the application of the energy equivalence method consists in determining the constants a, b and c such that:

$$\frac{1}{2nk}\left[\iint_{A_c}(\sigma(x,y))^2\,ds-\frac{N^2}{S}\right]+\frac{N}{nk}\left(\frac{N}{S}-a\right)=\frac{1}{2nk}\left(\frac{M_x^2I_y+M_y^2I_x+2M_xM_yI_{xy}}{I_xI_y-I_{xy}^2}\right) \tag{4.10}$$

The term on the left of this equality corresponds to the work of the rocking moment for the nonlinear model and the right one corresponds to the work of the rocking moment of the linear model. Eqns. 4.9 and 4.10 are not sufficient to determine the three constants a, b and c: a third equation is needed. As a last condition, it is assumed that the vectors $\left(\tilde{M}_x,\tilde{M}_y\right)$ and $\left(M_x,M_y\right)$ are parallel:

$$\frac{\tilde{M}_x}{\tilde{M}_y}=\frac{M_x}{M_y} \ \text{when } M_x=0 \quad \text{and} \quad \frac{\tilde{M}_y}{\tilde{M}_x}=\frac{M_y}{M_x} \ \text{otherwise} \tag{4.11}$$

The analytical solution of the system of Eqns. 4.9, 4.10 and 4.11 cannot be found in the general case. Hence, an iterative procedure is suggested to solve the problem (this procedure has been implemented in the SCILAB software):

I. Initialize the constants a, b and c:

$$a_0 = \frac{N}{S}; \qquad b_0 = \frac{M_y I_x + M_x I_{xy}}{I_x I_y - I_{xy}^2}; \qquad c_0 = -\frac{M_x I_y + M_y I_{xy}}{I_x I_y - I_{xy}^2}$$

The initial location of the line dividing the uplifted area from the one in contact with the soil is determined by the Eqn. 4.8.

II. Set i=1

III. Compute $A_{c,i}$ (raft area in contact with soil) from a_{i-1} , b_{i-1} and c_{i-1}

IV. Compute

$$\tilde{N}_i = \iint_{A_{c,i}} \sigma(x,y)ds; \qquad \tilde{M}_{x,i} = -\iint_{A_{c,i}} y\sigma(x,y)ds; \qquad \tilde{M}_{y,i} = \iint_{A_{c,i}} x\sigma(x,y)ds$$

$$\tilde{W}_i = \frac{1}{2nk}\left[\iint_{A_{c,i}} (\sigma(x,y))^2 ds - \frac{N^2}{S}\right] + \frac{N}{nk}\left(\frac{N}{S} - a_{i-1}\right)$$

V. Check

IF:

$\bullet\ \Delta N_i = N - \tilde{N}_i < \varepsilon$
$\bullet\ \Delta W_i = W - \tilde{W}_i < \varepsilon$
$\bullet\ \Delta\left(\frac{M_x}{M_y}\right)_i = \frac{M_x}{M_y} - \frac{\tilde{M}_{x,i}}{\tilde{M}_{y,i}} < \varepsilon$

when $M_x = 0$,

$\bullet\ \Delta N_i = N - \tilde{N}_i < \varepsilon$
$\bullet\ \Delta W_i = W - \tilde{W}_i < \varepsilon$
$\bullet\ \Delta\left(\frac{M_y}{M_x}\right)_i = \frac{M_y}{M_x} - \frac{\tilde{M}_{y,i}}{\tilde{M}_{x,i}} < \varepsilon$

otherwise

In these expressions, W is given by Eqn. 4.7 (the work of the rocking moment for the linear model).

STOP,
OTHERWISE:

VI. Compute a_i , b_i and c_i , set i=i+1 and GOTO **3.**

5. APPLICATION

5.1. Building and soil models

The structure analyzed in this study is a five-story reinforced concrete building. Two analyses are considered: (i) earthquake in one direction only (Subsection 5.2) and (ii) three component earthquake (Subsection 5.3). The building has an almost rectangular base raft and three slabs. It is 23.92 meters high and the base raft dimensions are: 23.6m x 23.6m x 1m (Fig. 5.1). The building mass is around 11000 tons. This building is modeled using ANSYS 11.0 computer program. The base raft, walls and slabs are modeled by shell elements SHELL 43. The beams and the columns are modeled by elements BEAM4. The material masses are modeled by elements MASS21. Every node of the base raft is connected to a fixed node by three springs: two horizontal springs and one vertical spring. Two sorts of springs are considered: (i) linear springs which keep permanently the contact with the base raft; (ii) nonlinear springs which allow the foundation uplift. The linear springs are modelled in ANSYS by COMBIN14 elements and nonlinear springs by COMBIN37 elements. In this study, the following soil impedances are used: K_x = 4.88 10^{10} N/m in the X direction, K_y = 4.49 10^{10} N/m in the Y direction, K_z = 7.73 10^{10} N/m in the Z direction. The stiffness of every spring is calculated from the global stiffness of the soil by multiplying it by the ratio of the surface allocated to this spring to the raft surface. Damping is supposed equal to 5% for both the structure and soil and is modelled according to the Rayleigh assumption with α=2.41 and β=0.00087.

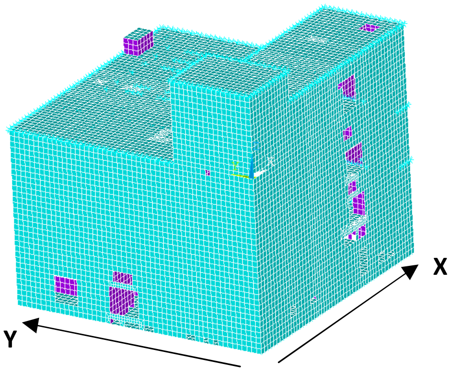


Figure 5.1. The building model

Table 5.1. Modal analysis results

Mode	Frequency	Partici. factor	Mass fraction
1	5.28	-306.31	0.009
2	5.45	2445.90	0.54
3	8.37	-65.33	0.0004
4	11.47	-602.40	0.034
5	12.58	1594.40	0.23
6	12.93	1205.90	0.13

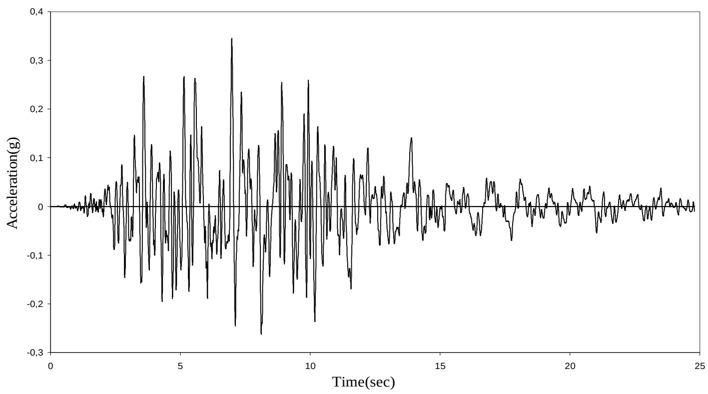
5.2. Seismic load applied in one lateral direction

The building is almost symmetric with respect to the axis parallel to Y passing through the geometric centre of the raft. Moreover, the fundamental mode in the Y direction (the 2^{nd} one, see Table 5.1) has a relatively high participating mass (54%). This direction is considered for the unidirectional study, because the basic assumptions of both pushover and classical energy equivalence methods are approximately fulfilled.

5.2.1. Time history analyses

Two time history analyses have been performed: the first one on the linear model (without uplift) and second one on the model with uplift.

(a)



(b)

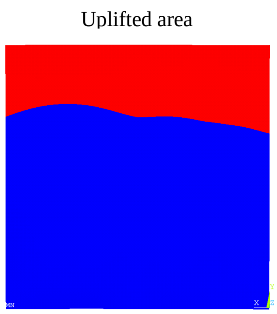


Figure 5.2. (a) Kobe ground motion; (b) Maximum raft uplift configuration

An accelerogram corresponding to the Kobe earthquake (January 17th, 1995) with PGA (Peak Ground Acceleration) of 0.345g has been used (Fig. 5.2.a). This earthquake is applied in the Y direction only. We are interested in the peak values of base shear V, rocking moment M_t, uplift percentage S* and roof displacement δ reached during the earthquake. Table 5.2 summarizes these values. Observe that the peaks of the different quantities are not simultaneous. The model where the uplift is taken into account is characterised by reduced values of V and M_t. The roof displacement is almost identical to the one obtained from the linear model. Fig. 5.2.b shows the maximum foundation uplift configuration. The line separating the uplifted part of the raft from the part in contact with the soil is not completely straight: this is due to the base raft flexibility.

Table 5.2. Time history analyses results

	Linear model	Non-linear model
Base shear (MN)	58.98	56.42
Rocking moment (MN.m)	796.53	692.21
Uplift surface (%)	0	26.74
Roof displacement (mm)	8.41	8.42

5.2.2. Pushover analysis

The load is supposed parallel to the Y direction, like in the previous time history analyses. In Fig. 5.3, both capacity curve of the linear and non-linear models are plotted, together with the response spectrum corresponding to the accelerogram of the Kobe earthquake (Fig. 5.2.a) for a damping ratio of 5% (damping ratios of both soil and structure are supposed equal to 5%):

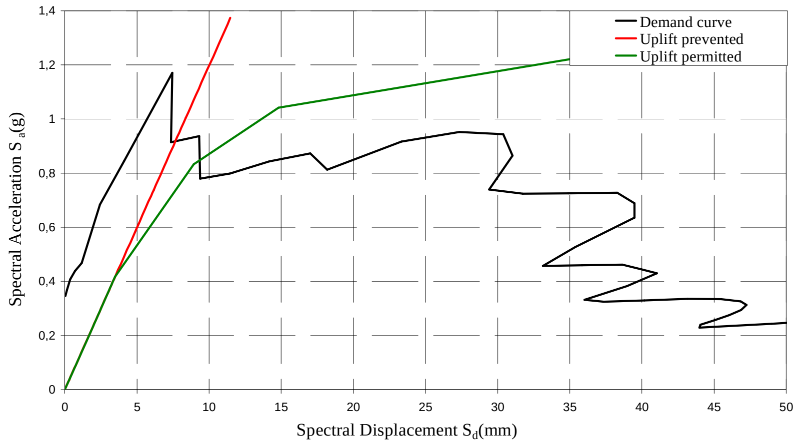


Figure 5.3. Capacity curves and demand curve (ADRS spectrum)

As already observed, the second mode is the dominant one in the Y-direction, with an effective mass equal to the 54 % of the total mass of the structure. Hence, the capacity curves have been built using an increasing load proportional to this mode. It is straightforward to verify that the initial slope of capacity curves corresponds to the square of the pulsation of this mode (having a frequency equal to 5.45 Hz, see Table 5.1). The intersection points of the capacity curves with the response spectrum correspond to the performance points of the linear and nonlinear models, since both building and soil (in compression) are supposed linear elastic, no additional damping due to hysteresis has to be considered. From performance points, the corresponding maximal base shear and the maximal roof displacement are computed for both models (Table 5.3). The static rocking moment is M_o = 57.10 MN.m and d = 14.43 m (obtained from Eqn. 3.5). Then, rocking moments at performance points for both linear and non-linear models can be calculated from Eqn. 3.6 and Table 5.3. This leads to M_t = 832.00 MNm for the linear model and M_t = 783.65 MNm for the non-linear model. In the non-linear case, the non-dimensional moment M* and the uplift percentage S*, given by Eqn. 3.7 and Eqn. 3.8 respectively, become equal to M* = 0.62 > 0.33 and S* = 42.24 %. The uplift percentage is quite different from the one computed by non-linear time history analysis (26.7 %; see Section 5.2.1). This difference may come from the fact that the fundamental mode participating mass is not large enough and other modes (the 5th and the 6th ones in the case of the building studied here) also participate to the dynamic response of the structure. Notice that the same discrepancy between uplift assessments obtained from time-history and pushover analyses has been found by Harden et al. (2006).

Table 5.3. Pushover analysis results

	Linear model	Nonlinear model
Base shear (MN)	53.70	50.35
Roof displacement (mm)	8.49	10.3

5.2.3. Energy equivalence method

The classical energy equivalence method (Eqns. 4.1 and 4.2) is applied to the building studied here. The considered input data is the maximum rocking moment obtained by the linear time history analysis. This leads to $S^* = 28.31$ % and $M_n = 665.36$ MN.m : thus, in spite of its simplicity, this method gives a good estimation of the raft uplift (6% error with respect to non-linear time history analysis; Table 5.2, 3rd row) and also a good estimation of the non-linear rocking moment (4% error; see Table 5.2, 2nd row).

5.3. Seismic load applied in the three directions of the space

In this case, the unidirectional pushover analysis cannot be applied. Conversely, the extended energy equivalence approach proposed in Section 4.2 can still be used, like the non-linear time-history analysis. Three different artificial accelerograms (Fig. 5.5), one for each direction, have been applied to both linear and non-linear models. They have been obtained from the spectrum of Fig. 5.4. The vertical accelerogram is reduced by the factor 2/3. The efforts obtained by time-history analysis of the linear model at the instant where the rocking moment vector (M_x, M_y) has maximum norm are used as input data for the extended energy equivalence method. These efforts are given in Table 5.4. In order to check the accuracy of the proposed extended energy equivalence method, the maximum rocking moments and the maximum uplift percentage obtained by this method are compared with the non-linear time-history analysis results. Table 5.5 resumes this comparison, showing that the extended energy equivalence method gives a good estimation of the uplift percentage (24% vs. 26%). The estimation of rocking moments is less accurate.

Table 5.4. Time history analysis results for the linear model

The vertical force N (MN)	106
The rocking moment Mx (MN.m)	640
The rocking moment My (MN.m)	-504

Table 5.5. Comparison between time history analysis and extended energy equivalence method

	Time history analysis	Extended energy equivalence method
The rocking moment Mx (MN.m)	-10.3	517
The rocking moment My (MN.m)	465	-407
Uplift surface (%)	24.06	26.18

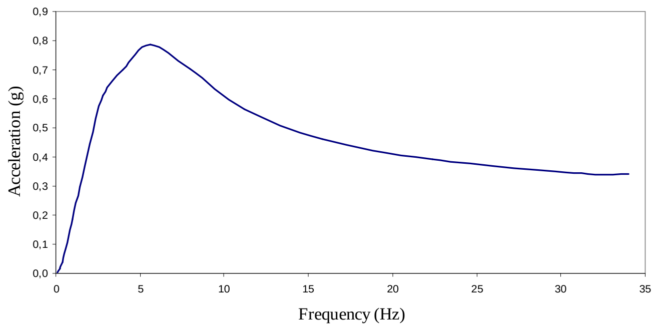


Figure 5.4. Response spectrum (5% damping) corresponding to Kobe earthquake (Fig. 3a) with PGA = 0.345 g

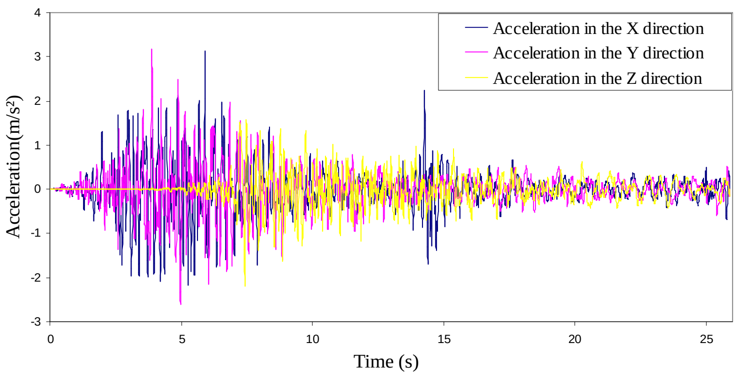


Figure 5.5. Artificial accelerograms obtained from the spectrum of Fig. 5.4

6. CONCLUSION

In this paper, three methods for the foundation uplift estimation under seismic actions have been compared: non-linear time-history analysis, pushover analysis and energy equivalence approach (also called "Betbeder's method" in the French literature). Moreover, an *extension* of the classical energy equivalence method has been proposed. The standard method is limited to the cases of circular and rectangular foundations with earthquake direction parallel to one of the sides. The extension concerns generic foundation shape and generic earthquake direction. For a seismic load applied on a building with an approximately rectangular base raft in the direction parallel to one of the sides, the uplift evaluation obtained by the standard energy equivalence method is found to be more accurate than the one obtained by the pushover analysis. In the case of generic earthquake direction, the extended energy equivalence method has also provided an accurate uplift estimation. Results presented here concern a single case study. Work is in progress to increase the number of case studies and to give a robust judgement on the accuracy of the extended energy equivalence method.

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