A STUDY OF SIMPLIFIED ANALYSIS METHOD OF THE HORIZONTAL SEISMIC ACTION ON STRUCTURES CONSIDERING SSDI EFFECTS

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SUMMARY

A simplified analysis model is put forward to estimate the earthquake responses of the interactive action of the ground soil-pile (box) foundation-upper structure. Using this model, the responses of more than 6000 high-rise building-ground soil dynamic interaction systems are analyzed, with the building story number 9, 15, 20, 25 respectively, the distribution characteristics of horizontal seismic action reduction coefficients considering SSDI effects are obtained. A computational method of fundamental natural period of SSDI system is established though theoretical analysis. In terms of response spectra theory, together with statistical results of 6000 calculation examples and available research conclusion home and abroad, the semi-empirical computational formula of the horizontal seismic action reduction coefficients are proposed, which can be used as a reference for seismic design of buildings considering SSDI effects.

INTRODUCTION

The research of SSDI is backward to the Reissner's vibrant research in circle foundation in the condition of surface rigidity and elasticity half-space in 1936. To 1950's, many researchers gained the circle foundation and rectangle foundation's transient and steady solution of plane, wheeling and distorting movement in the condition of stress boundary. In the middle of 1960's, Parmelee preliminary uncovered the basic characteristics of inertial dynamic interaction. After 1970's, the study of SSDI developed very active, the main points are the following:

Numerical discrete method The application of finite difference method, finite element method and boundary element method have offered a tool to analysis the SSDI of complex structure. The FE method is most suitable for modeling problem with complex geometry and loading, but the BE method is very easy to discuss the infinite boundary. O.C.Zienkiewicz etc developed the mixed method using the FE method and BE method, expanded a method to analysis SSDI[1].

Analysis method There are two main method to analysis the SSDI: frequency domain analysis and time domain analysis. In frequency domain, it is easy to relate the foundation impedance function with the frequency. The whole soil-structure system can be separated into a simpler set of two subsystem at an artificial boundary, that is, a structure with a near field subsystem and one with a far field subsystem, then study the reaction of the two subsystem respectively. The artificial boundary can be get using the FE method or wave transmitting theory, the boundary function is in the frequency. So most of researcher adopts the frequency domain method. But this method is difficult to deal with non-linear effects. Thus the study of the time domain method is becoming important. There are many artificial boundary for time domain analysis being putting forward, such as: Z.P.Liao's transmitting boundary and quiescence boundary, Smith's compound boundary coupling the Dirichlet boundary and Neumamn boundary used to reject the reflected wave in the boundary[2].

Vibration excitation The study of vibration excitation to consider the SSDI effect have two style: One is the vibration excitation of embedded foundation (including the pile-box foundation and raft-box foundation). The excitation amplitude and frequency in soil layer is different with in bedrock and surface. So the excitation is

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different in different position of the embedded foundation. The other is multiple excitation. F.Y.Cheng analyzed the structure’s three-dimensional earthquake response in the condition of multi-support and multi-direction excitations. X.L.Jiang etc studied the issue of earthquake response of neighboring building-foundation-soil system under multi-support excitations.

Approximate computational method The computation considering SSDI effect will be spend a lot of time, many researchers are developing the simplified computational method to approximately consider the effect of SSDI. The mixed method which Penzien puts forward is belong to this. In the same time, in order to settle for the needs of the seismic analysis of nuclear power reactor, some simplified computation model is putting forward to consider the effect of SSDI.[4],[5]

In-situ test and the conclusions of seismic observation A important aspect of SSDI is to obtain the observing records of building vibration and strong ground motion and to do the large scale in-situ test. But the scale of this test is large, it will be spent a lot of manpower, material resources, money and time, so the typical test is lack. Some apparent information has got with the observation and research of the earthquake engineering near century, but the farther research will be gone on. The main test conclusions are:

(1) If the dimension of the foundation in the same ground is added, the fundamental frequency of the SSDI system will be decreased, the natural period of vibration of structure will be extended, the damping will be increased. If the dimension of the foundation is reduced, the amplitude of vibration will be added, the damping will be decreased, the resonance peak will be cut. If the embedded depth of the foundation is extent, the fundamental frequency of the SSDI system will be increased, the amplitude of vibration will be decreased, the damping will be added. When the embedded depth reaches a specific depth, the damping will be added remarkably.

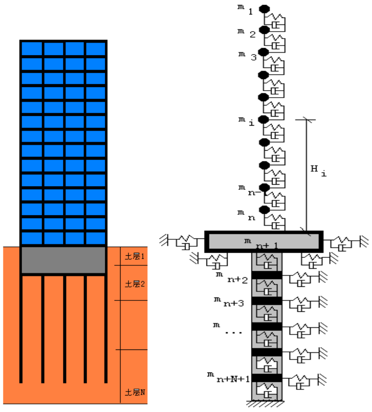

Figure 1 Simplified Analysis Model to Consider the Ground Soil-pile (Box) Foundation-Upper Structure Interaction

(2) The stiffness of upper-structure more smaller, the distortion of foundation more bigger, but the ratio of the distortion of foundation to the stiffness of upper-structure is still small. When the upper-structure is stiff, the frequency and damping of the SSDI system will change greater. When the upper-structure is flexible, the frequency and damping of the SSDI system will change littler. The frequency and damping of the SSDI system will change littler in high order mode.

(3) When the foundation and building is stiff, the interaction of neighboring building or neighboring foundation will be greater. However when the foundation and building is flexible, the interaction of neighboring building or neighboring foundation will be littler.

A WHOLE SIMPLIFIED ANALYSIS MODEL TO CONSIDER THE GROUND SOIL-PILE(BOX) FOUNDATION-UPPER STRUCTURE INTERACTION

Simplified Analysis Model

The whole simplified analysis model to consider the ground soil-pile (box) foundation-upper structure interaction is shown in figure 1, the differential equation is given by the equation (1).

$$M\ddot{X} + C\dot{X} + KX = -M\ddot{U}_g \qquad \text{(Eq.1)}$$

in which M is the mass matrix, define as the equation (2), K is the stiffness matrix, define as equation the (3), X, $\dot{X}$, $\ddot{X}$ and $\ddot{U}_g$ are the vectors of displacement, velocity, acceleration and ground motion excitation, shown as the equation (4).

$$M = \begin{bmatrix} m_1 & 0 & \cdots & 0 & m_1 & 0 & 0 & 0 & 0 & m_1H_1 \\ 0 & m_2 & \cdots & 0 & m_2 & 0 & 0 & 0 & 0 & m_2H_2 \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & \cdots & m_n & m_n & 0 & 0 & 0 & 0 & m_nH_n \\ m_1 & m_2 & \cdots & m_n & m_b + \sum\limits_{j=1}^{n} m_j & 0 & 0 & 0 & 0 & \sum\limits_{j=1}^{n} m_jH_j \\ 0 & 0 & \cdots & 0 & 0 & m_{n+1} & 0 & \cdots & 0 & 0 \\ 0 & 0 & \cdots & 0 & 0 & 0 & m_{n+2} & \cdots & 0 & 0 \\ 0 & 0 & \cdots & 0 & 0 & \vdots & \vdots & \ddots & \vdots & \vdots \\ 0 & 0 & \cdots & 0 & 0 & 0 & 0 & \cdots & m_{n+N} & 0 \\ m_1H_1 & m_2H_2 & \cdots & m_nH_n & \sum\limits_{j=1}^{n} m_jH_j & 0 & 0 & \cdots & 0 & I_b + \sum\limits_{j=1}^{n} m_jH_j^2 \end{bmatrix} \qquad \text{(Eq.2)}$$

$$K = \left[\begin{array}{ccccccccccccc} & k_1 & -k_2 & 0 & \vdots & 0 & 0 & 0 & 0 & 0 & \vdots & 0 & 0 \\ & -k_2 & k_1+k_2 & -k_2 & \vdots & 0 & 0 & 0 & 0 & 0 & \vdots & 0 & 0 \\ & 0 & -k_2 & k_2+k_3 & \vdots & 0 & 0 & 0 & 0 & 0 & \vdots & 0 & 0 \\ & \cdots & \cdots & \cdots & \ddots & \cdots & \cdots & \cdots & \cdots & \cdots & \ddots & \cdots & \cdots \\ & 0 & 0 & 0 & \vdots & k_{n-1}+k_n & -k_n & 0 & 0 & 0 & \vdots & 0 & 0 \\ & 0 & 0 & 0 & \vdots & -k_n & k_n+k_{xx} & -k_{xx} & 0 & 0 & \vdots & 0 & -k_{\theta x} \\ 0 & 0 & 0 & 0 & \vdots & 0 & -k_{xx} & k_{xx}+k_{p1} & -k_{p1} & 0 & \vdots & 0 & 0 \\ & 0 & 0 & 0 & \vdots & 0 & 0 & -k_{p1} & k_{p1}+k_{p2} & -k_{p2} & \vdots & 0 & 0 \\ & 0 & 0 & 0 & \vdots & 0 & 0 & 0 & -k_{p2} & k_{p2}+k_{p3} & \vdots & 0 & 0 \\ & \cdots & \cdots & \cdots & \ddots & \cdots & \cdots & \cdots & \cdots & \cdots & \ddots & \cdots & \cdots \\ & 0 & 0 & 0 & \vdots & 0 & 0 & 0 & 0 & 0 & \vdots & k_{pN-1}+k_{pN} & 0 \\ & 0 & 0 & 0 & \vdots & 0 & -k_{x\theta} & 0 & 0 & 0 & \vdots & 0 & k_{\theta\theta} \end{array}\right] \qquad \text{(Eq.3)}$$

$$X = \left\{\begin{matrix} x_{s1} \\ x_{s2} \\ \vdots \\ x_{sn-1} \\ x_{sn} \\ x_b \\ x_{p1} \\ x_{p2} \\ \vdots \\ x_{pN-1} \\ x_{pN} \\ \theta \end{matrix}\right\}, \quad \dot{X} = \left\{\begin{matrix} \dot{x}_{s1} \\ \dot{x}_{s2} \\ \vdots \\ \dot{x}_{sn-1} \\ \dot{x}_{sn} \\ \dot{x}_b \\ \dot{x}_{p1} \\ \dot{x}_{p1} \\ \vdots \\ \dot{x}_{pN-1} \\ \dot{x}_{pN-1} \\ \dot{\theta} \end{matrix}\right\}, \quad X = \left\{\begin{matrix} \ddot{x}_{s1} \\ \ddot{x}_{s2} \\ \vdots \\ \ddot{x}_{sn-1} \\ \ddot{x}_{sn} \\ \ddot{x}_b \\ \ddot{x}_{p1} \\ \ddot{x}_{p1} \\ \vdots \\ \ddot{x}_{pN-1} \\ \ddot{x}_{pN-1} \\ \ddot{\theta} \end{matrix}\right\}, \quad \ddot{U}_g = \left\{\begin{matrix} \ddot{\mu}_{gd} \\ \ddot{\mu}_{gd} \\ \\ \ddot{\mu}_{gd} \\ \ddot{\mu}_{gd} \\ \ddot{\mu}_{gd} \\ \ddot{\mu}_{gdb1} \\ \ddot{\mu}_{gdb2} \\ \vdots \\ \ddot{\mu}_{gdbN-1} \\ \ddot{\mu}_{gdbN} \\ 0 \end{matrix}\right\} \qquad \text{(Eq.4)}$$

The damping matrix is complicated. Assumed the damping matrix is constant and can be written as the linear combination of stiffness matrix K and mass matrix M in this paper, that is:

$$[C] = \alpha[M] + \beta[K] \qquad \text{(Eq.5)}$$

in which, α,β can be gotten in the structure dynamic literatures.

Semi-Degradation Three-lines Resilience Model

During discussing the solution of the kinetic equation of non-linear system, the stiffness matrix K_t is different in different time t. In order to find the variability of stiffness matrix K_t, the resilience model of system must be studied. Under the strong earthquake loading, the non-linear behavior of the ground soil-pile (box) foundation-upper structure interaction system main shows as the non-linear behavior of upper-structure and ground soil.

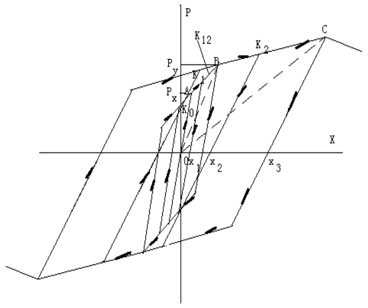

Figure 2 Semi-Degradation Three-lines Resilience Model

The semi-degradation three-lines resilience model is shown as Figure 2, it can describe the resilience behavior of upper-structure and ground soil. When the computational earthquake shear P is less than or equal to crazing loading P_x, the loading stiffness and unloading stiffness all are K_0. When the computational earthquake shear P is more than crazing loading P_x and less than bending loading P_y, the loading stiffness is K_1, the unloading stiffness is K_0, and modified the computational earthquake shear. When the computational earthquake shear P is more than bending loading P_y, the loading stiffness is K_2, the unloading stiffness is K_{12}, and modified the computational earthquake shear.

computational result

In order to quantificationally describe the horizontal seismic action effect induced by SSDI, the conception of horizontal seismic action reduction coefficient is established, it is defined as:

$$\text{horizontal seismic action reduction coefficient } \eta \quad \frac{\text{horizontal seismic action considering SSDI}}{\text{horizontal seismic action not considering SSDI}} \qquad \text{(Eq.6)}$$

According this method, we calculated more than 6000 examples which is the system of the ground soil-pile (box) foundation-upper structure dynamic interaction with the story number 9, 15, 20, 25 respectively and with more than 60 site conditions consist of different thickness of site soil layer, different type of site soil. The computational results are shown in figure 3, 4,5. In term of the numerical simulation and statistical result of more than 6000 calculated examples, the statistical upper limit of horizontal seismic action reduction coefficient relathe with the building height and type of site soil is putted forward. The result is convinced, but it is difficult to gain the expression of horizontal seismic action reduction coefficient. Because other researchers 's study of SSDI don't include the horizontal seismic action reduction, it is difficult to help the engineering design using their research, the engineering action is inapparent.

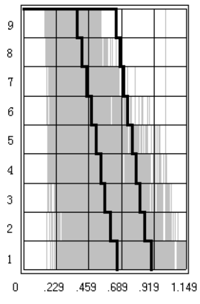

Figure3 Horizontal Seismic Action Reduction Coefficient Related with the Building Height(9 stories)

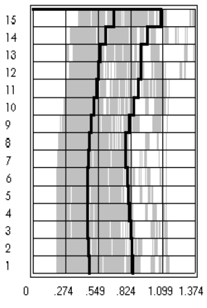

Figure4 Horizontal Seismic Action Reduction Coefficient Related with the Building Height (15stories)

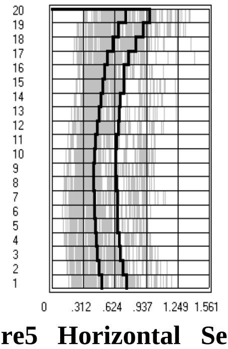

Figure5 Horizontal Seismic Action Reduction Coefficient Related with the Building Height (20stories)

CALCULATE THE FUNDAMENTAL NATURAL PERIOD OF VIBRATION OF SSDI SYSTEM

When considering the SSDI, one of effect is extended the system's fundamental natural period of vibration. Now assumed the upper-structure as single particle system (to high-building, only care for fundamental natural period of vibration), the mass is m, the stiffness is K_1, assumed the foundation as non-mass rigid body, the horizontal and rotational restricted whisht stiffness (the ground soil to foundation) are K_H, K_R. From this, we built a simplified analysis model as figure 6.

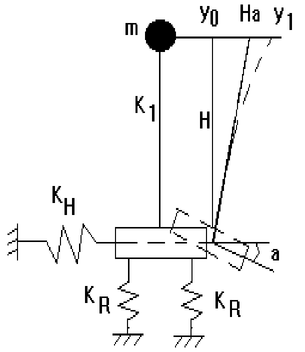

Figure 6 Single Particle System Simplified Analysis Model

The non-damping free vibration equation of the system is:

$$\begin{cases} m(\ddot{y}_0 + H\ddot{\phi} + \ddot{y}_1) + K_1 y_1 = 0 & (a) \\ K_H y_0 - K_1 y_1 = 0 & (b) \\ K_R\phi + m(\ddot{y}_0 + H\ddot{\phi} + \ddot{y}_1)H = 0 & (c) \end{cases} \qquad \text{(Eq.7)}$$

in which y_0 is the horizontal displacement of foundation, φis the rotational angle of foundation, y_1 is the relative displacement of the mass m to foundation.

From the equations (a),(c) of (7) , get the following:

$$\phi = \frac{K_1 H}{K_R} y_1 \qquad \text{(Eq.8)}$$

From the equation (8) and the equation (b) of (7), get the expression of y_0, replace y_0 in the equation (a) of (7), have:

$$m\ddot{y}_1 + K y_1 = 0 \qquad \text{(Eq.9)}$$

where $K = 1(1/K_1 + 1/K_H + 1/K_R)$. If regarded T_1, as the fundamental natural period of vibration of structure in the assumed condition of rigid foundation, T_g, as the fundamental natural period of vibration of the foundation horizontal vibration (or site fundamental period of vibration), T_R, as the fundamental natural period of vibration of the foundation rotational vibration, then:

$$T_1 = 2\pi\sqrt{m/K_1}, \quad T_g = 2\pi\sqrt{m/K_H}, \quad T_R = 2\pi\sqrt{m/K_R} \qquad \text{(Eq.10)}$$

Then the fundamental natural period of vibration of whole system is:

$$T'_1 = 2\pi\sqrt{m/k} = \sqrt{T_1^2 + T_g^2 + T_R^2} \qquad \text{(Eq.11)}$$

in the equation (11), the fundamental natural period of vibration T'_1, can express as the square root of the sum of the square of T_1, T_g, and T_R,. Because of T_g□0□T_R□0, T'_1 is always more than T_1,. Therefore if considered the SSDI, one of effect is extended the system‘s fundamental natural period of vibration.

ANALYSIS THE REDUCTION COEFFICIENT APPLIED THE RESPONSE SPECTRUM THEORY

If considered the SSDI, the fundamental natural period of vibration of SSDI system will be extended, and the horizontal seismic action of upper-structure will be decreased. The following is the analysis of the horizontal seismic action reduction coefficient applied the response spectrum theory.

Analysis the reduction coefficient applied the response spectrum theory

Assumed the fundamental natural period of vibration of structure in the condition of rigid foundation is T_1,, the fundamental natural period of vibration of SSDI extended to T'_1, according the response spectrum theory, the seismic influence coefficients are the equation 12.

$$\begin{cases} \left(T_g / T_1'\right)^{0.9} \alpha_{\max} \geq 0.2\alpha_{\max} \\ \left(T_g / T_1\right)^{0.9} \alpha_{\max} \geq 0.2\alpha_{\max} \end{cases} \qquad \text{(Eq.12)}$$

If ignored the influence of high order mode, the horizontal seismic action reduction coefficient η is:

$$\eta = \left(T_g / T_1'\right)^{0.9} \alpha_{\max} / \left(T_g / T_1\right)^{0.9} \alpha_{\max} = \left(T_1 / T_1'\right)^{0.9} \qquad \text{(Eq.13)}$$

The expression states: the horizontal seismic action reduction coefficient only relates with the ratio of T_1 to T'_1.

Although above conclusion only adapted to the single particle elasticity system and ignored the influence of the high order mode, they can offered theoretic base for the horizontal seismic action reduction coefficient η of the multi-particle elasticity system.

The semi-empirical formula of the horizontal seismic action reduction coefficient of SSDI

According to the analysis and the research, we can discover:

(1) To high building with piles-box (piles-raft) foundation, the distribution of the horizontal seismic action reduction coefficient related the story isn't a constant. The horizontal seismic action reduction coefficient of the top story in the high building is mostly not reduced, but increased. So the horizontal seismic action reduction coefficient in different story must have different value.

(2) Extension factor of fundamental natural period reflects structure behavior in the condition of rigid foundation, also reflects the dynamic interaction between soil and structure. It is a compositive index close related with the horizontal seismic action reduction coefficient.

(3) The horizontal seismic action reduction coefficient of i-th story is related with the height h_i from the ground to this story and the total height H.

(4) The condition of site soil has remarkable influence to the horizontal seismic action reduction coefficient.

(5) Because of the randomicity of ground vibration, the influence factors of the horizontal seismic action reduction coefficient are numerous. It should be take the lower limit of the horizontal seismic action reduction coefficient as 1.645 times of the average of reduction coefficient.

Based on the response spectrum theory, considered the site condition and the distributed characteristic of the horizontal seismic action reduction coefficient related with the story, combined with the conclusion about reduction coefficient in home and abroad and the result of this paper, the semi-empirical formula of the horizontal seismic action reduction coefficient of SSDI for high building is putted forward:

$$\eta_i = \left(T_1 / T_1^{'}\right)^{0.9(1-\mu)a(1-\lambda_i)} \qquad \text{(Eq.14)}$$

where: $T'_1$is the fundamental natural period of vibration of structure considered SSDI, s, T_1 is the fundamental natural period of vibration of structure in the assumed condition of rigid foundation, s, λ_1= h_i/H is height coefficient of i-th story, h_i is the height from the ground to i-th story, H is the total height of the building, a is the coefficient related with type of foundation and type of structure, to seismic structural wall structure, a=3/2, ($\eta_{groundfloor,\max,\mu=0,\lambda i=0} = 0.701$), to frame-seismic structural wall structure, a=1, ($\eta_{groundfloor,\max,\mu=0,\lambda i=0} = 0.789$), to frame structure, a=1/2, ($\eta_{groundfloor,\max,\mu=0,\lambda i=0} = 0.888$), b is the story distributed characteristic of reduction coefficient, usually b=1/2, $\mu$is site index, calculated as the following:

$$\mu = \gamma_G \mu_G + \gamma_d \mu_d \qquad \text{(Eq.15)}$$

ADVICE TO DESIGN

Generally don't consider the influence of SSDI on the work of structure seismic checking computation. But when the high building foundation is box foundation, stiff raft foundation, piles-box foundation or piles-raft foundation, and the natural period of vibration of structure T_1 is ranged from 6d/ V_{sm} to 1.2 T_g, and horizontal seismic action analysis as the rigid foundation assumed, the reduction coefficient can calculate as the following:

$$F'_i = \eta_i \bullet F_i \qquad \text{(Eq.16)}$$

in which η_i is the horizontal seismic action reduction coefficient considered SSDI, calculated as equation (14), F_i is the horizontal seismic action not considered SSDI, F'_i is the horizontal seismic action considered SSDI.

CONCLUSIONS

We can get some conclusions from this paper:

(1) The curve of horizontal seismic action reduction coefficient considered SSDI of the high building related the story is parabola, can't regard as a constant.

(2) The fundamental natural period of vibration of SSDI system is related with the site fundamental period of vibration T_g, the fundamental natural period of vibration of structure in the assumed condition of rigid foundation T_1 and the fundamental natural period of vibration of the foundation rotational vibration T_R. It can simplified calculate as the equation (11).

(3) Based on the response spectrum theory, combined with the statistical analysis of 6000 examples and the conclusion about reduction coefficient in home and abroad, the horizontal seismic action reduction coefficient considered of SSDI can be estimate from the equation (14).

(4) The high building's horizontal seismic action considering the SSDI effect can reduce as the equation (16) which gain in the condition of rigid foundation.

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