NONLINEAR SOIL-STRUCTURE INTERACTION THEORY FOR LOW-RISE REINFORCED CONCRETE BUILDINGS BASED ON THE FULL-SCALE SHAKE TABLE TEST AT E-DEFENSE

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ABSTRACT:

In this paper, nonlinear response analysis and soil-structure interaction theory are presented based on the simulation of the full-scale shake table test to quantify the input loss with the friction at the base. A simple bilinear model was adapted for the hysteresis model of the sliding base foundation. The general reduction in the responses of reinforced concrete buildings is investigated in case of a constant coefficient of friction at the base. The possessing energy of the building structure is conserved while the base slip has been occurred, because the ground acceleration does not act on the structural system. The energy can be evaluated from the velocity and base shear at the start of the base slip. While the base slip has been occurred, the ground acceleration does not act on the structural model because the sway spring doesn't resist to the transferred shear force at all, and free vibration has been generated between the superstructure and base foundation. A theoretical formula of upper-bound base shear value is derived from the maximum acceleration value, and friction coefficient, which may be used to determine the required lateral load-carrying capacity of the buildings to prevent damages under severe ground motion considering the effect of interaction.

KEYWORDS: soil-structural interaction, full-scale shaking test, theoretical upper-bound of base shear

1. BRIEF SUMMARY OF THE FULL-SCALE SHAKING TABLE TEST 2006

The shaking test of full-scale three-story reinforced concrete building structures were conducted with the world largest earthquake simulator 'E-Defense' from September to November 2006, as part of a five-year national project on seismic safety of urban areas, referred to as DaiDaiToku project in Japan. The plan and elevation of the test specimen are shown in Figure.1. The test specimen was constructed simulating typical plan and element of low-rise school buildings structures, designed following 1970' Building code of Japan. The failure mode of the specimen under an extreme motion is expected to be shear and axial collapse of short columns on the 1st floor in the longitudinal direction. The structure has three spans in the longitudinal (Y) direction, two spans in the orthogonal (X) directions. The span lengths are 4m in Y-direction, 2m and 6m in X-direction. The building structure has three stories, and total mass of the structures is up to 360 ton including steel weight on the roof. The RC structure has independent footing foundation with surrounding soil, and is not fixed to the shaking table directly. Those footings are attached to the concrete base plate with joint surface, so that the slip behavior would be simulated, when the foundation base shear exceeded the friction strength at the base. The width of footings is 1.0(m), and the height is 0.8(m). This would be almost minimum size for actual RC structure as an independent footing constructed on a sufficiently hard ground surface.

The base foundation of the structure didn't dislocate under JMA Kobe 50%, which was almost as the same level as the design spectrum earthquake motion in Japanese code. After that, the structure still remained minor damage under the original level of JMA KOBE (100%), due to the base slip behavior, although the ground motion was very severe to the strength of the RC structure. The foundation base shear, which was equivalent to the sum of the friction resistance and surrounding soil pressure, was evaluated from the accelerations and masses in each story. The hysteretic relations between the slip deformation and the foundation base shear represented an irregular bilinear shape with varying coefficient. The coefficient μ, the ratio of the foundation base shear to the normal force, was from 0.4 to 0.5 on the average in terms of the base slip duration at the shaking test. This is obviously lower than the value (from 0.7 to 0.8) at the start of base slip in the shaking test and the static loading test. It is verified experimentally that the maximum response and damage of the superstructure apparently reduced owing to the dislocation of the base structure under the severe earthquake motion with high acceleration. The objective of this study is to simulate the behavior with a simple model and to estimate the maximum response of the superstructure generally based on inelastic and dynamic theory.

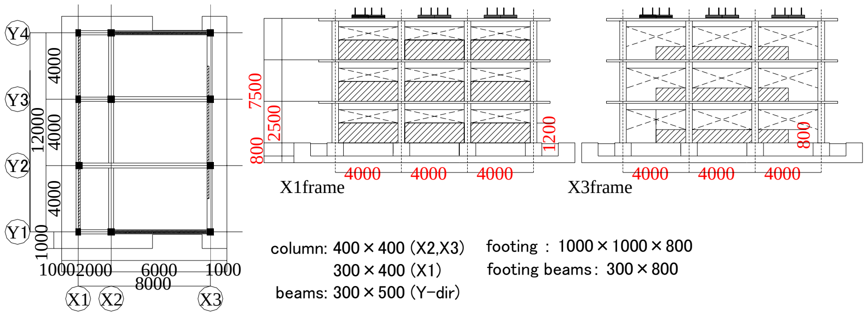


Figure 1 The specimen of the full-scale shaking test at E-Defense

3. THEORETICAL APPROACH OF BASE-SLIP SWAY MODEL

3.1. Sway-spring model

In order to generalize the upper-bound response of a building with base dislocation or slip behavior, the superstructure is idealized with an equivalent linear system, and installed a sway spring at the base foundation in the analysis model as shown in Figure 2. The linear stiffness and equivalent viscous damping coefficient were assumed with pre-yielding behavior of the superstructure. The bilinear model is used for the hysteretic relations of the sway spring. The yielding strength of the sway spring is same in the positive and the negative direction, and the coefficient μ is varied as analytical parameter in terms of the yielding shear divided by the total mass of the system. The elastic stiffness was assumed as higher enough than that of the super structure, which is derived from bending moment resistance of footing beams of the shaking test specimen (K_t =2686000 (kN/m)), while the stiffness after yielding was factored by α_y (α_y=10^{-8}) to be negligibly small value. The damping coefficient is 0.03 of critical, which is proportional to the tangent stiffness matrix. The unbalance force for damping and stiffness matrix is released at next step. The time step interval for numerical integration is 0.0025(s).

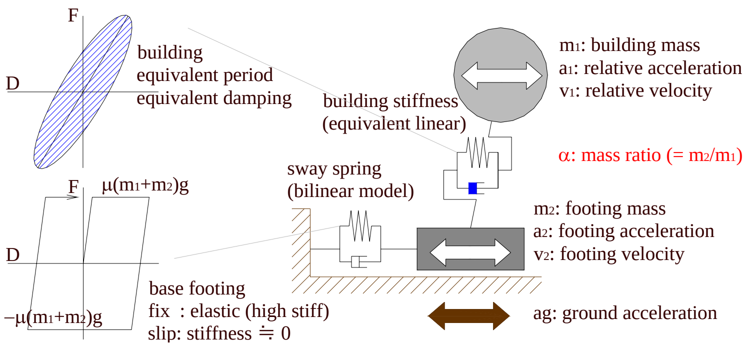


Figure 2 The sway spring model

3.2. Concept for proposed estimation

While the base slip has occurred, the ground acceleration does not act on the structural model because the sway spring doesn't resist to the transferred shear force at all, and free vibration has been generated between the superstructure and the base foundation. Therefore, the total energy of the building structure has been conserved from the start of the base slip behavior. The energy can be evaluated from the velocity and the base shear of the building structure at the time of slip start. In the following investigation, the possible total energy, as the sum of the kinetic energy and the strain energy in the spring, is estimated from the combination of the both responses of the structure and the base foundation in all the cases.

3.3. Combination of two responses

The base shear force at the start of base slip doesn't exceed an upper-bound value, due to the maximum input acceleration, because the foundation base shear is limited by the friction coefficient. The increment of the relative velocity of the superstructure at that time can be derived from an integration of relative acceleration during the time at which the base is being fixed. When the relative acceleration changes periodically, the velocity depends on the sum of two relative accelerations: one at the opposite structural response peak and the other at the start of the base slip behavior.

The total energy of the structure increases when the acceleration ag and the velocity v0 of the ground are in the same direction. When the base shear in the first story exceeds the foundation strength, the ground acceleration act in the opposite direction to the structural response, and the total energy will decrease after the first story base shear reaches the base friction strength. The foundation base shear should be the largest (friction strength) then in order to record the maximum integration value of the relative acceleration. If the input acceleration and velocity are supposed to turn over at a time accidentally, the energy will be increasing. In that case, the base slip behavior doesn't occur at that cycle because the foundation base shear decreases. Therefore, the first story base shear is smaller than the base friction strength in the direction of response increasing, where the total energy of the structure is the value at the start of the base slip. The same approach can apply to the opposite peak response of the structure. If the foundation base shear was equal to the base slip strength, the relative acceleration of the structure would reach the largest value, when the structural response attain the opposite response peak. When the structure reaches the opposite response peak before termination of the base slip, the relative velocity of the structure is derived by integrating the acceleration of the structure relative to the base, not to the ground. Because relative accelerations; (a_1- a_2) and (a_2-ag) respond in opposite phase for free vibration and the relative acceleration of the base foundation (a_2-ag) has to be higher value in order to terminate the base slip, the integrated velocity during the base slip duration is generally very small, compared to the velocity by the integration of the relative acceleration during the periodic behavior with the fixed base. Therefore, the response peak of the structure in the opposite direction is supposed to coincide with the termination of the base-slip, which is assumed in the following analysis. The combinations of the response conditions of the structure and the base foundation is limited to the three cases as shown in Figure 3 to attain the maximum total energy of the structure at the start of the base slip.

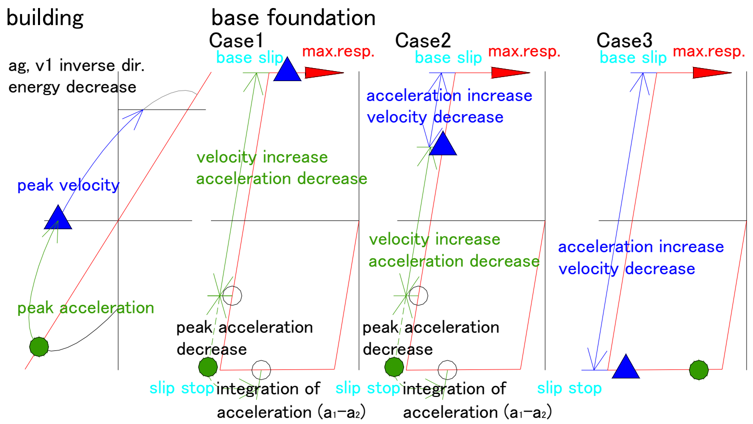


Figure 3 the order relation between responses of building and foundation

3.4. Derivation for v_0 (velocity of the structure when base slip start)

The relative accelerations of the structure at the termination and the restart of the base slip are derived from the ground accelerations ag_1, ag_2 at these times respectively (Eqn. 3.1, 3.2). The interval for the base-fixed response of the structure is defined as $(T_b/4)\ \beta\gamma$ as shown in Eqn. 3.3, where T_b is the period of the superstructure, β is the reduction factor of the friction coefficient μ, expressed as Eqn. 3.4, and γ is the ratio of the time duration to the point 0 which is expressed using two ground accelerations ag_1, ag_2 as Eqn. 3.5. The value v0 is the structural velocity at the start of the base slip can be derived as Eqn. 3.6. The relative acceleration at the start of the base slip is higher than $-\mu(1+\alpha)g$, because the base shear does not exceed the friction strength, where α is the mass ratio ($=m_2/m_1$), and g is the gravity acceleration. When the relative acceleration crosses over 0, the identical v0 exists in the other transition model, where the base slip starts before the peak velocity of the structure, because the acceleration on both sides cancels each other, as shown Figure 5. The base shear at the start of the base slip is slightly different in those two cases, but this difference of the strain energy would be negligible compared to the kinetic energy, when the ground acceleration level is relatively high for the friction strength coefficient μ, because the potential energy is limited by the friction strength.

$$\textit{When Base slip stop}\quad a_{11} = -\mu(1+\alpha)g - ag_1(1+\alpha) \tag{3.1}$$

$$\textit{When Base slip start}\quad a_{12} = \mu(1+\alpha)g + C_b v/m_1 - ag_2(1+\alpha) \tag{3.2}$$

$$\textit{Base fix duration}\quad T_S = (T_b/4)\beta\gamma \quad (T_b/4: \textit{building } 1/4\,\textit{cycle period}) \tag{3.3}$$

$(\textit{reponse peak} \sim \textit{base slip}) / (\textit{bulding } 1/4\ \textit{cycle period})\ \beta$

$$\beta = \frac{a_{11}-a_{12}}{a_{11}} = \frac{-2\mu(1+\alpha)g - C_b v/m_1 + (1+\alpha)(ag_1-ag_2)}{-\mu(1+\alpha)g - ag_1(1+\alpha)} \tag{3.4}$$

$(\textit{shear for acceleration}) / (\textit{foundation base shear})\ \gamma\ (ag_1 > ag_2)$

$$\gamma = \frac{2\mu(m_1+m_2)g}{2\mu(m_1+m_2)g - C_b v - (m_1+m_2)(ag_1-ag_2)} \tag{3.5}$$

$\textit{velocity when base slip start}\ \ v0$

$$v0 = (a_{11}+a_{12})\times\frac{T_S}{2} = \{(ag_1+ag_2)(1+\alpha) - C_b v0/m_1\}(T_b/4)\frac{\mu g}{ag_1+\mu g} \tag{3.6}$$

$$(1+\frac{2hK_b}{\omega m_1}\frac{\mu g}{ag_1+\mu g}(T_b/4))v0 = (1+h\pi\frac{\mu g}{ag_1+\mu g})v0 = (ag_1+ag_2)(1+\alpha)\frac{\mu g}{ag_1+\mu g}(T_b/4) \tag{3.7}$$

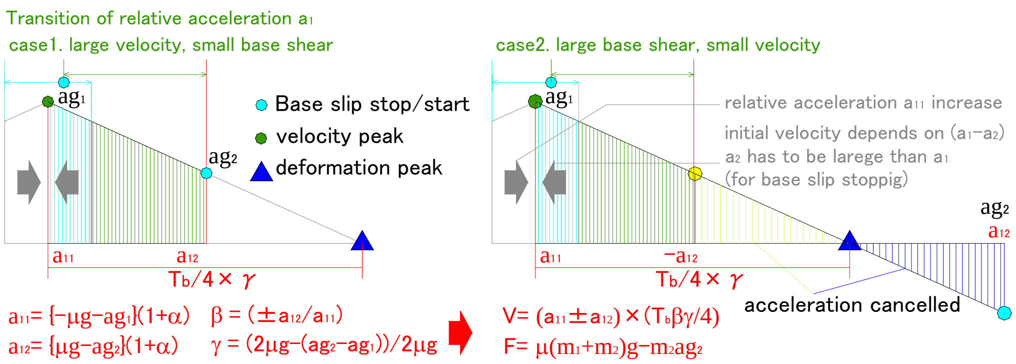


Figure 4 the transition relative acceleration (without damping)

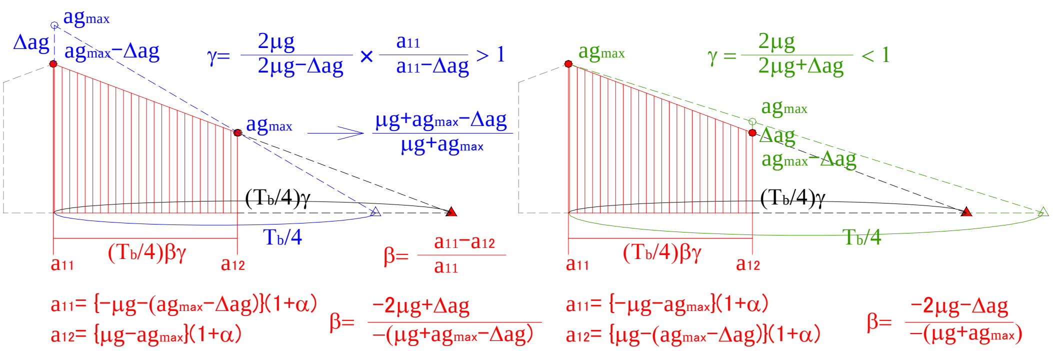


Figure 5 Change of base-fixed time for ground acceleration difference

The factor β indicates the ratio of the friction strength to the ground acceleration during the time of the base being fixed. The factor γ indicates the difference in contribution of the ground acceleration between two points. Those factors can be derived from the linearly change of the relative acceleration in Eqn. 3.4, 3.5. The difference of the ground accelerations does not influence the product βγ as shown in Figure 5. Substituting βγ into Eqn. 3.6, the velocity v0 can be evaluated as Eqn. 3.7.

3.5. Derivation of the upper-bound base shear coefficient C_{limit}

The upper-bound base shear coefficient (C_{limit}) is estimated from the possessing energy, as the sum of kinetic energy and the strain energy at the start of the base slip. The potential energy can be derived from the ground acceleration at that time ag_2. While the increment of the base shear during the base slip for the kineic energy represents ΔF, an equation of the conservation of energy without damping is given as Eqn. 3.8. Evaluated in terms of shear coefficient, C_{limit} is described as SRSS (square root of square sum) for Ck and Cv as Eqn. 3.9. A quarter cycle damping energy dissipation reduces Cv in the time history analysis as shown in Eqn. 3.10. Substituting with v0 given by Eqn. 3.7, Cv and Ck can expressed as a function of two input accelerations ag_1, ag_2, the mass ratio α and the friction strength coefficient μ in Eqn. 3.11, 3.12.

$$F0: \ initial\ base\ shear\ whe\ base\ slip\ start$$

$$\Delta F: \ increment\ of\ shear\ force\ during\ base\ slip$$

$$0.5 m_1 v0^2 = 0.5(F0+\Delta F)^2/Kb - 0.5F0^2/Kb \tag{3.8}$$

$$(F0+\Delta F) = \sqrt{m_1 Kbv0^2 + F0^2}$$

$$C_{limit} = \sqrt{\left(\frac{v0\omega_b}{g}\right)^2 + C_K{}^2}$$

$$C_{limit} = \sqrt{C_V{}^2 + C_K{}^2} \tag{3.9}$$

$$With\ 1/4\ cycle\ damping\ energy\ dissipation$$

$$1/2 m_1 v0^2 \cong 1/2 KbX_{max}{}^2 + (\pi/4)CbX_{max}v_0$$

$$X_{max} = \left(\sqrt{1+\left(\frac{\pi}{2}\right)^2 h^2} - \left(\frac{\pi}{2}\right)h\right)\frac{v0}{\omega_b}$$

$$\max.base\ shear\ coefficient\ C_{max}\ during\ base\ slip\ behaviour$$

$$C_V = KbX_{max}/m_1 g = X_{max}\omega^2{}_b/g$$

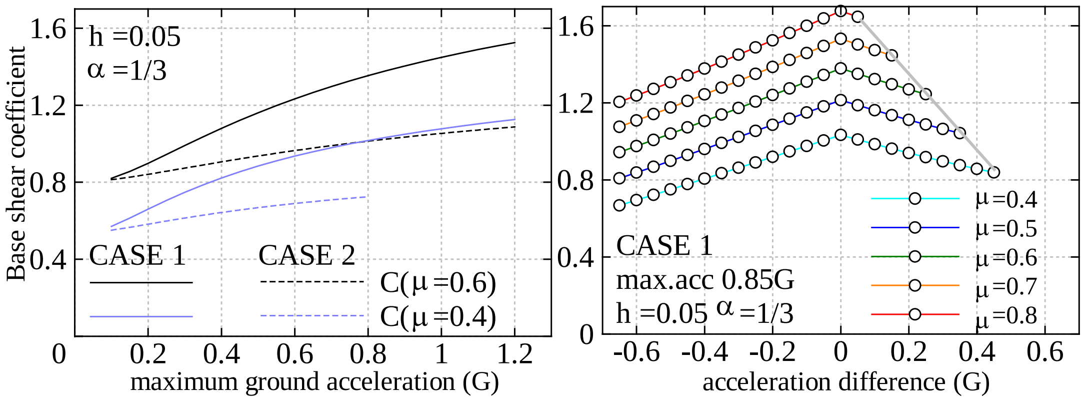
$$C_V = \left(\sqrt{1+\left(\frac{\pi}{2}\right)^2 h^2} - \left(\frac{\pi}{2}\right)h\right)\frac{v0\omega_b}{g} \tag{3.10}$$

$$C_V = \pi\mu\left(\sqrt{1+\left(\frac{\pi}{2}\right)^2 h^2} - \left(\frac{\pi}{2}\right)h\right)(1+\alpha)\frac{(ag_1+ag_2)}{2(ag_1+\mu g)} \bigg/ \left(1+h\pi\frac{\mu g}{ag_1+\mu g}\right) \tag{3.11}$$

$$C_K = \mu(1+\alpha) - \alpha\frac{ag_2}{g} \tag{3.12}$$

An example of the relations between C_{limit} and the ground acceleration is shown in Figure 6(a), where two accelerations ag_1, ag_2 are supposed to be the maximum acceleration value ag_{max} of the input earthquake record. The friction strength coefficients μ are 0.4 and 0.6, the mass ratio α is 1/3, and the damping coefficient is 0.05 in this example. The upper bound of the base shear coefficient C_{limit} becomes higher with the increase of the maximum ground acceleration and the friction strength.

The relations between C_{limit} and the difference of the two acceleration Δag are shown in Figure 6(b). The higher ground acceleration (ag_1 or ag_2) is assumed as the maximum value, while the other acceleration is assumed smaller in the figure. The velocity v0 is proportional to a sum of the two ground accelerations in Eqn. 3.7, and βγ depends only on the higher value of these accelerations, so that C_{limit} decreases when the absolute value of Δag as well as the kinetic energy increases. From the studies above, C_{limit} may be defined with the maximum ground acceleration ag_{max} as given by Eqn. 3.13~3.15.



(a) C_{limit} vs. maximum acceleration. (b) C_{limit} vs. Δacc.

Figure 6 Change of base-fixed time for ground acceleration difference

$$C_k = (\mu(1+\alpha) - \alpha \frac{ag_{\max}}{g}) \tag{3.13}$$

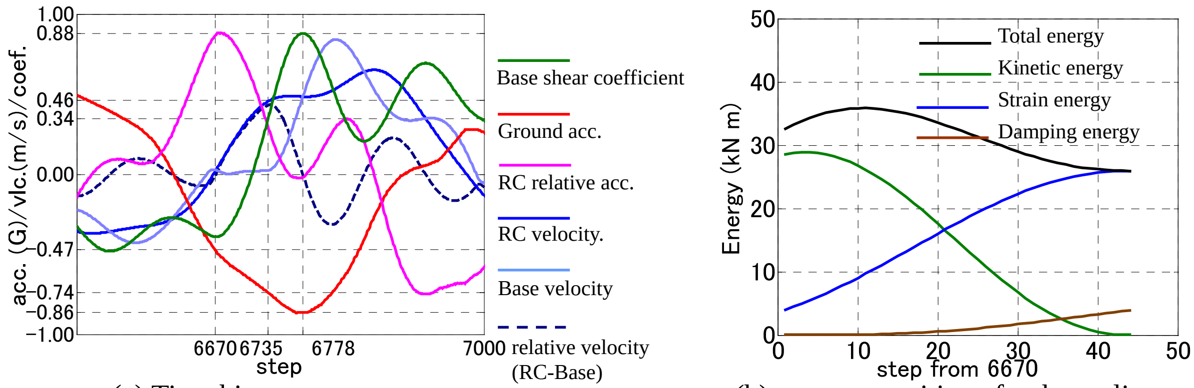
$$C_v = \pi\mu(\sqrt{1+\left(\frac{\pi}{2}\right)^2 h^2} - \left(\frac{\pi}{2}\right)h)(1+\alpha)\frac{ag_{\max}}{(ag_{\max}+\mu g)} / \left(1 + h\pi \frac{\mu g}{(ag_{\max}+\mu g)}\right) \tag{3.14}$$

$$C_{limit} = \sqrt{C_v^{\,2} + C_k^{\,2}} \tag{3.15}$$

4. VERIFICATION OF THE APPROACH BY NONLINEAR DYNAMIC ANALYSIS

4.1. Time-history response for analytical examples

An example time history response analysis with the sway spring model is shown in Figure 7 under the ground acceleration record of JMA Kobe(NS). The equivalent period of the structure is 0.3(s) with the mass ratio α of 1/3, the friction strength μ of 0.4, and the damping coefficient of 0.05 in the analysis model, which is similar case with the full-scale shaking test of the three-story RC structure. The base foundation has been slipping when the base shear of the structure attained the maximum value, and the ground acceleration is almost peak value during the time of the previous interval at the base fixed. The kinetic energy is relatively high compared to the strain energy at the start of the base slip. The maximum dynamic base shear coefficient is 0.88, which is about 85% of C_{limit}.



(a) Time history response (b) energy transition after base slip

Figure 7 the time history response of sway spring model

4.2. Nonlinear Response Spectrum

The maximum responses of the structure are evaluated by the time-history analyses with the sway model in various analytical cases and compared with the theoretical upper-bound C_{limit}. The analytical parameters were varied, such as the friction coefficient μ, the mass ratio α, and the ground acceleration records. The maximum base shear responses of the structure are shown in Figure 8 in cases with different friction strength such as μ=0.4, 0.6, and 0.8, calculated under the BCJ-L4, which is twice as high as the acceleration Level 2, the design level of the very rare earthquake motion. The mass ratio α is 1/3, and damping coefficient is 0.05 and the elastic fundamental period of the structure is varied. The linear response spectrum, which corresponds to the case with the fixed base, is also shown in the figure. The maximum responses under the extreme ground motion records in recent earthquakes are compared in case of μ =0.4 as shown in Figure 9. The maximum base shear responses reduce by the base slip behavior in all the analytical examples, and the maximum value is almost independent to the frequency of the structure so that the reduction is prominent around the specific frequency range, where the linear response shows high value in case with the fixed base. The nonlinear response shows almost constant value a little less than the theoretical value C_{limit}, especially when the friction strength coefficient μ is lower. On the other hand, the responses with slip become close to the linear responses, and C_{limit} overestimate the dynamic responses outside of the resonance frequency range, when the friction strength coefficient is higher. The maximum response coefficients are around 1.0 under any earthquake record in Figure 9, while the theoretical upper bounds are also almost identical among those earthquakes. The maximum responses and the theoretical C_{limit} are compared in all the analytical examples as shown in Figure 10. The dynamic responses are always smaller, and distributed in a range of 0 ~ -20% error from the theoretical upper bound C_{limit}. The stiffness of the regression line between those two values is almost 0.9.

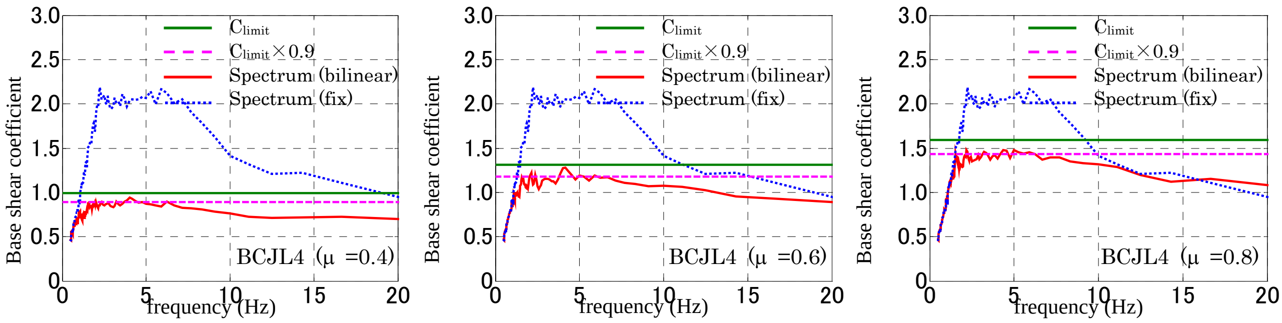


Figure 8 Comparisons between C_{limit} and nonlinear spectrums (friction strength coefficient)

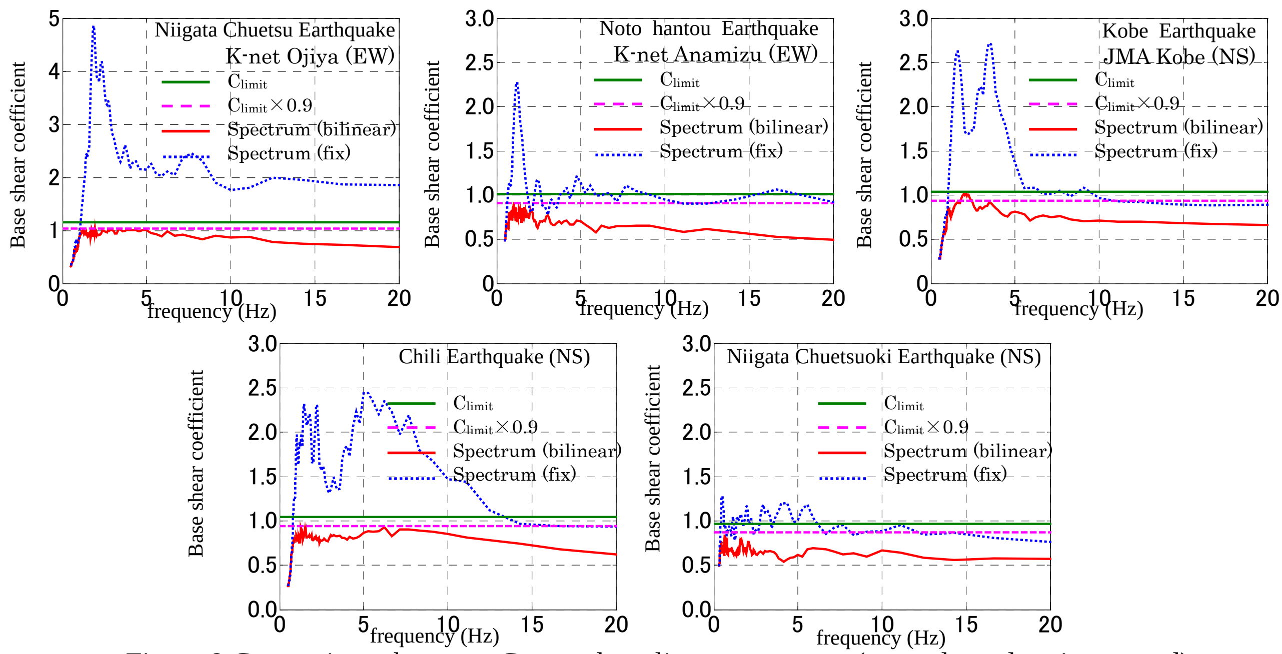


Figure 9 Comparisons between C_{limit} and nonlinear spectrums (ground acceleration record)

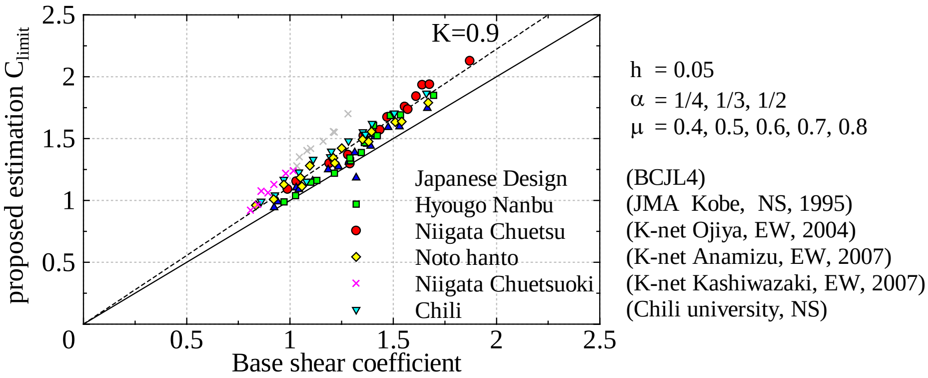


Figure 10 Comparisons between C_{limit} and time-history responses

5. CONCLUSION

The obvious reduction of damage to the superstructure was verified through the full-scale shake table test with the slip behavior at the base foundation. The behavior was simulated using an idealized two mass model with specific focus on the maximum responses of the superstructure. An equivalent linear model is used for the superstructure and the perfectly plastic model for the hysteretic relation of the base friction. Theoretical estimation of the upper bound response of the superstructure is presented considering the generalized response characteristics of the model. The kinetic energy and the strain energy of the superstructure are being conserved during the slip behavior and the upper bound energy or velocity can be derived from the ground acceleration levels. From possible theoretical verification on the acceleration conditions at the time when the base is being fixed, the possible upper bound relative velocity of the superstructure could be derived from the maximum ground acceleration, which is independent to the fundamental period of the superstructure. The numerical time history analyses on the maximum responses with the base slip behavior were carried out for various cases to verify the theoretical upper bounds. A good correlation is observed between the theory and the analyses with constant ratio. While ground compliance with input acceleration frequency has long been well quantified by conventional linear interaction model, the maximum response with friction could be estimated without frequency-dependent relations. If the surrounding soil shows nonlinear behavior or the base surface shows the slip behavior under an extreme ground motion, lateral yielding strength at the base foundation would be the most important factor to the response of the superstructure.

ACKNOWLEDGMENT

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