DYNAMIC RESPONSE CHARACTERISTICS OF BUILDINGS SUBJECTED TO PULSE-LIKE GROUND MOTIONS

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ABSTRACT

We have investigated the fundamental but very important response characteristics of buildings against the pulse-like ground motions which are predicted in the near-fault region of inland shallow earthquake. Especially, the soil structure interaction effects (SSI effects), contribution of each vibration mode, propagation of pulse-like wave in a building, and effects of viscous and hysteresis dampers are discussed in detail. In conclusion, the natural periods normalized by the pulse period $T_p$ are very important characteristic values to evaluate the maximum response and building damage as well as the peak ground velocity or peak ground displacement.

INTRODUCTION

It is expected that destructive pulse-like ground motions are predicted to occur in the near-fault region of massive inland shallow earthquake and many tall buildings may suffer severe damage including collapse. However, uncertainty in estimating ground motion characteristics cannot be avoided. For example, Figure 1 shows the empirically obtained average relationships between the moment magnitude $M_w$ and predominant period $T_p$ observed in the near-fault regions. We call $T_p$ as pulse period in this paper. In addition to the uncertainty of the moment magnitude of a predicted earthquake, uncertainty also exists in estimating the peak ground velocity $V_p$ and pulse period $T_p$. As we discussed in this paper, these characteristic values of near-fault ground motions influence in the response of buildings strongly.

In this paper, we have clarified the response characteristics of buildings against the pulse-like ground motions. Sinusoidal pulse and Ricker wavelet are used for acceleration input wave to idealize the pulse-like ground motions. We have employed the peak velocity $V_p$ and pulse period $T_p$ as the characteristic values of pulse-like ground motions and used as analytical variables.

Two kinds of analysis models are used to evaluate the response characteristics of buildings. One analysis model is a sway-rocking model (SR model in Fig. 2) of buildings with spread foundation on the soil surface in order to consider the dynamic soil-structure interaction (SSI) effects. We investigate the variation in the response of soil and buildings when the predominant period of pulse-like ground motions $T_p$, the total height of building $H$ and the average shear wave velocity of soil are parametrically changed.

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Another analysis model is an uniform transversely vibrating beam with a fixed and free ends with fundamental natural period $T_1$. First, the modal analysis is conducted analytically to investigate the contribution of vibration modes to the maximum drift angle $R_{\text{max}}$. The effects of the type of damper such as hysteretic damper or viscous damper on $R_{\text{max}}$ are also studied. Next, linear and nonlinear response analyses are carried out to investigate the height $H_m$ where the maximum drift angle has produced from the viewpoint of pulse like deformation propagation through the building and its damage concentration.

**SOIL-STRUCTURE INTERACTION EFFECTS**

Building height is $H=3.5N$ if the number of stories is $N$ and story height is 3.5m. The foundation is a square, $L=30$m on a side, and is built on the semi-infinite soil with shear wave velocity $V_s$. The building-soil interaction system is modeled by a Sway Rocking model (SR model), as shown in Fig. 2.

Upperstructure is treated as an equivalent SDOF system, and its natural period $T_0$ with damping factor $h_0$ of 0.03 is given by Eq.(1) if foundation is fixed.

$$T_0=\alpha N$$  \hspace{1cm} (1)

where proportional constant $\alpha=0.07$. Weight per unit area is set to 1t/m$^2$, and massless rigid foundation is assumed. If acceleration of gravity is $g$, the total mass of a building will be $m=NL^2/g$. If the fundamental mode shape of the upperstructure is assumed to be linear, the equivalent mass $m_v$ and the equivalent height $H_v$ are given by

$$m_v = \frac{1.5(N+1)}{2N+1}m, \quad H_v = \frac{2N+1}{3N}H.$$  \hspace{1cm} (2)
The impedance functions of horizontal and rotational spring at the foundation are estimated based on the central displacement under uniform loading at the surface of the soil as follows.

\[
\begin{align*}
    k_H(\omega) &= k_{H0} - \omega^2 m_{H0} + i\omega c_{H0} \\
    k_R(\omega) &= k_{R0} - \omega^2 m_{R0} + i\omega c_R(\omega) \\
    k_{H0} &= \frac{2\pi}{2(1-\nu)} G r_H^3, \quad m_{H0} = \frac{2\pi}{2(1-\nu)} G r_H^3, \quad c_{H0} = \frac{2\pi}{2(1-\nu)} \rho V_s r_H^2 \\
    k_{R0} &= \frac{\pi}{2(1-\nu)} G r_R^3, \quad m_{R0} = \frac{\pi}{2(1-\nu)} G r_R^3, \quad c_R(\omega) = \frac{\pi}{2(1-\nu)} \frac{\Gamma_1}{12} \rho V_s r_R^6 \omega^2,
\end{align*}
\]

where unit volume weight \( \gamma = \rho g = 1.9 \text{ t/m}^3 \), \( G = \rho V_s^2 \) and \( (\nu, \Gamma_1, \Gamma_2) = (1/3, 1.212, 0.912) \). Moreover, \( r_H \) and \( r_R \) are the radius of a circle with equivalent cross-section area and second moment of area, respectively, to a square of one side \( L \).

The \( u_G \), \( u_{GR} \), \( u_{GR} \), and \( u_B \) denotes the ground displacement, relative horizontal displacement and rotational displacement of the foundation, and the deformation of the upperstructure, respectively. Displacement amplification ratio \( |u_B/u_G| \) for \( V_s = 100 \text{ m/s} \) and fixed foundation model (FIX) is shown in Fig. 3. In this figure, the natural frequency of the fixed foundation model is \( f_0 (=1/T_0) \). By considering the SSI effects, equivalent natural period \( T_{e0} \) increases and is approximated by the following equation.

\[
T_{e0} = T_0 \sqrt{1 + \frac{k_s}{k_{H0}} + \frac{k_s H^2}{k_{R0}}}
\]

Figure 3. Transfer functions (\( V_s = 100 \text{ m/s} \))

Input ground motions

Sinusoidal pulses (Eq.(5)) or Ricker wavelets (Eq.(6)) is used for pulse type input ground motions as shown in Fig. 4.

\[
\ddot{u}_p(t) = A_p \sin \omega_p t,
\]

where \( \omega_p = 2\pi / T_p \) and \( A_p = \pi V_p / T_p \).

\[
\ddot{u}_c(t) = A_p (\tau^2 - 1) \exp(-\tau^2 / 2),
\]

where \( \tau = \sqrt{2\pi(t-T_p) / T_p} \), \( A_p = \sqrt{2\pi V_p / T_p} \).

The \( T_p \) is pulse period, \( A_p \) and \( V_p \) are the peak acceleration and velocity, respectively.
Linear response analysis

Frequency response analysis is conducted and the linear response characteristics of SSI system under pulse type ground motions of $V_p=100\text{cm/s}$ are investigated. The influence of SSI effects to the maximum response displacement $D_{\text{max}}$ normalized by the peak ground motion $D_p$ with pulse period $T_p=1\text{s}$ is shown in Fig. 5. The difference of response between SR model and FIX model increases as $V_s$ decreases. This tendency is evident in the low-rise building of $N=5-10$.

Then, Figure 6 shows the variation of $D_{\text{max}}/D_p$ with pulse period $T_p$. The difference in the maximum response of SR model and FIX model varies with pulse period $T_p$ and the number of stories $N$. It is thought that the difference may correspond to that in the fundamental natural period. Therefore, relationship between $T/\varphi$ and $D_{\text{max}}/D_p$ is shown in Fig. 7. Pulse type ground motions of $T_p=1\text{s}$ or $3\text{s}$ are input to buildings on the soil of $V_s=400, 200, 100 \text{ m/s}$. Displacement response spectra for the damping ratio of 0.03 is also shown in the same figure. Results obtained using SR model agrees very well with normalized displacement response spectra.
Nonlinear response analysis

In this subsection, influence of SSI effects is studied if the nonlinearity of buildings is taken into account. Let the restoring force characteristics of upperstructure be a trilinear type as shown in Fig. 8.

In this study, the yield base shear coefficient $C_y$ is set to be in inverse proportion to the number of stories $N$.

$$C_y = \beta / N$$  \hspace{1cm} (7)

If $m_e$ and $H_e$ are the effective mass and effective height of the building, $R_y (=1/150)$ is the yield shear deformation angle, yield shear force $Q_y$ and yield deformation $D_y$ are expressed as follows.

$$Q_y = C_y m_e g$$ \hspace{1cm} (8)
\[ D_y = H_y R_y \]

Proportional constant \( \beta \) in Eq.(7) is related to \( \alpha = 0.07 \) in Eq.(1) as

\[ \alpha = \sqrt[2]{\frac{4\pi^2d}{9 \gamma g} \frac{2N+1}{N}} \approx 0.145 / \sqrt{\beta} \]

Therefore, the results for \( \beta = 3 \) (\( \alpha = 0.083 \)) are shown in the following.

Figure 9 shows the relationship between \( T_{eo}/T_p \) and \( D_{max}/D_p \) obtained from nonlinear response analyses for SR (SSI) model and fixed foundation (FIX) model. In this figure, \( T_{eo} \) is the equivalent natural period of linear SSI system calculated from Eq.(4). For \( T_{eo}/T_p < 1.5 \), normalized displacement response spectra underestimate the maximum response nonlinear response of buildings due to the underestimation of equivalent natural period. However, normalized displacement response spectra gives us a good estimation of the maximum displacement response for \( T_{eo}/T_p > 1.5 \) because the maximum displacement response can be determined by the peak ground displacement \( D_p \).

The maximum total response angles \( R_{max} = \frac{D_{max}}{H_v} \) and shear deformation angles of upperstructure \( R_{0max} = \frac{\text{max}(u_B)}{H_v} \) of linear SR model (SSI) and fixed foundation model (FIX) to sinusoidal pulse waves of \( T_p=1s \) are shown by thin lines in Fig. 10. The \( R_{max} = R_{0max} \) for the fixed foundation model.

It is pointed out that \( R_{max} \) increases by considering SSI effects because of the increase in equivalent natural period \( T_{eo} \). However the shear deformation of upper structure \( R_{0max} \) itself slightly is smaller than \( R_{max} \) (FIX) for low-rise buildings (see \( N < 10 \)). On the other hand, increase in \( R_{max} \) by the SSI effects is not so large for \( N > 15 \), but the reduction in \( R_{0max} \) becomes larger.

![Figure 9](image1.png) \( R_{max} \) and maximum displacement response

![Figure 10](image2.png) Maximum shear deformation angle for linear (L) and nonlinear (NL) soil-structure interaction analyses
The maximum response angle $R_{\text{max}}$ and maximum shear deformation angle of upperstructure $R_{0\text{max}}$ of nonlinear SSI systems to sinusoidal pulse waves is shown by bold lines in Fig. 10. It is pointed out that the SSI effects becomes smaller by considering the nonlinearity of buildings especially in $R_{\text{max}}$. In addition, $R_{\text{max}}$ decreases with the number of stories. This is because the maximum displacement response $D_{\text{max}}$ approaches by considering the nonlinearity of the upperstructure but equivalent height $H_{\text{eq}}$ increases with $N$.

When changing the peak ground velocity $V_p$ with 100 or 200cm/s and the pulse period $T_p$ with 1s, 2s, 3s, the $R_{\text{max}}$ and $R_{0\text{max}}$ are also shown in Fig. 11. As $V_p$ increases, the difference of $R_{\text{max}}$ for SR model and the fixed foundation model decreases. Namely, the SSI effects are not so large if $V_p$ increases and the nonlinearity of upperstructure progresses.

If $V_p$ is constant, the peak ground displacement $D_p$ and the maximum response angle $R_{\text{max}}$ for high-rise buildings increase with $T_p$. However, the maximum response angle $R_{\text{max}}$ or $R_{0\text{max}}$ drastically reduce if the number of stories $N$ is less than a critical value $N_c$. For example, $N_c=15$ is for $V_p=200\text{cm/s}$ and $T_p=3s$ as shown in Fig.11(c). Then, $N_c$ increases with $T_p$ and contrarily decreases with $V_p$. This characterizes the damage tendency of buildings near the epicentral area. Therefore, when changing pulse period $T_p$, the height of buildings suffering damage may change drastically.

![Figure 11. Maximum shear deformation angle considering nonlinear soil-structure interaction](image)

**MAXIMUM RESPONSE OF UNIFORM SHEAR-BEAM MODEL**

In this section, buildings of height $H$ fixed at the foundation is modeled by a uniform shear-beam model as shown in Fig. 12. We discuss the maximum response of the uniform shear-beam model subjected to Ricker wavelets.
Modal analysis of undamped system

First, the modal analysis is conducted analytically to investigate the contribution of vibration modes to the maximum drift angle $R_{\text{max}}$. In this subsection, we consider an undamped shear-beam model.

If the natural period $T_1$ of the fundamental mode is proportional to the height $H$ as

$$T_1 = \alpha_1 H$$  \hspace{1cm} (10)

the $s$-th natural period $T_s$ is expressed as

$$T_s = T_1 / (2s - 1).$$  \hspace{1cm} (11)

The natural mode $\phi_s(x)$ and its participation factor $\beta_s$ are given as follows.

$$\phi_s(x) = \sin((2s-1)x\pi / 2H)$$  \hspace{1cm} (12)

$$\beta_s = 4 / (2s - 1)\pi$$  \hspace{1cm} (13)

Here, $\psi_s(x)$ is called the drift angle mode in this paper and is obtained by normalizing the function obtained from the differentiation of the natural mode from height $x$.

$$\psi_s(x) = \cos((2s-1)x\pi / 2H)$$  \hspace{1cm} (14)

The maximum response of the $s$-th mode $R_{\text{max}}$ is given by using the displacement response spectra $S_d$ as

$$sR_{\text{max}}(x) = \beta_s \cdot (d\phi_s(x) / dx) \cdot S_d(\tau)$$  \hspace{1cm} (15)

where $\tau$ is the normalized $s$-th mode natural period $\tau = T_s / T_p$. Above equation can be written as

$$sR_{\text{max}}(x) = sR_{\text{eff}} \cdot \psi_s(x)$$  \hspace{1cm} (16)

The $R_{\text{eff}}$ is the contribution of $s$-th mode to the maximum drift angle. If considering the wave propagation in infinite shear-beam, the maximum deformation angle $R_{\text{0max}}$ can be easily obtained as follows.

$$R_{\text{0max}} = \alpha_1 V_p / 4$$  \hspace{1cm} (17)

Then, using this value, $R_{\text{eff}}$ is normalized as

$$sR_{\text{eff}} / R_{\text{0max}} = 4\sqrt{2\pi} S_d(s, \tau) / \pi \cdot \tau \cdot D_p.$$  \hspace{1cm} (18)

It is pointed out that the contribution of the first mode is dominant in the maximum drift angle if $\tau = T_1 / T_p < 2$. In other words, contribution of higher mode cannot be negligible if $T_1 / T_p > 2$. The $sR_{\text{eff}} / R_{\text{0max}}$ is shown in Fig. 13.
Wave propagation in undamped shear-beam

Next, wave propagation behavior in the undamped shear-beam is demonstrated. Shear-beam is modeled by using an equivalent lumped mass-spring system. Namely, mass and stiffness are constant. Assuming a building of 40 stories, the fundamental natural period $T_1$ is set to be 4.8s. In order to compare the contribution of higher mode, $T_1/T_p$ is set to be 1.0 or 4.0 as shown in Fig. 14. By showing the waveform of shear drift angle at all floors, wave propagation behavior is investigated. If $T_1/T_p=1.0$, the building is shaking in the fundamental mode. However, if $T_1/T_p=4.0$, we cannot distinguish each mode but the propagation of a wavelet. Then, the maximum drift angle $\max R_{\max}$ occur due to the interference of an upward wave from the bottom and a downward wave reflected at the top. Therefore, the maximum drift angle occur at a middle to upper floor if $T_1/T_p$ is large.

The $H_m$ and $t_m$ are, respectively, the height and time of the maximum drift angle $\max R_{\max}$ has produced. Figure 15 shows $\max R_{\max}$, $H_m$, and $t_m$ with respect to the non-dimensional fundamental natural period $\tau = T_1/T_p$. For $\tau > 2$, $\max R_{\max}$, $H_m$, and $t_m$ can be estimated as follows, respectively.

\[
\max R_{\max} / R_{\max} = 2 \quad (19)
\]

\[
H_m / H = 1 - 1 / \tau \quad (20)
\]
\[
\frac{t_m}{T_p} = \frac{\tau}{4} + 1
\]  
(21)

By using these equations, we can roughly estimate the maximum drift angle, height and time of occurrence.

Figure 15. Value, occurrence position, and occurrence time of maximum response

Maximum response and response reduction effects by dampers

Effects of the type of damper such as hysteretic dampers or viscous dampers on reduction in \( R_{\text{max}} \) are also studied. In addition, nonlinear time-domain analyses are carried out to investigate the height \( H_m \) where the maximum drift angle has produced from the view point of pulse like deformation propagating through the building.

Figure 16. Restoring force characteristics of main structure (left) and hysteretic dampers (right)

In this subsection, nonlinear response analyses are carried out. Lumped mass system are used for building model. Then, mass, stiffness, and amount of dampers are uniform in the vertical direction. Ratio of mass and stiffness are determined to match the fundamental natural period \( T_1 \). Yield drift angle of the main structure \( R_y \) is set to be 1/150. Viscous dampers are installed so that the damping factor for fundamental natural period is 0.04. As for hysteretic dampers, on the other hand, the restoring force characteristics of dampers are bilinear type as shown in Fig. 16. The yield shear force of dampers are 0.2 times as large as that of main structure \( Q_y \), while yield drift angle of the dampers are 0.1 \( R_y \). Ricker wavelet whose \( V_p = 100\text{cm/s} \) or \( 200\text{cm/s} \) is used for input ground motion.

The maximum drift angle are shown in Figs. 17 and 18. Results for the case of no damper and with dampers are shown in these figures.

First, height \( H_m \) of the maximum response occurrence is discussed for the case of no damper (see dashed lines in Fig. 17). For \( \frac{T_1}{T_p} = 1.2 \), \( H_m/H \) is 0, that is, the maximum drift angle occurred at the first floor regardless of \( V_p \). As for \( \frac{T_1}{T_p} = 4.0 \), however, \( H_m/H \) is nearly equal to 0.7 for \( V_p = 100\text{cm/s} \), but \( H_m/H \) is 0 for \( V_p = 200\text{cm/s} \). Therefore, the \( H_m \) for nonlinear analysis depends on \( V_p \) as well as \( T_1/T_p \). Especially, \( H_m \) suddenly shifts to the lower floor from the middle to upper floor as \( V_p \) increases if \( T_1/T_p >> 1 \). The reason for this phenomenon is as follows. The \( H_m \) for linear model is shown in Fig. 15(b). Therefore, for small \( T_1/T_p \), the maximum response always occur at the first floor. And for large \( T_1/T_p \), the maximum response occur upper intermediate story if the building is linear. However, deformation propagates from the bottom of the building just after the input of pulse-like ground
motions. Therefore, even if \( T_1/T_p \) is large enough, deformation concentrates gradually around the bottom of the building as \( V_p \) increases. Then, sudden shift of \( H_m \) to the lower floor from the middle to upper floor occurs as \( V_p \) increases.

Next, the reduction effects in the maximum response by dampers are discussed. The maximum response is reduced by viscous dampers regardless of \( V_p \) and reduction ratio of viscous damper becomes larger as \( \tau = T_1/T_p \) increases as shown in Fig. 17. However, the maximum response for \( V_p = 200 \text{cm/s} \) and \( \tau = 1.2 \) slightly increases by installing hysteretic dampers as shown in Fig. 18(a). It is considered to be the reason for this that the equivalent natural period decreases by installing the hysteretic damper and approaches the peak period of displacement response spectra shown in Fig. 7.

**CONCLUSIONS**

Based on our studies, we have pointed out very important response characteristics of buildings against the pulse-like ground motions as follows.

a. Results obtained using SR model agrees very well with normalized displacement response spectra. The linear soil-structure interaction effects are dependent on the ratio of equivalent natural period to pulse period \( T_{eq}/T_p \).

b. The SSI effects are not so large if \( V_p \) increases and the nonlinearity of upperstructure progresses. If \( V_p \) is constant, the maximum drift angle for high-rise buildings increase with \( T_p \). However, the maximum drift angle drastically reduce if the number of stories is less than a critical value. Therefore, when changing pulse period \( T_p \), the height of buildings suffering severe damage may change drastically.

c. The maximum drift angle of linear tall buildings can be evaluated from \( V_p \) and \( T_1/T_p \). Then, the occurrence time of the maximum response is also expressed by using \( T_1 \) and \( T_p \). Next, the contribution of each vibration mode to \( R_{max} \) as well as the height \( H_m \) can be formulated using \( T_1/T_p \).

d. The \( H_m \) for nonlinear analysis depends on \( V_p \) as well as \( T_1/T_p \). Especially, \( H_m \) makes the sudden shift to the lower floor from the middle to upper floor as \( V_p \) increases if \( T_1/T_p >> 1 \). This phenomenon can be understood using the theory of pulse propagation in an uniform transversely vibrating beam.

e. The maximum response is reduced by viscous dampers regardless of \( V_p \) and reduction ratio of viscous damper becomes larger as \( T_1/T_p \) increases. However, the maximum response for \( V_p = 200 \text{cm/s} \) and \( \tau = 1.2 \) slightly increases by installing hysteretic dampers. It is considered to be the reason for this that the equivalent natural period decreases by installing the hysteretic dampers.
and approaches the peak period of displacement response spectra. In conclusion, the natural periods normalized by the pulse period $T_p$ are very important characteristic values to evaluate the maximum response and building damage as well as the peak ground velocity $V_p$ or peak ground displacement $D_p$.

REFERENCES


