ON THE GENERATION OF THE DESIGN EARTHQUAKE GROUND MOTION TIME HISTORY

by

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ABSTRACT

In design, it is common that the design motion is provided in the form of response spectrum.
It is possible to calculate the maximum response with the design spectra using the response spectral method such as the SRSS method, if the building is assumed linear during the earthquake. However, for such types of severe ground motions, the building will exceed its elastic limit. The approximate method like SRSS is not sufficient for the reliable response values. In such case we need to generate the design motion time histories.
There is an issue there, what kind of time histories should we make use.
There may be two options.

(1) The past strong motion records
(2) The simulated ground motion considering the earthquake activity in the area.

It is known that in Japan, specific buildings such as the high rise buildings or base-isolated buildings that necessitate the verification of the realistic behavior during earthquake for the reason that there are still not sufficient design experiences for the types of buildings.
In this paper, the variations of simulated ground motions with respect to their properties in ground motion characteristics and structural responses.

KEY WORDS: Simulated motions
Phase angles, Spectrum-compatible motions, Envelope function, Duration time, Variations

1. INTRODUCTION

The conventionally used various standard design spectrum, is a result of statistical analysis of collected ground motion records for various conditions.
There are two types of design spectra. One is an envelope or integration of many expected ground motions that are possible to occur in certain period of time. The other is one that is constructed based on the earthquake magnitude, distance and./or soil condition that is also expected to occur.
From the viewpoint of time history generation, the latter assumption is suitable because the time history is primarily based on the image of the expected earthquake event. The duration time, for example, depends on the earthquake magnitude.
However, the former type is, in reality, frequently found as the standard spectrum in engineering practice.
The time history includes many factors related to variation representing the accidental occurrence or irreproducibility of earthquake event.
As a fundamental rule, several time histories are used for analyses. However, limited number of time histories is used in reality due to the limit in time and cost.
In recent revision of the notification in the Japan Building Standard Law, it is suggested that several statistically independent time histories should be used for the dynamic analysis.
Figure 1 shows the variation of story drift for seven simulated input motions that are compatible to the design spectrum. It is clearly seen that the variation of story drift among 7 generated motions are rather larger than expected.
In this paper, some notes and considerations of the

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variation in the simulated ground motion will be discussed.

2. TYPES OF TIME HISTORIES

There are three types of motions considered in the current design of specific buildings.
(1) Standard type includes the well-known El Centro/1940 NS component, Taft/1952 EW Component and Hachinohe harbor for Tokachi-oki earthquake/1968 EW component. These motions become a kind of standard in Japan.
The motion of this type has no variation and is mainly used to examine the overall validity of the design. Because those standard motions are used in most of the design practice. Therefore, the designer can check the level of seismic performance in reference to other design examples.
(2) Site-specific type includes the modified motions recorded at the site or nearby sites, and the simulated motions based on the event that has the largest influence to the building concerned.
The ground motion of this type can be evaluated as the motion due to the selected specific earthquake. The motion has variation based on the rupture process uncertainty of the causative fault that is specified as of the largest influence in the area.
(3) Design spectrum-compatible type includes two options. One is the motion that has clear definition of corresponding to an earthquake event. The other is not necessarily related with some specific event, since the spectrum is determined from the statistical analysis of variety of motions. Here, we need a strategy to fix the ground motion parameters to generate time histories. The motion of this type had been frequently used in the past. The technique for generating ground motion of the type is available. (Ref.1)
We will mainly focus on type (3) in this paper.

3. VARIATION IN DESIGN MOTIONS

The Primary factors causing the ground motion variation might be as follows;

a) Seismological factors
It is relevant to earthquake occurrence or fault rupture. In engineering – these factors are involved in phase scatter all together.
b) Variation of time history properties
It includes response spectra, peak amplitudes, energy spectra, etc.
c) Variation of the structural response
It involves non-linear response, responses of multi-story buildings

4. GENERATION OF SPECTRUM COMPATIBLE MOTIONS

4.1 Wave forms

The commonly used waveform for the time history of ground motion is as follows;

$$a(t) = E(t)\sum_i A_i \cos(\omega_i t + \phi_i) \qquad (1)$$

where, $a(t)$ is acceleration time history with uniform time interval, $E(t)$ is the envelope function to make wave non-stationary. A_i , ω_i , ϕ_i are amplitude, circular frequency and phase angle of the i-th component, respectively

The time history can be generated by the inverse Fourier transform. The response spectra of the generated motion are calculated. If the convergence to the target (design) spectra is not good, the amplitude of each component is modified in accordance with the corresponding spectral value. This process is repeated until a good agreement is attained. The final time history will be served as the design motion for the verification in dynamic analysis.
The time history generation scheme using uniformly random phase angles and envelope function does not always promise the convergence to the target response spectrum for a larger number of cyclic computation. The poor convergence is sometimes improved by changing the initial set of phase angles.
It is known that the usage of the phase angles of the recorded motion generates a time history similar

waveform appearance with the original recorded motion. In this case, the envelope function is not necessary. The expression (1) becomes as follows removing the envelope function.

$$a(t) = \sum_i A_i \cos(\omega_i t + \phi^*_i) \qquad (2)$$

where, ϕ_i^* is the phase angle of the recorded motion.

The target response spectrum is generally given for 5 % of critical damping. It is necessary for the generated motion to be compatible to only the 5% damping response spectrum. It is not generally required, however, that the generated motion is compatible to the spectral values with damping other than 5% of critical. This condition seems too loose, turns out to result in actually restricted by the fix of duration.

4.2 Duration time and envelope function

The duration time and envelope function are necessary in the simulation process of ground motion.
In some cases, the velocity and displacement time histories have drift in baselines. One should be careful for it in computing peak amplitudes. There are number of definitions of duration time.

The three plausible definitions of duration time T_d are ;

(1) The cumulative time in which the amplitude exceeds certain level of threshold.
(2) The elapse time from moment the amplitude exceeds a specified level first to the last time it falls below the level.
(3) The times corresponding to 5% and 95% of cumulative energy. The cumulative energy is the proportional to the integration of squared amplitude.

Physically, the duration time is related with the time required for the fault to complete the rupture. In addition, the delays and advances of seismic waves make the waveform longer, if there are many paths for the seismic waves.
An example of the equation for duration time is as follows.

$$T_d = 10^{0.31M - 0.774} \qquad (3)$$

Where, M is earthquake magnitude

The most essential value in design is the maximum response. In addition, strong motion records are digitized with time interval of 0.005 to 0.02 second. The spectrum-compatible motion can be defined with any of the time interval; it should be specified as smaller when one needs to evaluate the response with prevailing higher modes.

The property of the design ground motion specified in the notification No.1461 in the revised Japanese Building Standard Law should be as follows;
The simulated motion should be compatible in acceleration response spectrum with 5% critical damping specified in the notification No.1461. The compatibility to the target spectrum (design spectrum) should be checked in the period range between 0.02 and 5 second. However, under some specific conditions such as the deep sediment in the ground surface (here within the scope as deep as up to layer with shear wave velocity of 3,000m/s), much longer component than 5-second should be considered, since the deep subsurface conditions sometimes generate surface waves that dominates in longer period component. The typical areas in Japan include the plain areas (Kanto plain, Osaka plain, Nohbi plain etc., please refer the BCJ guideline, 1992)

4.3 Compatibility to Target

There are several indices proposed to check if the motion is compatible to the target spectrum. Here, the recommended values for each index is added.

(1) Smallest Spectral Ratio, $\varepsilon_{\min}$

$$\varepsilon_{\min} = \left\{ \frac{S_a(T_i, h)}{DS_a(T_i, h)} \right\}_{\min} \geq 0.85$$

where, $S_a(T_i, h)$ is the acceleration response spectrum for period T_i, and damping ξ.

$DS_a(T_i, h)$ is the target spectrum.

(2) Coefficient of Variation, ν

$$\nu = \sqrt{\frac{\sum(\varepsilon_i - 1.0)^2}{N}} \le 0.05 \qquad (4)$$

(3) Average Error, $|1-\varepsilon_{ave}|$

$$|1-\varepsilon_{ave}| \le 0.02 \qquad (5)$$

where, $$\varepsilon_{ave} = \frac{\sum \varepsilon_i}{N} \qquad (6)$$

$$\varepsilon_i = \frac{S_a(T_i, h)}{DS_a(T_i, h)} \qquad (7)$$

N: Number of Periods where the error is examined

The following two values are recommended to confirm the damage potential of the motion.

(4) Spectral Intensity Ratio, SI_{ratio}

Here the integration will be done for between 1 and 5 second since the motion is mainly use in the design of long period buildings, although the Housner's proposal for the Spectral Intensity is the integration between 0.1 and 2.5 seconds.

$$SI_{ratio} = \frac{\int_1^5 pS_v(T)dT}{\int_1^5 DpS_v(T)dT} \ge 1.0 \qquad (8)$$

(5) Energy Spectrum

$$V_E = \sqrt{\frac{2E}{M}} \qquad (9)$$

5. EVALUATION OF VARIATIONS OF THE GENERATED GROUND MOTIONS

Time histories compatible to the design spectra are generated and their variations are examined.

5.1 Nonstationary time history with envelope function and uniformly random phase angles

1) Effect of duration time

Four types of duration time, i.e,.10, 20, 60, 120 seconds were used to generate the ground motions. The design spectra used here is an acceleration response spectrum shown in Fig.3. For each duration time, thirty time histories were generated. The non-stationary property, given with three parts, i.e., build-up, constant, and decay, are identical of a proportion to total duration time. The wave forms are shown in Fig.4. The frequency distribution of peak acceleration and velocity are shown in Fig.5. The left hand figure shows the frequency distribution of the peak accelerations. It is seen that shorter duration time gives larger variations. The variation for the 120-second motion is much less than the shorter duration cases. The right hand figure shows the same distribution for peak velocities. It is also seen that the longer duration time shows less variations with peak amplitudes.

2) Variation of response spectra

In Fig.6, the averages of the ratio to the target spectra (5%) are shown for durations of 10 and 120 seconds for 1, 2, 10, 20 % damping. The variation of the acc spectral values with damping at the longer period range becomes less for shorter duration time histories. This is due to that the wave number is not enough for the response to grow up for shorter duration and longer period. As a result, the response value is less varied with damping.

In Japan, the following formula is commonly used to modify the response spectrum with damping other than 5%.

$$F_h = \frac{1.5}{1+10h} \qquad (10)$$

Fig.7 shows coincidence between equation (10) and the ratio computed from the generated ground motions. It is seen that for shorter period the ratio becomes nearly unity for 10% and 1 to 1.2 for 20% damping, for longer period the ratio is much larger with shorter duration and less in longer duration and almost constant for intermediate period.

Fig.8 shows the ratio between the response spectrum and the pseudo response spectrum for displacement. It is seen that for longer period the difference becomes larger. In addition, the comparison was made for mean energy spectra for motions with different duration time in Fig.9. It is seen that the energy spectral value for 120 second duration becomes approximately double of the 10 second duration, and the spectrum is sensitive to the duration time.

3) Nonlinear response characteristics

Time history analysis is mainly utilized in nonlinear response analysis. Here, we used the 1dof bi-linear model to compare the non-linear response properties.

The parameters used are the skeleton curves, stiffness ratio, building period, stiffness ratio. We investigated the influence of these parameters on the peak displacement response with motions of 4 types of duration times. Figure 10 shows the influence of yield strength and building period on the relation of peak displacement and duration time. It can be commonly said that the displacement response depends on duration time. The longer building period gives larger variation. The smaller yield strength gives larger variation in peak displacement. For the cases of elastic building period less than 1 second, the response displacement does not depend too much on the duration time.

Figure11 shows the relation between the displacement response and duration time for several stiffness ratios. Although the displacement response and variation become larger when the stiffness ratio is very small, the response becomes smaller and almost same regardless of the stiffness ration when the ration is greater than 0.1 and as the duration becomes longer. It is also seen that the displacement response does not depend on the strength except for the duration of 10 second.

5.2 Nonstationary time history with phase angle of recorded motion

Figure 12 shows the superimposed plot of the acceleration response spectra for damping of 2, 5, 20 % with 16 generated spectrum-compatible motions. In this simulation, the duration of 120 second was used. In addition, the phase angles of the recorded motion are used to give the nonstationarity. The name of the records , minimum error, coefficient of variation, average error are listed in Table 1 with the resulted PGA, PGV and PGD.

In each drawing of Fig.12, the spectra derived using the equation (10) normalized to the 5% spectrum is also drawn. It is seen from the figure, the expected average spectra for ten generated motions slip out of the spectra computed with F_h. It can be pointed out that the assumed wave form with duration does not promise the relation F_h.

CONCLUSIONS

The variation of the average of the peak amplitude, PGA and PGV and the linear and nonlinear response spectral values were computed with numbers of simulated waves with various duration times.

The variation should be related with the randomness involved in earthquake occurrence, such as rupture process, complexity of wave propagation and the surface soils. However, to incorporate all these uncertainties in the earthquake simulation is almost impossible since the geophysical and/or seismological investigation on the earthquake wave forming process is still under development. Therefore, it is necessary for engineers to incorporate the uncertainties all together in evaluating the future strong ground motions. The number of time histories applied to the design analysis is quite limited. The importance is to know the possibility of loads or responses under such total uncertainties. In addition, the right position of time history analysis in design practice should be reestablished.

REFERENCE

(1) Y.Ohsaki, R.Iwasaki, T.Masao, I.Okawa,"Phase Characteristics of Earthquake Accelerogram and Its Application", Trans. of 5th Int. Conf. On SMiRT, K1/4, Vol.K(a)

(2) Building Research Institute, Building Center of Japan (1992), "A Technique for Evaluation of Design Earthquake Ground Motion", Building Center of Japan

(3) K.Yamagishi, M.Tanigaki, I.Okawa, "Average and Deviation Response Evaluation for Various Simulated Seismic Motions Using Random Phase Angles (Part 1,2)", Summary of AIJ Annual Meeting, 2000

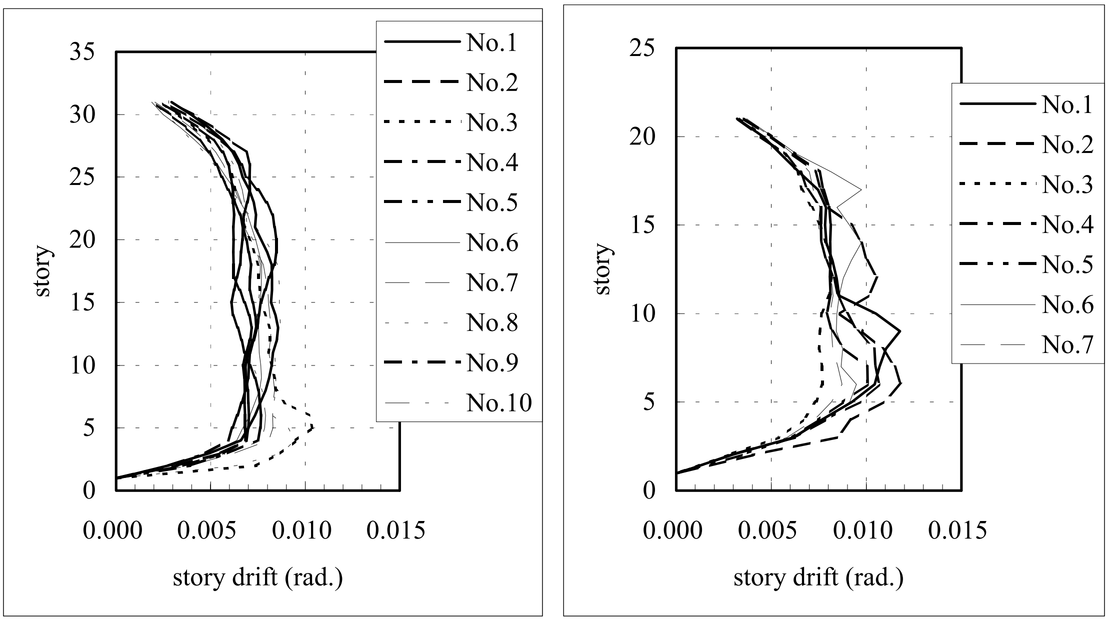


Fig.1 The distribution of computed story drift for the spectrum compatible motions
(Left: 10 BCJ level 2 motions, Right: 7 BSL Notification No1461 motions)

Table 1 List of recorded motion for use of phase angle and resulte peak amplitudes

Symbol	Min. error	COV	Ave. error	Δt	n	PGA (cm/s/s)	PGV (cm/s)	PGD (cm)
ELC40NS	0.831	0.044	0.011	0.02	2674	288.1	46.3	55.2
TAF52EW	0.874	0.062	0.008	0.02	2718	407.1	44.9	49.8
HAC68EW	0.863	0.056	0.000	0.02	2550	328.3	43.1	36.6
HAC68NS	0.866	0.075	0.062	0.02	2550	316.6	48.4	42.7
KSR93NS	0.864	0.005	0.008	0.01	8192	326.1	46.9	49.5
KSR93EW	0.860	0.069	0.010	0.01	8192	315.5	50.1	37.5
HKD93NS	0.856	0.050	0.010	0.01	15100	329.0	54.8	30.3
HKD93EW	0.869	0.049	0.011	0.01	15100	368.5	46.8	38.5
KSR94NS	0.861	0.056	0.004	0.01	16384	350.8	49.3	38.5
KSR94EW	0.866	0.044	0.006	0.01	16384	375.0	48.2	40.5
HCN94NS	0.883	0.050	0.010	0.01	8192	435.0	61.8	38.2
HCN94EW	0.861	0.054	0.007	0.01	8192	366.2	53.5	44.2
HCN95NS	0.858	0.041	0.005	0.01	8192	349.7	55.6	53.5
HCN95EW	0.892	0.044	0.008	0.01	8192	322.7	61.6	54.7
KOB95NS	0.854	0.054	0.011	0.02	4096	389.8	47.4	40.6
KOB95EW	0.850	0.064	0.006	0.02	4096	396.1	60.1	44.3

PGA:Peak Ground Acceleration, PGV:Peak Ground Velocity, PGD:Peak Ground Displacement

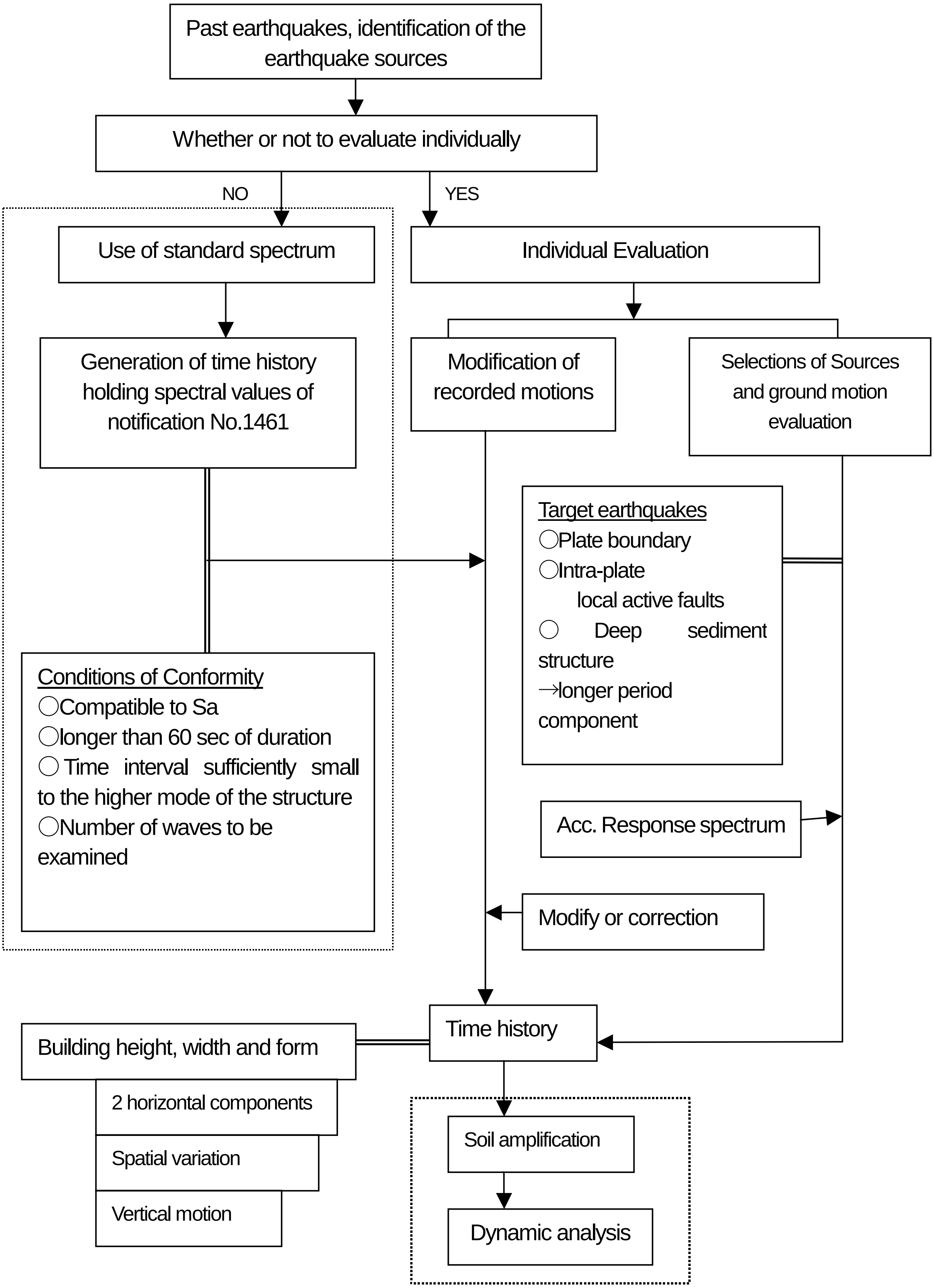


Fig.2 The flow chart for evaluating the design ground motion time history

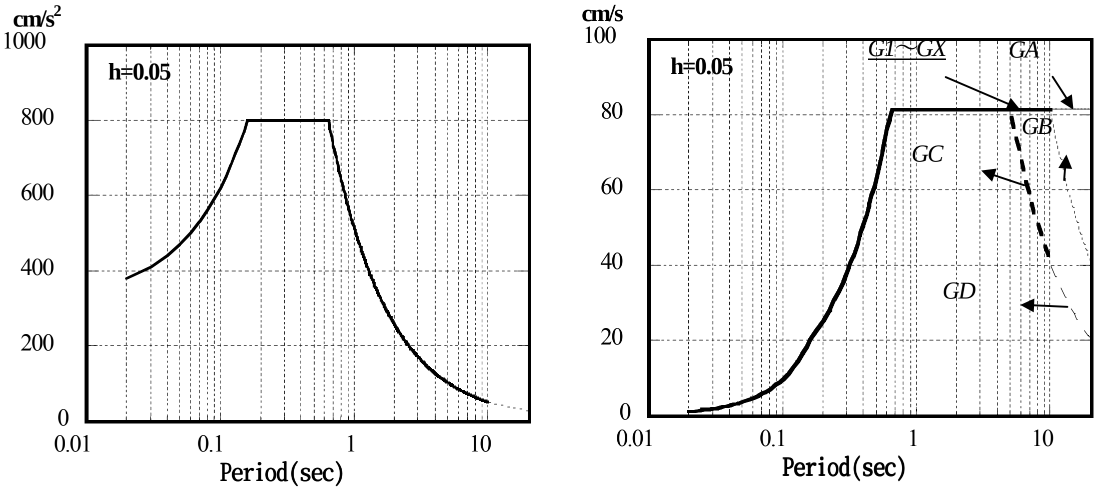


Fig.3 Target response spectrum (Left: Acceleration, Right: Velocity)

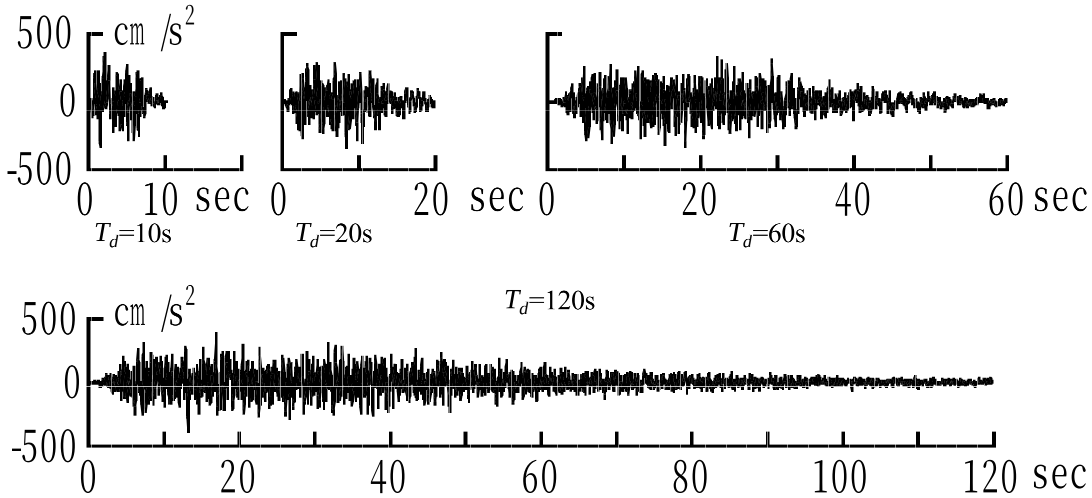


Fig.4 Example of time history (10, 20, 60, 120 seconds)

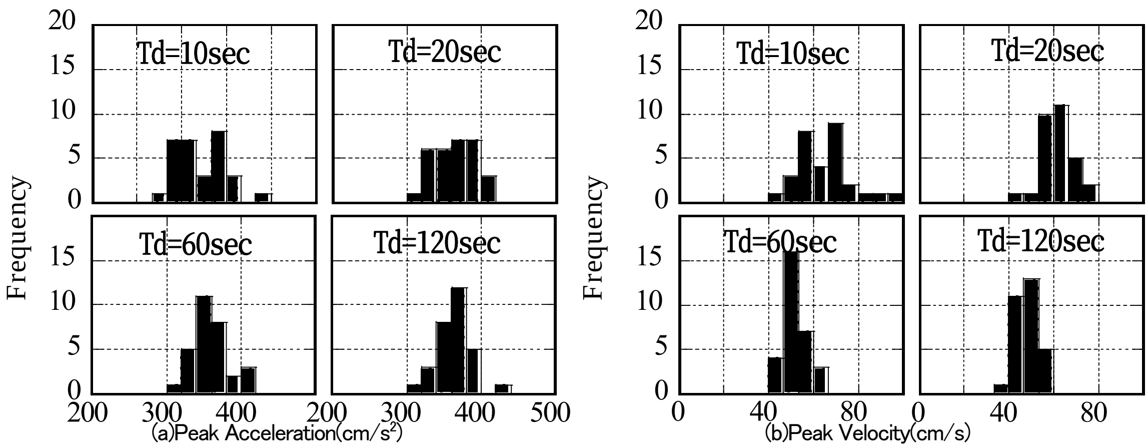


Fig.5 Frequency distribution of peak acceleration and peak velocity of generated time histories

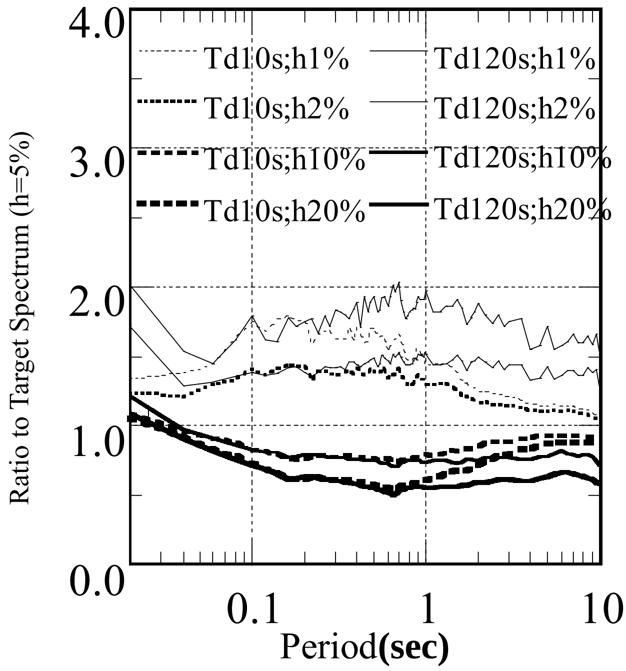


Fig.6 Ratio of response spectrum to the 5% damping spectrum

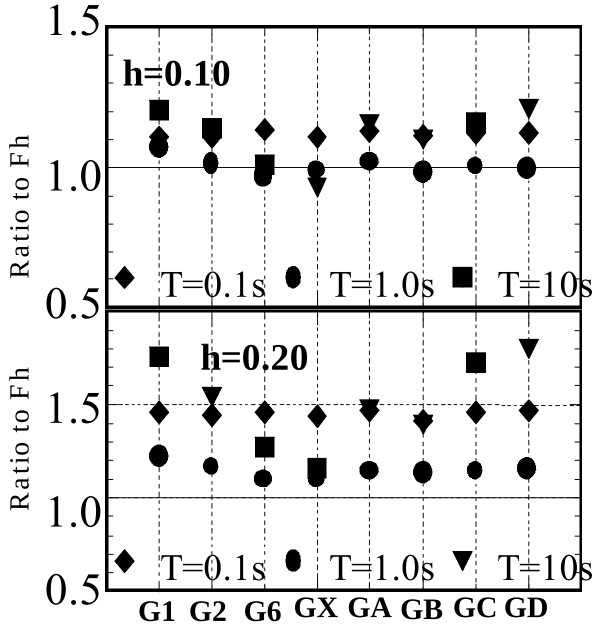


Fig.7 Comparison of F_h values at 0.1, 1, 10 seconds for various durations

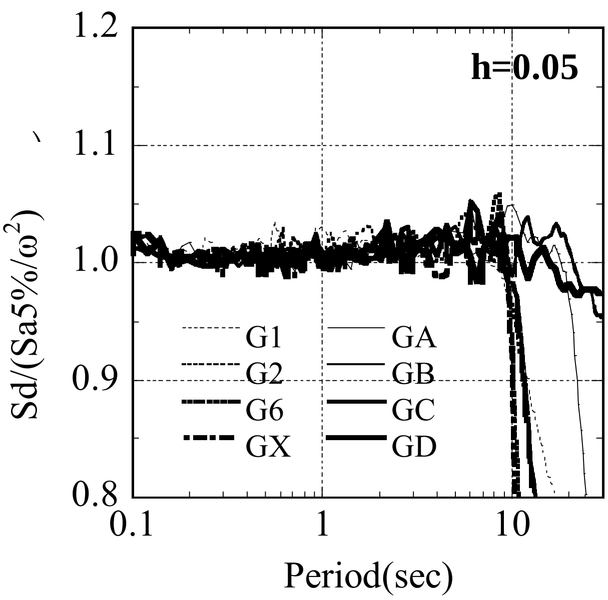


Fig.8 (lower) Ratio of the displacement spectrum to the pseudo displacement spectrum

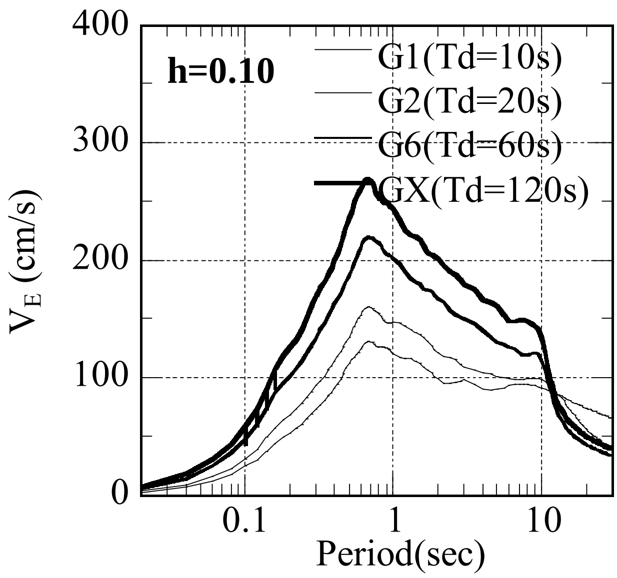
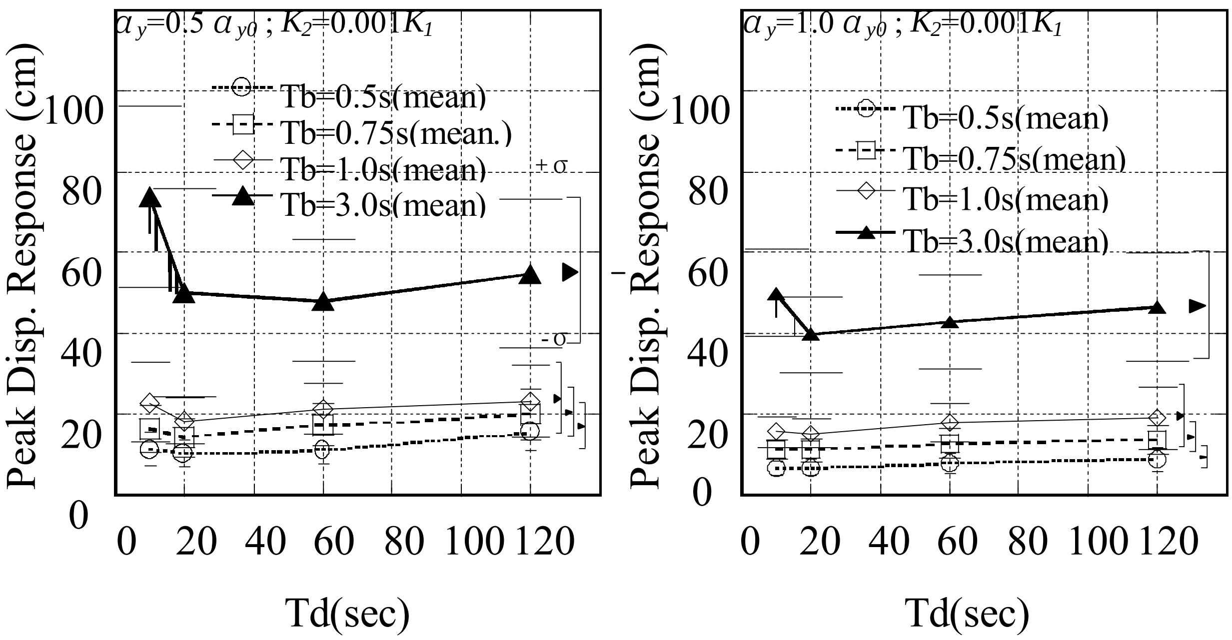
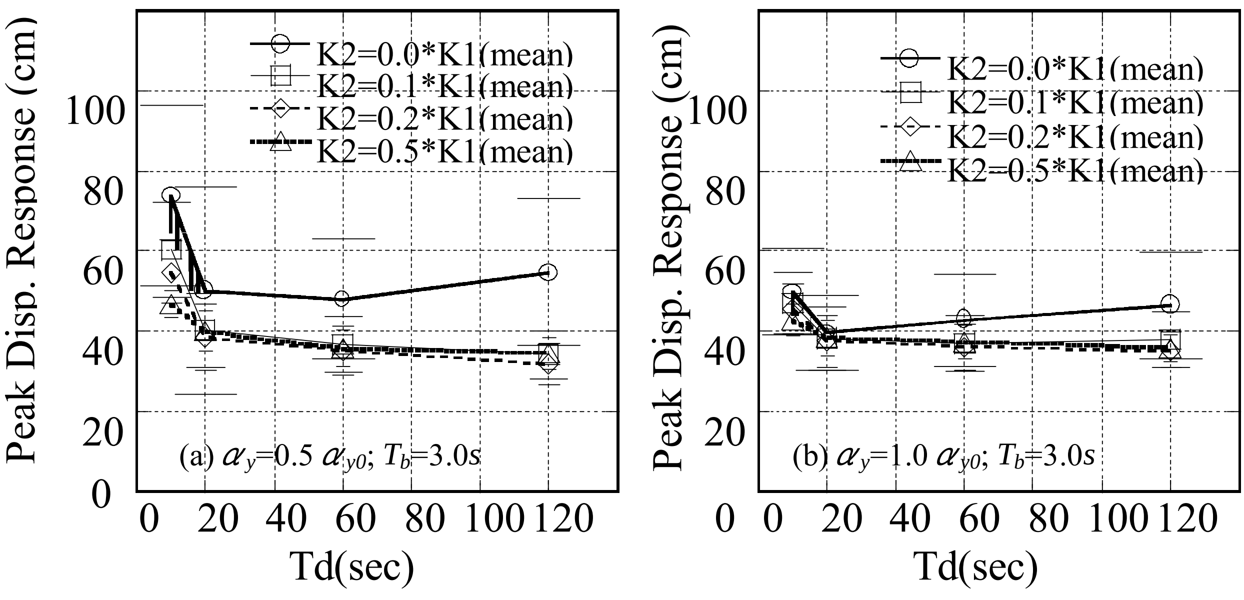


Fig.9 Comparison of energy spectra of the simulated motions



(a)*Bi-Linear* model(α_y=0.5 α_{y0}) (b)*Bi-Linear* model(α_y=1.0 α_{y0})

Fig.10 Relation between duration time and peak displacement response for bi-linear model (1)



Duration time and peak disp. response for *Bi-Linear* model

Fig.11 Relation between duration time and peak displacement response for bi-linear model (2)

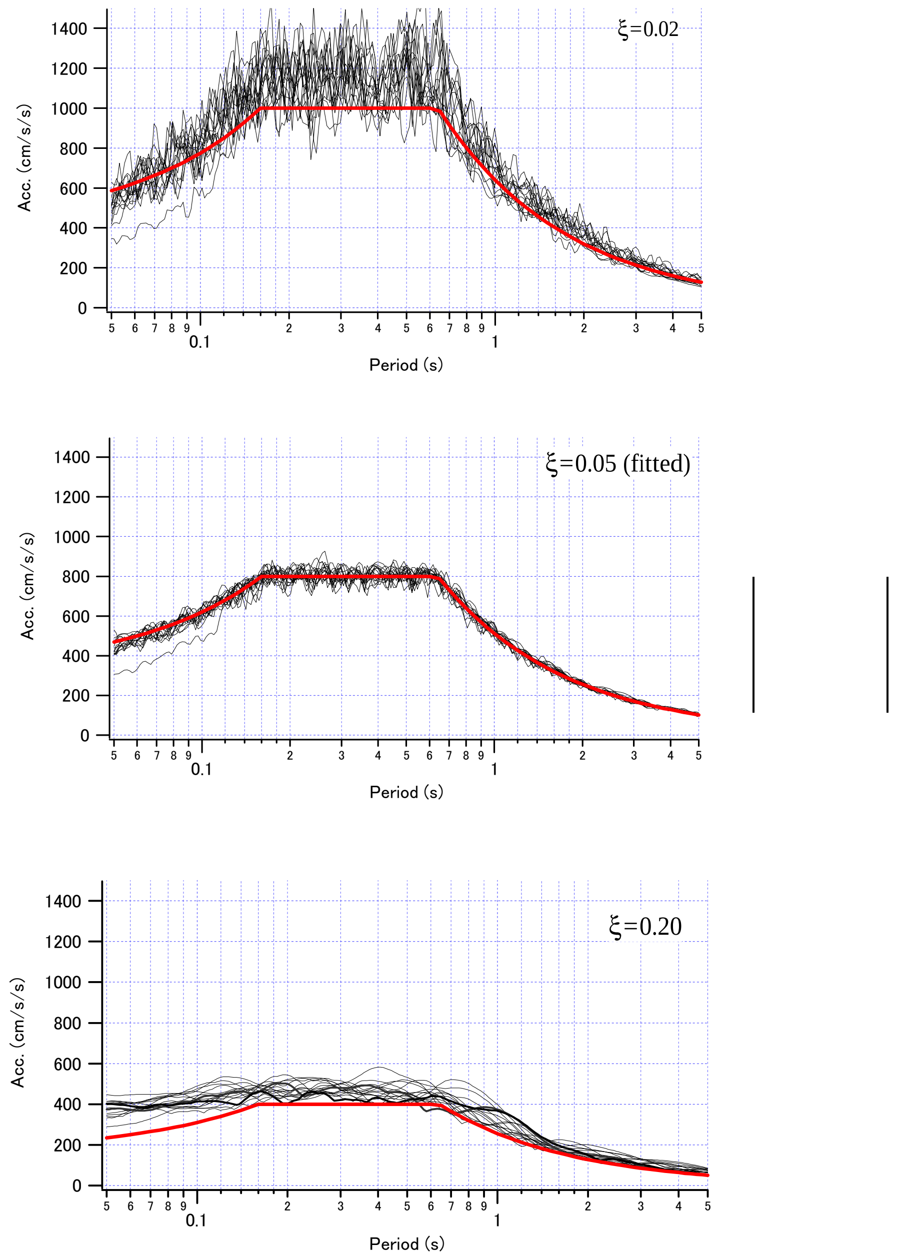


Fig.12 Computed response spectra and expected mean values