STUDY ON SEISMIC RESPONSE CONTROL SYSTEM FOR HEAVY, RIGID BUILDINGS OF REINFORCED CONCRETE: PART-2 INFLUENCE OF DIVERSIFICATION OF DESIGN FACTORS ON SEISMIC RESPONSE

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SUMMARY

A series of dynamic analyses for "a seismic response control building with soft upper steel frame", introduced in Part-1, is performed in order to investigate the influence of the diversification of design factors listed below. From these analyses, the influence of each factor has been evaluated as follows:

- the velocity of S-wave, 'Vs' of a support rock ground: the decrease of Vs yields greater responses at upper steel frames
- earthquake input waves: responses of the structure caused by all waves studied are within the design criteria
- weight of the roof: as the weight decreases the overturning moment of the building increases
- yielding shear force of the steel frame: considering non-linearity, the force on the viscous dampers decreases but the response of the structure remains the same as in the linear case
- rigidity (compressive strength) of concrete: little influence on the responses
- thickness of the outer box wall: the overturning moment increases a little, but this change contributes to decrease the response of the reinforced concrete containment vessel

Through these analyses, the seismic reponse controlled reactor building demonstrated enough aseismatic capacity against each of the parameters considered in this study.

INTRODUCTION

A new seismic response control system for a heavy, rigid structure of reinforced concrete such as a nuclear reactor building has been developed. "The seismic response control building with soft upper steel frame", as introduced in Part-1 of this series of study, is an application of the 'passive control' using a part of the main structure as the control system. So the parameters of the system are not easy to change once the system is installed. In this paper, the influences of the diversification of various design factors on the seismic responses of the building are discussed. Design factors considered here include: the velocity of S-wave (Vs) of a support rock ground, earthquake input waves, weight of the roof, non-linearity of the upper steel frame, rigidity (compressive strength) of the concrete organizing the main body of the building, and the thickness of the outer box wall.

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Firstly the influence on the mechanisms of seismic control system caused by the changes of these factors will be investigated through the eigenvalue analyses. Then the structural design and the safety margin will be checked by the seismic response analyses using the extreme design earthquakes.

THE ASSUMPTION OF THE DIVERSIFICATION OF DESIGN FACTORS

Six design factors were investigated in this study. The range of each factor's diversification was assumed wider than that of an ordinary design in order to ensure a safety margin for the new structural system.

The six design factors considered in this study are listed below, and the diversification range of each factor are assumed as follows.

a. velocity of S-wave,'Vs' of a support rock ground:
Vs=1,250 m/sec as a standard velocity of S-wave of the support rock ground, and the range of Vs=1,000~ 1,500 m/sec is investigated considering the differences in geological formation within the area of a site

b. earthquake input waves:
'The extreme design earthquake' (S2-D) shown in the Figure 2-1 as a standard input wave and two other earthquake input waves (HE and KB) which have different amplitude envelopes, but the acceleration target spectra of those waves are fitted with that of S2-D are used. Table 2-1 shows the characteristics of these earthquakes.

c. weight of the roof:
Changes of the weight of the snow on the roof are considered. ±0 as a standard weight, ±15% of total weight of the roof are assumed, that is about ±0.3% of total weight of the building

d. yielding shear force 'Qy' of the steel frame
The restoring force characteristics of the shear deformation of the upper steel frame are modeled as a trilinear skeleton curve as in Figure 2-2. The standard case is defined as Qy=1.0xQmax, that means the shear force at the first turning point Qy equals the maximum shear force Qmax in the steel frame caused by the S2-D. In other words, the steel frame remains linear on the restoring force characteristics of the shear deformation. Two other skeleton curves are defined so that Qy equals to 0.7xQmax and 0.5xQmax. The case of Qy=0.7xQmax corresponds to the state where the frame is linear under the maximum design earthquake, S1.

e. rigidity (compressive strength) of concrete (Fc) organizing the main body of the building
Fc=32.4 N/mm^2, where the design standard strength, is the standard case, the increase of the Fc up to the average compressive strength of concrete in the actual building (assume Fc=45.0 N/mm^2) and the decrease of the Fc by the non-linearity of the shear wall (assume Fc=25.5 N/mm^2) are considered.

f. thickness of the outer box wall
Increase of the thickness of the shear wall by the design change is considered. The original thickness as a standard thickness, +20% of original thickness is assumed.

Dynamic analysis models corresponding to the diversification of these design factors are prepared and listed in Table 2-2. The parameters of the seismic control system, 'K' and 'C', are set by using the results of the Part-1, those are K=1/6K$_0$ and C=20 (tf sec/cm).

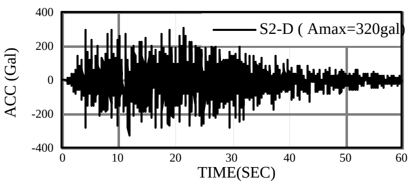


Figure 2-1: S2-D

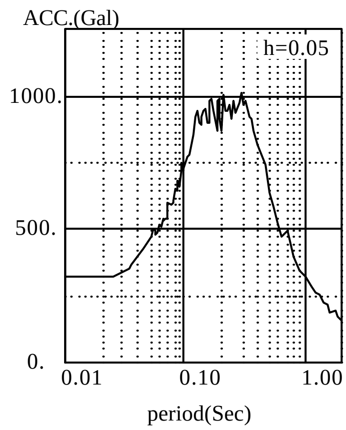
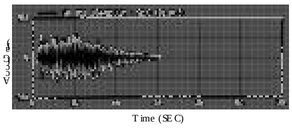
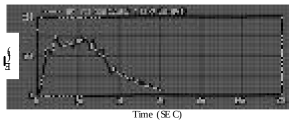
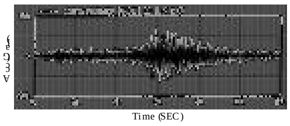
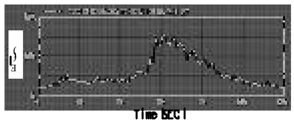
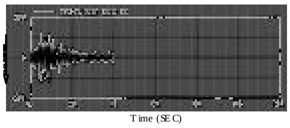
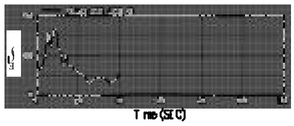


Table 2-1: earthquake input waves

case	observed				standardized imput wave (gal)	Time History acceleration wave	amplitude envelope
	yr, mo, d	M	D(Km)	acc(gal)			
S2-D (artificial)	—	—	—	—	320	 Time (SEC)	 Time (SEC)
HE (observed)	94.10.4	8.1	571	48	350	 Time (SEC)	 Time (SEC)
KB (observed)	95.1.17	7.2	15.5	818	325	 Time (SEC)	 Time (SEC)

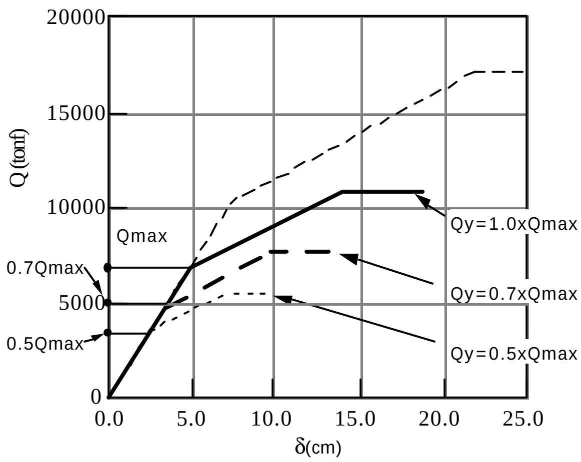
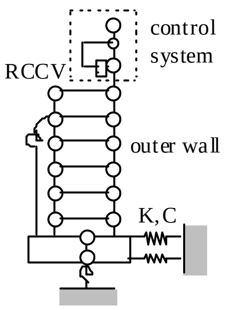
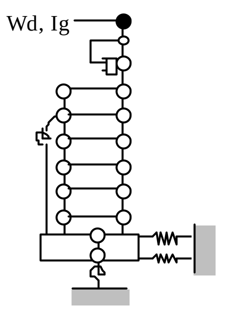
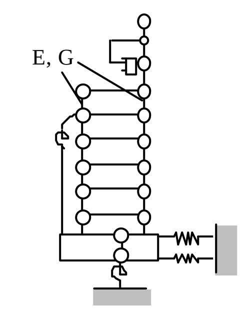
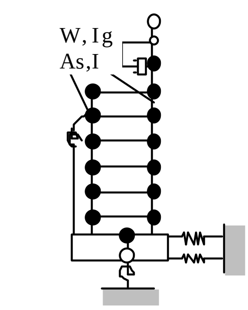


Figure 2-2: triliniear skeleton curves for the steel frame

Table 2-2: calculation models

: standard case

case	a, 'Vs' of suppot ground			c, 'Wr' weight of the roof			e, 'Fc' rigidity of concrete			f, 't' thickness of wall	
paramenters	Vs 1000	Vs 1250	Vs 1500	Wr –15%	Wr ±0%	Wr +15%	Fc 25.2 N/mm2	Fc 32.4 N/mm2	Fc 45.0 N/mm2	original t	t +20%
model											

EVALUATION OF THE INFLUENCE OF EACH DESIGN FACTOR

EIGENVALUE ANALYSES

"The seismic response control building with soft upper steel frame" is a kind of 'tuned mass type' seismic control system. The influence of the diversification of the design factors on seismic response of the buildings can be estimated by investigating the relationship between the natural period of the 'soft' upper steel frame and that of the 'hard' lower reinforced concrete (RC) structure. Those natural periods correspond to the first and the second order natural periods of the building, respectively in this study. A series of eigenvalue analyses were conducted using the dynamic analysis model in the Table 2-2 in order to investigate the influence of the diversification of design factors on the first-order and the second-order vibration modes. Among six design factors, "b. earthquake input waves" and "d. yielding shear force 'Qy' of the steel frame" are excluded from these analyses because they don't affect the natural vibration mode of the building.

Eigenvalues and the natural vibration mode of each case are shown in Table 3-1. According to these results, seismic response reduction produced by the control system can be expected in all cases. This is because the peculiar natural vibration modes of the controlled building have been observed; specifically the upper steel frame deforms in the first-order mode and the upper frame moves the opposite direction of the lower structure in the second-order mode. The fluctuation of the first and the second natural periods is shown in Table 3-2. The change in weight of the roof is the primary factor which affects the fluctuation of the first-order period. The changes in weight of the roof from –15% to +15% cause the same result in the first-order period as the change of the steel frame stiffness from $K=1/5K_0$ to $K=1/7K_0$. 'Vs' of a support rock ground and the rigidity (compressive strength) of concrete organizing the main body of the building also have a little influence on the first-order period. For the second-order natural period, these two factors play a very important role.

From the results of these eigenvalue analyses, the following findings have been obtained.

- Changes in weight of the roof is the primary factor which affect the first-order natural vibration mode of the seismic response controlled building where the deformation predominates in the upper steel frame.
- 'Vs' of a support rock ground and the rigidity (compressive strength) of concrete are important for the second-order natural vibration mode where the deformation of lower RC structures is predominant.

Table 3-1 the results of eigenvalue analyses

β : participation factor

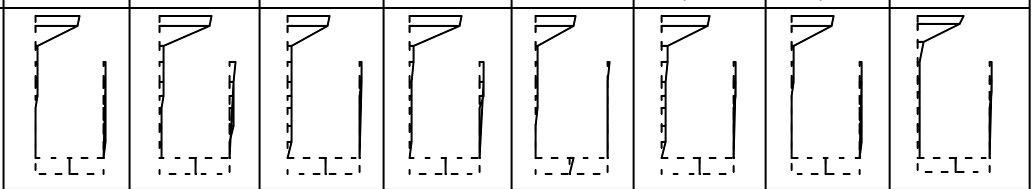
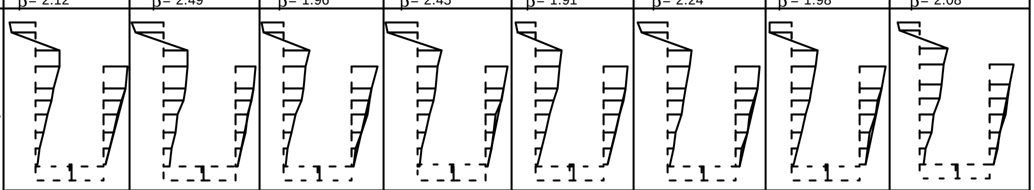
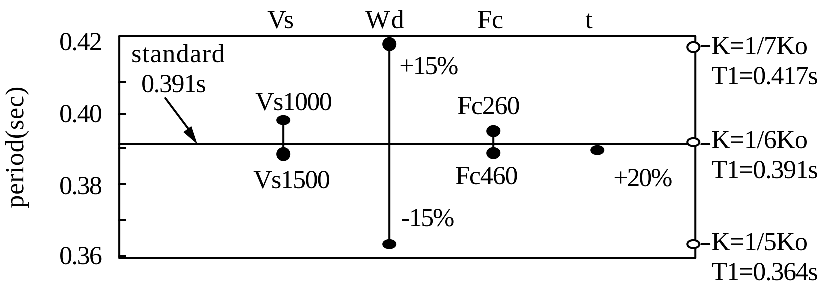
	standard case	a, 'Vs' of suppot ground		c, 'Wr' weight of the roof		e, 'Fc' rigidity of concrete		f, 't' thickness of wall
		Vs 1000	Vs 1500	Wr –15%	Wr +15%	Fc 25.2 N/mm^2	Fc 45.0 N/mm^2	t +20%
1st-order								
	T1= 0.391sec β= 2.12	T1= 0.397sec β= 2.49	T1=0.389sec β= 1.96	T1= 0.363sec β= 2.45	T1= 0.417sec β= 1.91	T1= 0.394sec β= 2.24	T1= 0.388sec β= 1.98	T1= 0.389sec β= 2.08
2nd-order								
	T2= 0.253sec β= 1.32	T2=0.276sec β= 1.56	T2=0.241sec β= 1.34	T2= 0.252sec β= 1.52	T2= 0.254sec β= 1.35	T2= 0.262sec β= 1.30	T2= 0.242sec β= 1.36	T2= 0.249sec β= 1.34

Table 3-2 fluctuation of 1st. and 2nd. natural period

<table>
<tr><td>1st-order natural vibration mode</td><td>
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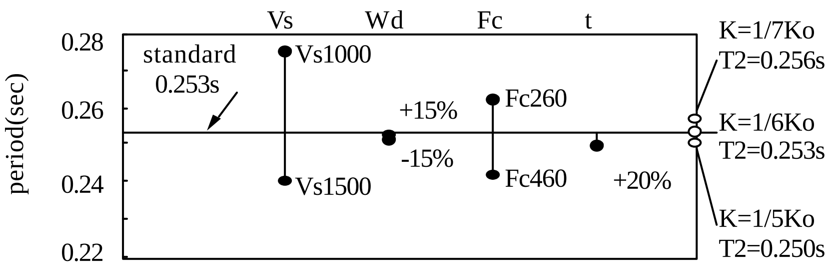
<tr><td>2nd-order natural vibration mode</td><td>
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3.2 SEISMIC RESPONSE ANALYSES

Using the calculation models presented in Table 2-2, seismic response analyses for each design factors were conducted. Comparison in each case of the maximum overturning moment of the building, Md, and shear force in the upper steel frame, Qs, representing the seismic response of the whole building and the seismic response control system, respectively, is shown in Table 3-3. The influences of the diversification of each design factors on the seismic responses is as follows:

a. velocity of S-wave, ‘Vs’ of a support rock ground

The influence of Vs, ranging 1,000 ~ 1,500 (m/sec), on the maximum response overturning moment of the building, Md, is quite small, increasing 4% as the Vs changes from Vs=1,250 to Vs=1,500, and decreasing 4% as it changes to Vs=1,000. The fluctuation of the maximum response shear force of the steel frame, Qs, is larger than that of the overturning. The increase/decrease tendency of the Qs, by changing the value of Vs, is different from that of Md, that is, a 12% increase in case of Vs=1,000 and a 5% decrease in the case of Vs=1,500 as compared with the case of Vs=1,250. This is because the increase/decrease of Vs represents the relative decrease/increase of the stiffness of the upper steel frame against the stiffness of lower structure including the support rock ground. According to the fluctuation balance of Qs, the stiffness of the steel frame, K, should be designed lower to prevent excess in the steel frame response.

b. earthquake input waves

The differences in the maximum Md caused by the variation of amplitude envelopes for input waves were not significant but those in the maximum Qs reached 27%. In this study, design criterion of Qs was satisfied in all cases, however, the design margin should be carefully maintained considering these conditions.

c. weight of the roof, ‘Wr’

According to the result of the analysis, the response of the whole building increased 7% by the decrease of the roof weight, and decreased 2% by the increase of the weight. The response of the upper frame decreased a little in both cases. That is because the increase/decrease of the weight of the roof causes the same response with the decrease/increase of the stiffness, K. The maximum value of Md increases if the Wr is changed to be heavier or lighter because the optimum ‘K’ was designed so that the overturning moment would be minimum.

d. yielding shear force ‘Qy’ of the steel frame

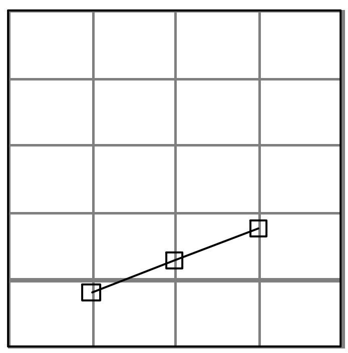
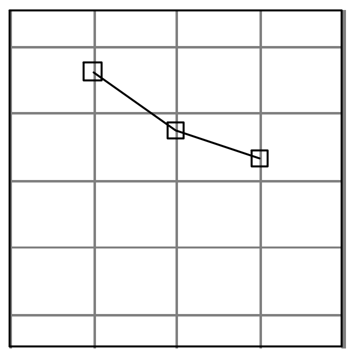
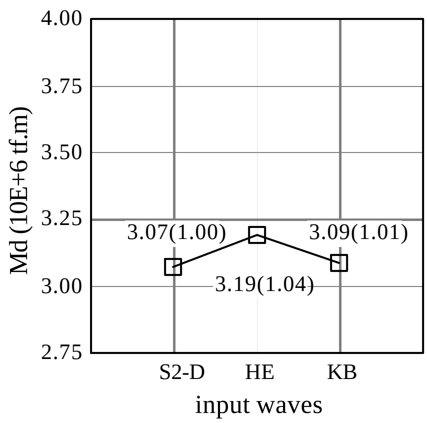
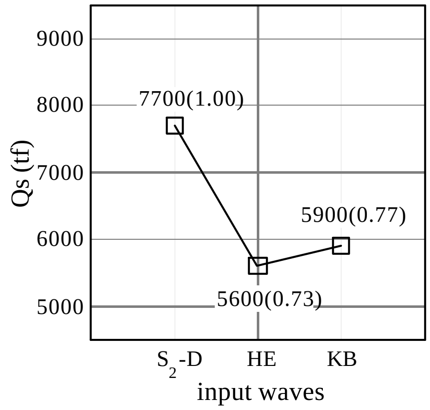
As the design yielding shear force, Qy decreases, the maximum shear force in steel frame, Qmax also decreases, however, Md, the response of whole building, remains unchanged. This is because the hysteretic damping of the steel frame takes over a part of the dynamic energy consumed in the viscous dampers after the steel frame yields.

e. rigidity (compressive strength) of concrete (Fc) organizing the main body of the building
As the rigidity of the concrete increases/decreases, the overturning moment increases/decreases by about 1~2%, and the shear force on the steel frame decreases/increases by 6%. The change of the compressive strength, Fc, represents the change of the rigidity balance between the upper steel frame and the lower RC structure just like the change of velocity of S-wave of a support rock ground.

f. thickness of the outer box wall
By increasing the thickness of the shear wall by +20% of original thickness, the overturning moment increases by 5%. But the stability of the building, such as the contact rate of the foundation, doesn’t change because the total weight of the building also increases 5% at the same time.

All the maximum responses, considering the diversification of design factors assumed in this study, satisfy the design criteria on every part of the structure.

Table 3-3 comparison of the maximum responses in each case

output / parameters	Overturning moment(Md)	Shear force steel frame(Qs)
a. 'Vs' of a support rock		
b. earthquake input		

<table>
<tr><th>output
parameters</th><th>Overturning moment(Md)</th><th>Shear force steel frame(Qs)</th></tr>
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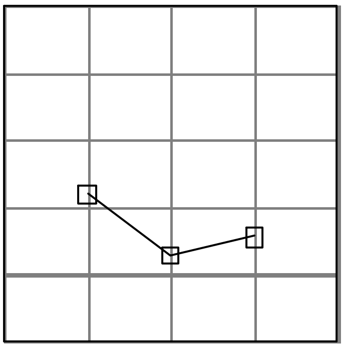
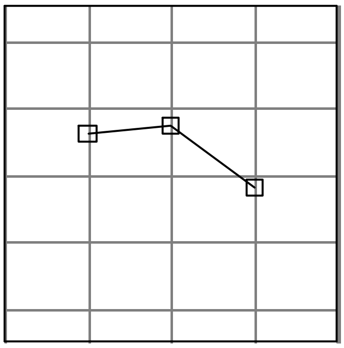
'Wr'
weight of the roof</td><td></td><td></td></tr>
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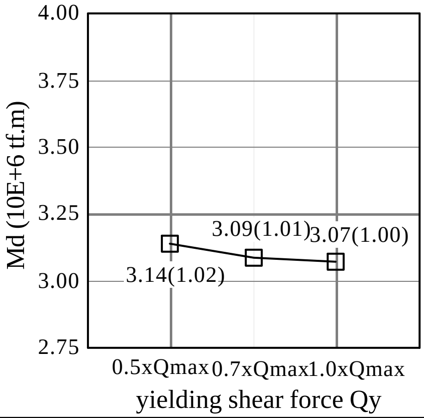
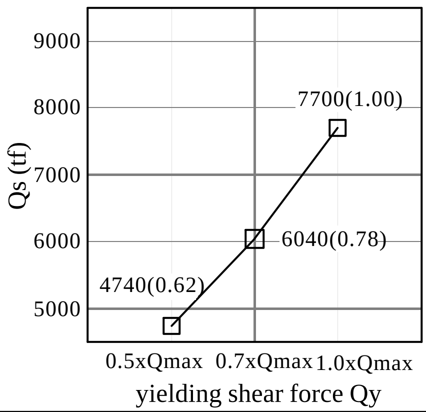
yeilding shear force 'Qy' of</td><td>
</td><td>
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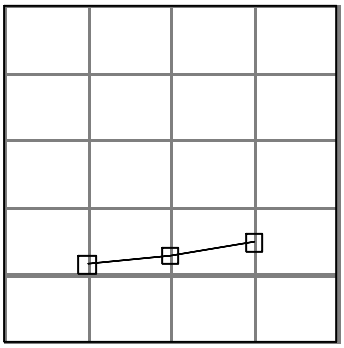
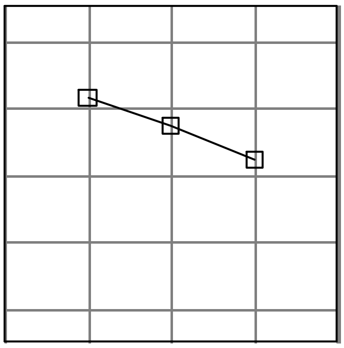
<tr><td>e.

compressive strength 'Fc' of concrete</td><td></td><td></td></tr>
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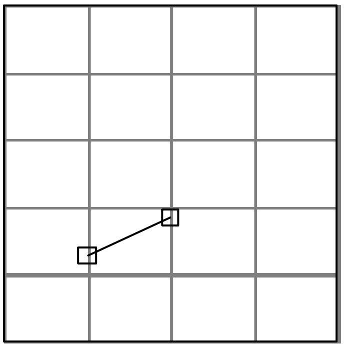
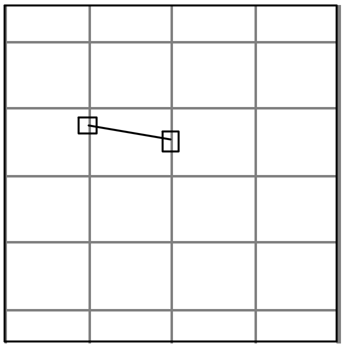
thickness of the outer wall 't'</td><td></td><td></td></tr>
</table>

CONCLUSIONS

A series of dynamic analyses for “a seismic response controlled building with soft upper steel frame”, introduced in Part-1, is performed in order to investigate the influence of the diversification of design factors listed below. From these analyses, the influence of each factor has been evaluated as follows:

- the velocity of S-wave, ‘Vs’ of a support rock ground: the decrease of Vs yields greater responses on upper steel frames
- earthquake input waves: responses of the structure caused by all waves studied are within the design criteria
- weight of the roof: as the weight decreases the overturning moment of the building increases
- yielding shear force ‘Qy’ of the steel frame: considering non-linearity, the force on the viscous dampers decreases but the response of the structure remains the same as in the linear case
- rigidity (compressive strength) of concrete: little influence on the responses
- thickness of the outer box wall: the overturning moment increases a little, but the stability of the building, the contact rate of the foundation, during the earthquake is not affected by this change

Through these analyses, the seismic reponse controlled reactor building demonstrated enough aseismatic capacity against each of the parameters considered in this study.

REFERENCES

Fukushima Y., Sugawara R., et al. (1995), *Study on Seismic Response Control System of Nuclear Reactor Building Part.2 Influence of Bed Rock and Building Characteristics Variation*, AIJ annual report (1995) in Japanese

Matsumoto H., Ogihara M., et al. (1996), *Study on Seismic Response Control System of Nuclear Reactor Building Part. 3 Response Effect on Steel Frame Non-Elasticity*, AIJ annual report (1996) in Japanese

Matsumoto H., Sugawara R., et al. (1995), *Study on Seismic Response Control System of Nuclear Reactor Building Part. 1 Seismic Response Control Method and Response Reduction*, AIJ annual report (1995) in Japanese