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PERFORMANCE–BASED SEISMIC DESIGN METHOD OF ULTRA-HIGH RISE REINFORCED CONCRETE BUILDINGS

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SUMMARY

This paper describes a seismic design method proposed by authors for ultra-high rise reinforced concrete buildings based on structural performance.
For three earthquake design levels which are expressed in terms of mean recurrence interval, three performance levels of buildings are established. The ground motions in the structural design are determined from the method proposed by the author. Detailed seismic criteria to estimate damages of structural members are established and compared with the damage level computed using static analyses and dynamic response analyses with a frame model. The applicable range of concrete compressive strength is 100 N/mm^2.

The authors examined whether a 60-story RC building could be designed using the proposed design method. Its structural system is composed of a frame structure and mega-frame structure made of mega-beams and mega-columns.

The damage for each of the three earthquake design levels is less than the criteria through the trial design. It was shown that a 60-story RC building can be designed, and that the proposed design method is reasonable and rational for ultra-high rise reinforced concrete buildings.

1. INTRODUCTION

It has become possible to build higher reinforced concrete buildings as stronger materials have been developed. Recently, buildings nearly over 40-stories high have been built using high strength concrete whose compressive strength fc is about 60MPa, but it is probably impossible to build a 60-story building using such concrete. On the other hand, there is a global trend toward designing structures based on performance, yet there is no performance-based seismic design method that is authorized in Japan.

An object of this paper is proposing a performance-based seismic design method for ultra-high rise reinforced concrete buildings that are 60 stories (200 meters) high using ultra high strength concrete whose compressive strength fc is 100 N/mm^2. The design method has three earthquake design levels and satisfies building performance levels. There was no rational method for defining earthquake motions based on performance. This paper defined a new design method, particularly for multiple dwelling houses. The authors propose a structural system that is based on suitable in use and scale of the building. The structural system is composed of a frame structure and mega-frame structure made of mega-beams and mega-columns. The Structural performance of the building was investigated through the trial design using the proposed seismic design method.

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2. ABSTRACT OF PROPOSED PERFORMANCE-BASED SEISMIC DESIGN METHOD

2.1 Outlines of Design Method

The service period of the building is defined as 100 years in this design method. Three earthquake design levels are defined from the recurrence interval of earthquakes in the period. Each ground motion and each static earthquake load is defined for each design level (section 3). Buildings have to satisfy three performance levels for three earthquake levels. The building performance is assessed by computing analyses which are non-linear static analyses and dynamic response analyses with a three-dimensional frame model, and non-linear dynamic response analyses with a multi-lumped mass model.

2.2 Limit Status and Criteria

The relationships between the earthquake design levels and the building performance levels are shown in Table 1. That is, the building damage must not exceed the damage in the serviceability limit state due to an earthquake happening a few times in the service period, the building damage must not exceed that in the restorability limit state due to a huge earthquake possibly happening one time in the service period, and the building damage must not exceed that in the ultimate limit state due to the largest huge earthquake at the site. The definition of each limit state is as follows.

Serviceability limit state : Building can not continue to be used,
Restorability limit state : Building damaged, difficult to reuse after restoring ,
Ultimate limit state : Difficult to maintain safety:

Criteria in each limit state for structural members are shown in Table 2. The criteria values, considering of importance of this building, are a little more severe than the values generally used.

Serviceability limit state: Members must not undergo a yielding hinge or shear cracking. Standard story drift angle is less than 1/200rad,
Restorability limit state : Members are allowed a yielding hinge, but restricted ductility factor. Standard story drift angle is less than 1/120rad,
Ultimate limit state : Members have sufficient strength and ductility not to fail to cause collapse. Standard story drift angle is less than 1/75rad:

2.3 Design Force Amplifications at Ultimate State Limit Design Stage

In the frame model analyses, main reinforcement strength of the members allowed yielding hinge is the specified yield strength value when estimating the story drift angle, while one of the members allowed yielding hinge is upper limit yield strength when estimating for strength and ductile design in members. Loading angle is two structure principal axis and diagonal angle which shows the maximum load on the structure. The coefficients of design force amplification at the ultimate limit design stage are shown in Table 3. The design force amplification factors are gotten as follows.

A) Dispersion in material strength,
B) Lateral loading of diagonal angle on structure,
C) Dynamic effect of vertical members, such as columns and shear walls in frame structure,
D) Dispersion in flexural strength formula,
E) Ground motion in the vertical direction:

Table 1: Relationships between the earthquake design levels and the building performance levels

Building limit state		Serviceability	Restorability	Ultimate
Performance level		Can continue to be used	Reuse after restoring	maintain safety
Earthquake frequency	A few times in the service period	Yes		
	Possibly one times in the service period		Yes	
	A maximum huge earthquake at the site			Yes

Table 2: Criteria in each limit state for structural members

<table>
<tr><th colspan="2">Limit state</th><th>Serviceability</th><th>Restorability</th><th>Ultimate</th></tr>
<tr><td rowspan="9">Damage state for structural members</td><td>Column
(not yielding)</td><td rowspan="4">$M<M_u$
$Q<{}_sQ_{cr}$</td><td rowspan="2">$M< M_u$</td><td>1) $M<M_u$
2) Not shear failure
3) Not bond failure
4) Not compressive failure</td></tr>
<tr><td>Column
(allowed yielding in the ultimate limit state)</td><td rowspan="2">1) Not exceed limit deflection for bending.
2) Not shear failure after yielding
3) Shear strength is higher than design shear force though bond failure occurred
4) Not compressive failure</td></tr>
<tr><td>Column
(allowed yielding)</td><td>$\mu < 1.5$</td></tr>
<tr><td>Beam</td><td>$\mu < 3.0$</td><td>1) Not shear failure after yielding
2) Shear strength is higher than design shear force though bond failure occurred</td></tr>
<tr><td>Beam-column joint</td><td>$M<M_u$
$Q<{}_sQ_{cr}$</td><td>$\gamma < 0.002$</td><td>1) Not shear failure
2) Not bond failure in the beam bar</td></tr>
<tr><td>Shear wall and mega-column
(not yielding)</td><td rowspan="4">$M<M_u$
$Q<{}_sQ_{cr}$</td><td rowspan="3">$M<M_u$
$\gamma < 0.002$</td><td>1) $M<M_u$
2) Not compressive failure
3) Not shear failure</td></tr>
<tr><td>Shear wall
(allowed yielding in the ultimate limit state)</td><td rowspan="2">1) Not exceed limit deflection for bending
2) Not shear failure after yielding
3) Not compressive failure</td></tr>
<tr><td>Shear wall and mega-column
(allowed yielding)</td></tr>
<tr><td>Mega-beam</td><td>$\mu < 1.5$</td><td>1) Not shear failure after yielding
2) Shear strength is higher than design shear force though bond failure occurred</td></tr>
</table>

M_u: Ultimate bending moment, ${}_sQ_{cr}$: Cracking shear force, γ : Shear deformation angle, μ : Ductility factor

Table 3: Coefficients of design force amplification at the ultimate limit design stage

<table>
<tr><th colspan="2">Analysis case</th><th>Static analyses using specified yield strength value for yielding member reinforcement</th><th>Dynamic analyses using upper limit strength value for yielding member reinforcement</th></tr>
<tr><td rowspan="4">Member not allowed yielding</td><td>Bending moment for column, mega-column and shear wall</td><td>1.1 × (rate of dynamic amplification)</td><td>1.1 × (value of (upper limit strength / specified yield strength)) for the adjoining yielding member</td></tr>
<tr><td>Axial force for column</td><td>1.2 times for the dead load value
1.1 times for the additional load value</td><td>1.2 times for the dead load value
1.4 times for the additional load value</td></tr>
<tr><td>Shear force for column, mega-column and shear wall</td><td>1.1 × (rate of dynamic amplification)</td><td>1.1 × (value of (upper limit strength / specified yield strength)) for the adjoining yielding member</td></tr>
<tr><td>Bond stress for column reinforcement</td><td>1.1 × (rate of dynamic amplification) for the bond stress in analysis of Navier's hypothesis)</td><td>1.1 × (value of (upper limit strength / specified yield strength)) for the adjoining yielding member for the bond stress in analysis of Navier's hypothesis</td></tr>
<tr><td rowspan="5">Member allowed yielding
and
member allowed yielding in the ultimate limit state</td><td>Axial force for column</td><td>1.2 times for the dead load value
1.1 times for the additional load value</td><td>1.2 times for the dead load value
1.4 times for the additional load value</td></tr>
<tr><td>Shear force after yielding for beam</td><td>1.1 times for the shear force</td><td>1.1 × (value of (upper limit strength / specified yield strength))</td></tr>
<tr><td>Shear force after yielding for column, mega-column and shear wall</td><td>1.1 × (rate of dynamic amplification)</td><td>1.1 × (value of (upper limit strength / specified yield strength))</td></tr>
<tr><td>Bond stress for beam reinforcement</td><td colspan="2">1.1 times for the stress of reinforcement at compression side in analysis of Navier's hypothesis using upper limit strength value</td></tr>
<tr><td>Bond stress for column reinforcement</td><td colspan="2">1.1 × (dynamic amplification) for the reinforcement stress of compression side in analysis of Navier's hypothesis using upper limit strength value</td></tr>
<tr><td>Beam-column joint</td><td>Shear failure after beam yielding</td><td>Additional ratio for shear strength*</td><td>Additional ratio for shear strength * × (value of (upper limit strength / specified yield strength)) for the adjoining yielding member</td></tr>
</table>

* Additional ratio for shear strength: ref.(2), pp250-253

The effects of A) and B) are automatically counted in the analyses. Therefore, the amplification should consider the other three factors. The amplification of factor C) is adopted in vertical members in case of the static analyses. The amplification coefficient is determined by comparing with the static frame analyses using specified yield strength and dynamic frame analyses using specified yield strength. The dispersion of factor D) is mainly that in the beam flexural strength formula, and the ratio of past experimental values and calculated values (equation (1)) is 1.11, so the coefficient is fixed to be 1.1.

$$M_b=0.9A_t \cdot \sigma_y \cdot d \tag{1}$$

Where,

M_b : Ultimate bending moment

A_t : Area of reinforcement at tension side

σ_y : Actual yield strength of reinforcement

d : Distance from extreme compression fiber to centroid of longitudinal tension reinforcement

The amplification of factor E) is, for a column, an additional axial load, and its coefficient is determined from the ratio of beam shear force amplification estimated 1.25 (= ratio of reinforcement strength (= value of upper limit yield strength / specified yield strength)) and from the ratio of axial load amplification in vertical ground motion estimated 0.2.

2.4 Formula of Member Using High Strength Material

The range of specified concrete compressive strength is from 24 N/mm^2 to 100 N/mm^2, and the upper limit of specified reinforcement yield strength is 685MPa. Most formulas are adopted from formulas those given in ref.(1), but the upper limit of concrete compressive strength in the original formula in ref.(1) is 60 N/mm^2. In this study, we inspected whether the formula could be an underestimate of past experimental values. Several formula which could not underestimate them were modified to be underestimates by the authors.

3. EVALUATION METHOD OF GROUND MOTIONS

3.1 Ground Motions for Dynamic Analyses

An evaluation method of ground motions for dynamic analyses is proposed. If the characteristics of both earthquake activity and ground conditions around the site is clarified, and if prediction is highly accurate, it is possible to estimate the ground motions of seismic design from a definite simulation of the earthquake motion based on the fault model. But in the existing studies, it is difficult to determine the ground motions from a definite method. On the other hand, probability and statistical method for evaluating the ground motion is not certain, because there the historic earthquake data period is too short to predict long recurrence interval earthquakes, there are few data on M8 earthquakes, etc. So, we decide the ground motion from both the definite method and probability and statistical method.

The flow chart to decide on the ground motions used in dynamic analyses is shown in Figure 1. The ground motion level should be decided from its probability of excess in a building service period. In this study, the probability of exceeding the serviceability limit state, the restorability limit state and the ultimate limit state are fixed 80%, 20% and 5% in the service period.

3.2 Evaluation based on Probabilistic or Statistical Method for Ground Motions

The recording term of past earthquakes is not long enough to decide the ground motion for the ultimate limit state, but it is possible to decide the ground motions for the serviceability limit state and restorability limit state. The ground motions based on past earthquake data are made, after deciding uniform hazard spectrum at bedrock, by considering the amplification characteristics of the ground at the site. The ground motion for the serviceability limit state is represented an artificial wave fitting uniform hazard spectrum in 50 years at the input point of the building. The ground motion for the restorability limit state is made from standard waveforms of ground motion and maximum velocity of that is the largest one chosen from uniform hazard spectrum at the foundation bed of the building and 50 cm/sec that is generally used. Three waveforms of ground motions are chosen from past standard records (El-Centro 1940NS, Taft 1952EW, Hachinohe 1968NS etc.) and recorded waveform at the site.

3.3 Evaluation Based on Fault Model

In case of ground motions from the definite method, the ground motions, assuming a definite fault from past earthquakes and earthquake forecasted to occur in the near future, should reflect the dynamic characteristics of the ground at the site. These ground motions should be used for restorability limit state and ultimate limit state. In a inter-plate earthquake, the judgment as to which state to use is based on recurrence interval and destroy

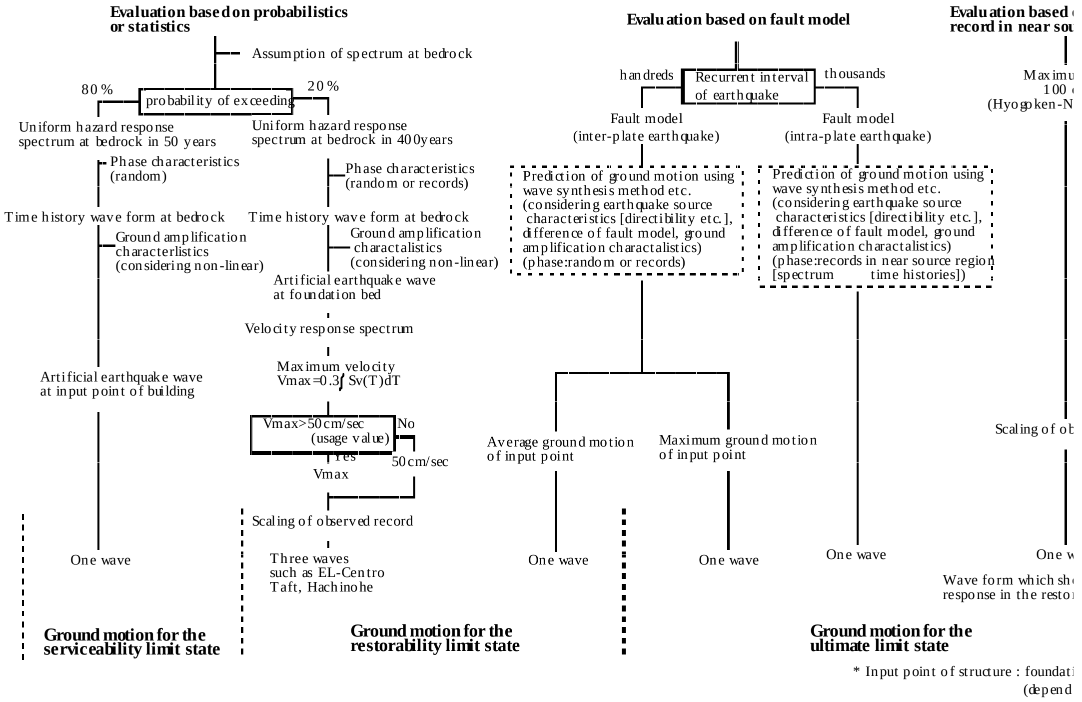


Figure 1: Flow chart to decide on the ground motions in dynamic analyses
(service period of building : 100 years)

type of fault, etc. But dispersion in forecasting of ground motions is wide, therefore, multiple forecasts of ground motion should be calculated, and its average ground motion is used for the restorability limit state, and its biggest ground motion, whose recurrence interval is very long, is used for the ultimate limit state. On the other hand, an intra-plate earthquake is used for the ultimate limit state, for the recurrence interval of an intra-plate earthquake is generally considered to be more than 1000 years. Making a time history waveform from the forecast spectrum, its form includes a short time, large-amplitude pulse wave, which is characteristic by the site. Further, the phase used for making the time history waveform must be selected to response most severely for the building.

3.4 Evaluation Based on Strong Ground Motion Record In Near Source Region

In the Hyogoken-Nanbu earthquake, the maximum velocity of ground motion indicated about 100cm/sec. As investigation and study on the fault are not complete, The standard ground motion by 100 cm/sec is used as one of the ground motions for ultimate limit state based on observed data. The waveform is the earthquake record which shows the maximum response in the restorability limit state.

4. ANALYSES OF 60-STORY BUILDING FOR TRIAL DESIGN

4.1 Outline of the Building

The authors examined whether a 60-story class RC building could be designed using the proposed design method. The typical floor plan is shown in Figure 2, the framing plan and elevation in Figure 3, and the section of typical members in Figure 4. Standard story height is 3.1m, and the aspect ratio is less than 4.0. The structural system is composed of a frame structure and mega-frame structure made of mega-beams and mega-columns. Mega-frame, both of two directions, consist of two planes of structure, and one plane of the structure is composed of 2-story, 1-span frame which consist of two mega-columns and two mega-beams. The applicable range of concrete compressive strength is 100 N/mm^2, and that of main reinforcement yield strength is 685 N/mm^2. The building data are shown in Table 4.

4.2 Analytical Method

The 3-D nonlinear seismic response frame analysis program DREAM-3D developed by Obayashi Corporation was used. Lateral loading directions are the X-direction and Y-direction. Bending moment – plastic rotation angle of beam-ends is calculated by a rigid plastic spring model of beam-ends. Two-directional bending deflection and axial deflection considering axial force fluctuations are calculated by a multi-spring model (MS model) of columns-ends. The analytical model of mega-beam and mega-column is shown in Figure 5. The relationship of story shear force (Q) and story deflection (δ) is modeled as a tri-linear from a monotonically

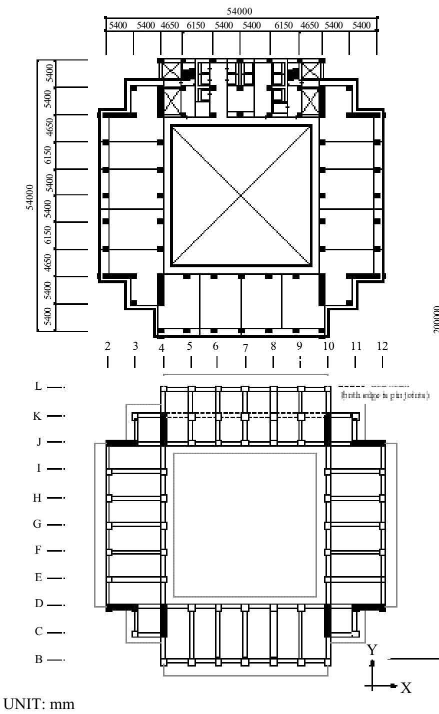


Figure 2: Typical floor plan

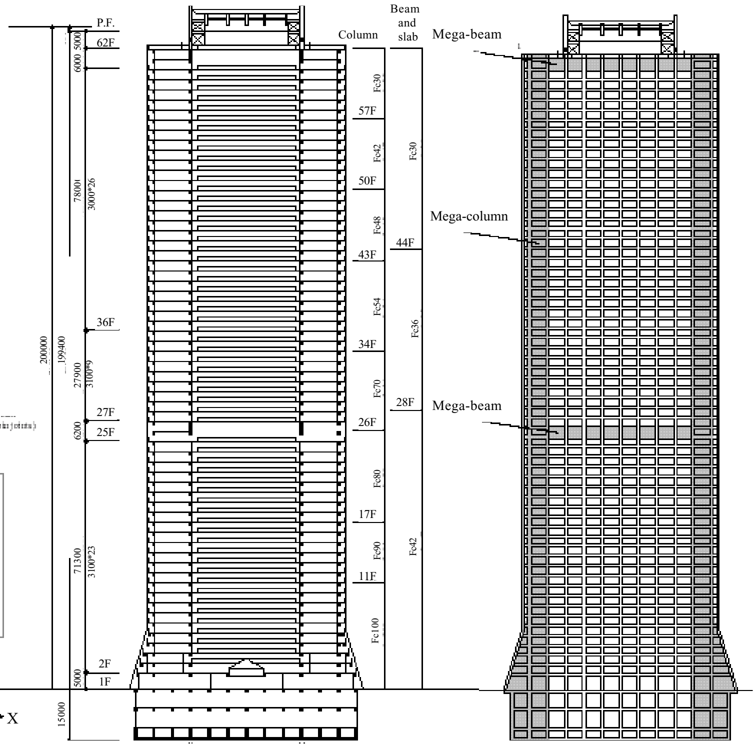


Figure 3: Framing plan and Elevation

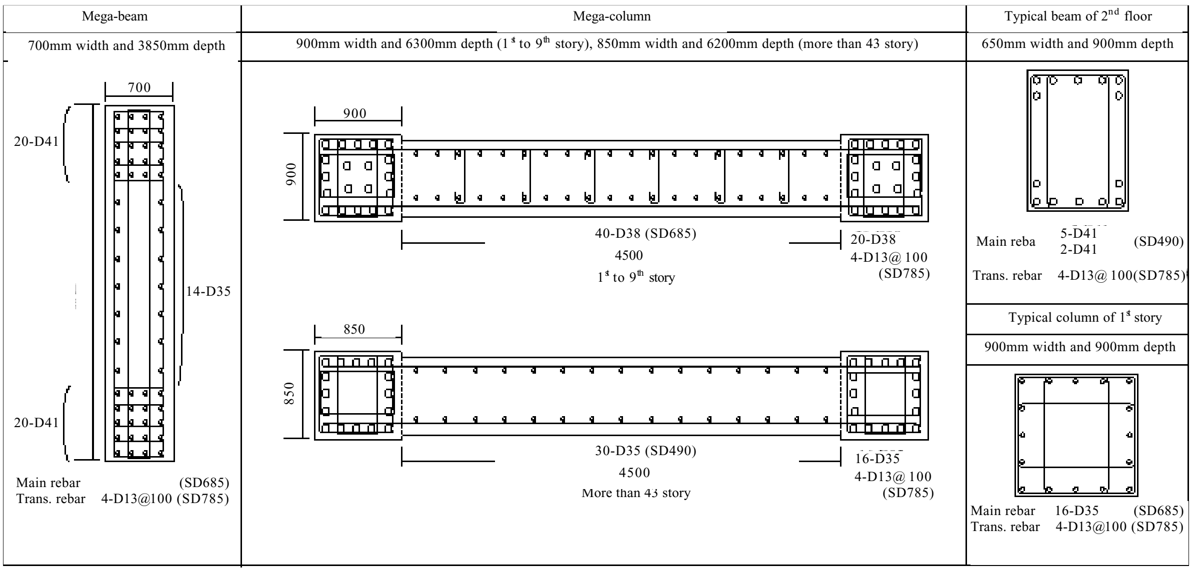


Figure 4: Section of typical members UNIT: mm

Table 4: Building data

Total floor area	99,200m^2
Number of story	64-story above the ground and P.H. 1 story
Eaves height	200m
Dwelling unit number	794
Rentable floor area ratio	72.9 %

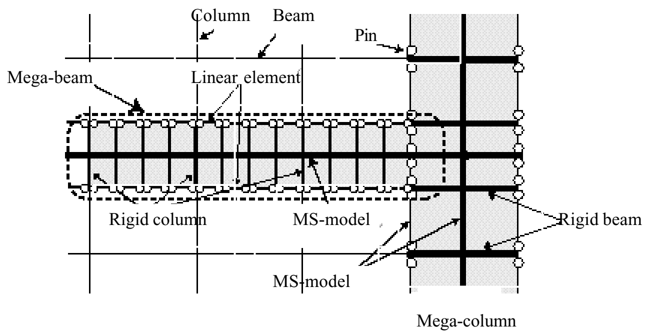


Figure 5: Analytical model of mega-beam and mega-column

Table 5: The ground motions for analyses

		Serviceability	Restorability	Ultimate
Artificial ground motions Three wave 1)	Maximum acceleration(gal) Maximum velocity(cm/sec)	180 14.2	288 41.2	471 69.5
Well-known ground motions Six waves 2)	Maximum velocity(cm/sec)	-	50	100 3)

1) Ground motion for the serviceability is historical earthquake,
ground motions for the ultimate are trench case models
2) Well-known seismic waves (NewRC1-3,Hachinohe NS, Taft, EL-Centro)

Table 6: Natural period

Mode number	1	2	3
X-direction	3.89	1.49	0.92
Y-direction	3.84	1.48	0.92

UNIT: sec.

Table 7: Base shear of maximum response

(a) Artificial seismic waves

		NewRC1	NewRC2	NewRC3	EL-Centro	Hachinohe	Taft
X-direction	50cm/sec	8360 (0.054)	8360 (0.054)	9750 (0.063)	8320 (0.054)	7900 (0.051)	7070 (0.046)
Y-direction	50cm/sec	9400 (0.061)	8420 (0.054)	8520 (0.055)	8420 (0.055)	8010 (0.052)	7220 (0.047)

( ) base shear coefficient

(b) Well-known seismic waves

	Serviceability	Restorability	Ultimate	
			Artificial	Taft
X-direction	3990 (0.026)	9960 (0.064)	10750 (0.070)	10650 (0.069
Y-direction	4080 (0.026)	9420 (0.061)	10870 (0.070)	10630 (0.069

loading analysis of Ai distribution. The distribution form of lateral loading is modeled for each story shear force to be more than the maximum response shear forces of each story in the dynamic analytical (multi-lumped mass Model) results of ground motions (six well-known earthquakes, two artificial earthquakes: maximum velocity changed to 25cm/sec). Each story Q-δ model is modeled to be tri-linear of the shear type from results of 3-D frame static nonlinear analysis. Hysteresis rule is Takeda model, and the damping is internal viscous type. Input seismic waves are three artificial ground motions and six well-known ones. The well-known ground motions for the restorability limit state are standardized to be 50cm/sec. Seismic waveform being standardized to be 100cm/sec for the ultimate limit state is the waveform which shows maximum response among the waveforms used in the restorability limit state analyses. The ground motions for analyses are shown in Table 5.

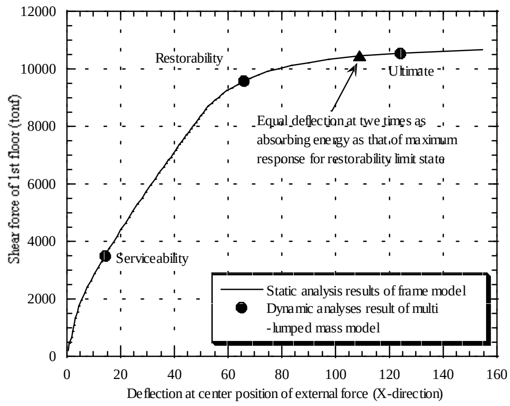


Figure 6: Lateral deflection at center position of external force in the X direction

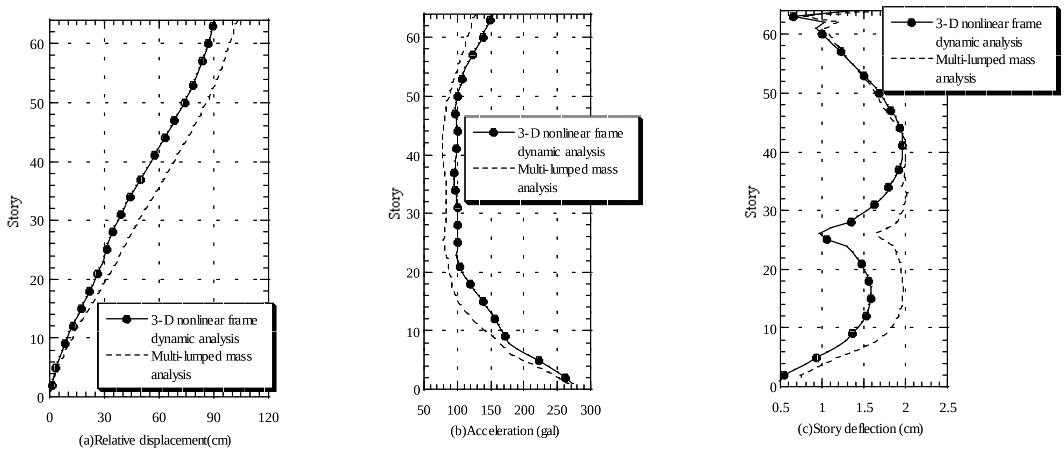
4.3 Analytical Results

Table 6 shows the natural period of the building in the X and Y direction. Table 7(a), (b) show base shear of maximum response. The shear force of artificial ground motions was largest in case of both the restorability and ultimate limit states. From static analysis, the relationship with story shear force of 1st floor and lateral deflection at the center position of external force in the X direction is shown in Figure 6. The marks ● in the Figure 6 are the same at lateral deflections of dynamic analysis for each limit state. Maximum value of ductility factor at the beams in the restorability limit state is 1.69, which is less than the criterion. Further, all members do not yield in the serviceability limit state, and the maximum value of ductility factor at the beams in the ultimate limit state is 2.88, which is less than the criterion too. Mega-columns in the tension side and some columns which join mega-beams yield, but the ductility factors are all less than the criteria. Figure 7 shows comparisons between 3-D nonlinear frame dynamic analysis and multi-jumped mass dynamic analyses at maximum values of relative displacement, absolute acceleration, and relative story displacement of the ground motions for restorability limit state. Figure 8 shows maximum values of relative displacement, absolute acceleration, and relative story displacement from multi-lumped mass dynamic analyses of the artificial ground motions. Each maximum relative story displacement does not exceed the criteria drawn in Figure 8(c).

Through the examinations on whether each member satisfied the criteria, it was confirmed that all members in all limit states satisfied the criteria shown in Table 2.

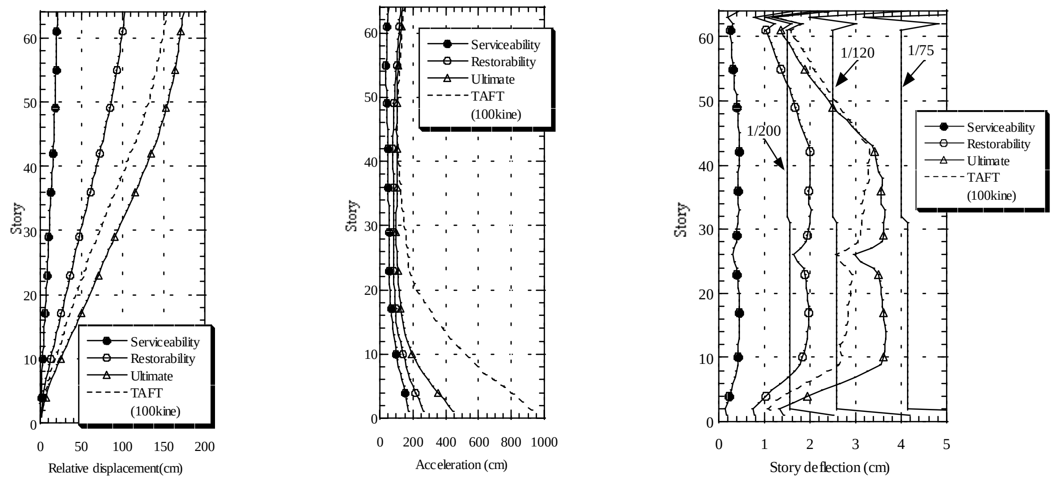
5 CONCLUSIONS

This paper proposed a method of evaluating ground motions of three levels for dynamic analysis of performance-based seismic design, and a seismic design method that is regulated the three performance levels. The seismic design method having definite criteria and detailed verification method is for 60-story reinforced concrete buildings. The trial design of a 64-story RC building composed of a frame structure and mega-frame structure showed that a 60-story class RC building could be designed, and that the proposed design method is reasonable and rational for ultra-high rise reinforced concrete buildings.



(a) Relative displacement (b) Acceleration (c) Story deflection

Figure 7: Comparisons between 3-D nonlinear seismic response frame analysis and multi-lumped mass seismic response analysis of the ground motions for the restorability limit state



(a) Relative displacement (b) Acceleration (c) Story deflection

Figure 8: Maximum values from multi-lumped mass seismic response analyses of the artificial ground motions

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