

# Analytical Simulation of Shake-Table Responses of a Torsionally-Eccentric Piloti-Type High-Rise RC Building Model

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## **ABSTRACT :**

A series of shaking table tests on a 1:12 scale model using scaled Taft N21E earthquake records were conducted to investigate the seismic performance of a 17-story high-rise reinforced concrete building structure with a high degree of torsional eccentricity and soft-story irregularities in the bottom two stories. The main characteristics of behaviors were: (1) The sudden change of the predominant vibration mode from the mode of translation and torsion to the torsional mode after the flexible side underwent large inelastic deformation. (2) The abrupt increase in the torsional stiffness in this change of modes. (3) The warping behavior of wall in the torsional mode. And (4) the unilateral overturning moment in the direction transverse to the table excitations.

In this study, efforts were devoted to simulate the above characteristics using a nonlinear analysis program, Perform3D. The advantages and limitations are presented with the nonlinear models available in this software as they are related to the correlation between analysis and test results.

**KEYWORDS:** torsional irregularity, reinforced concrete building, shaking table test, nonlinear dynamic analysis

## **1. INTRODUCTION**

Performance-based seismic evaluation and design has become practical with remarkable developments in the experimental and analytic seismic engineering research. In performance-based engineering, the estimation of the available capacity of structural components as well as of the whole structure is crucial to its successful implementation. Also, the demand on component forces and deformations by the expected earthquake ground motions should be reasonably predicted. This prediction or estimation is largely enabled by nonlinear dynamic or static analysis. Current techniques of nonlinear time-history analysis or static analysis are well developed. However, nonlinear analysis should be calibrated whenever possible as analysis reliability has a crucial role in performance-based engineering. The objective of this paper is to investigate the correlation between the results of previously conducted earthquake simulation tests of a 1:12 scale 17-story high-rise RC building model<sup>[1]</sup> and the nonlinear time-history analysis performed by using Perform $3D^{[2]}$ . The reason why the authors have chosen this software is that it has good pre- and post-process and a user-friendly environment, particularly with currently available rehabilitation guides for existing building structures such as FEMA  $356^{[3]}$  or ASCE/SEI 41- $06^{[4]}$ .

#### 2. EARTHQUAKE SIMULATION TESTS OF A 1:12 SCALE 17-STORY RC BUILDING MODEL

The prototype was selected based on an inventory study of multi-purpose high-rise buildings in Korea<sup>[5]</sup>. The structure consists of a lower 2-story 2-bay $\times$ 2-bay frame and upper 15-story wall system. This prototype building structure was designed for the earthquake load as follows:

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$$V = C_s \times W = \left(\frac{AI_E C}{R}\right) W = (0.048) \times (23,770) = 1,135 \text{ kN}$$
(2.1)

$$C = \left(\frac{S}{1.2\sqrt{T}}\right) = 0.868\tag{2.2}$$

$$T = 0.0488(h_n)^{3/4} = 0.920 \text{ sec}$$
(2.3)

where  $C_S$  is the seismic coefficient, W the effective weight of building, A the zone factor (0.11),  $I_E$  the importance factor (1.5), R the response modification factor (3.0), S the soil factor (1.0), and  $h_n$  the height of the structure. The structure was analyzed using an equivalent lateral force procedure according to the Korean Building Code 2000<sup>[6]</sup>. The fundamental period T was estimated using Eq. (2.3) for the other structures defined in this code. The overall dimensions of the prototype and details of columns and wall are shown in Figure 1 and Figure 2 respectively.

The lower frame has an infilled shear wall at one of the exterior frames. Herein, the exterior frame containing the shear wall is defined as stiff frame and the other exterior frame as flexible frame. The prototype was evaluated regarding irregularities according to the Uniform Building Code (1997)<sup>[7]</sup>. The results in Table 1 show that the prototype has the stiffness, strength and torsional irregularities simultaneously at the bottom two stories.

#### 2.1. Model specimen and Earthquake Simulation Test Setup

After considering capacity of the available shaking table, the scale of specimen was determined as 1:12<sup>[8]</sup>. The adopted similitude requirements correspond to the modified replica model in Table 2. Accelerations, displacements, and local deformations such as plastic hinge rotations, shear deformation in walls, and elongation of the first-story columns were recorded using accelerometers and a displacement transducers in Figure 3. Custom-made load cells were installed at mid-height in all of the first-story columns to measure shear and axial forces. The overview of the shaking table test set-up is shown in Figure 4. Detailed information on the experiment and corresponding interpretation of test results are given in reference [1]. The adopted table input accelerogram is Taft N21E component. The peak ground acceleration (PGA) of each run of the shaking table test is given in Table 3. Input earthquake records were compressed along the time.

run of the shaking table test is given in Table 3. Input earthquake records were compressed along the time axis by  $1:\sqrt{24}$  scale according to the similitude requirements in Table 2. Output table accelerograms were used as base input for the subsequent analytical simulation.

#### **3. ANALYTICAL MODELING**

#### 3.1. Introduction

Performance-based evaluation and design is now well advanced and is at the stage of being implemented into practice, particularly now that the rehabilitation of existing building structures has developed guidelines, such as FEMA 356 and ASCE/SEI 41-06. FEMA  $440^{[9]}$  provides detailed information on the practical application of nonlinear static analysis. Therefore, practicing engineers may have to use nonlinear analysis software more frequently than ever. However, generally, most of the codes or guidelines that allow for nonlinear static or dynamic analysis also require a peer review process, particularly when nonlinear dynamic analysis is adopted as a design basis. This implies that nonlinear dynamic analysis must fully consider the structural and member modeling and applied earthquake input motions. Blind use of these procedures could lead to unsafe and unreasonable design. Perform3D requires the input of data through input windows including the definition of nodal points, structural elements, and final load applications. Figure 5 shows the entire structural model.

General relationships between the action and deformation of structural components in Figure 6 conform to the behavior model given in seismic rehabilitation guides such as FEMA 356. The important points, Y, U, L,



R, and X are defined as follows:

- Y : The first yield point (where significant nonlinear behavior begins)
- U(1) : The ultimate strength point (Initiating point of perfect plasticity)
- L(2) : The ductile limit point (Initiating loss of strength)
- R(3) : The residual strength point (Initiating yielding after loss of strength)

X : Point of deformation leading to final collapse

In this study, a trilinear model consisting of only Y, U, and X was adopted. U, L, R are sometimes denoted as 1, 2, and 3 in this program. Each structural member consists of several components modeled to reflect their actual linear and nonlinear behavior in experiments and in responses to actual earthquakes.

#### 3.2 Modeling of Beams

Beams are composed of three types of components including a stiff end zone, an elastic beam element in the middle part, and a plastic hinge representing most of inelastic deformations, as shown in Figure 7.

A plastic hinge has no length except for one spring. The relationship between the hinge's rotation and acting bending moment is modeled in Figure 8. The stiffness in the joint region was assumed to be 10 times the stiffness of the middle part of the beam.

#### 3.3 Modeling of Columns

Columns were modeled in the same way as the beam except that the end of column connected to the foundation is assumed to have no stiff end zone. The behavior of the plastic hinges is defined by the yield surface in Figure 9. According to the reference [10], this surface can be represented using three points, compressive yield point PC, tensile yield point PT and compressive strength and yield moment at balanced failure point PB. Applied values for PC, PT, and PB are shown in Table 4 with the corresponding values in the prototype.

The shape of yield surface is defined by Eq. (3.1) and (3.2). The values of shape parameters  $\alpha$ ,  $\beta$ , and  $\gamma$  needed in these equations are given in Table 5.

$$\left(\frac{M_2}{M_{YP2}}\right)^{\gamma} + \left(\frac{M_3}{M_{YP3}}\right)^{\gamma} = 1$$
(3.1)

$$\left(\frac{P-P_B}{P_{Y0}-P_B}\right)^{\alpha} + \left(\frac{M}{M_{YB}}\right)^{\beta} = 1$$
(3.2)

Where  $M_{yp2}$  and  $M_{yp3}$  are yield moments about major axis 2 and 3 respectively and  $P_{y0}$  is  $\frac{1}{2}(PC - PT)$ .

The action – deformation relationship at the states of PT, PC, and PB can be characterized by determining the deformations DU and DX, corresponding to points U and X in Figure 6 in Table 6.

#### 3.4 Modeling of Stiffness Degradation

The applied values of input parameters are given in Table 7. These values were determined through trial-and-error procedure, thereby simulating the stiffness degradation of the experimental results most closely.

## 3.5 Modeling of Walls

The element models, general wall, shear wall and infilled wall are available in the program. In this study, the wall was modeled as infilled between the boundary columns. Action and deformation in this model are defined in Figure 10 and an elastic perfect plastic model was adopted with initial elastic stiffness, yield strength, and maximum deformation of 40kN/mm, 80kN, and 5mm, respectively.



#### 3.6 Implementation of Time History Analysis

Gravity load analysis was conducted before time history analysis. During 78 seconds of earthquake motion initiating from Taft030 and leading to Taft120, as shown in Figure 11, the model was analysed through the step-by-step procedure. Each base input excitation was the same as the output of table motion in each test and separated with neighbor input sufficiently so that there would be no inertial forces when a new round of analysis begins. However, since the analysis was continuously conducted for the whole series of input motions, the damage caused by the preceding run could be taken into account in the subsequent run of analysis. Time step was determined as 0.002 seconds and output was obtained for every five steps in order to avoid excessive amounts of data.

#### 4. CORRELATION BETWEEN EXPERIMENT AND ANALYSIS

#### 4.1 Global Structural Actions

Seismic coefficients obtained from tests appear to be generally less than those from the analysis, as shown in Figure 12. The fundamental period measured through a white-noise test after Taft022 was 0.188 seconds, with the same estimate reached using the empirical Eq. (2.3). The three natural mode shapes obtained by analysis are shown in Figure 13 with the corresponding periods. In this figure, the second mode shape and corresponding period match the fundamental mode shape and period of the experiment. The correlation of experiment and analysis regarding the case of Taft080 will be discussed hereon while the case of Taft030 will be addressed wherever appropriate.

The time histories for the analysis and experiment of base shear, overturning moment, and torsional moment are compared in Figure 14 (a), (b), and (c). Analytical results correlate very well with experimental behavior regarding base shear and overturning moment. In Figure 14 (c), MT indicates the total torsional moment whereas MT-P denotes the torsional moment contributed by the frames parallel with the direction of excitations. Therefore, the difference between MT and MT-P is the contribution to the torsional moment by the frames transverse to the direction of excitation. This contribution amounts to 50~100% in experiment, whereas in analysis it is almost negligible.

#### 4.2 Time Histories of Bent Base Shear in Stiff and Flexible Frames

Time histories of bent base shear in stiff and flexible frames are compared for Taft030 and Taft080 in Figure 15(a) and (b). Under Taft030, the structure behaved in the elastic range and both stiff and flexible frames have base shears almost all along the time in the same direction. In other words, the translation-and-torsional mode (the second mode) prevailed. The analysis shows good correlation with the experimental results. Also, under Taft080, the translation-and-torsional coupled mode (the second mode) governs initially for a short time, but with inelastic deformation and large stiffness degradation, which will be discussed in 4.4, the torsional mode (the third mode) prevailed in experiment. The analysis describes these phenomena very well. However, the amount of base shear in the stiff frame in analysis tends to be higher than in experiment.

#### 4.3 Global Deformation

The upper portion of the experimental model was constructed as a concrete box with steel plates added to satisfy the mass similitude requirement. The rigidity of the upper wall system is much higher than that of the lower frame structure. Therefore, the global deformation of the whole model can be described with three major deformations: shear, overturning, and torsional deformation of the lower frame defined in Figure 16. Figure 17 compares the experimental and analytical time histories of these global deformations under Taft080. The trends of the experimental results are well simulated by the analysis. It is noted also that the maximum amount of deformation in analysis tends to be smaller than in experiment.

## 4.4 Hysteretic Curves of Bent Base Shear and Lateral Displacement at the Top of Flexible Frame

Experimental and analytical hysteretic relationships between bent base shear and lateral displacement at the top of flexible frame are compared in Figure 18(a) and (c), for Taft030 and Taft080. In Figure 18(c), the analysis shows the stiffness degradation as the intensity increased from Taft030 to Taft080. In Figure 18(b)



and (d), analysis describes the excursion into the inelastic range and the gradual stiffness degradation throughout from Taft030 up to Taft120.

#### 4.5 Interrelationship between Global Deformations

Global deformations defined in Figure 16 have some relationships among themselves demonstrating the governing mode of vibrations. To investigate the correlation between experiment and analysis in this regard, Figure 19(a) compares the experimental and analytical time histories of shear deformation at stiff frame( $\theta_{1,stiff}$ ) and overturning deformation( $\theta_2$ ). The analysis simulates very well the experiment in that the overturning behavior is phasing with the shear deformation in stiff frame. Also, the shear deformation in flexible frame( $\theta_{1,flex}$ ) and the torsional deformation( $\theta_3$ ) are always in phase regardless of any governing mode of vibration in Figure 19(b). This phenomenon is well described in the analysis. However, it can be clearly noted in Figure 19(c) that the mode is governed by the relationship between overturning deformation( $\theta_3$ ). Initially  $\theta_2$  and  $\theta_3$  are in phase under the translation-and torsion coupled mode, but afterwards these deformations become out of phase under the torsional mode. The analysis clearly this phenomenon in the experiment.

#### 4.6 Hysteretic Relationship Between Torsional Moment vs. Torsional Deformation

Figure 20 shows the hysteretic curves between torsional moment and deformation under Taft080. The initial and later portions of these hysteretic curves have a much flatter slope (about 1200kN·m/rad) when the translation-and-torsion coupled mode governs, than the middle portion of the curves (4~6s) with approximate slope being 4500kN·m/rad under torsional mode. The analysis clearly shows this change of torsional stiffness as the governing mode of vibration changes.

#### 4.7 Interaction Between Base Shear and Base Torsional Moment

*De la Llera and Chopra*<sup>[11, 12]</sup> used an interactive diagram between base shear and torsional moment to clearly show the three-dimensional collapse caused by simultaneous application of shear and torsion. Figure 21 shows this interactive hysteretic diagram for the Taft030 in linear elastic range. The slope in hysteretic curves between torsion vs. shear in the experiment is a little larger in the positive direction than in analysis. Figure 22, however, shows that the governing slope in hysteretic curves under Taft080 changes abruptly around 3 seconds from the positive to negative, and then returns to the initial slope in analysis. Both the experiment and analysis show that the maximum damaging combination of base shear and base torsional moment can be in translation-and-torsion coupled mode or torsional mode. Though the experimental hysteretic curves look more chaotic, the analysis describes this interaction between base shear and base torsional moment very well.

## 4.8 Warping Behavior of Wall in Torsional Mode

The time histories of column elongations,  $\Delta_{Col1}$  and  $\Delta_{Col2}$  in Figure 23(a), show that Column 2 had a deformation of the opposite sign to that of Column 1, during the translation-and-torsion coupled mode, while it remained elongated by a small amount during the torsional mode of vibration in the experiment. The initial and latter portions of the response in time histories reveal the flexural behavior of the wall whereas the middle portion, around t = 5 seconds, represents the warping phenomenon due to torsion. However, the analysis could not simulate these phenomena at all, as shown in Figure 23(b).

#### 4.9 Overturning Moment Contributed by Axial Forces in Columns

Axial forces were measured in seven out of nine columns marked with circles in Figure 24(a). The contributions of the measured axial forces to the overturning moment under Taft030, with the response of the



model being in the linear elastic range, are shown in Figure 24(a). It is interesting to note that while the overturning moment in the direction of excitation is reversed cyclic, in the direction transverse to the excitation it is not reversed cyclic but has unilateral bias in one direction. This bias of the overturning moment in the transverse direction could not be simulated in analysis as shown in Figure 24(b).

Performance-based seismic design and evaluation have now become a norm throughout the world. This new and comprehensive design and evaluation procedure could have been successful only if reliable prediction or estimation of supply and demand in member force and deformations were possible. Engineers should have a reliable tool of analysis to predict the inelastic and failure stage of the structures.

The objective of this study is to calibrate the input parameters for nonlinear analysis and to check the reliability of one of available nonlinear dynamic analysis program, Perform $3D^{[2]}$ . For this purpose, the experimental data obtained through earthquake simulation tests of 1:12 scale 17-story RC building model<sup>[1]</sup> were used.

The findings from this investigation are as follows:

(1) The base shear, overturning moment, and torsional moment of the whole structure were simulated very closely to the experimental results. However, the torsion contributed by the frames transverse to the direction of excitation was almost negligible in analysis while that was up to  $50\sim100\%$  of the total torsion in experiment.

(2) The bent base shear in stiff and flexible frames could be reliably simulated in analysis in both elastic and inelastic ranges of behavior. Generally, the value of the bent base shear in stiff frame were larger in analysis than in experiment. The phase of base shear in stiff and flexible frames could be well simulated by analysis showing the change of the governing mode of vibrations.

(3) The deformation of the whole structure could be described with shear deformation in flexible and stiff frames ( $\theta_{1,flex}$  and  $\theta_{1,stiff}$ ), overturning deformation( $\theta_2$ ), and torsional deformation( $\theta_3$ ) in the lower frames. The analysis shows good correlation with the experiment with respect to these global deformations.

But, generally, the amount of deformation in analysis tends to be smaller than that in the experiment.

(4) The hysteretic relationship between bent base shear and the lateral displacement at top of the lower flexible frame showed large inelastic excursions and stiffness degradations in experiment. These phenomena could be reasonably approximated in analysis.

(5) The experiment showed that  $\theta_{1,stiff}$  follows the phase of  $\theta_2$ , whereas  $\theta_{1,flex}$  is in phase with  $\theta_3$ , throughout the response regardless of the governing mode of vibration. The governing mode of vibration could be clearly recognized by observing if overturning deformation,  $\theta_2$ , and torsional deformation,  $\theta_3$ , is in

phase or out of phase. The analysis simulated the relations between  $\theta_{1,stiff}$  and  $\theta_2$ , and between  $\theta_{1,flex}$ 

and  $\theta_3$  and change of governing modes very well.

(6) The change of the governing modes throughout the response caused changes in the torsional stiffness. This phenomenon could be reliably simulated by analysis.

(7) A Base-Shear-Torsion Diagram could be useful in checking the governing mode of vibration leading to the collapse. The governing mode of vibration can be clearly recognized by observing the governing direction of curves in this diagram. The translation-and-torsion coupled mode shows a flat positive slope while the torsional mode reveals a relatively large negative slope in the diagram. The analysis approximated these behaviors reasonably.

(8) In the experiment two boundary columns in the wall showed elongation and shortening due to flexural behavior of the wall under the mode of translation and torsion, but elongations due only to warping behavior under the torsional mode. However, the analysis could not simulate this behavior at all.

(9) The overturning moment transverse to the direction of excitation was found to be unilateral in one direction in experiment. However, this bias in overturning moment could not be simulated at all in analysis. On the contrary, the analysis shows reversed cyclic overturning moment in the direction transverse to the direction of excitation.



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Table 1 Evaluation of Irregularity for Prototype					
Irregularity	Criteria	Evaluation at lower stories			
Stiffness irrelgularity	$\frac{k_i^*}{k_{i+1}} < 0.7, \frac{\Delta_i^*}{\Delta_{i+1}} > 1.3$	$\frac{k_2}{k_3} = 0.43 < 0.7, \frac{\Delta_2}{\Delta_3} = 4.65 > 1.3$			
Discontinuity in capacity	$\frac{F_{1}^{\dagger}}{F_{i+1}} < 0.8$	$\frac{F_2}{F_3} = 0.40 < 0.8$			
Torsional irregularity	$\frac{\delta_{\max}^{\$}}{\delta_{avg}^{\$}} > 1.2$	$\frac{\delta_{\max}}{\delta_{avg}} = 1.72 > 1.2$ (at transfer floor)			

Story lateral stiffness at *i*th story

<sup>†</sup> Story lateral capacity at *i*th story

<sup>‡</sup> Interstoy drift at *i*th story

§ Maximum and average displacement at the critical floor

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Table 2 Similitude Law				
Item	Dimension	True replica model	Modified replica model	
Length, l	L	1/12	1/12	
Area, A	$L^2$	1/144	1/144	
Mass, M	M	1/144	1/288	
Force, F	$MLT^{-2}$	1/144	1/144	
Acceleration, $\ddot{x}$	$LT^{-2}$	1	2	
Frequency, f	$T^{-1}$	$\sqrt{12}$	$\sqrt{24}$	
Time, t	Т	$1/\sqrt{12}$	$1/\sqrt{24}$	

# able ? Similitude

## Table 4 Values of Parameters, PC, PT, and PB

	Point	Prototype (kN, kN∙ mm)	1:12 Model (kN, kN· mm)	
Column	PC	(20,192, 0)	(140, 0)	
	PT	(-4,080, 0)	(-28, 0)	
	PB	(6,862, 2,511,000)	(48, 1, 453)	
Corner Column	PC	(23,096, 0)	(160, 0)	
	PT	(-7,140, 0)	(50, 0)	
	PB	( 6,809, 3,165,000)	(47, 1,832)	
Girder	Bending	( 0, 362,800)	( 0, 210)	

#### Table 6 Action-Deformations in Columns

State of Actions	PT	PC	PB
Deformation Type	Elongation (mm)	Shortening (mm)	Plastic Rotation (rad)
Deformation DU	2	0.5	0.05
Deformation DX	10	5	0.1



Figure 1 Prototype building (unit : mm): plan and elevation

Table 3 Test Program				
	PGA(g)			
Test	Model	Prototype	Remark	
Taft022	0.22	0.11	Design earthquake (IE=1.0)	
Taft030	0.3	0.15	Design earthquake (IE=1.5)	
Taft040	0.4	0.2		
Taft060	0.6	0.3		
Taft080	0.8	0.4	Design earthquake in a highly seismic region	
Taft120	1.2	0.6	Maximum considered earthquake in a highly seismic region	

#### Table 5 Values of $\alpha$ , $\beta$ , and $\gamma$

PB				
Column	Corner Column	α	β	γ
0.34	0.29	2	1.1	1.4

#### Table 7 Cyclic Degradation

	Column		Beam		
	Axis 2, 3 Deformation (Rad)	Energy Factor	Deformation (Rad)	Energy Factor	
Y	-	0.35	-	0.4	
1	0.005	0.25	0.005	0.3	
2	0.01	0.2	0.02	0.15	
3	0.04	0.15	0.04	0.1	
Х	-	0.05	-	0.05	



Figure 2 Prototype Building (unit : mm) columns and wall



Figure 3 Experimental Arrangement (unit : mm): front view, side view, plan, and instrumentation for wall and columns

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Figure 4 Overview of Shaking Table Test Setup



Bending Moment (kN.mm)

210

200

-0.02

Figure 5 Overall Analytical Model Figure 6 Action-Deformation Relationship

0.06

Plastic Rotation Angle (rad)



Figure 7 Beam Components

Figure 8 Plastic-Hinge and Relationship of Bending Moment and Plastic Rotation Angle

-0.06





Zero length

) м

Jθ

м ( -

Figure 9 Concrete Type PMM Yield Surface



Figure 11 Input Base Accelerograms used for Analysis



1st mode(0.15s) 2nd mode(0.13s) 3rd mode(0.08s) Figure 13 Mode Shape and Natural Period (Analysis)



0.02

-200 -210

Figure 10 Infill Panel Shear Model



Figure 12 Natural Period & CS





Figure 14 Global Structural Force Response: (a) base shear; (b) overturning moment; and (c) torsional moment



(b) Taft080 Figure 15 Time Histories of Bent Base Shear



Figure 16 Definition of: (a) shear deformation ( $\theta_1$ ); (b) overturning deformation ( $\theta_2$ ); and (c) torsional deformation( $\theta_3$ )



Figure 17 Comparison of Time Histories of Shear, Overturning, and Torsional Deformations between Analysis and Experiment





Figure 20 Relation Between Torsional Moment and Deformation (Taft080)









