Analysis and Distributed Hybrid Simulation of Shear-Sensitive RC Bridges Subjected to Horizontal and Vertical Earthquake Ground Motion

by

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ABSTRACT

The paper is concerned with the assessment of the effect of vertical earthquake ground motion on the shear capacity and imposed demand on RC bridge piers. Two specific aspects are studied, namely the effect of difference in arrival time between vertical and horizontal ground shaking, and the ratio of peak vertical to horizontal acceleration. A bridge damaged in the 1994 Northridge earthquake is selected as a prototype structure for analytical and experimental investigations. For the analytical investigation, various vertical and horizontal peak ground acceleration ratios and arrival time intervals are considered and results are compared with the case of horizontal-only excitation. Plans for testing using distributed hybrid (experimental -analytical) simulations are outlined and a pilot test is described. It is conclusively observed that the effect of vertical ground motion on the measured response is significant.

KEYWORDS: vertical earthquake motion, distributed hybrid simulation, RC bridge piers, shear failure.

1. INTRODUCTION

In recent years, moderate-to-large magnitude earthquakes, e.g., the Loma Prieta (1989) and Northridge earthquakes (1994) in California and the Hyogo-ken Nanbu earthquake (1995) in Kobe, Japan, have caused significant damage to RC bridges. In these past earthquakes, shear behavior of concrete piers is one of the major causes of damage. Previous investigations (e.g., Papazoglou and Elnashai, 1996), have attributed the observed failure to the reduction of shear strength caused by vertical ground motion effect. In the mean time, modern codes neglect or underestimate the effect of vertical ground motion, to the detriment of structures especially in the vicinity of active faults. Many studies report data showing that the vertical peak acceleration may be even higher than the horizontal value. Examples of the latter studies are by Abrahamson and Litehiser (1989), Ambraseys and Simpson (1996), Elnashai and Papazoglou (1997), Collier and Elnashai (2001), Elgamal and He (2004). Moreover, dependence of response on the arrival time (coincidence or otherwise) of peak vertical and horizontal ground motion is an important parameter that has not been investigated. It is likely that the above ground motion features are dependent on the source distance, earthquake magnitude, travel path, and site condition. In this paper, the vertical-to-horizontal peak ground acceleration ratio (V/H) and time interval between the arrival of vertical and horizontal large amplitude acceleration cycles are the primary focus of the study of the effect of vertical ground motion on shear response of RC bridge piers.

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Piers of bridges and building columns are subjected not only to the axial actions due to dead and live loads but also to combined varying axial force, moments and shear under earthquake load. Since axial load affects shear and moment capacity of reinforced concrete elements, failure analysis should carefully consider input motion components. The combined effect of over-turning and multi-axial input leads to significant variation in axial loads on columns, leading to changes in the balance between their supply and demand in axial, moment and shear that do not lend themselves to prediction by simple models. Experimental investigations of the above-described factors are the best way forward. However, experiments are expensive and time-consuming, and should therefore be steered by extensive analysis. Also, laboratories are restricted by scale and capacity, especially when dealing with problems of even medium span bridges.

To overcome the above difficulties in testing, advanced distributed testing and hybrid testing-analysis methods, employing pseudo-dynamic techniques and sub-structuring are in increasing use, as described hereafter. By deploying the new approaches of combining tests and analysis, and by optimal use of advanced test control and advanced analysis, important effects such as the influence of axial force variation on the shear deformation and failure of RC members, especially bridge piers, can be investigated.

2. SHEAR STRENGTH MODEL

In this paper, the shear strength model proposed by Priestley et al (1994) is used. The model is simple to implement in FE analysis, agrees well with tests and assumes that the strength consists of three independent components as followings;

$$V_n = V_c + V_s + V_p \qquad (1)$$

where, V_c is the contribution of concrete shear resisting mechanism, V_s is the contribution of the truss mechanism provided by shear reinforcement and V_p represents the shear resistance of the arch mechanism, provided by axial force. V_c is given by;

$$V_c = k\sqrt{f_c^{'}}A_e \qquad (2)$$

where, $A_e (= 0.8A_{gross})$ is the effective shear area and k depends on the instantaneous displacement or curvature ductility factor. V_s is based on the truss mechanism using a 30° angle of the inclined flexure shear cracking. Thus,

$$V_s = \frac{A_v f_y D^{'}}{s}\cot 30^o \text{ (rectangular column)} \quad (3.a)$$

$$V_s = \frac{\pi}{2}\frac{A_v f_y D^{'}}{s}\cot 30^o \text{ (circular column)} \quad (3.b)$$

where, A_v is the total transverse reinforcement area per layer and D’ is the distance between centers of the peripheral hoop in the direction parallel to the applied shear force.

The shear strength enhancement by axial force is considered to result from an inclined compression strut, given by

$$V_p = P\tan\alpha = \frac{D-c}{2a}P \qquad (4)$$

Where, D is section depth or diameter, c is the compression zone depth which can be determined from flexural analysis, and a is the shear span which is L/2 for a column in reversed bending and L for a cantilever column.

3. VERTICAL GROUND MOTION

3.1 Nature of Vertical Motion

Characteristics of the vertical component of ground motion are significantly different from those of the horizontal component. The vertical component of ground motion is associated with the arrival of vertically propagating P-waves, while the horizontal component is caused by S-waves. Thus, the vertical component of ground motion has much higher frequency content than the horizontal component. The high frequency content leads to large amplifications in the short period range, which often coincide with the vertical period of RC members, thus causing large response values, especially with regard to forces, as opposed to displacements.

3.2 Ratio of Peak Accelerations (V/H)

The significance or otherwise of vertical components of ground motion is often characterized by the vertical-to-horizontal, V/H peak ground acceleration ratio. Many codes suggest scaling of a single spectral shape, originally derived for horizontal component and the average V/H ratio is taken as 2/3. This procedure was originally proposed by Newmark et al (1973). As a result, all components of motion have the same frequency contents in almost all design codes. The frequency content, however, is demonstrably different, as discussed in section 3.1 above. Also, the 2/3 rule for V/H is unconservative in the near-field. Many recent studies such as Abrahamson and Litehiser (1989), Ambraseys and Simpson (1996), Elgamal and He (2004), Bozorgnia and Campbell (2004) amongst others, provide evidence of the lack of conservatism of the 2/3 scaling factor. Moreover, such ratios are sensitive to the source distance and structural period. The 2/3 ratio is too low in near source areas and too high for structures at large distances. Selected records from with V/H in excess of 2/3 are given in Table 1.

3.3 Time Interval between Peak Vertical and Horizontal Ground Motion

The time interval between the arrival of peak vertical and horizontal motion is an important parameter for structural response. The latter parameter is dependent on the magnitude and source distance. Elnashai and Collier (2001) showed that the time interval should be taken as zero for a distance of 5 km from the source. The interaction between vertical and horizontal peaks has some significance within a radius of 25 km. The early arrival of the vertical motion may cause shakedown of the structure prior to the arrival of horizontal motion, thus affecting significantly the structural response. On the other hand, the coincidence of vertical and horizontal peaks would cause levels of distress in structural members that cannot be predicted by simplified methods. Therefore, inclusion of realistic input motion in both vertical and horizontal directions is necessary.

4. MODEL STRUCTURE FOR STUDY

4.1 Prototype Structure and Damage Description

The prototype structure for analytical and experimental investigation is selected as Collector-Distributor 36 of the Santa Monica (I10) Freeway, which was damaged in the Northridge earthquake of 17 January 1994. The Collector-Distributor 36 forms part of a pair of off-ramps from the eastbound carriageway on the I-10 freeway at La Cienega-Venice Under-crossing and was designed and constructed between 1962 and 1965. The structural configuration is shown in figure 1. The ramp was located about 25km to the south-east of the epicenter. From the bifurcation point just to the west of bent 5 of the Under-crossing, the RC ramp was carried first over the multi-column bent 5, then over three single piers (6, 7 and 8) and finally over the pier wall of bent 9 to the east abutment. The deck consisted of a 3-cell continuous box girder which was rigidly connected to the supporting structure 10.

There was no visible damage on either the ramp deck or the abutment. However, the piers experienced varying levels of damage. In particular, pier 6 experienced spectacular failure and was the most damaged of all the columns supporting the ramp 10. As shown in figure 2, shear failure occurred in the lower half of the pier. The concrete cover completely spalled over the height and the concrete core disintegrated. Moreover, all the reinforcement bars buckled symmetrically and the transverse hoops opened, leaving the pier with large permanent axial deformation. There is evidence that the collapse of this pier is partly attributed to the instantaneous reduction of shear strength caused by vertical motion and the resulting fluctuation of the pier axial load. The nearest strong-motion records suggest a maximum horizontal peak ground acceleration of up to 0.37g and peak vertical acceleration of up to 0.23g.

4.2 Model Structure

To utilize the current NEES experimental facilities and for simplification, the bridge is assumed to have three piers. The overall structural configuration is similar to the real structure as shown in figure 3. Masses are placed on the deck since the hybrid test will be conducted for the piers under static conditions while the dynamic response will be obtained from an analytical model of the deck. The consistent mass values are shown in table 2. The initial load which was calculated with deck self-weight applied to the top of piers are shown in table 3, while the pier section detailed are shown in figure 4. Prototype material properties are used and are as follows;

Concrete;

- Compressive Strength: 34.5MPa
- Tensile Strength: 1.94 MPa
- Crushing strain: 0.0025

Reinforcement;

- Yield strength: 413 MPa
- Ultimate Strength: 670.86 MPa

5. ANALYTICAL INVESTIGATION

Inelastic dynamic analyses were performed using the Mid-America Earthquake Center program Zeus-NL (Elnashai et al 2002). In this study, only the time intervals between vertical and horizontal peaks and V/H ratio are considered in the pre-testing analytical investigation.

5.1 Selected Ground Motion

To investigate the effect of vertical ground motion on the shear capacity of piers, the 6 earthquake records shown in the table 1 are selected. An analytical investigation is undertaken with varying V/H and time interval between vertical and horizontal peaks. The V/H ratios are considered in the range of 0.5 to 2.0 with increments of 0.1. The arrival time is considered in the range of 0.0 sec to 5.0 sec with 0.5 sec. increments.

5.2 Analyses with Original Records

The effect of the vertical component was assessed by comparing the elastic horizontal and vertical periods to the periods of horizontal and vertical vibration for each ground motion and their combinations. The fundamental periods of the analytical model are 0.269 sec horizontal and 0.077 sec vertical, from eigenvalue analysis. Table 4 shows that the horizontal and vertical periods of vibration increase significantly when the vertical ground motion is considered. Also, compared with the case that each component of ground motion is applied separately, periods are elongated. For instance, for the Northridge (Arleta Fire), the horizontal and vertical periods increase by 21% and 41%, respectively.

Table 5 summarizes the results of inelastic dynamic analysis with the original records. The table shows that the contribution to the axial force imposed on pier 1 by vertical ground motion increases significantly from 47% to 81%. In the case of the Kobe earthquake (Port Island Array, Figure 5), the axial force increased by 81%. Capacity and demand analysis indicate that except for the Kobe earthquake - Port Island - shear failure can be expected, as shown in table 6. Shear demand is slightly affected by vertical motion. However, the shear capacity decreases by 4.5%-16.3%. For Kobe record (Port Island Array), the shear demand exceeds the capacity only when the vertical ground motion is considered; in other words failure would occur only if vertical motion is included in the assessment. As shown in figure 6 and table 6, shear demand increases by 16.6 % whilst shear capacity decreases by about the same amount. The change in capacity and demand is caused by increase in axial force and its variation. Thus, if only horizontal ground motion is considered, the pier would be deemed safe.

5.3 Effect of V/H ratio on Shear Capacity

Due to the scarcity of viable earthquake records, the original records were parametrically manipulated. It is appreciated that the resulting signals do not represent the physics of any seismo-tectonic environment. However, the variability of records in general is such that scaling of records for V/H ratio is acceptable in the context of the current targeted investigation. For a fixed time interval and PGA of horizontal ground motion, 16 V/H ratios per earthquake record are considered in the range of 0.5 to 2.0 with an increment of 0.1. The results are compared with the result from horizontal ground motion analysis.

The effect of varying amplitude of vertical component on the periods of vibration was investigated. As shown in figure 7, the period is elongated for both components as the vertical amplitude increases. Here, zero for V/H ratio means that the vertical ground motion is not considered in the analysis. Although the data shows scatter due to input motion variation, it indicates that the slope of rate of period increase is steeper up to a V/H Ratio of 1.0. Figure 8 indicates that the variation of axial force and contribution of vertical ground motion to the axial force increase as the V/H ratio increases for all earthquake records. In particular, there is a significant increase for the Kobe record at Port Island. From figure 9, it is likely that there is no significant change of shear demand compared to response of horizontal ground motion only. The Kobe record shows that shear demand increases by up to 18%, while for Northridge records the demand decreases marginally. In contrast, shear capacity is reduced by 5%-36%.

5.4 Effect of Time Interval on Shear Capacity

The study of Collier and Elnashai (2001) indicated that horizontal and vertical ground motion peaks can be coincident when the distance from source is less than 5 km. Within 25 km from the source, the arrival time interval is less than 5 sec. Thus, in this paper, the 11 cases of arrival time intervals for each record are also studied in the range 0.0 to 5.0 sec with an increment of 0.5 sec, by shifting the horizontal record along the time axis. The original recorded V/H ratios are maintained throughout the arrival time increment study. The results are bench-marked versus the response under the horizontal ground motion only.

The effect of arrival time interval on the period of vibration was studied by comparing with the result from the case of the coinciding vertical and horizontal peaks. As shown in figure 10, it is difficult to determine the effect of arrival time interval on the dominant inelastic period due to the proximity of peaks in a normal Fourier Amplitude Spectrum plot. Thus, inelastic periods were evaluated from a moving widow Discrete Fourier Transform analysis. To overcome problems of discontinuities, the Hanning window method was used for each segment. Since these periods are obtained from each segment of data, the values are not strictly dominant periods, but rather they give a trend of period shift. Figure 11 indicates that the horizontal period is more elongated when the time interval is small. This effect is shown clearly when the 0.0 sec and 5.0 sec for time interval are compared.

As shown in figure 12, changes in arrival time interval have no noticeable effect on the axial force. Therefore, the fluctuation of axial force is mainly affected by the amplitude of vertical ground motion and not its arrival time. With regard to shear response (Figure 13), the demand is not significantly affected, while the capacity tends to increase slightly as the arrival time interval increases. For example, under the Northridge - Arleta Fire and Santa Monica – records capacity changes in the range of 5% to 20% and 3% to 10%, respectively, are observed.

The overall outcome of the brief analytical investigation discussed above is that in the vicinity of active faults, where V/H is likely to be high and the arrival time interval is likely to be zero or very short, shear capacity and demand assessment **must** take vertical ground motion into account.

6. PLAN FOR EXPERIMENTAL INVESTIGATION OF SHEAR SENSITIVITY

6.1 Introduction to the MUST-SIM Facility

In this section, to consider the effect of vertical ground motion on the shear capacity of RC pier, an experimental plan is introduced. The Multi-Axial Full-Scale Sub-Structured Testing and Simulation facility (MUST-SIM) at the University of Illinois at Urbana-Champaign will be used for the experimental portion of this investigation. MUST-SIM is one of the fifteen NEES experiment sites that provides distributed experimental-computational simulation capabilities to the earthquake engineering community. The MUST-SIM facility has many advanced features, including the following: i) 6-DOF load and position control at 3 specimen connection points, ii) Three dense non-contact measurement systems, iii) Data fusion and high end visualization capabilities. The facility is well-suited to run the Pseudo Dynamic (PSD) tests used for this shear sensitively investigation. The concept of using PSD testing for a bridge structure using the MUST-SIM facilities is shown in figure 14.

6.1.1 Reaction Wall

An important feature of the UIUC NEES experimental site is the large reaction wall, used for anchoring test specimens and loading devices. This L-shaped post-tensioned concrete strong wall of 15.2 × 9.1 × 8.5 × 1.5 m (length × width × height × thickness, respectively) enables testing of full scale sub-structures, as shown in Figure 15.

6.1.2 Load and Boundary Condition Boxes (LBCBs)

Through use of the three LBCBs that are part of the MUST-SIM facility, researchers can displace a test specimen in 6 DOF, easily applying combinations of shear, axial force, and moment (Figure 15). Each LBCB is a self-reacting assembly of actuators and swivel joints, with control software capable of imposing any combination of six actions (forces and moments) and six deformations (displacements and rotations) to test specimens connected to its loading platform. The LBCBs are capable of imposing motions on the test structures from the results of concurrently-running numerical models of the surrounding structure-foundation-soil system employing pseudo-dynamic and sub-structuring testing methods.

6.1.3 1/5th Scale Laboratory

A fully functional 1/5th scale laboratory includes 1/5th scale reaction structure with 1/5th scale LBCB (Figure 16) and dedicated servo-controllers. The 1/5th scale laboratory allows users with diverse research backgrounds to have full access to the MUST-SIM facility and to understand the capability and limitations of the facility. Also, the laboratory will provide the pre-test verifications before using the large scale facility.

6.1.4 UI-SIMCOR

Recognizing the need for a central control system for multi-site testing, the University of Illinois simulation coordinator, UI-SIMCOR was developed for multi-site substructure PSD test and simulation. During the development of this coordination system, the following key components were sought:

- Integration scheme for PSD tests.
- Communication amongst sub-structured components.
- Sub-structuring (sub-division) of the complex system.

One of notable advantages in UI-SIMCOR is that it allows all sub-structured components to be analyzed or physically loaded statically. The dynamic components of structural tests are contributed by UI-SIMCOR through a PSD algorithm. The α-Operator Splitting method is used as the integration scheme. Another significant advantage of the simulation coordinator is the ease with which it allows integrated response to be determined from numerous separate subdivisions of the overall system. Distant geographically distributed sub-structured components can be integrated and tested as a fully interacting system, allowing multiple laboratories to be used for large and complex tests.

6.2 Multi-Site Soil-Structure-Foundation Interaction Test (MISST)

The Multi-Site soil-structure foundation interaction test (MISST) will use the MUST-SIM, Lehigh University and Rensselaer Polytechnic Institute (RPI) facilities to investigate the effect of vertical ground motion and soil-structure-interaction on earthquake response of bridges. Due to the complexity of the system and the size and capacity requirements for testing, only component tests have been undertaken to date. The MISST simulation is intended to provide a framework for testing complex bridge systems including their underlying soil, and varying axial force.

The MISST structure is based on the Collector-Distributor 36 of the Santa Monica Freeway, described in previous sections of this paper. In MISST, two large scale NEES structural sites (UIUC and Lehigh University), one NEES geotechnical site (RPI) and computational simulation modules (NCSA) will be used in concert through coordinated sub-structuring to perform a PSD test on the entire bridge system. This five-site execution of MISST will use NEESgrid and UI-SIMCOR for communication and control.

6.2.1 Test Setup for Large Scale Bridge Simulation

The structure is subdivided into 5 static modules (Figure 17) as mentioned above. The dynamic characteristics of these components are accounted for in UI-SIMCOR. Two components are analytical models whilst the remaining three are experimental. The test components are:

- Module 1: Four decks and second pier including soil 2 – NCSA
- Module 2: First pier – UIUC
- Module 3: Third pier – Lehigh
- Module 4: Soil 1 – RPI
- Module 5: Soil 3 – NCSA

Due to load limitations at the experimental facilities, half scaled piers were designed for use at the UIUC and Lehigh sites. The applied similitude law and each scale factor are shown in table 7. This similitude law was verified by a comparison of push-over analysis results using Zeus-NL (Figure 18). The RPI scale is about 1:50.

6.2.2 Test Setup for Small Scale Test

Additional sub-structured tests will be performed utilizing the 1/5 scale MUST-SIM laboratory. The small scale testing will serve to determine the specific test parameters for the large scale and to verify the capabilities of the small scale facility. The structure will be subdivided as above and piers 1 and 3 will be experimentally tested at UIUC. The model piers will be 1/16 and 1/20 scaled representations of the prototype. Exact similitude cannot be fulfilled with the prototype structure due to difficulties in obtaining suitable reinforcing steel. Therefore, the small scale piers were designed to have similar axial-moment capacity when compared to the prototype. The relaxation of similitude requirements was deemed acceptable for the scope of the current project, where the investigation focuses on the difference in behavior between testing with and without vertical motion effects. Shear strength of the model piers will be controlled by stirrup spacing. Although several configurations will be tested, a representative 1/10 section is shown in figure 19 along with the small scale testing setup.

Wire commonly used in welded mesh reinforcement will be utilized for reinforcement in the model piers. Sizes D2.5 and W1.4 will be used for longitudinal and transverse reinforcement respectively. A micro-concrete mix will used to model the prototype concrete. Work is in progress to create a micro-concrete mix that can represent the compressive stress-strain relationship and tensile strength of the prototype concrete.

6.2.3 Test Plan and Expected Outcome

The effect of vertical ground motion and soil structure interaction on the bridge structure will be investigated experimentally through small and large scale test. The test specimens will experience the interaction of moment, lateral force and varying axial force under selected earthquake records from analytical models. It is anticipated that effects of SSI on the period elongation (but not radiation damping) will be accounted for. Moreover, shear deformation and failure will be represented. Finally, the effects of vertical ground motion and SSI will be assessed and appropriate equations considering those effects will be proposed.

6.3 Preliminary Test

A preliminary test using a 1/2 scale pier was completed to reproduce the shear failure observed in the Santa Monica Freeway bridge and to verify the MUST-SIM facility. The sections of the prototype and half scale piers are shown in figure 4.

A shear capacity of 533.78 kN was calculated by using the shear equation suggested by Priestly et al (1994). The test results are shown in figure 20. Significant failure due to reduction of shear capacity was observed at 507kN corresponding to a displacement of 51.3 mm. An analytical model has been created in Zeus-NL that employs a shear spring based upon the Modified Compression Field Theory. Figure 20 provides a comparison between the analytical and experimental behavior. The pier behaved as predicted and the failure mode observed is similar to that seen in the Santa Monica Bridge shown in figure 2.

7. CLOSURE

There is ample evidence that neglecting vertical earthquake input in assessment of structures, especially reinforced concrete structures, leads to unquantifiable errors. It is however noted that the majority of studies on the effect of vertical motion on RC structures are analytical. In this paper, new analytical results, as prelude to laboratory testing, are presented. The effect of vertical-horizontal motion interaction on inelastic periods of a reinforced concrete bridge is assessed, alongside the effect on axial force amplitude and direction. It is concluded that inelastic periods of vibration are significantly affected by including vertical ground motion, thus potentially affecting the demand in unexpected ways, dependent on the frequency content of the input motion. Periods vary by up to 40% while axial force levels vary by up to 80% when vertical-horizontal interaction is taken into account. A short parametric study is conducted on the effect of the arrival time of large vertical and horizontal shaking cycles. It is concluded that for the structure considered and the motion set used, the arrival time has minimal effect on the periods of response, but a rather important effect on the shear capacity, up to 20% is observed. Plans for the testing of a complex bridge system taking into account the effect of vertical input motion and soil-structure interaction are outlined. Use is made of the advanced features of the UIUC NEES site in terms of multiple load and boundary condition points with 6 DOF capabilities. The NEES sites at Lehigh and RPI are also utilized, with a pier tested at Lehigh and foundation interaction tested at RPI. A plan of employing the $1/5^{th}$ scale MUST-SIM laboratory to narrow down the test range in preparation for the large scale test is described. The deployment of distributed testing utilizing the most suitable structural and geotechnical NEES sites, alongside advanced analytical simulation, as described in the paper, provides new insight into the seismic response of complex structural-geotechnical systems.

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Table 1 Selected earthquakes for analytical investigation

Earthquake	Ms	Date	Station	PGA (g)		V/H	Time interval, T_p (sec)
				Long.	Vert.		
Northridge	6.7	17/01/94	Arleta Fire St.	0.308	0.552	1.79	2.78
Northridge	6.7	17/01/94	Santa Monica Hosp.	0.370	0.230	0.62	0.08
Kobe Japan	7.2	17/01/95	Port Island Array	0.349	0.569	1.63	1.92
Kobe Japan	7.2	17/01/95	Kobe University	0.276	0.431	1.56	1.10
Loma Prieta	7.17	18/10/89	Corralitos	0.470	0.434	0.92	1.46
Loma Prieta	7.17	18/10/89	Capitola	0.397	0.538	1.36	-1.48

Table 2 Consistent mass value for the each member ((N. sec^2/mm)/mm)

	Piers	Wall	Deck
$\bar{m}$	3.16334E-3	1.05087E-2	9.17425E-3

Table 3 Initial load (kN)

	Pier 1	Pier 2	Pier 3
Initial load	2288.82	2515.62	2834.56

Table 4 Period of vibration for each earthquake record

Earthquake	Horizontal Period (sec)				Vertical Period (sec)			
	0.1 HGM	HGM	HGM+ VGM	RI (%)	0.1 VGM	VGM	HGM+ VGM	RI (%)
Northridge - A. F.	0.2805	0.3792	0.4762	26	0.0868	0.1599	0.2250	41
Northridge - S. M.	0.2805	0.3723	0.4095	10	0.0869	0.1575	0.2048	30
Kobe Japan - P.I.A.	0.2805	0.3470	0.4550	31	0.0875	0.3470	0.3656	5
Kobe Japan - K.U.	0.2805	0.3592	0.3863	8	0.0898	0.1271	0.1950	53
Loma Prieta - Cor.	0.2844	0.4550	0.6205	36	0.0871	0.2767	0.3357	21
Loma Prieta - Cap.	0.2805	0.4653	0.6205	33	0.0922	0.1896	0.2226	17

* HGM: Horizontal ground motion, VGM: Vertical ground motion,
* RI: Ratio of period variation

Table 5 Variation of axial force on pier 1

Earthquake	HGM		HGM+VGM		Range of axial force variation			Contribution of VGM to axial force (%)
	Max.	Min.	Max.	Min.	HGM	HGM+ VGM	RI (%)	
Northridge - A. F.	-1305	-3375	-100	-4525	2070	4624	123	55
Northridge - S. M.	-1493	-3109	-899	-3940	1617	3041	88	47
Kobe Japan - P.I.A.	-1923	-2704	-257	-4475	781	4218	440	81
Kobe Japan - K.U.	-1472	-3206	45	-4656	1734	4701	171	63
Loma Prieta - Cor.	-1360	-3400	15	-4541	2040	4556	123	55
Loma Prieta - Cap.	-1020	-3470	448	-5219	2450	5667	131	57

* RI: Ratio of axial force variation

Table 6 Shear capacity and demand of pier 1 for original records

Earthquake records	HGM		HGM+VGM		Rate of increase	
	Demand (kN)	Capacity (kN)	Demand (kN)	Capacity (kN)	Demand (%)	Capacity (%)
Northridge - A. F.	1993	1752	1839	1624	-7.70	-7.28
Northridge - S. M.	2040	1814	2041	1708	0.03	-5.88
Kobe Japan - P.I.A.	1703	1917	1986	1604	16.63	-16.33
Kobe Japan - K.U.	2019	1775	2016	1694	-0.13	-4.56
Loma Prieta - Cor.	2319	1803	2369	1722	2.17	-4.48
Loma Prieta - Cap.	2333	1896	2282	1725	-2.19	-9.03

Table 7 Material properties and scale factor for experimental investigation

Items	Prototype	Half Scaled model	
		Scale factor	Material Properties
Length	6575 mm	1/2	3048 mm
Concrete Area	1.167 mm^2 (D = 1.219 mm)	1/4	0.2918 mm^2 (D = 0.6095 mm)
Rebar	34.9mm dia	1/2	19.05 mm (#6)
	12.7 mm dia	1/2	9.525 mm(#3)
Ec	29000 MPa	1	29000 MPa
Es	210000 MPa	1	210000 MPa
f'c	34.5 MPa	1	34.5 MPa
Fy	413 MPa	1	413 MPa
Displacement		1/2	
Rotation		1	
Load		1/4	
Moment		1/8	

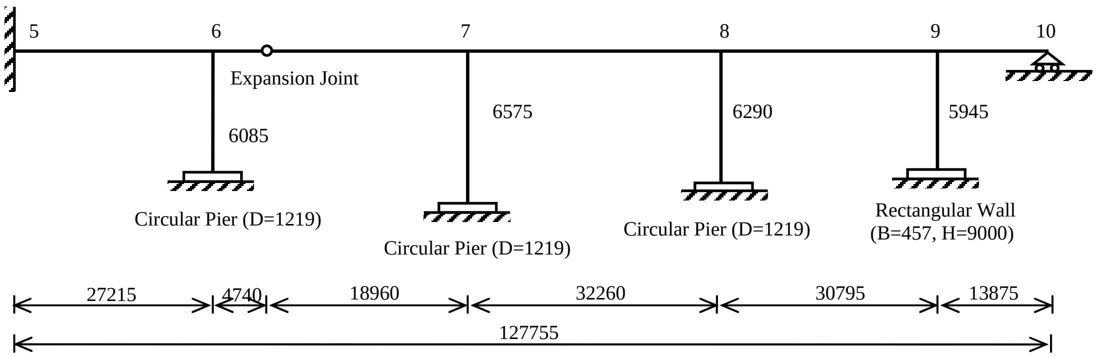


Figure 1 Layout of Santa Monica Freeway (unit: mm)

Figure 2. Pier 6 of Collector Distributor 36.
Shear failure caused by fluctuation of axial load

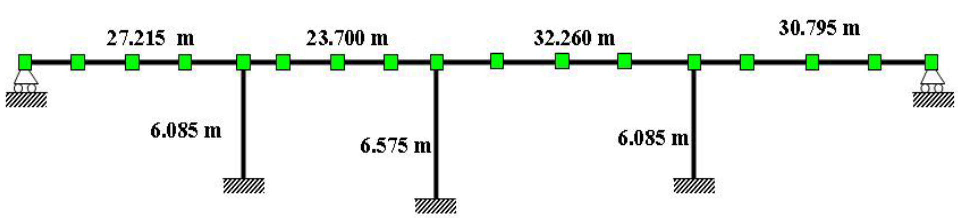
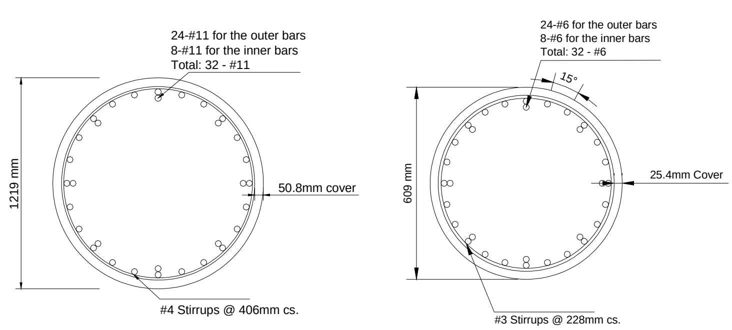
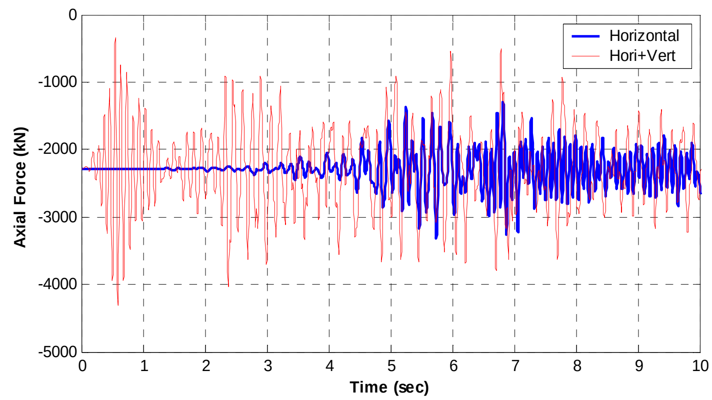


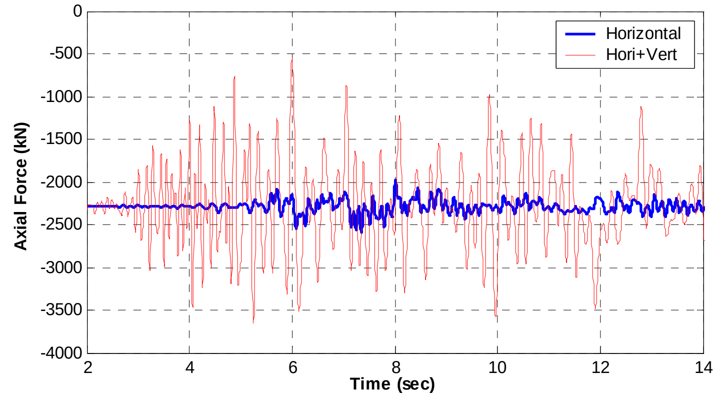
Figure 3. Layout of Model Structure



a. Prototype b. Half scaled model for test
Figure 4. Section properties

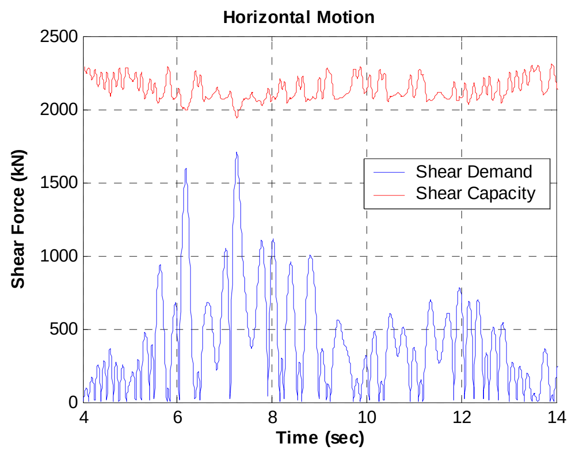


a. Northridge (Arleta Fire) earthquake (T_p =2.78, V/H=1.8)

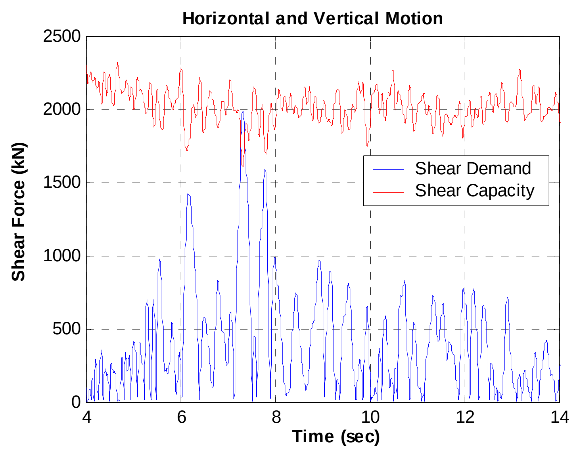


b. Kobe (Port Island Array) earthquake (T_p =1.9, V/H=1.6)

Figure 5 Comparison of axial force variation on pier 1

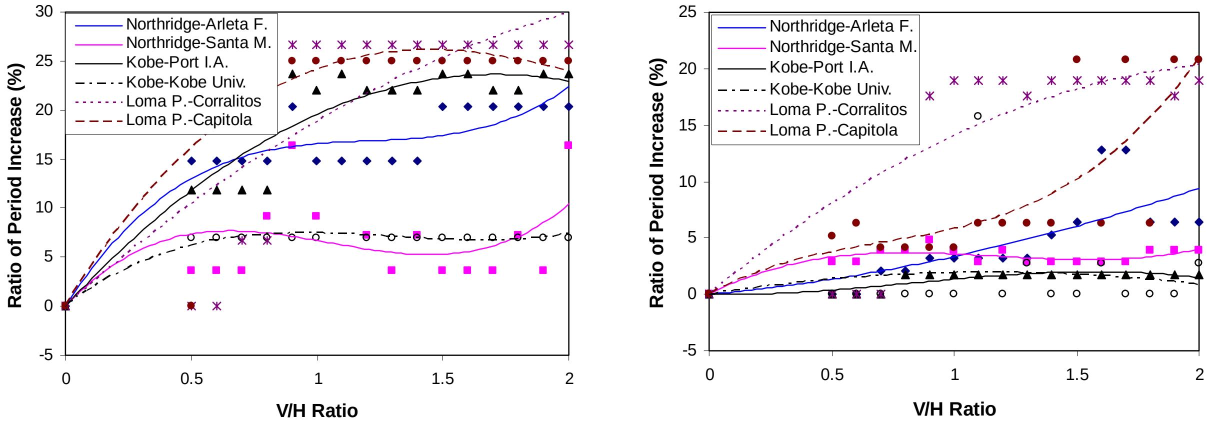


a. Horizontal ground motion



b. Horizontal and Vertical Ground motion

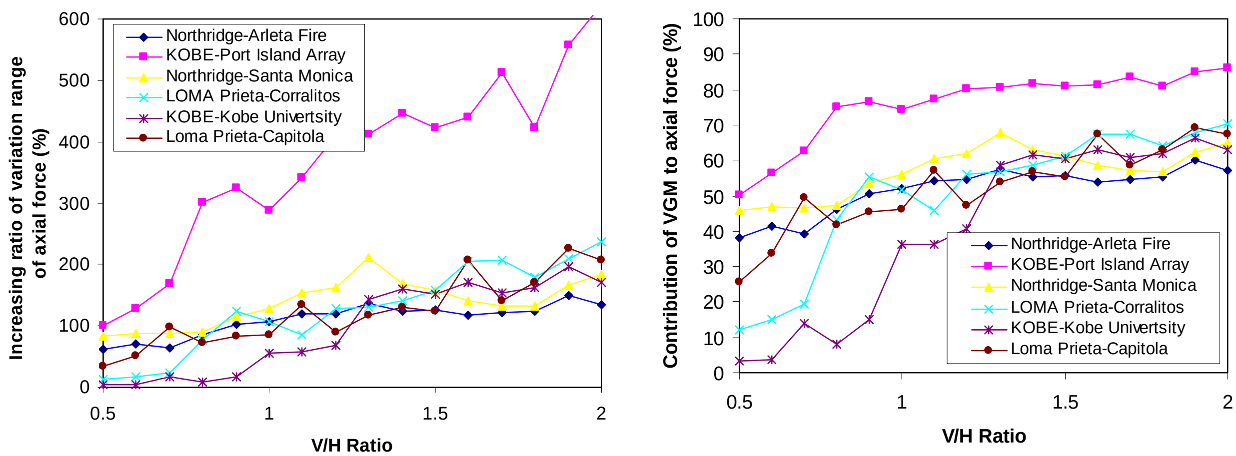
Figure 6. Shear capacity and demand of pier 1 for KOBE (Port Island Array) earthquake (T_p =1.9, V/H=1.6)



a. Horizontal Period of vibration

b. Vertical Period of vibration

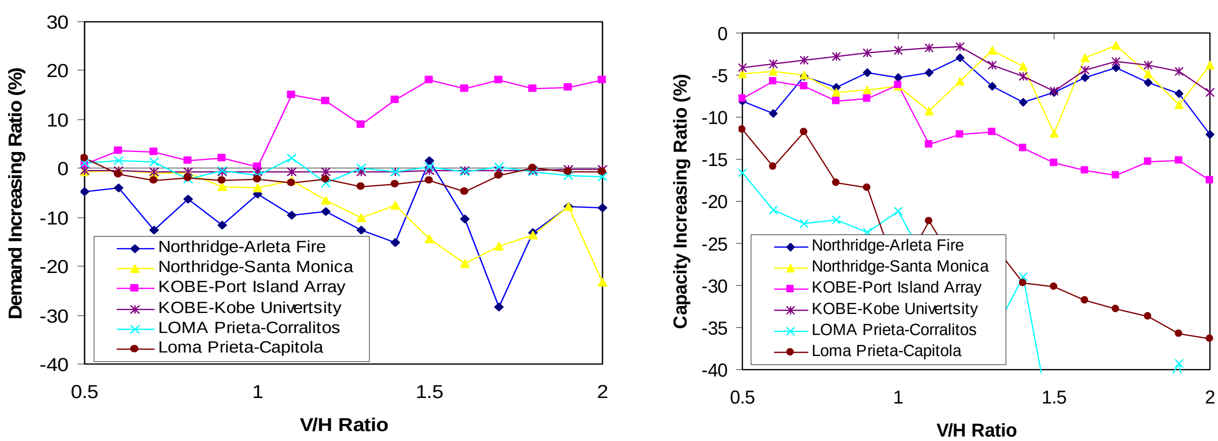
Figure 7. Period change by V/H Ratio



a. Variation of shear demand

b. Variation of shear capacity

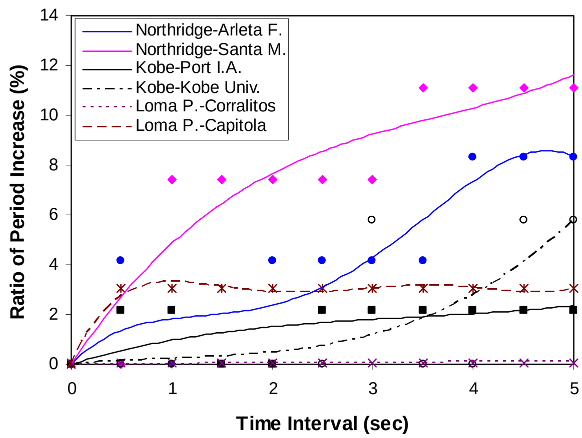
Figure 8. Change of axial force by V/H Ratio for pier 1



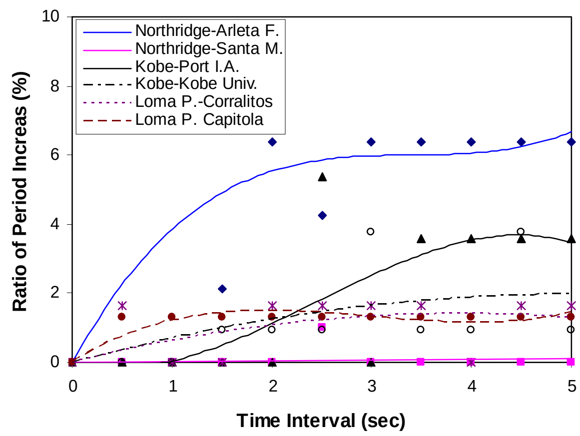
a. Variation of shear demand

b. Variation of shear capacity

Figure 9. Shear demand and capacity by V/H Ratio for pier 1

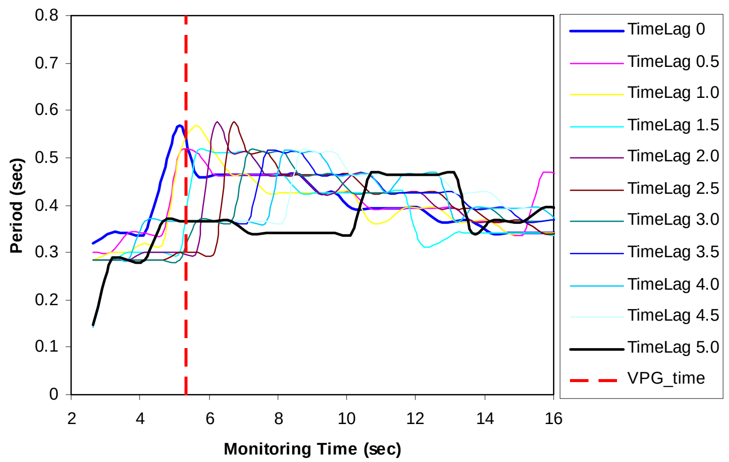


a. Horizontal Period of vibration

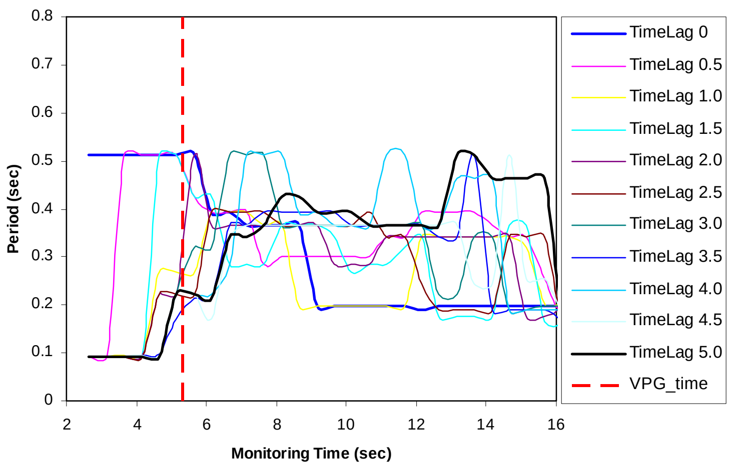


b. Vertical Period of vibration

Figure 10. Period change by arrival time interval

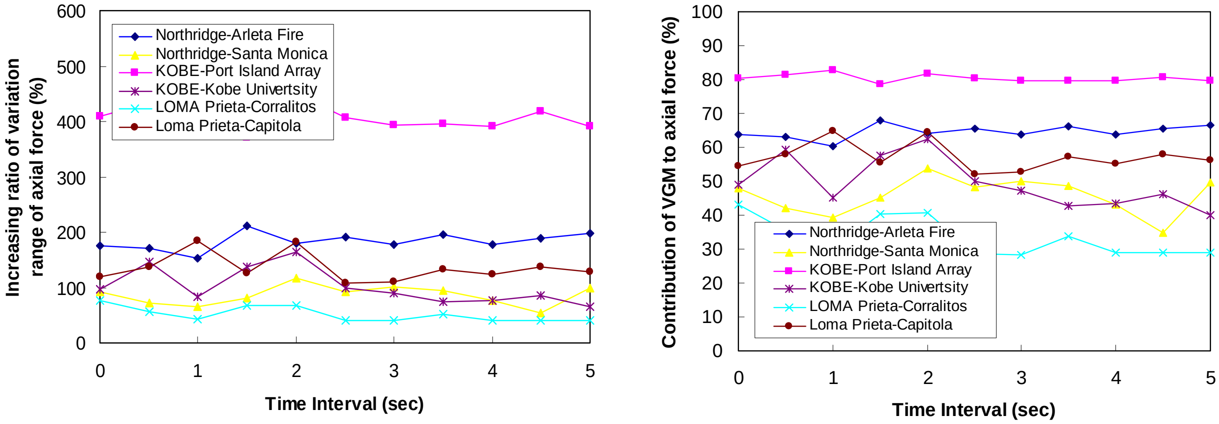


a. Horizontal Period of vibration



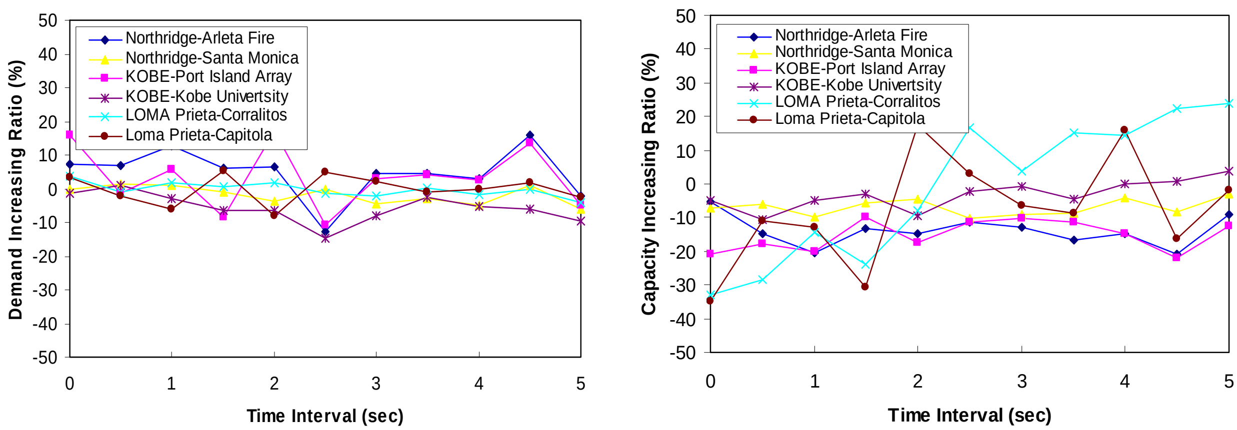
b. Vertical Period of vibration

Figure 11. Period change versus time,
Kobe, Port Island Array



a. Variation of axial force b. Contribution of vertical ground motion

Figure 12. Change of axial force by time interval for pier 1



a. Variation of shear demand b. Variation of shear capacity

Figure 13. Shear demand and capacity by arrival time interval for pier 1

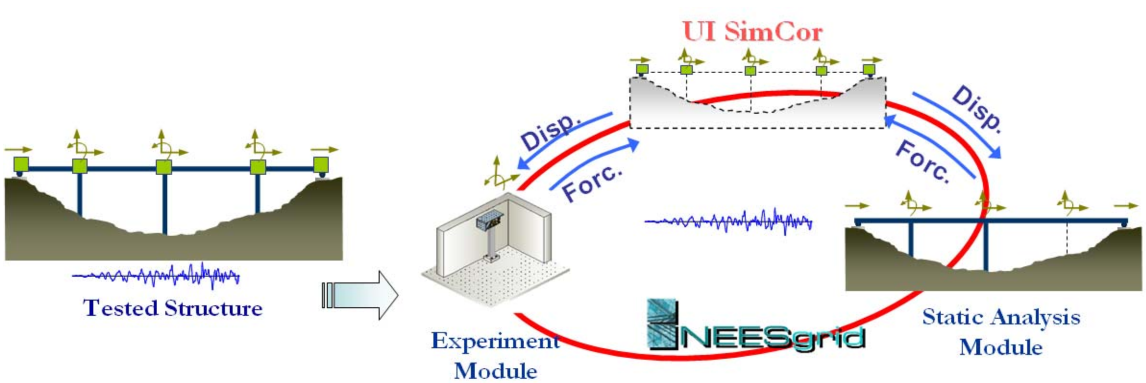


Figure 14. PSD test using sub-structuring scheme with MUST-SIM facilities

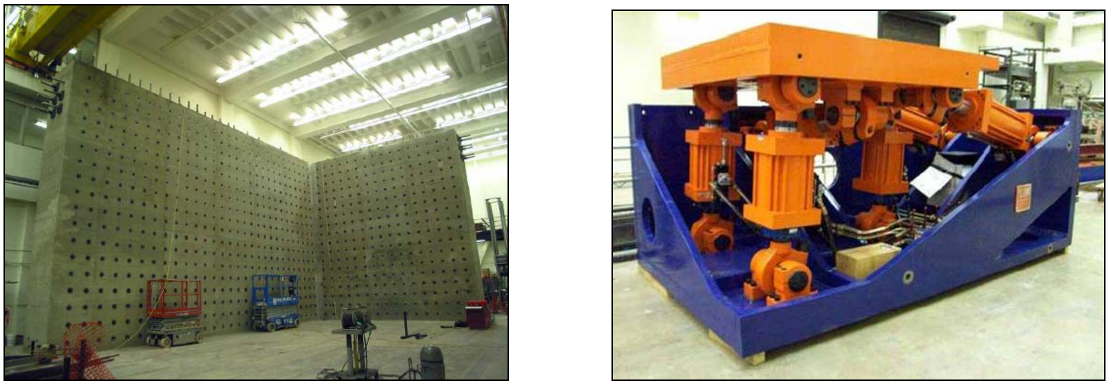
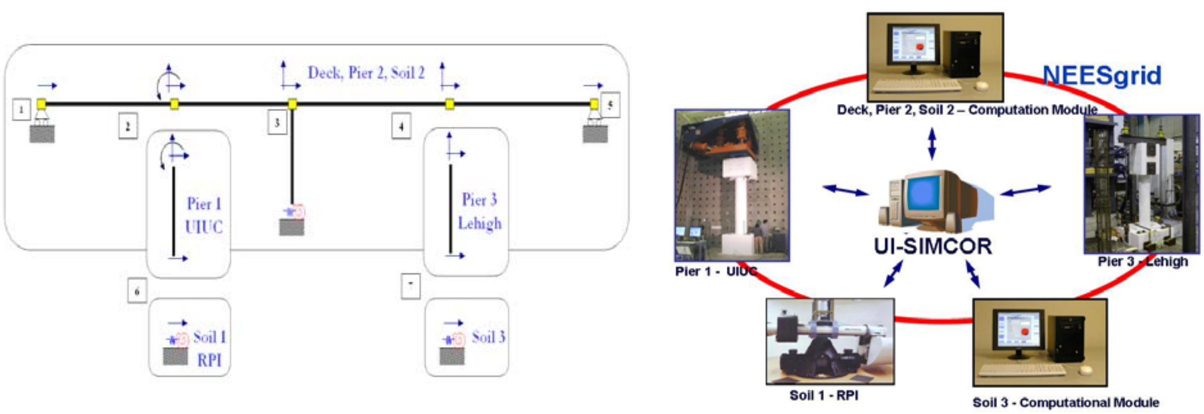


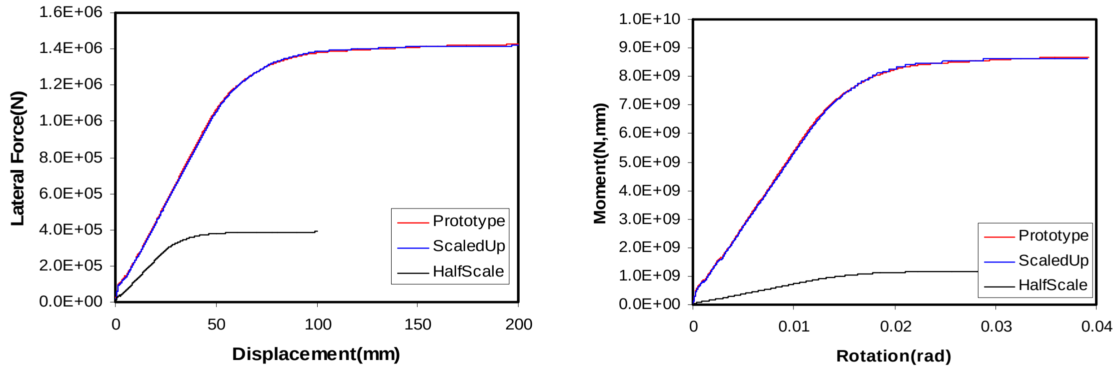
Figure 15. Reaction wall and LBCB in the MUST-SIM facilities

Figure 16. 1/5th scaled reaction wall and LBCB in the MUST-SIM facilities



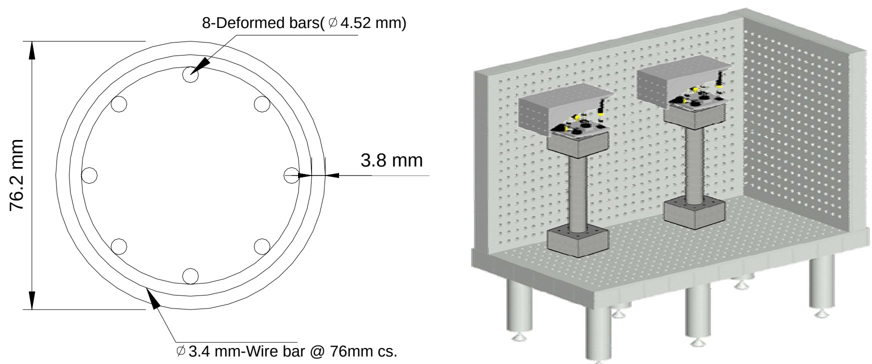
a. Substructure b. Distributed hybrid simulation test

Figure 17. Substructure configuration of MISST



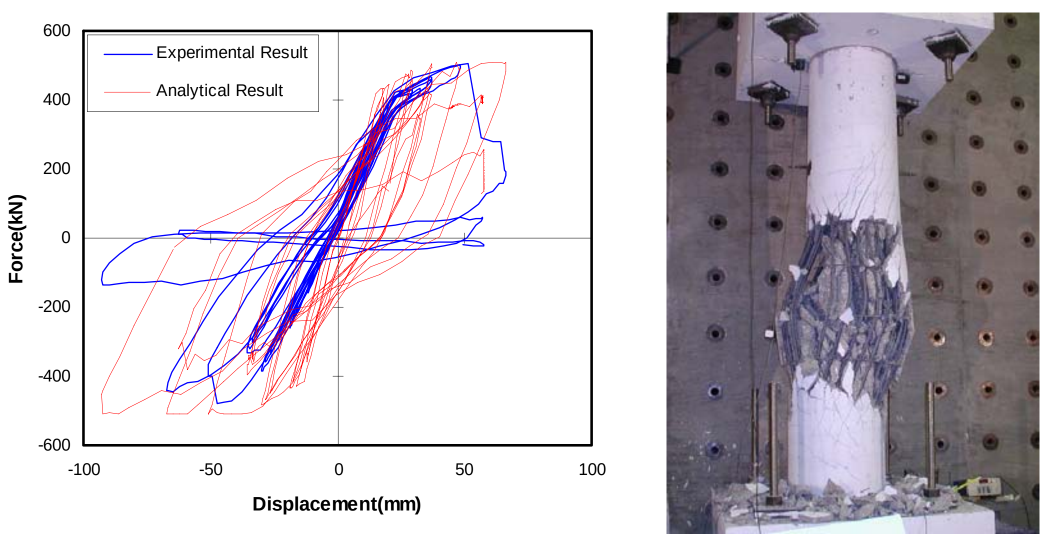
a. Force and displacement b. Moment and rotation

Figure 18. Verification of scale factor using Push-Over analysis for ½ scaled pier



a. section of 1/8th scaled pier b. Test setup for small test

Figure 19. Section properties



a. Comparison with analytical result b. Failure in the laboratory

Figure 20. Preliminary pier test result