

Response of Bridge Structures subjected to Spatially Varying Earthquake Ground Motions

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Acknowledgement

My supervisor: the late Profs Joe Penzien and Bruce Bolt

My former PhD students: A/P Kaiming Bi, Prof. Chao Li and Dr. Bipin Shrestha,

My current PhD student: Miss Haimei Ling



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NATIONAL NATURAL SCIENCE FOUNDATION OF CHINA



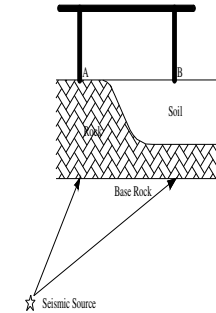
Australian Government

Australian Research Council

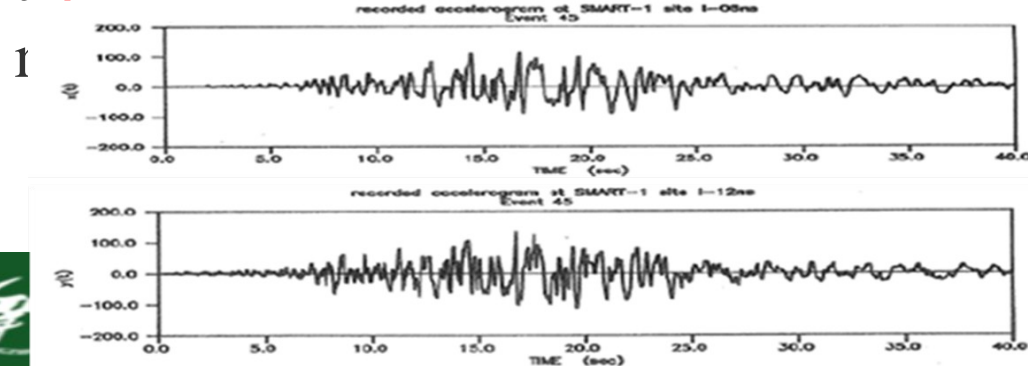
Background

Ground motion spatial variation (GMSV) is inevitable
Source, path and local site conditions make seismic ground motions vary spatially

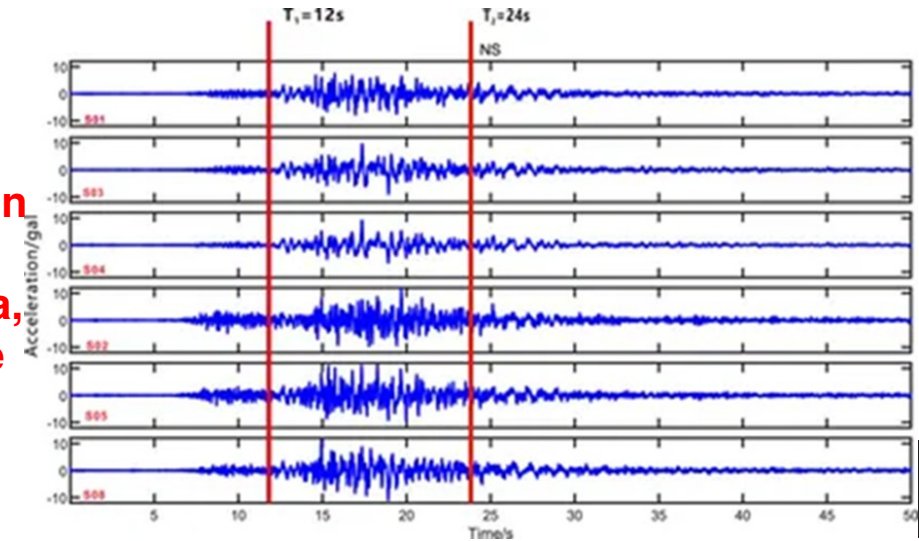
1. **Wave passage effect**: seismic waves arrive at different locations at different times
2. **Incoherence effect**: seismic waves scatter in the heterogeneous medium of the ground
3. **Local site effect**: differences in local soil conditions at each station may alter the amplitude and frequency content of the base



Recorded ground motions at SMART-1 array in Taiwan, separation distance 200 m

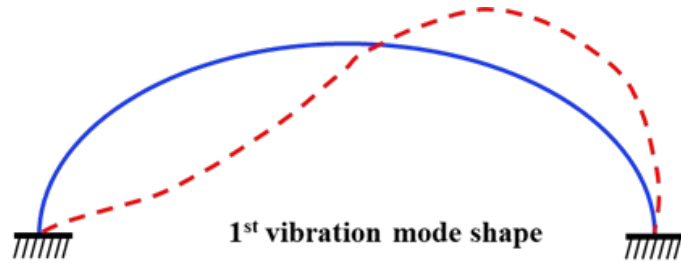


Recorded motions in 2021 Lushan earthquake in China, separation distance 100 m

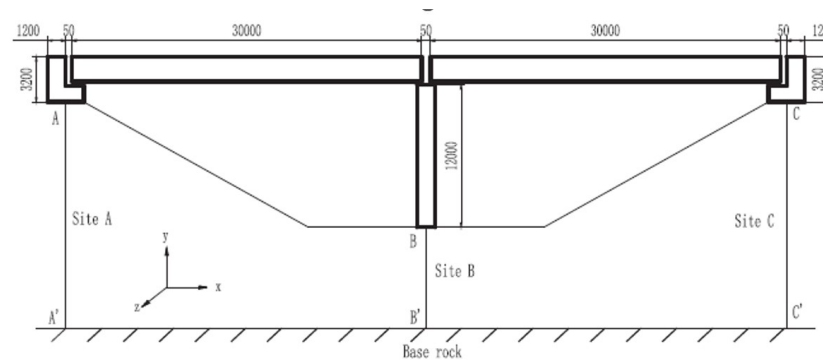
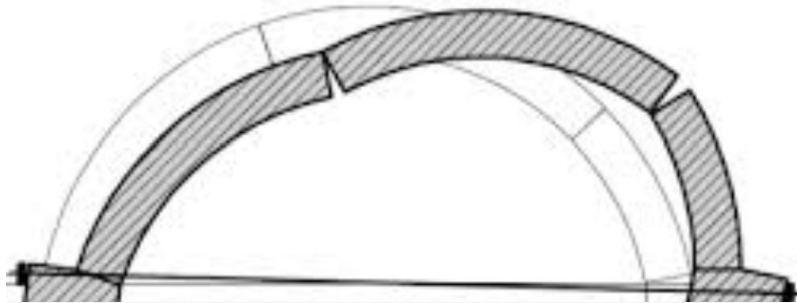


Spatially varying earthquake ground motions result in different structural responses.

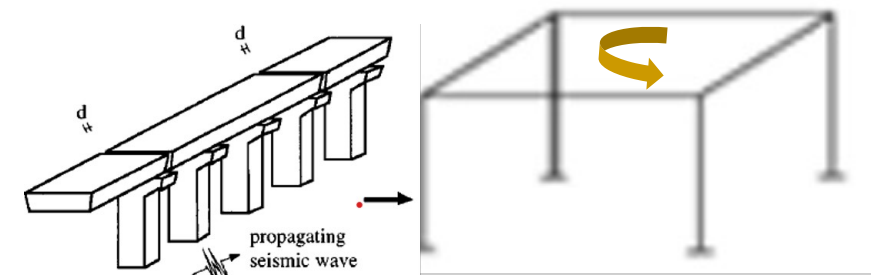
They excite asymmetric modes that will not be excited by uniform ground motions; and 2) cause relative responses between adjacent structures



1st vibration mode shape
Asymmetric fundamental mode of arch structure



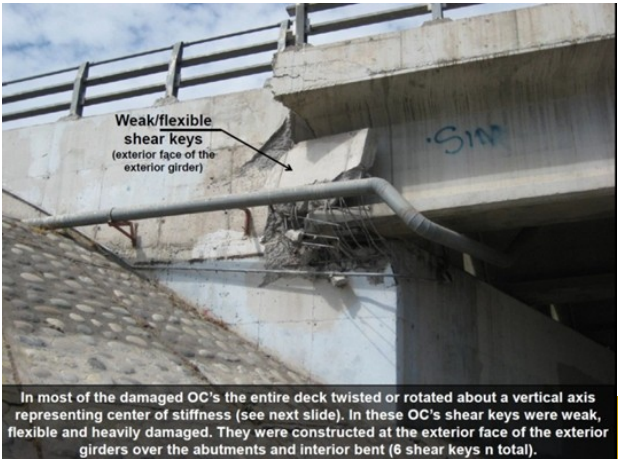
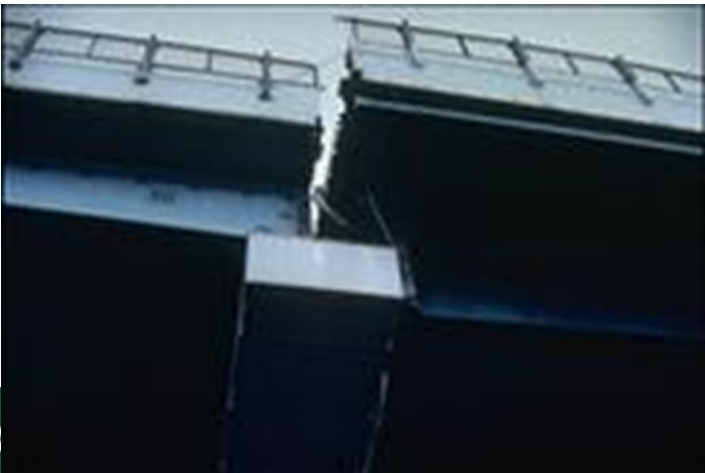
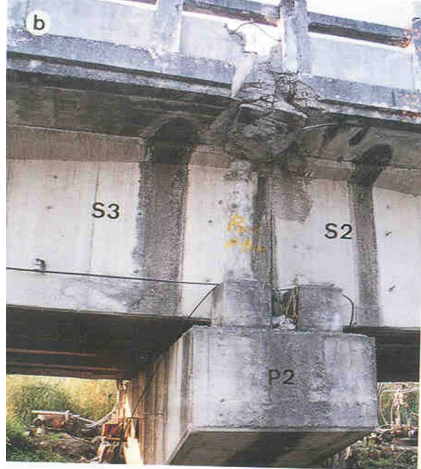
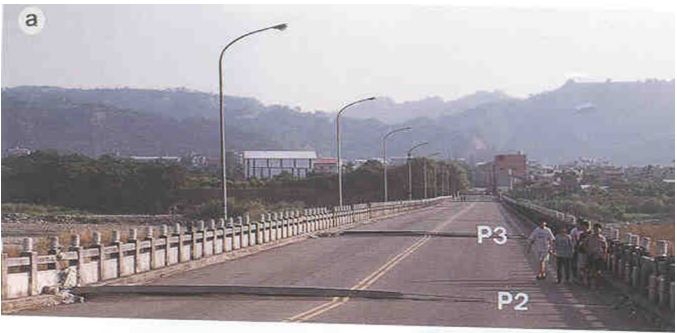
Relative displacement



Torsional response



Damage associated to relative displacement and torsional response



In most of the damaged OC's the entire deck twisted or rotated about a vertical axis representing center of stiffness (see next slide). In these OC's shear keys were weak, flexible and heavily damaged. They were constructed at the exterior face of the exterior girders over the abutments and interior bent (6 shear keys in total).

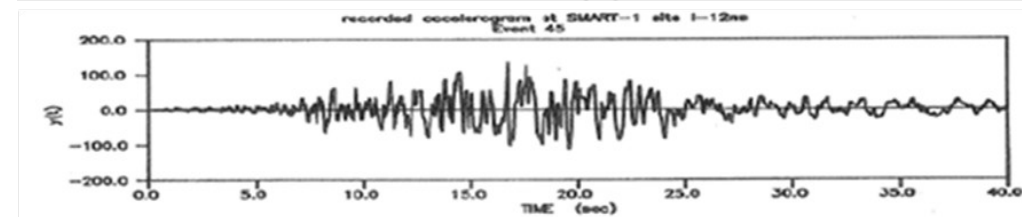
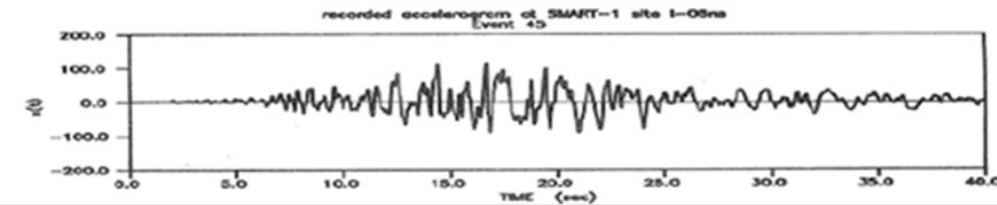
Early Research

1. Bogdanoff NL, Goldberg JE, Schiff AJ, 'The Effect of Ground Transmission time on the response of long structures', Bull. Seismol. Soc. Am, 1965
2. Newmark NM, 'Torsional Response of Symmetric Buildings', Proceedings 4WCEE 1969
3. Sandi H, 'Conventional Seismic Forces Corresponding to Non-synchronous Ground Motion', Proc of 3rd European Symp Earthquake Engineering, Sofia, Bulgaria 1970.
4. Morgan JR, Hall WJ and Newmark NM, 'Seismic Response Arising from Travelling Waves', J. Str Eng, ASCE
5. Wilson E, SAP4,
6. Penzien J. NEABS
7. Etc, etc

$a_1(t)$

$a_2(t)$

$$a_2(t) = a_1(t - d/v)$$



Only considered seismic wave passage effect, neglected loss of coherency between spatial ground motions



Most seismic design codes point out the importance of considering ground motion spatial variations, but do not provide detailed analysis and design procedures

AASHTO, Caltran, Eurocode8, Fib, ISO3010, MCCER-ATC, NCHRP, NZ Transportation Agency

Seismic design code (spatial

GB 50011-2010
P
建筑抗震设计规范
Code for seismic design of buildings
2010-05-31 发布
2010-12-01 实施
中华人民共和国住房和城乡建设部
中华人民共和国国家质量监督检验检疫总局 联合发布

AASHTO
Standard Specifications for Highway Bridges
17th Edition - 2002
Images of bridges

MCCER/ATC-49
Recommended LRFD guidelines for the seismic design of highway bridges
Part II: Commentary and Appendices
Image of bridge structure

Japanese code
STANDARD SPECIFICATIONS for HIGHWAY BRIDGES
SIXTEENTH EDITION 1996
Image of bridge structure

EURO code
EUROPEAN STANDARD
DRAFT prEN 1998-2
Eurocode 8 - Design of structures for earthquake resistance - Part 2: Bridges
Image of bridge structure

Italian code
GAZZETTA UFFICIALE DELLA REPUBBLICA ITALIANA
Ministero delle Infrastrutture e dei Trasporti
Decreto 17 gennaio 2018
Aggiornamento delle «Norme tecniche per le costruzioni»
Image of bridge structure

ISO 3010
INTERNATIONAL STANDARD
Third edition 2017-03
Bases for design of structures — Seismic actions on structures
Bases du calcul des constructions — Actions sismiques sur les structures

Necessary

Seating length

Simplified method

Ground motion simulation

seat length requirement

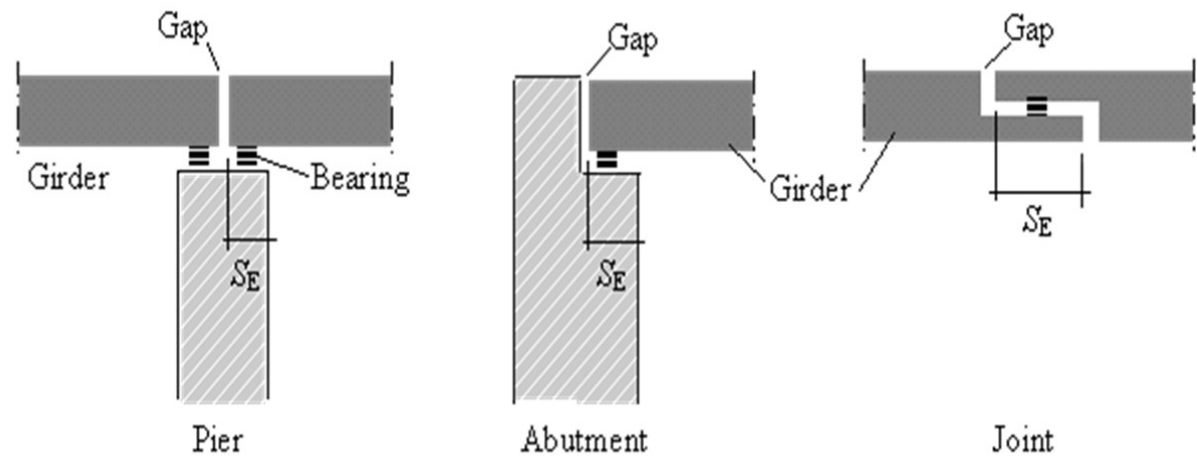


Figure 1(a)-(c) Seating length on (a) pier, (b) abutment and (c) at a joint



Caltran



ATC - 6



Taiwan

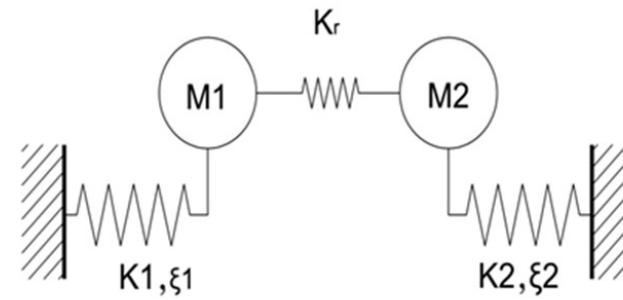
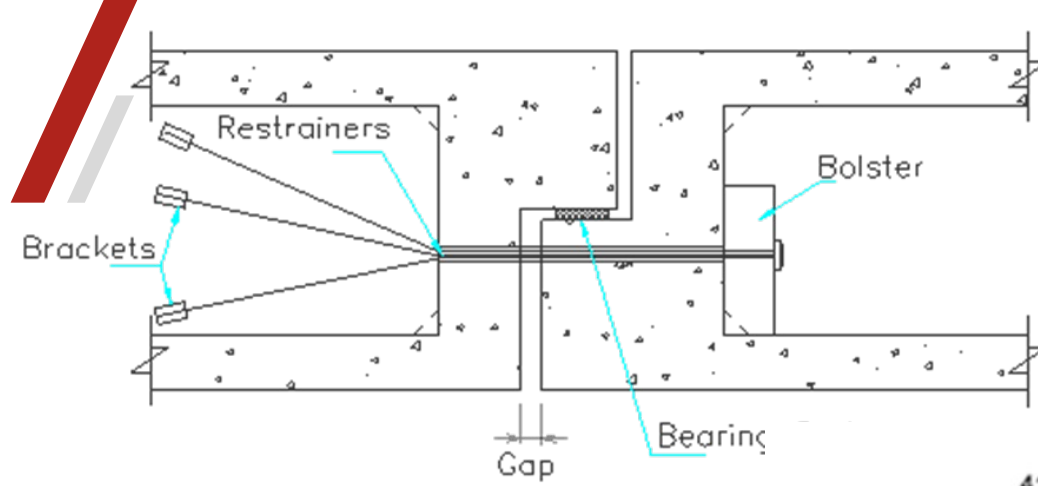


Japan

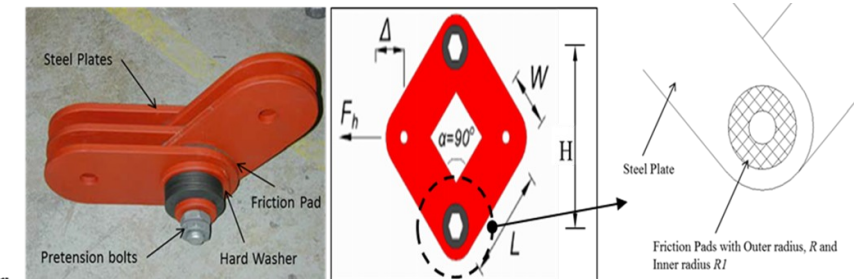
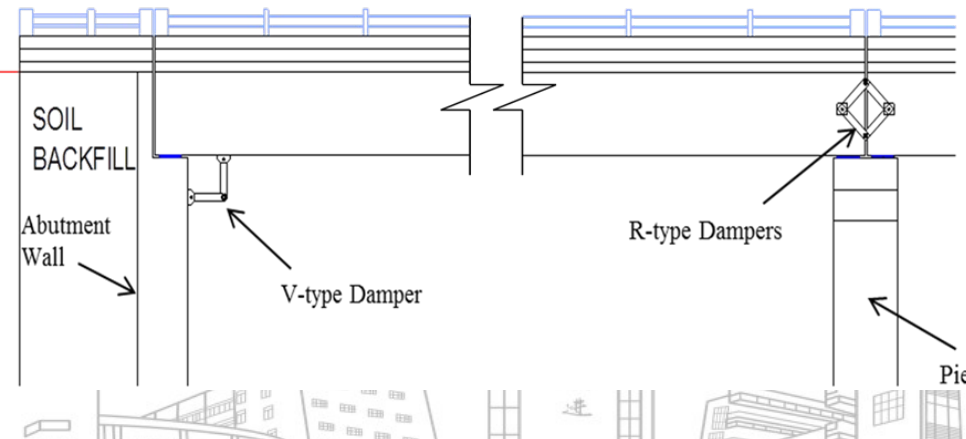
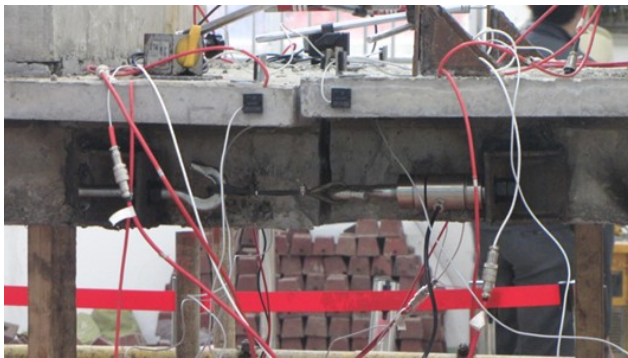
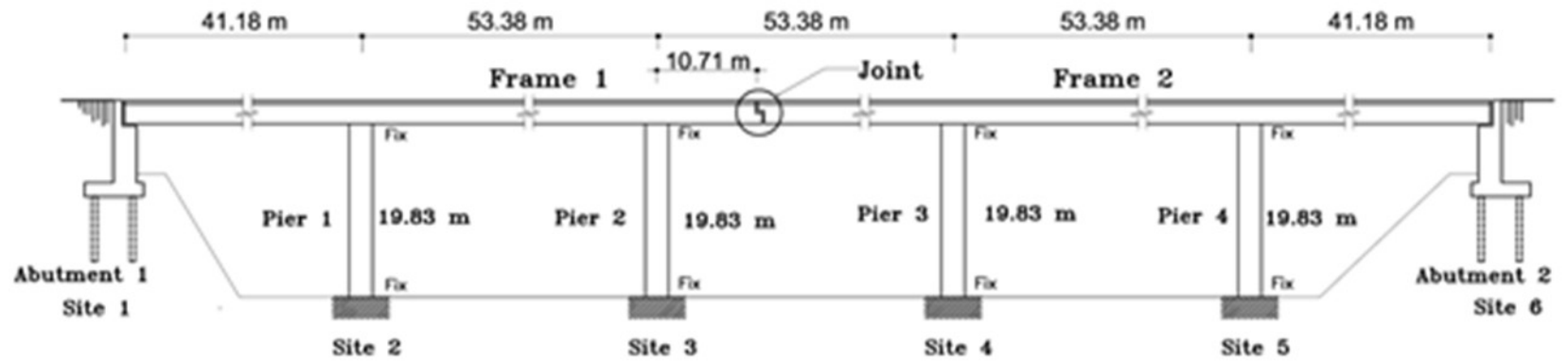
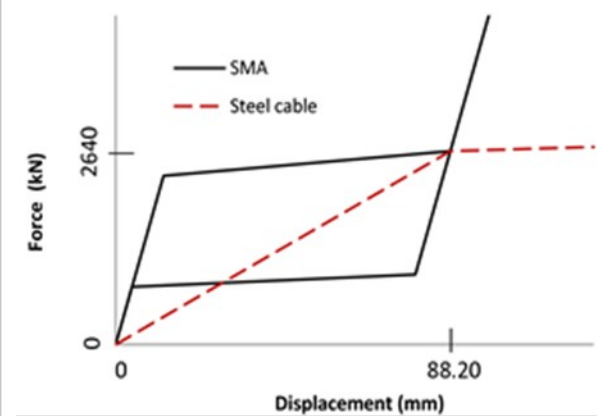


$$u_G = \epsilon_G L$$

$\epsilon_G = 0.0025$ (Hard soil)
 0.00375 (Medium soil)
 0.005 (Soft soil)



DesRoches and Fenves (2000) proposed a method to estimate the required cable restrainer stiffness to avoid unseating

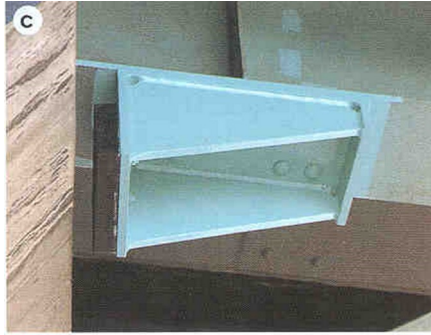


Bipin Shrestha, H. Hao and Kaiming Bi, 'Effectiveness of using Rubber Bumper and Restrainer on Mitigating Pounding and Unseating Damage of Bridge Structures subjected to Spatially Varying Ground Motions', Engineering Structures, 2014.

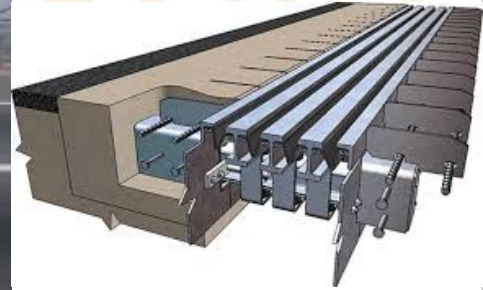
Practice



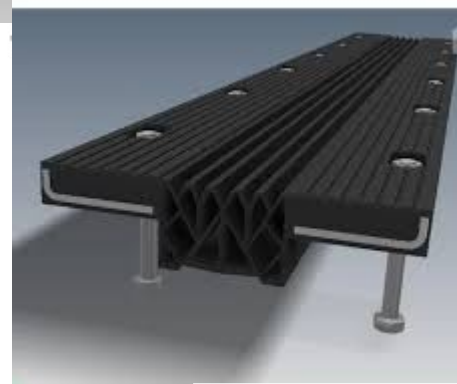
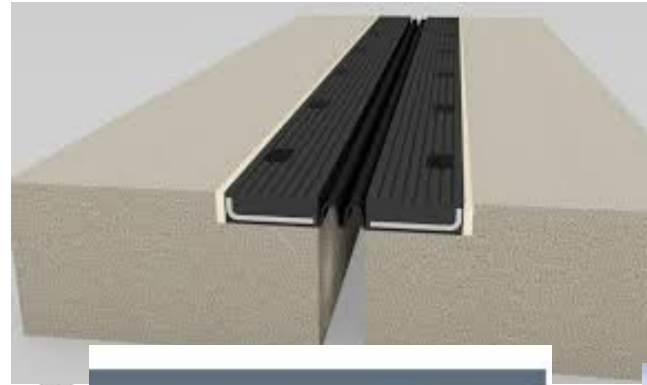
Extending seat length



Stopper



Modular expansion joint



Rubber expansion joint



Restrainer

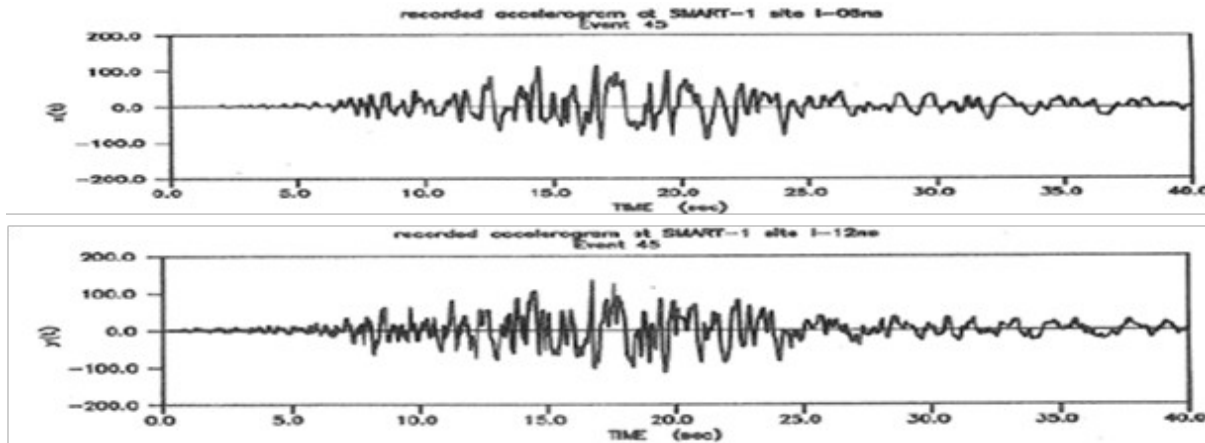
Ava bridge collapse, 2025



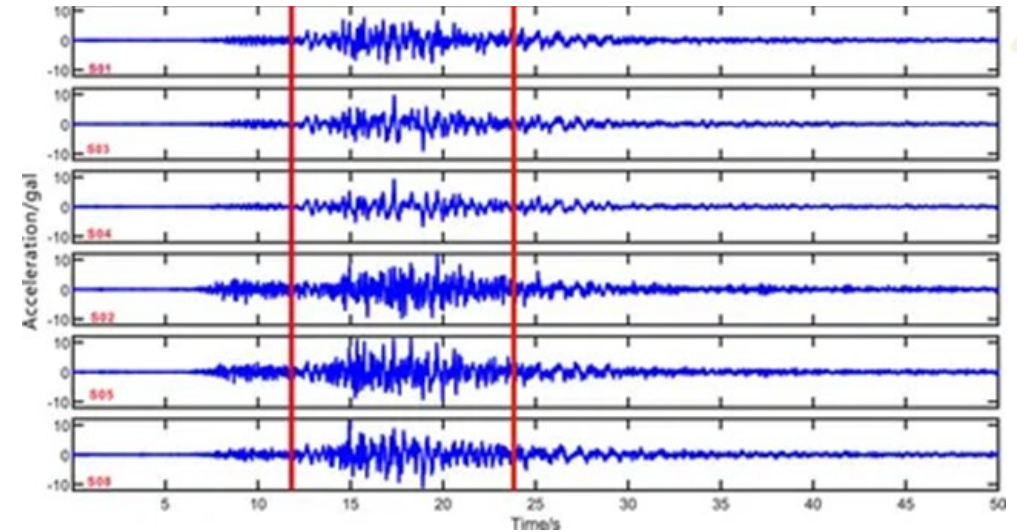
After over 60 years efforts, it is still a challenge to reliably design structures to resist spatially varying ground motions

Characteristics and modelling of ground motion spatial variations

Recorded ground motion at SMART-1 array, 200 m



Recorded ground motions at Zhigong, separation distance 100 m



Auto and Cross Power Spectrum

$$S_{x_1 x_1}(\omega) = \frac{1}{T} \sum_{m=-M}^M W_m X_1\left(\omega + \frac{2\pi m}{T}\right) X_1^*\left(\omega + \frac{2\pi m}{T}\right)$$

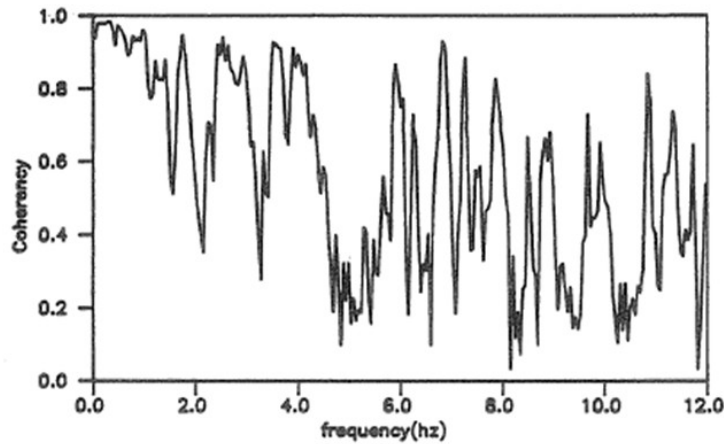
$$S_{x_1 x_2}(i\omega) = \frac{1}{T} \sum_{m=-M}^M W_m X_1\left(\omega + \frac{2\pi m}{T}\right) X_2^*\left(\omega + \frac{2\pi m}{T}\right)$$

Coherency and phase
fu

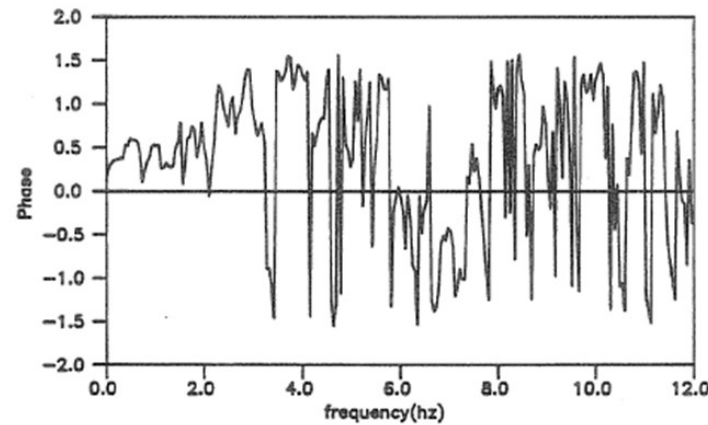
$$\gamma_{x_1 x_2}(i\omega) = \frac{S_{x_1 x_2}(i\omega)}{[S_{x_1 x_1}(\omega) S_{x_2 x_2}(\omega)]^{\frac{1}{2}}}$$

$$\begin{aligned} \phi_{x_1 x_2}(\omega) &= \tan^{-1} \left(\frac{\text{Im}[S_{x_1 x_2}(i\omega)]}{\text{Re}[S_{x_1 x_2}(i\omega)]} \right) \\ &= \tan^{-1} \left(\frac{\text{Im}[\gamma_{x_1 x_2}(i\omega)]}{\text{Re}[\gamma_{x_1 x_2}(i\omega)]} \right) \end{aligned}$$

$$\gamma_{x_1 x_2}(i\omega) = \frac{S_{x_1 x_2}(i\omega)}{[S_{x_1 x_1}(\omega) S_{x_2 x_2}(\omega)]^{\frac{1}{2}}}$$



$$\begin{aligned}\phi_{x_1 x_2}(\omega) &= \tan^{-1} \left(\frac{\text{Im}[S_{x_1 x_2}(i\omega)]}{\text{Re}[S_{x_1 x_2}(i\omega)]} \right) \\ &= \tan^{-1} \left(\frac{\text{Im}[\gamma_{x_1 x_2}(i\omega)]}{\text{Re}[\gamma_{x_1 x_2}(i\omega)]} \right)\end{aligned}$$



$$\gamma_{jk}(i\omega) = |\gamma_{jk}(i\omega)| \exp(i\theta_{jk}(\omega)) = |\gamma_{jk}(i\omega)| \exp\left(-i \frac{\omega d}{v_{app}}\right)$$

Coherency loss function $|\gamma_{jk}(i\omega)|$, Phase shift $\theta_{jk}(\omega)$, which can be easily determined if apparent wave velocity V_{app} is determined

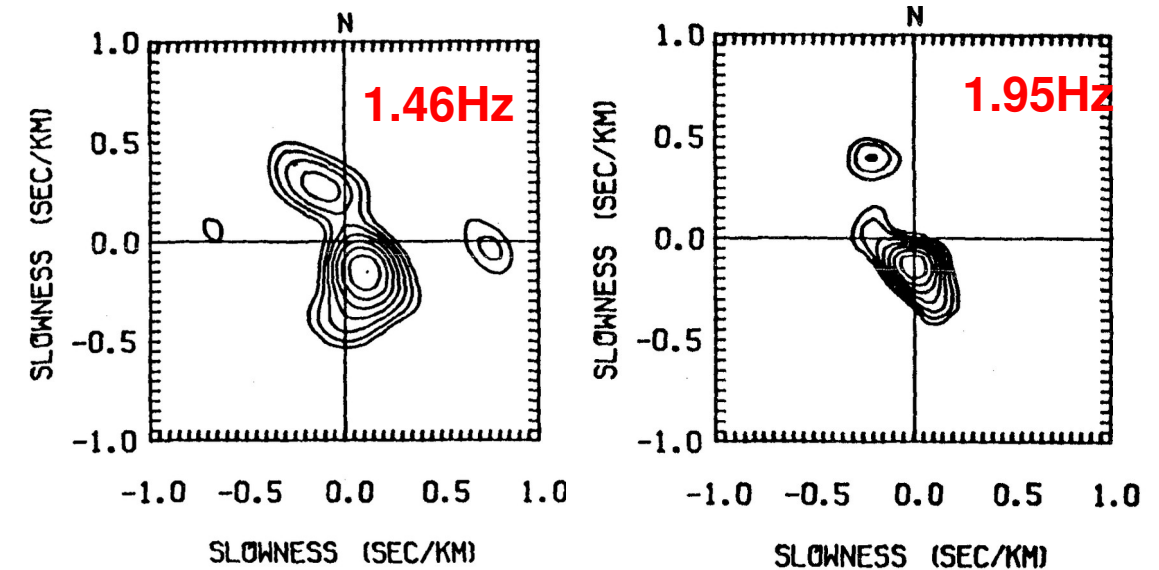
Spatial ground motion apparent wave velocity V_{app}

Assuming ground motion consists of mainly shear wave .

$$v_{app} = \frac{V_s}{\cos \alpha} \quad \text{Apparent wave velocity}$$



Ground motion incident angle to the site, which in practice is difficult to be determined



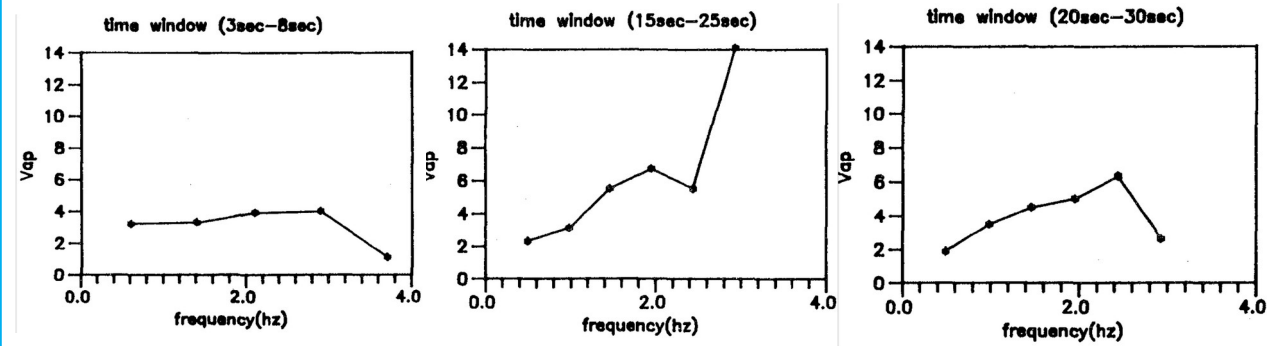
F-K analysis, Event 45, SMART-1 Array, 15-25 sec window

Assume uniform site, two extreme values of apparent velocity

1) Horizontal incident : $V_{app} = V_s$

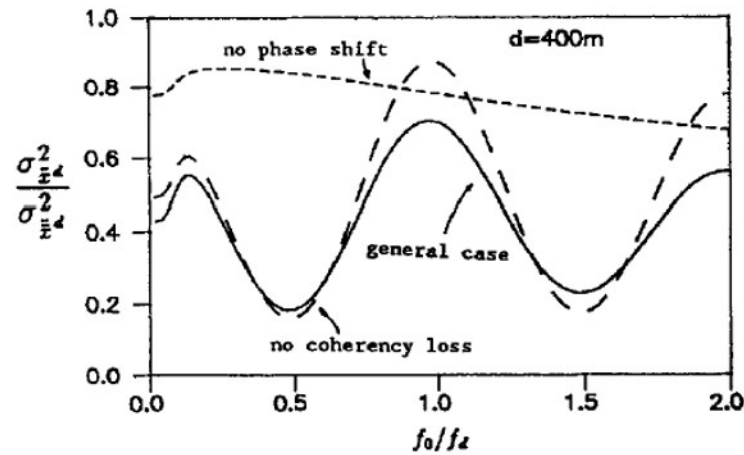
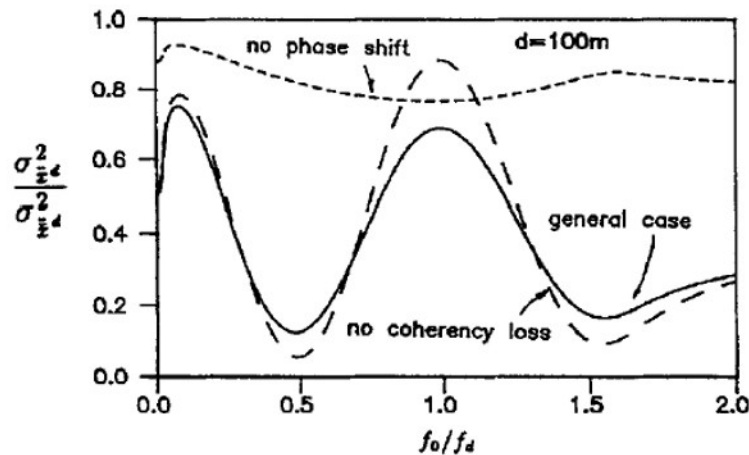


2) vertical incident : $V_{app} = \infty$, no phase shift



V_{app} is time, frequency and azimuth dependent.

Currently, no reliable model or methodology to predict seismic wave propagation apparent velocity yet. In practice a constant apparent velocity is usually assumed, e.g., **1000 m/s**, or the extreme values V_s and ∞ . This simplification may bring uncertainties in modelling spatial ground motion time delays. It is commonly understood that the influence of spatial ground motion phase shift on structural responses depends on the dimensionless parameter **fd/V_{app}**



Hong Hao, 'Response of multiply-supported rigid plate to spatially correlated seismic excitations', International Journal of earthquake engineering and structural dynamics, Vo1. 20, 1991.

$fd/V_{app}=1.0, 2.0, \dots$: spatial ground motion phase shift and vibration mode is in-phase

$fd/V_{app}=0.5, 1.5, \dots$: spatial ground motion phase shift and vibration mode is out-of-phase

They correspond to the extreme values of phase shift effect, which can be used in analysis to define the range of phase shift influences

Coherency loss :



-lagged coherency, depicts incoherence effect

1) theoretical model : based on wave propagation in random field

i) B. J. Uscinski, $\gamma_{ij} = E_0^2 \exp\{-l[1 - \exp\{-(\xi^2 + \eta^2)/r_0^2\}]\}$

ii) Kazimierz

Sobczyk $\gamma_{ij}(i\omega) = |\gamma_{ij}(i\omega)| \exp(-i\omega d_{ij} \cos \alpha / v_{app}) = \exp(-\beta \omega d_{ij}^2 / v_{app}) \cdot \exp(-i\omega d_{ij} \cos \alpha / v_{app})$

- 1) Provide the fundamentals of wave coherency loss during propagations in random field
- 2) Highly idealized, not easy to reliably determine the respective parameters

2) Semi-empirical model

- i) Der Kiureghian,
- ii) Luco and Wong
- iii) Yang and Cheng,
- iv) Somerville,

Needs spatial ground motion records to fit and determine the respective parameters

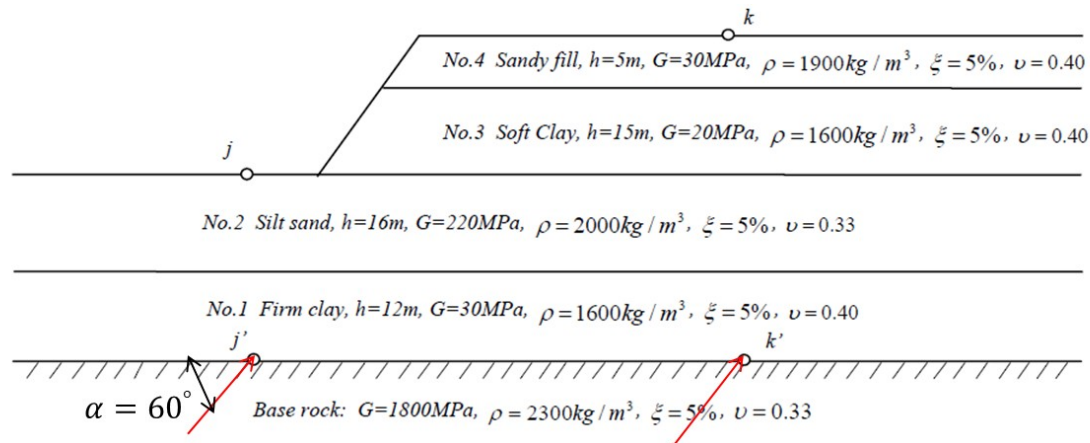
$$|\gamma(\xi_{kl}, \omega_i)| = \cos[\beta(\xi_{kl}, \omega_i)] \exp \left\{ -\frac{1}{2} \alpha^2(\xi_{kl}, \omega_i) \right\}$$

$$|\gamma(\xi, \omega)| = e^{-(\frac{v \omega \xi}{V_s})^2} = e^{-\alpha^2 \omega^2 \xi^2} \quad \left| \begin{array}{l} v = \mu \left(\frac{R}{r_0} \right)^{1/2} ; \quad \alpha = \frac{v}{V_s} \end{array} \right.$$

$$|\gamma(\xi_{kl}, \omega_i)| = \cos[\beta(\xi_{kl}, \omega_i)] \exp \left\{ -\frac{1}{2} \alpha^2(\xi_{kl}, \omega_i) \right\}$$

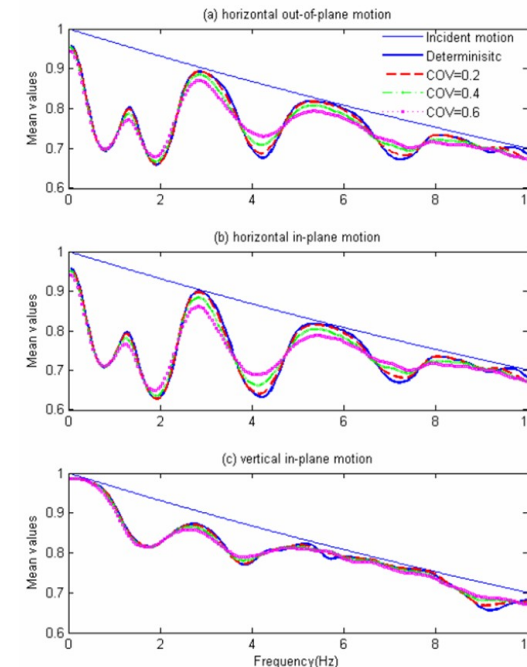
$$\gamma_{total}(\omega, d) = \gamma_{source}(\omega, d) \cdot \gamma_{path}(\omega, d) \quad \left| \gamma_{path}(\omega, \xi) \right| = \tanh([a - b \ln(\xi)] \cdot \exp[-c\omega] + \xi)$$

$$\left| \gamma_{source}(\omega, \xi) \right| = \tanh(e \exp[-m\omega - n\xi] + p)$$



Model wave propagation in random local site

$$G = \bar{G} + \sigma_G \sqrt{\lambda(Z)} \phi = \bar{G} (1 + COV \times \sqrt{\lambda(Z)} \phi) \quad \lambda(Z) = \frac{1}{2(Z/\delta_u)^2} [2(Z/\delta_u) + e^{-2(Z/\delta_u)} - 1]$$



Kaiming Bi and Hong Hao, 'Influence of irregular topography and random soil properties on coherency loss of spatial seismic ground motions', *Earthquake Engineering & Structural Dynamics*, 2011, Vol. 40, pp1045-1061

3. Empirical model

	Location	Array Name	Owner	Number of Stations	Dates of Operation	Site Characterization	Station Spacing	Layout	Events
	AMERICA								
1	Canada	Yellowknife Seismic Array	CTBT	19	1962 ~present		min: ~2.5 km; max: ~22.5 km	cross with unequal arms	>10,000 seismic events and nuclear tests
2	Mexico	Jardin Array	UNAM	7	4/1/1994 ~05/14/1994	hill and intermediate zone on edge of Vally of Mexico lake-bed zone of Valley of Mexico	min: ~150m; max: ~565m	grid; aperture ~500m	24 (3.0~M-6.2)
3	Mexico	Roma Microarray	UNAM	3 (total 5)	1993 ~ present		min: ~92m; max: ~140m	triangular	> 4 (5.7~Mw<7.5; distant)
4	CA, USA	San Jose Dense Seismic Array	USGS	52 (initially 41)	April 1999 ~ present	alluvium	1km (average); min: ~107m; max: ~13.3km	grid	11 (M2.5~M7.1)
5	CA, USA	EPRI Parkfield Dense Seismic Array	EPRI	13 (total 21)	1987	alluvium	min: 10m max: 150m	Y-shaped (120m)	
6	CA, USA	USGS Parkfield Small Aperture Array (UPSAR)	USGS	14	May 1989~	Paso Robles	25m-952m	distributed stations 1km ²	625 (05/1989 ~12/1990)
7	CA, USA	Turkey Flat Strong motion Array							
8	CA, USA	Gilroy Array		7					
9	CA, USA	Gamer Valley Array (GVDA)	UC Santa Barbara	5 (11 total)	1992 ~ present	alluvium	66 m	linear	1162 (to 10/01) weak and strong events
10	CA, USA	ZAYA Portable Array		30	May. 1991	rock	15 m (average) (min: 13.8 m, max: 14.2 m)	grid (45 m x 45 m)	5 chemical explosions
11	CA, USA	Small-Aperture Array at Pinyon Flat		25	1991	thin weathered layer over crystalline basement	radii: 150m, 320m, 735m, 1.48km	four concentric rings	
12	CA, USA	High-frequency Array at Pinyon Flat		58	4/18/1990 ~05/27/1990	decomposed granite	min: 7m max: ~390m	7x7 element grid @7m & 11 element linear array	324
13	CA, USA	San Fernando Valley Portable Arrays (five)		Parking Garage (4), Mulholland (5), Sherman oaks (8), White Oak (3), I-10 (5)	1/18/1994 ~ 02/24/1994			aperture of 200 m ~ 500 m	Northridge Earthquake aftershocks
14	CA, USA	San Fernando Valley Dense Array	SCEC / USGS	21	1994	silt, clay, sand	min: 0.5 km max: 5km	distributed (5 km x 5 km)	25 (2.5~ML<4.0)

	Location	Array Name	Owner	Number of Stations	Dates of Operation	Site Characterization	Station Spacing	Layout	Events
15	CA, USA	Sunnyvale Array		4	October. 1989	alluvium valley	min: ~ 85 m max: 335 m	triangle (sides ~ 300 m)	Loma Prieta Earthquake aftershocks
16	CA, USA	El Centro Differential Array		7			station distance 60~1000 ft	linear (350m)	Imperial Valley Earthquake
17	CA, USA	Borrego Valley Data Array (BVDA)		13 (total 18)	1993 ~	alluvium	max: ~ 4km	two linear arrays	150 (1.5~M-6.8)
18	CA, USA	UCSB/SCEC Dense Strong-motion Array	UCSB	5+5	June. 1992	alluvium, rock	min: ~85m; max: ~450m	two arrays (~ 3 km apart)	5 Landers aftershocks
19	CA, USA	San Bernardino Valley Array	GEOS	SB (3); SG (3); NA (4) (total 14)	June. 1992	alluvium	200 m (average)	each array configuration ~ triangular	40 Landers-Big Bear aftershocks
20	CA, USA	Tarzana Nursery Array	USGS	21	1994	shales	sensors at 15~50m apart	cross; 13 stations along length (580m); 9 stations cross (380m)	20 Northridge aftershocks
21	CA, USA	Hector Mines (Two) Arrays	UC Santa Barbara	40+22	October. 1999		Bullion Wash: 9m - 1.1km; Bul. Mountain: 9m - 2km	Bul. Wash: two "lines" (75deg) & grid; Bul. Mount: two "lines" (50deg)	
22	CA, USA	Hollister Differential Array		4		soil	min: 61m max: 256m		1 (M=5.3)
23	CA, USA	Stanford		4		soil	min: 32m max: 185m		4 (M=4)
24	CA, USA	Coalinga		7		rock	min: 48m max: 313m		
25	IN, USA	Glendora Array	NSWC	102	September. 1998	mined spoils & soils	min: 10m	63 radially from explosion & 39 moving, high density ones	20 under water explosions
26	NV, USA	Colwick Array		9			min: 100 m; max: 400 m	nested triangles (sides: 100, 200 & 400m)	explosion
27	TX, USA	Lajitas Seismic Array	SMU	8 vertical, one three-component center		limestone		two concentric rings of radii 1 & 4 km	seismic & explosions
	ANTARCTICA								
28	Antarctica	VNA2 Array	Alfred Wegener Institute	15 (vertical component)	February 1997 ~ present	ice rises (ice on solid rock)	200 m - 1.96 km	3 concentric rings	

	Location	Array Name	Owner	Number of Stations	Dates of Operation	Site Characterization	Station Spacing	Layout	Events
	ASIA								
29	Armenia	Garni Dense 3-D Array	USGS	10	July 1990 ~	tunnel	100m (average)	linear with three additional stations	11 (09/90 ~ 11/90)
30	China	Tangshan Array	Caltech	13 (21 total)	April 1981 ~ December 1984	soil / rock	min area 0.3 km ²	along and normal to Lulong fault	416 (1.2~ML<5.7)
31	Japan	Chiba Dense Array	U. of Tokyo	15		loam	5m-310m	cross array and distributed stations	
32	Japan	Yokohama Dense Array		150	May 1997~		2km (average)	distributed stations 25km x 20km	29 (Mw4.0~Mw7.3)
33	Japan	Fuchu Strong Motion Array	NEID	6 (total 10)	1983 ~	alluvium	min: ~ 3km max: ~8.5km	two ~ linear arrays	> 6
34	Japan	Ashigara Valley Array, Japan	ERI	20 (23 total)		rock (6), alluvium (14)		distributed stations	4 (2.9~Mj<5.1)
35	Japan	Osaka Basin Array, Japan							
36	Japan	Tokachi Basin Array, Japan		5					
37	Japan	Kushiro Array, Japan	JESG						
38	Kazakhstan	Makanchu Seismic Array	KazNDC	9			min: 430m max: 4.8km	distributed (3.5km x 5km)	
39	Kazakhstan	Karatau Seismic Array	KazNDC	9				two concentric circles (500m & 2km);	
40	Kazakhstan	Kurchatov Seismic Array	KazNDC	22			min: 2.25 km max: 22.5km	cross (11 stations / arm)	
41	Kazakhstan	Akbulak Seismic Array	KazNDC	10	2003 ~ present		min: ~130m max: 3.8km	~ two concentric circles	
42	Taiwan	Strong Motion Array in Taiwan, Phase I (SMART I)	UCB, IES	37	1980~1991	alluvium	104m - 4km	three concentric rings	60 (2.7~ML<6.5)
43	Taiwan	Strong Motion Array in Taiwan, Phase II (SMART II)	IES	45	December 1990 ~ present	firm gravel deposits	2km (average)	distributed stations	63 (to 3/02) (2.6~ML<7.3)
44	Taiwan	Lotung Large Scale Seismic Test (LLSST)	TaiPower / IES	15 (37 total)	1985 ~ 1990	alluvium		Y-shaped	30 (10/85~ 07/90) (3.7~ML<6.5)

Empirical Coherency loss models

Proposer	Model
Novak and Hindy (1979, the 1 st one in earthquake engineering)	$ \gamma(\xi, \omega) = \exp\left[-\kappa\left(\frac{\omega\xi}{V_s}\right)^{\nu}\right]$
Loh and his co-workers	$ \gamma(\xi, \omega) = \exp(-a(\omega)\xi)$ $ \gamma(\xi, \omega) = \exp\left(-a\frac{\omega\xi}{2\pi c}\right)$ $ \gamma(\xi, \omega) = \exp((-a - b\omega^2)\xi)$ $ \gamma(\xi, \omega) = \exp((-a - b\omega)\xi^{\nu})$ $ \gamma(\xi, \omega, \theta) = \exp((-a_1 - b_1\omega^2) \xi \cos \theta)\exp((-a_2 - b_2\omega^2) \xi \sin \theta)$
Harichandran and Vanmarcke	$ \gamma(\xi, f) = A \exp\left(-\frac{2 B \xi}{a v(f)}\right) + (1 - A) \exp\left(-\frac{2 B \xi}{v(f)}\right)$ $v(f) = k \left[1 + \left(\frac{f}{f_0}\right)^b\right]^{-1/2}; \quad B = (1 - A + a A)$

Harichandran	$ \gamma(\xi, f) = A \exp\left\{-\frac{2\xi(1 - A)}{(a k)} \left[1 + \left(\frac{f}{f_0}\right)^b\right]^{1/2}\right\} + (1 - A)$
Abrahamson et al (LSST data)	$ \gamma(\xi, f) = \tanh\left\{(2.54 - 0.012\xi) \left[\exp[(-0.115 - 0.00084\xi)f] + \frac{f^{-0.878}}{3}\right] + 0.35\right\}$
Abrahamson	$ \gamma^H(\xi, f) = \tanh\left\{\frac{c_1^H(\xi)}{1 + c_2^H(\xi)f + c_4^H(\xi)f^2} + (4.80 - c_1^H(\xi)) \exp[c_3^H(\xi)f] + 0.35\right\}$ $ \gamma^V(\xi, \omega) = \tanh\left\{\frac{c_1^V(\xi)}{1 + c_2^V(\xi)f} + (4.65 - c_1^V(\xi)) \exp[c_3^V(\xi)f] + 0.35\right\}$
Menke et al	$ \gamma(\xi, f) = \exp(-\alpha f \xi)$
etc	

There are about 100 empirical models from different researchers, and they vary substantially.

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JULY 1989

EARTHQUAKE ENGINEERING RESEARCH CENTER

EFFECTS OF SPATIAL VARIATION OF GROUND MOTIONS ON LARGE MULTI-SUPPORTED STRUCTURES

by
HONG HAO

PREFACE

by
BRUCE A. BOLT
and
JOSEPH PENZIEN

Report to the National Science Foundation



COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA AT BERKELEY

DYNAMICS OF STRUCTURES

Second
Edition

Ray W. Clough
Joseph Penzien



$$\gamma_{ij}(d_{ij}^L, d_{ij}^T, f) = \exp(-\beta_1 d_{ij}^L - \beta_2 d_{ij}^T) \times \exp\left[-(\alpha_1 d_{ij}^{L1/2} + \alpha_2 d_{ij}^{T1/2})f^2\right] \times \exp\left(i 2\pi f \frac{d_{ij}^L}{v_{app}}\right) \quad (12)$$

$$\alpha_1(f) = \frac{a}{f} + bf + c \quad \alpha_2(f) = \frac{d}{f} + ef + g$$

Event	20	22	23	24	25	29
β_1	5.350×10^{-4}	1.130×10^{-4}	5.290×10^{-4}	2.622×10^{-4}	2.390×10^{-4}	3.550×10^{-4}
β_2	3.670×10^{-4}	3.710×10^{-4}	1.860×10^{-4}	1.211×10^{-4}	1.820×10^{-4}	6.310×10^{-4}
Event	30	31	33	36	37	40
β_1	2.250×10^{-4}	4.620×10^{-4}	2.810×10^{-4}	3.530×10^{-4}	7.910×10^{-4}	9.323×10^{-5}
β_2	5.100×10^{-4}	4.820×10^{-4}	3.710×10^{-4}	2.830×10^{-4}	6.830×10^{-4}	1.421×10^{-4}
Event	41	45	46	47	48	
β_1	3.062×10^{-4}	1.109×10^{-4}	1.193×10^{-3}	7.420×10^{-4}	1.391×10^{-3}	
β_2	6.894×10^{-4}	6.730×10^{-5}	9.010×10^{-4}	1.202×10^{-3}	4.723×10^{-4}	

No guide on how to select an appropriate coherency loss model, arbitrary selection may lead to different structural response predictions

To establish a guide for coherency loss model selection, available coherency loss functions were collected and classified into three groups according to the site conditions, statistical analyses were conducted to determine the mean and standard deviation coherency loss functions.

Three site conditions :

1) **Baseroack**

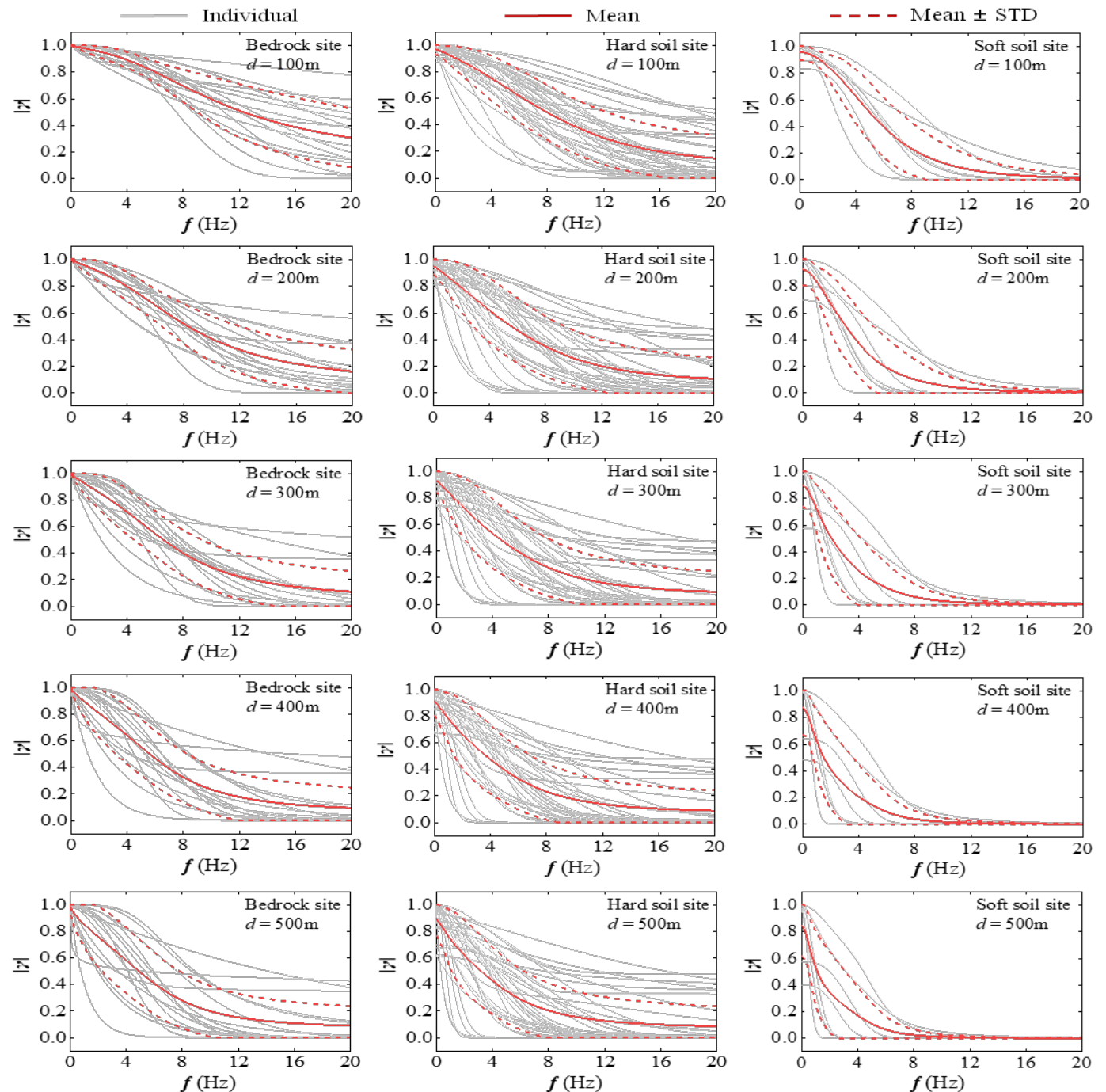
$V_s \geq 750$ m/s;

2) **Hard soil**

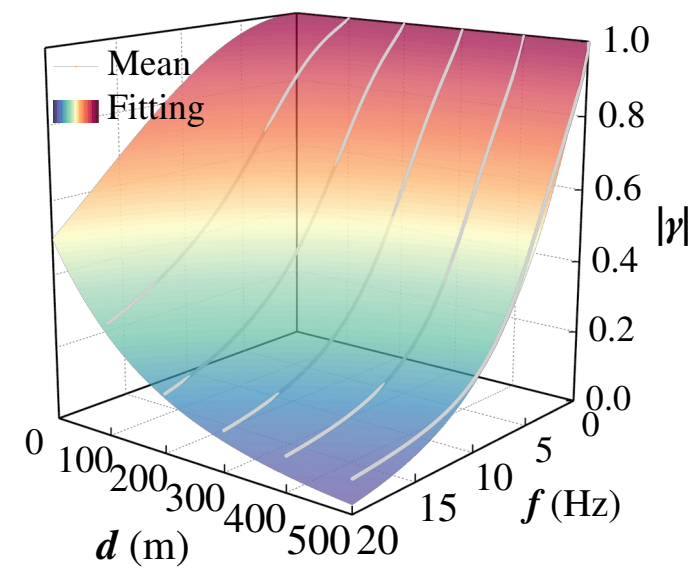
$250 \text{ m/s} < V_s < 750 \text{ m/s};$

3) **Soft soil**

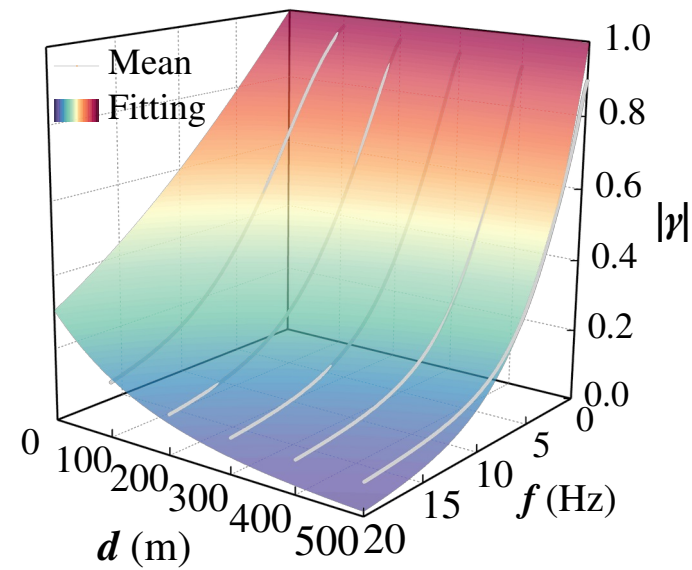
$V_s < 250$ m/s



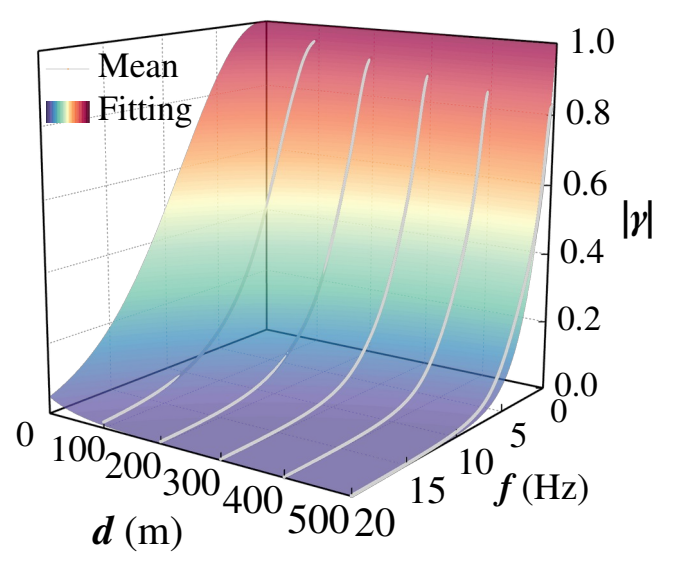
Statistical coherency loss functions



Baserock



Hard
soil



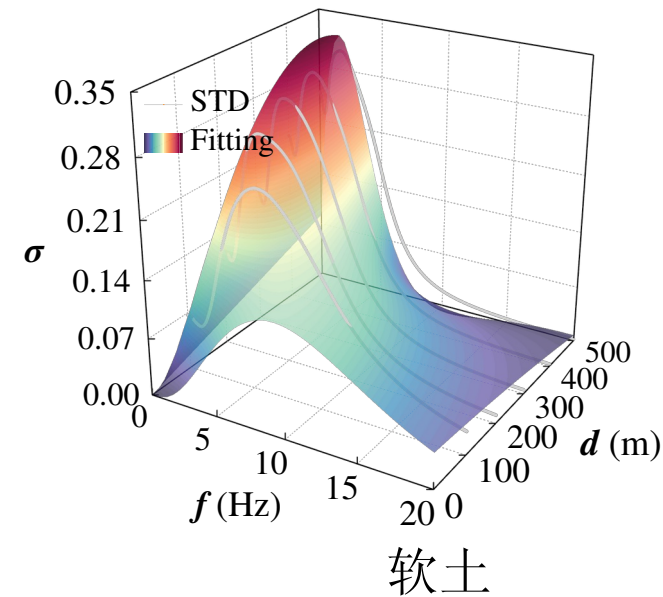
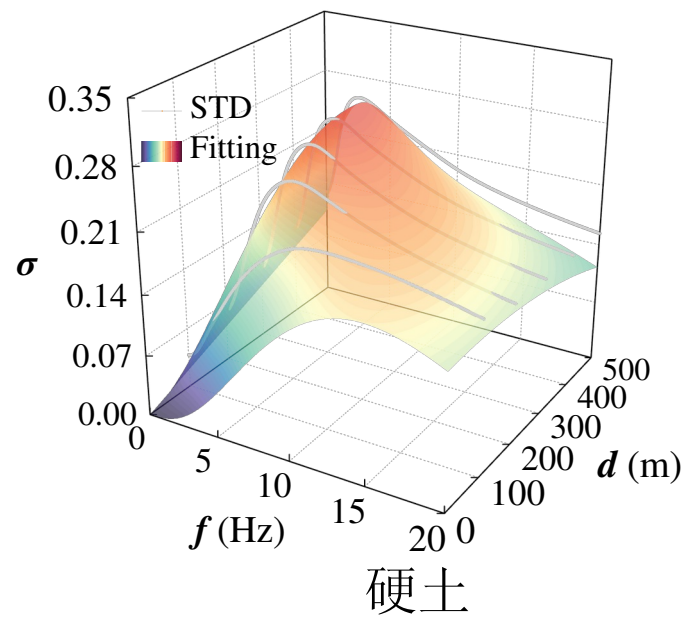
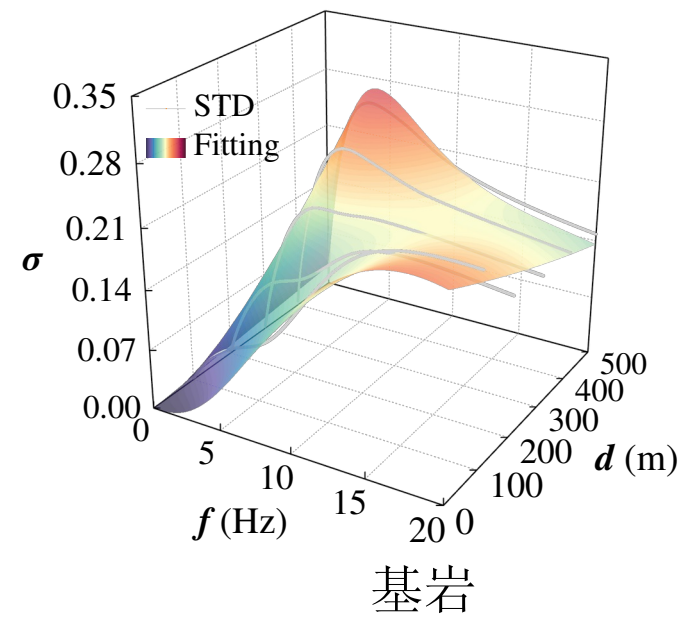
Soft
soil

Mean coherency loss function

$$|\gamma(f, d)| = 1/\exp (a_1f^2 + a_2f + a_3fd)$$

Paramet ers	Bedrock site	Hard soil site	Soft soil site
	1.83×10^{-3}	8.05×10^{-4}	7.97×10^{-3}
	-1.73×10^{-3}	4.32×10^{-2}	$-7.70 \times 10^{-}$

Standard deviations



$$\sigma = (b_2f^3 + b_3f^2d + b_4fd^2 + b_5fd + b_6d)/\exp(b_1f)$$

Parameter

s							
Bedrock site	1.70×10^{-1}	8.60×10^{-4}	-3.19×10^{-5}	1.07×10^{-7}	2.99×10^{-4}	4.68×10^{-5}	0.95
Hard soil site	2.03×10^{-1}	1.11×10^{-3}	-2.71×10^{-5}	-7.01×10^{-7}	6.50×10^{-4}	2.06×10^{-4}	0.87

$$|\gamma(f, d)| = |\gamma(f, d)|_{mean} \pm \sigma_\gamma$$

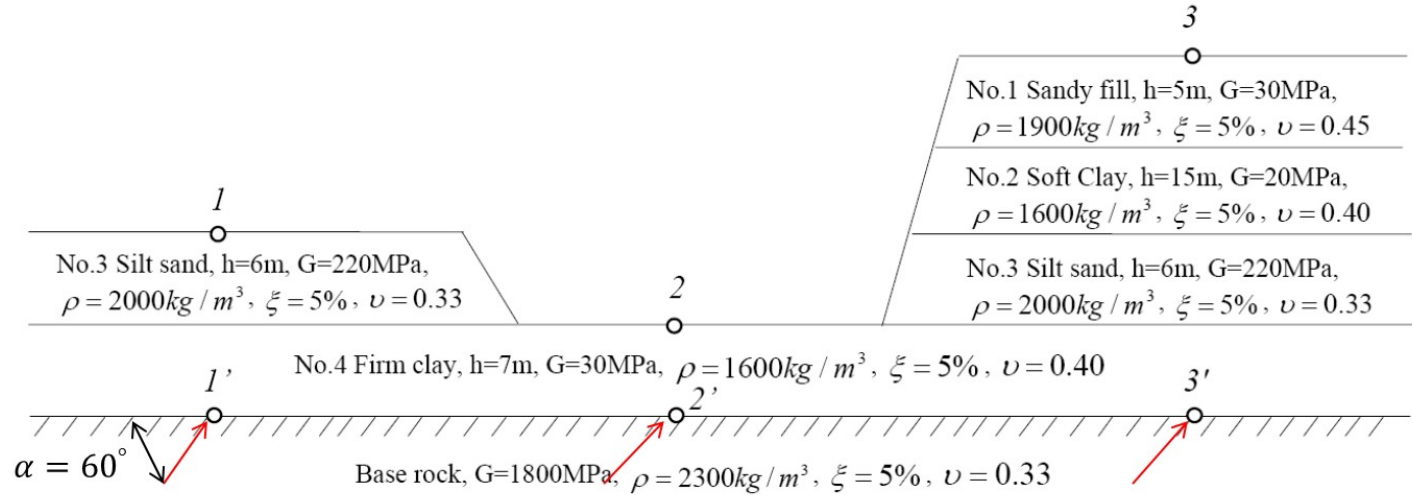
Haimen Ling, Hong Hao, Chao Li, Kaiming Bi, Xiaojun Fang, 'Statistical Analysis of Earthquake Ground Motion Coherency Loss and a Simplified Evaluation Method of Its Effect on Structural Responses', SDEE, Volume 194, 2025, Soft soil

$$|\gamma(f, d)| = |\gamma(f, d)|_{mean} \pm \sigma_{\gamma}$$

This model defines the statistical variation range of coherency loss functions, overcomes the dilemma of arbitrarily selecting a coherency loss function in modelling earthquake ground motion spatial variations. It provides a guide in modelling spatial ground motion coherency loss, i.e., mean, or mean plus/minus one standard deviation

It facilitates a prediction of the influence range of spatial ground motion coherency loss on structural responses

Predictions of spatial ground motion

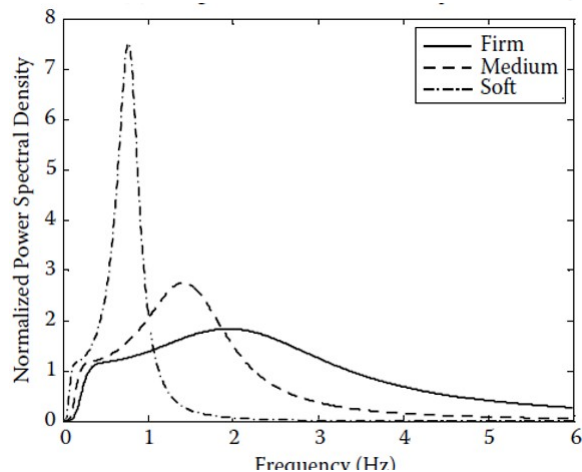


- 1) determine baserock motion
- 2) Consider P, SH, SV incident to local site, calculate site responses

Ruiz-Penzien

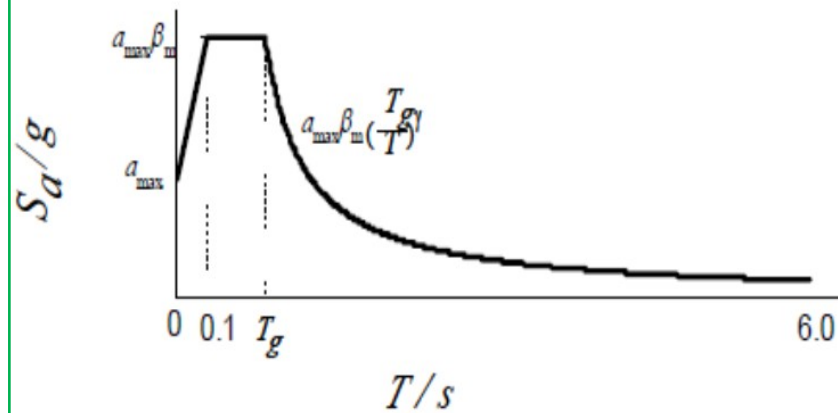
model

$$S_{CP}(\omega) = S_0 \frac{1 + 4\xi_g^2 \left(\frac{\omega}{\omega_g}\right)^2}{\left[1 - \left(\frac{\omega}{\omega_g}\right)^2\right]^2 + 4\xi_g^2 \left(\frac{\omega}{\omega_g}\right)^2} \frac{\left(\frac{\omega}{\omega_f}\right)^4}{\left[1 - \left(\frac{\omega}{\omega_f}\right)^2\right]^2 + 4\xi_f^2 \left(\frac{\omega}{\omega_f}\right)^2}$$



Design response spectrum

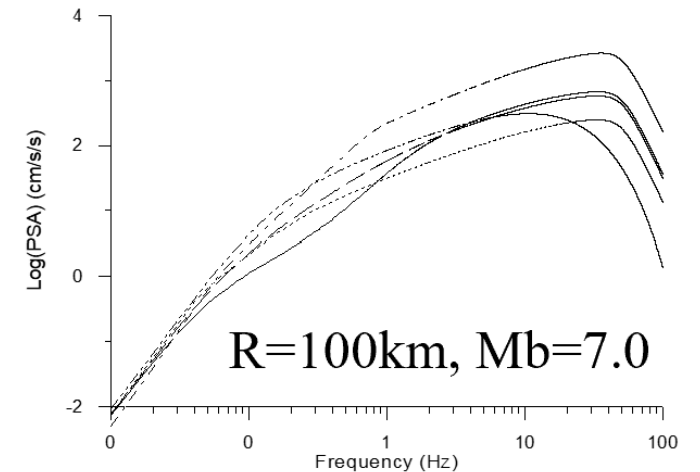
$$S(\omega) = -\frac{\xi}{\pi\omega} RSA^2(\omega) / \ln\left(-\frac{\pi}{\omega T} \ln p\right)$$



Attenuation

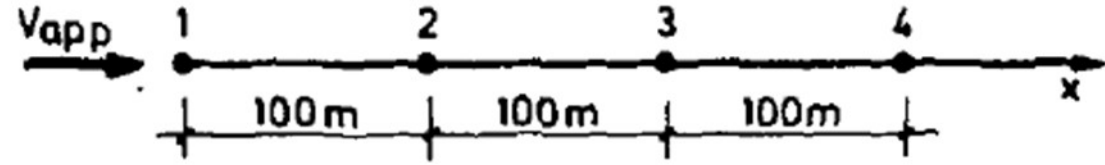
model

$$Y(M_0, R, f) = E(M_0, f) P(R, f) G(f) I(f)$$



Spatial ground motion simulation

i) Flat site



$$S(i\omega) = \begin{bmatrix} S_{11}(\omega) & S_{12}(i\omega) & \cdots & S_{1n}(i\omega) \\ S_{21}(i\omega) & S_{22}(\omega) & \cdots & S_{2n}(i\omega) \\ \cdots & \cdots & \cdots & \cdots \\ S_{n1}(i\omega) & S_{n2}(i\omega) & \cdots & S_{nn}(\omega) \end{bmatrix} = \begin{bmatrix} \gamma_{11}(\bar{\omega}) & \gamma_{12}(i\bar{\omega}) & \cdots & \gamma_{1n}(i\bar{\omega}) \\ \gamma_{21}(i\bar{\omega}) & \gamma_{22}(\bar{\omega}) & \cdots & \gamma_{2n}(i\bar{\omega}) \\ \cdots & \cdots & \cdots & \cdots \\ \gamma_{n1}(i\bar{\omega}) & \gamma_{n2}(i\bar{\omega}) & \cdots & \gamma_{nn}(\bar{\omega}) \end{bmatrix} S_0(\bar{\omega})$$

$$S(\bar{\omega}) = L(i\bar{\omega}) L^H(i\bar{\omega}) S_0(\omega)$$

$$L(i\bar{\omega}) = \begin{bmatrix} l_{11}(\bar{\omega}) & 0 & \cdots & 0 \\ l_{21}(i\bar{\omega}) & l_{22}(\bar{\omega}) & \cdots & 0 \\ \cdots & \cdots & \cdots & \cdots \\ l_{n1}(i\bar{\omega}) & l_{n2}(i\bar{\omega}) & \cdots & l_{nn}(\bar{\omega}) \end{bmatrix}$$

$$l_{ii}(\bar{\omega}) = \left[\gamma_{ii}(\bar{\omega}) - \sum_{k=1}^{i-1} l_{ik}(i\bar{\omega}) l_{ik}^*(i\bar{\omega}) \right]^{1/2}, \quad l_{ij}(i\bar{\omega}) = \frac{\gamma_{ij}(i\bar{\omega}) - \sum_{k=1}^{j-1} l_{ik}(i\bar{\omega}) l_{jk}^*(i\bar{\omega})}{l_{jj}(\bar{\omega})}$$

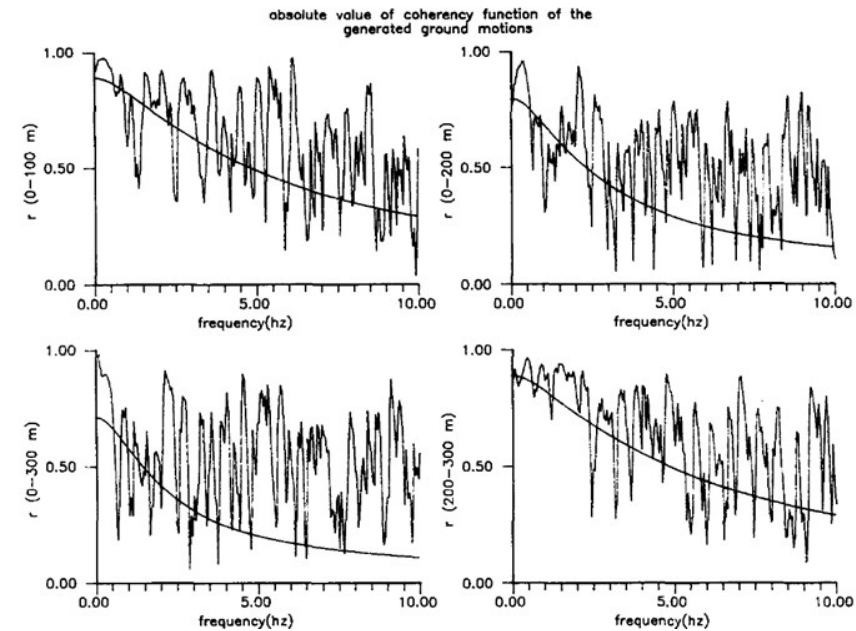
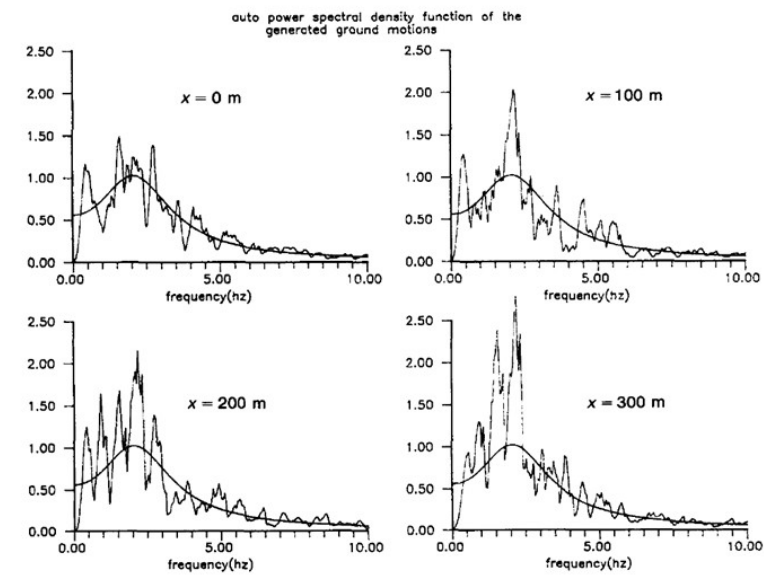
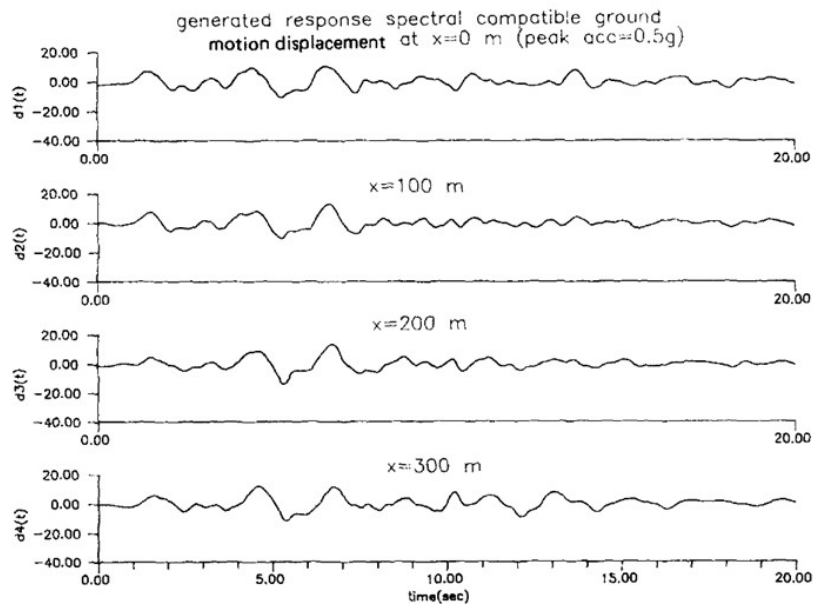
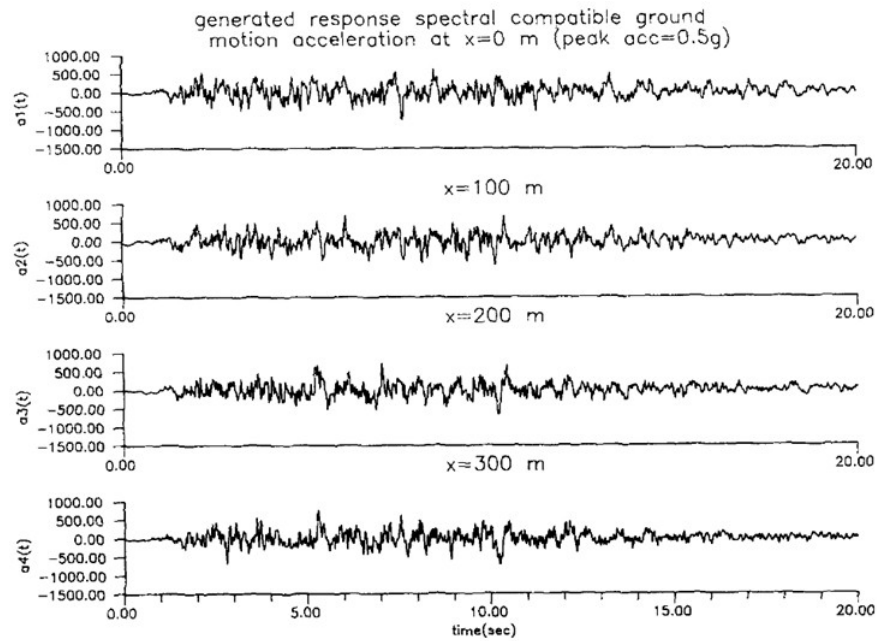
$$i = 1, 2, \dots, n;$$

$$j = 1, 2, \dots, i.$$

$$u_i(t) = \sum_{m=1}^i \sum_{n=1}^N A_{im}(\bar{\omega}_n) \times \cos[\bar{\omega}_n t + \beta_{im}(\bar{\omega}_n) + \phi_{mn}(\bar{\omega}_n)]$$

$$A_{ij}(\bar{\omega}) = \sqrt{4S_0(\omega) \Delta\omega} |l_{ij}(i\bar{\omega})|, \quad 0 \leq \bar{\omega} \leq \bar{\omega}_N;$$

$$\beta_{ij}(\bar{\omega}) = \tan^{-1} \left(\frac{\text{Im}[l_{ij}(i\bar{\omega})]}{\text{Re}[l_{ij}(i\bar{\omega})]} \right), \quad 0 \leq \bar{\omega} \leq \bar{\omega}_N.$$

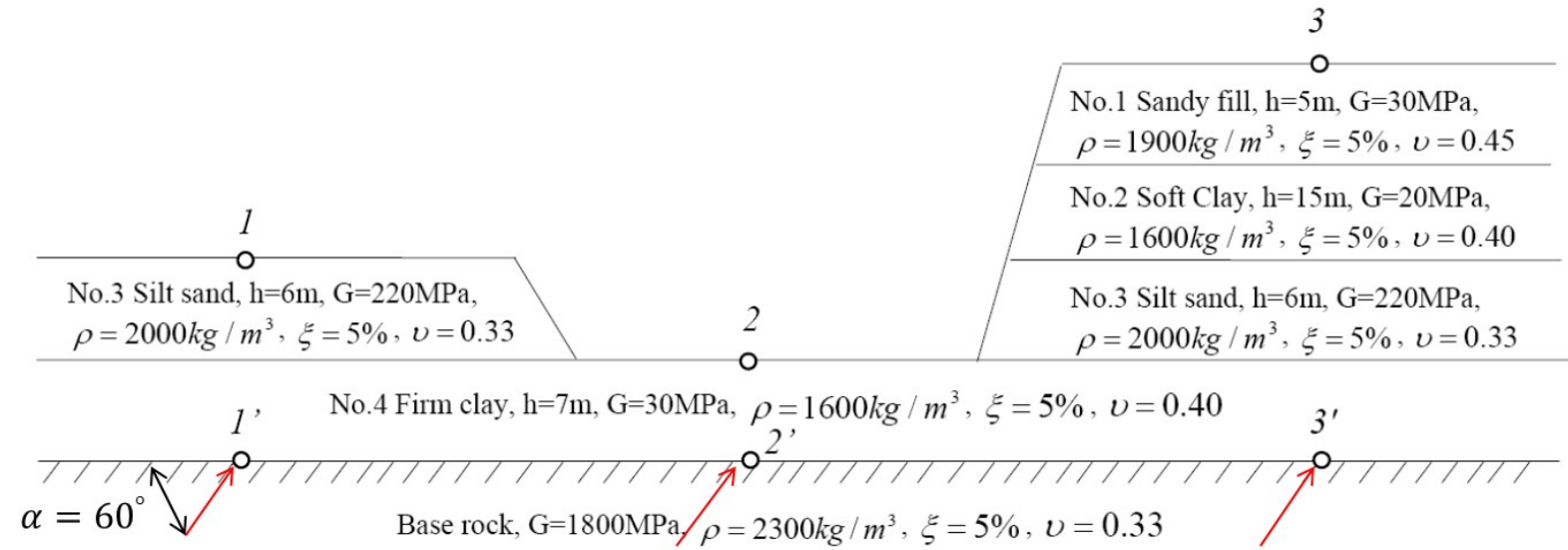


Hong Hao, C.S. Oliveira and J. Penzien, 'Multiple station ground motion processing and simulation based on SMART-1 data', International journal of nuclear engineering and design, Vol. 111, No.3, pp293-310, Elsevier Science Publishers, Holland, Feb. 1989.

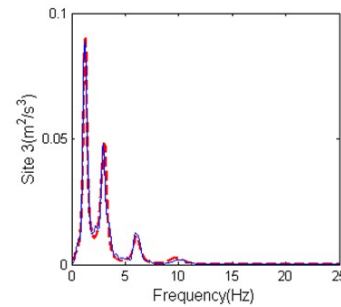
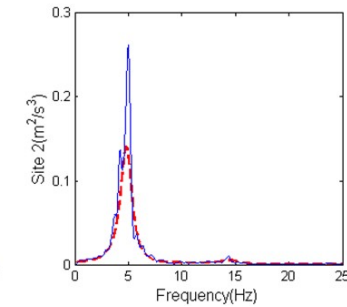
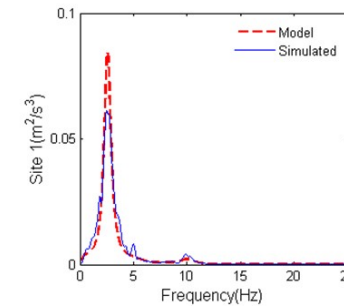
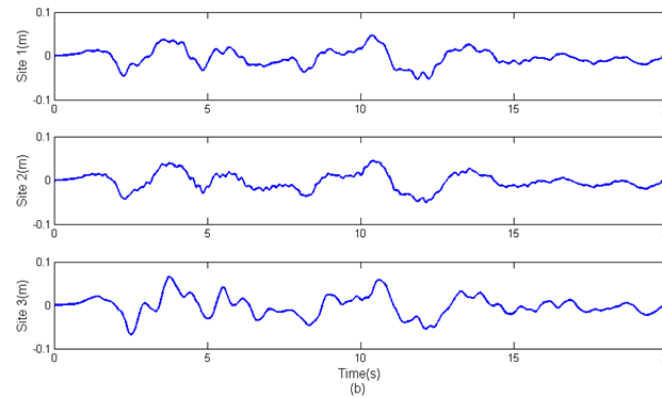
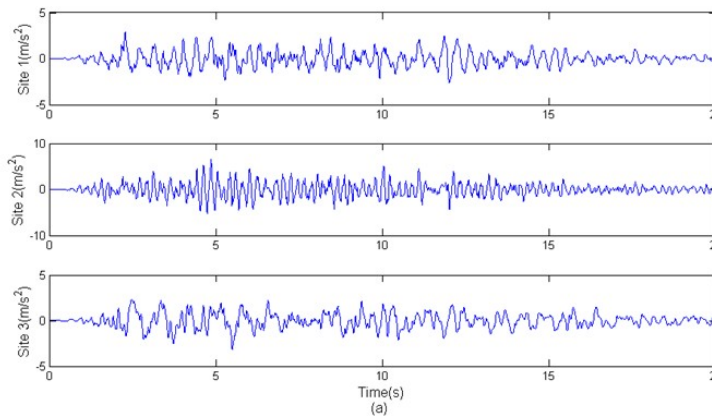
ii) Uneven site

Assume different incident angles and wave types, calculate site responses

$$S_{jj}(\omega) = |H_j(i\omega)|^2 S_g(\omega) \quad j = 1, 2, \dots, n$$

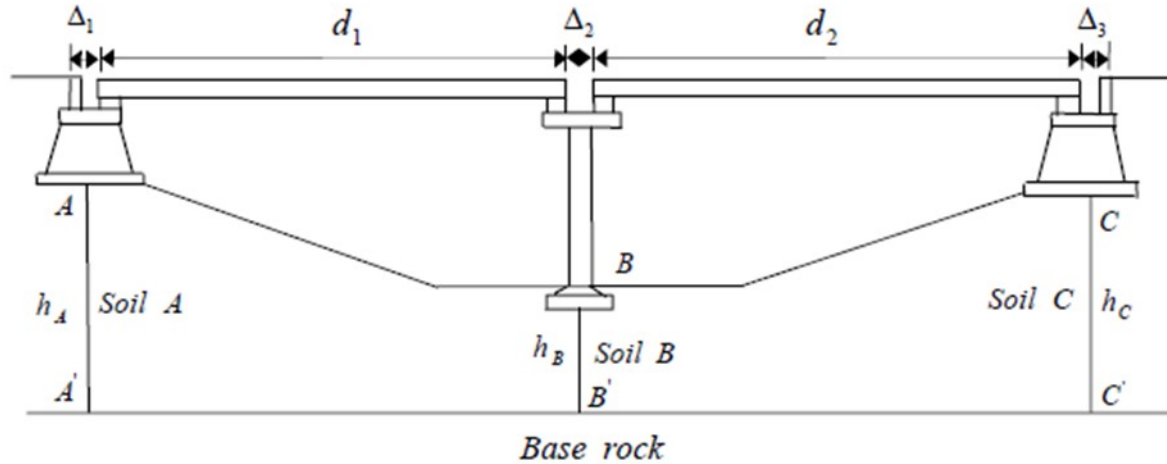


$$S_{jk}(i\omega) = H_j(i\omega)H_k^*(i\omega)S_g(\omega)\gamma_{j'k'}(d_{j'k'}, i\omega) \quad j, k = 1, 2, \dots, n$$



Kaiming Bi and Hong Hao, 'Modelling and simulation of spatially varying earthquake ground motions at a canyon site with multiple soil layers Probabilistic Engineering Mechanics', International Journal of Probabilistic Engineering Mechanics, Vol.29, pp92-104, 2012

Structural response analysis : 1) Stochastic analysis



Relative response power spectrum

$$S_{u_s}(\bar{\omega}) = \left\{ \left[H_1 H_1^* / 2 - \frac{1}{\bar{\omega}^2} \text{real}(H_1) + \frac{1}{2\bar{\omega}^4} \right] \left[1 + |\gamma_{12}| \cos \frac{\bar{\omega} d_{12}^I}{v} \right] + \left[H_2 H_2^* - \frac{2}{\bar{\omega}^2} \text{real}(H_2) + \frac{1}{\bar{\omega}^4} \right] - \left[\text{real}(H_1 H_2^*) - \frac{1}{\bar{\omega}^2} \text{real}(H_1) - \frac{1}{\bar{\omega}^2} \text{real}(H_2) + \frac{1}{\bar{\omega}^4} \right] \left[|\gamma_{13}| \cos \frac{\bar{\omega} d_{13}^I}{v} + |\gamma_{23}| \cos \frac{\bar{\omega} d_{23}^I}{v} \right] \right\} S_g(\bar{\omega})$$

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{K}_{ss}\mathbf{U} = \mathbf{M}\mathbf{K}_{cc}^{-1}\mathbf{K}_{sb}\ddot{\mathbf{V}}_g \quad \text{Dynamic}$$

$$u_1^{qs}(t) = \frac{1}{2}(v_1 + v_2)$$

Pseudo-static

$$u_4^{qs}(t) = v_3$$

$$S_{kl}(i\bar{\omega}) = S_g(\bar{\omega})\gamma_{kl}(i\bar{\omega}, d_{kl}) \quad \text{Cross power spectrum}$$

spectral moment Zero mean cross rate Shape factor

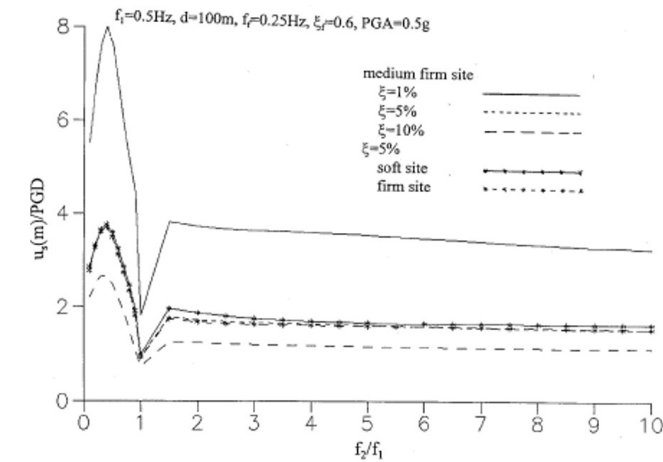
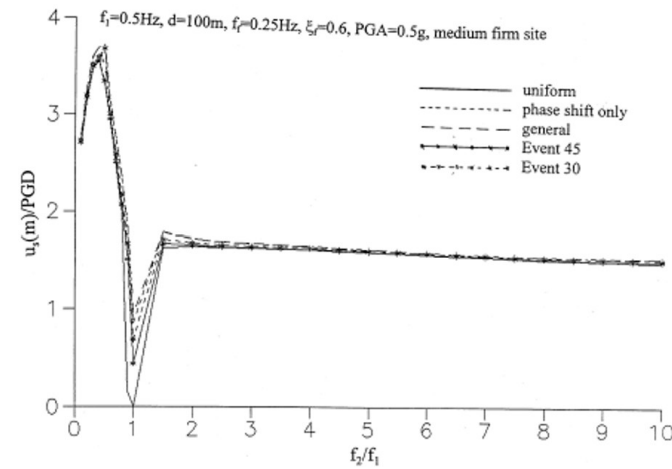
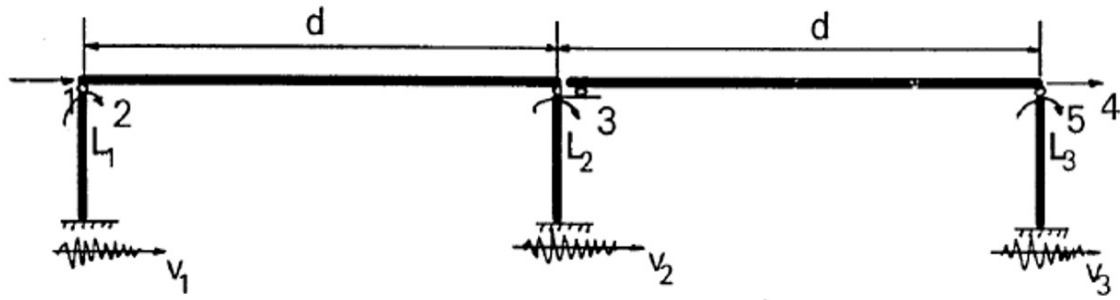
$$\lambda_m \approx \int_0^{\omega_c} \omega^m S(\omega) d\omega \quad v = \frac{1}{\pi} \sqrt{\frac{\lambda_2}{\lambda_0}} \quad \delta = \sqrt{1 - \frac{\lambda_1^2}{\lambda_0 \lambda_2}}$$

Mean peak response

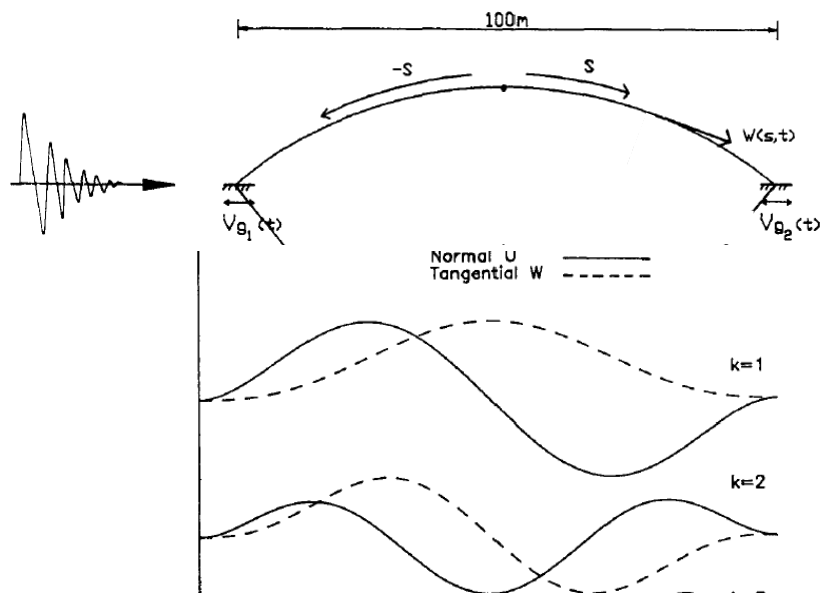
$$x_{\max} = \left(\sqrt{2 \ln v_e T} + \frac{0.5772}{\sqrt{2 \ln v_e T}} \right) \sigma \quad \sigma = \sqrt{\lambda_0}$$

$$v_e T = \begin{cases} \max(2.1, 2\delta T) & 0 \leq \delta < 0.1 \\ (1.63\delta^{0.45} - 0.38)vT & 0.1 \leq \delta < 0.69 \\ vT & \delta \geq 0.69 \end{cases}$$

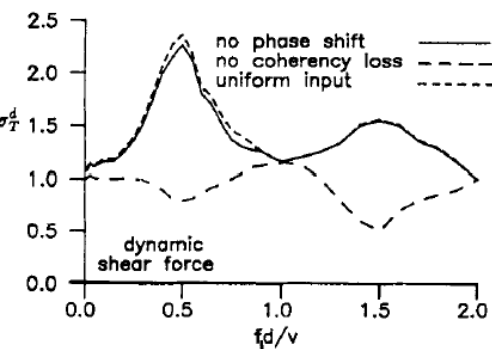
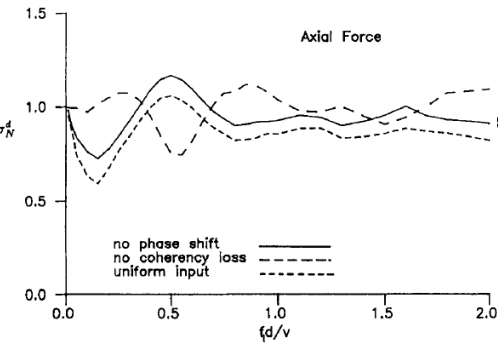
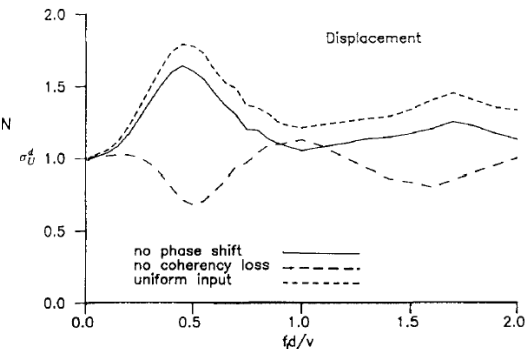
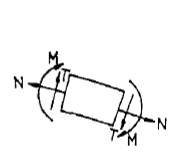
Hong Hao, A parametric study of required seating lengths for bridge decks during earthquake, International Journal of Earthquake Engineering and Structural Dynamics, Vol. 27, pp91-103, 1998.



Hong Hao, "A parametric study of required seating lengths for bridge decks during earthquake", International Journal of Earthquake Engineering and Structural Dynamics, Vol. 27, pp91-103, 1998.

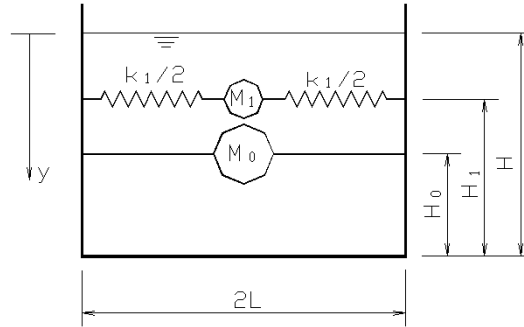
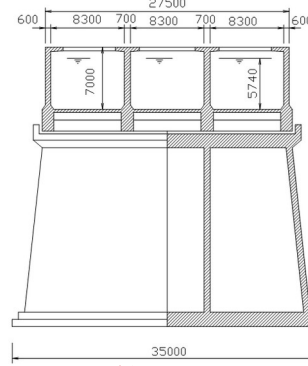
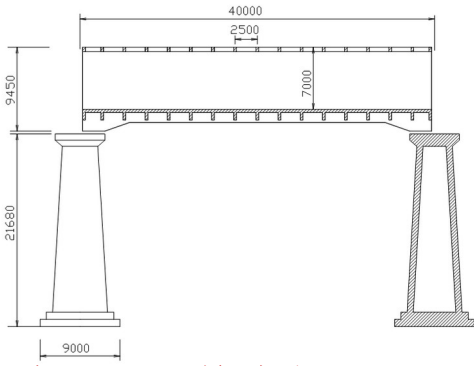


Mode shapes



Hong Hao, "Arch responses to correlated multiple excitations" International Journal of Earthquake engineering and Structural Dynamics, Vol. 22, pp389-404, USA, May 1993.

Time-history analysis, an elevated aqueduct $PGA=70\text{ cm/s}^2$, 极端 $PGA=400\text{ cm/s}^2$

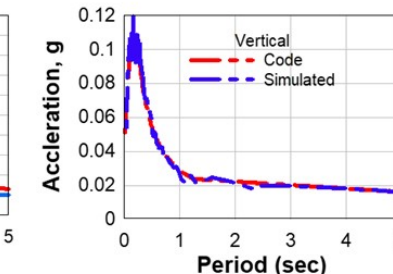
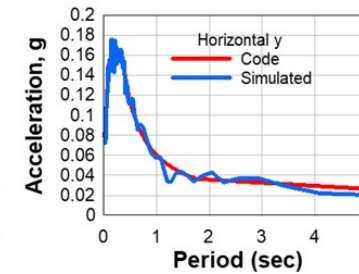
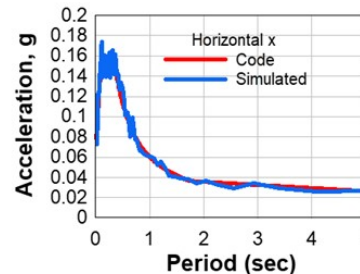
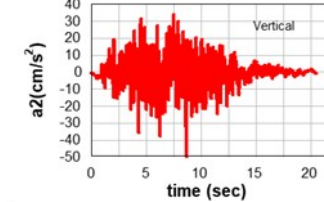
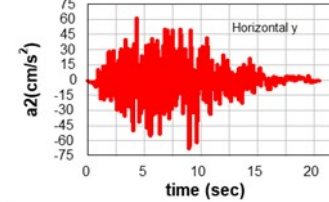
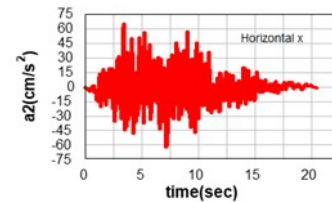
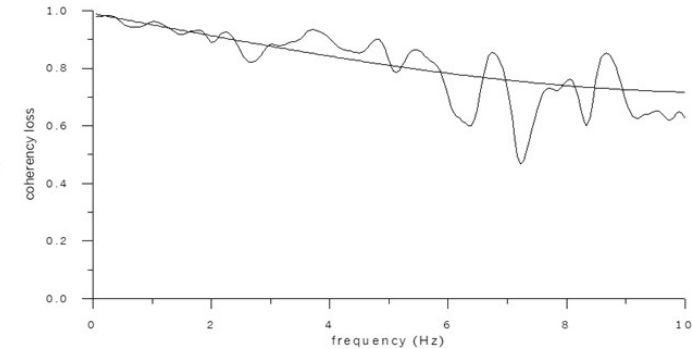
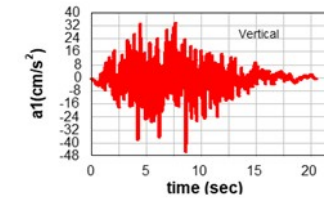
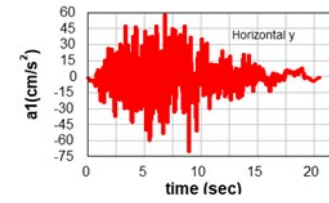
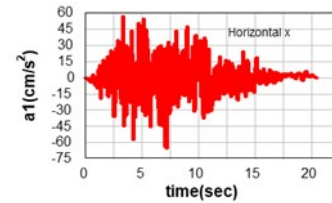
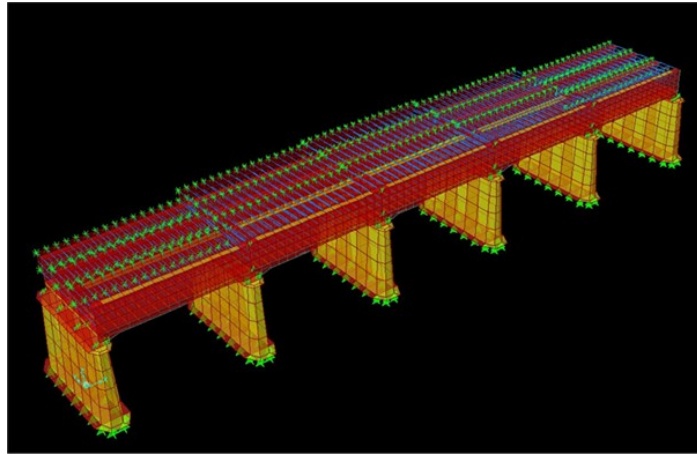


1 Comparison of measured and calculated aqueduct vibration frequencies

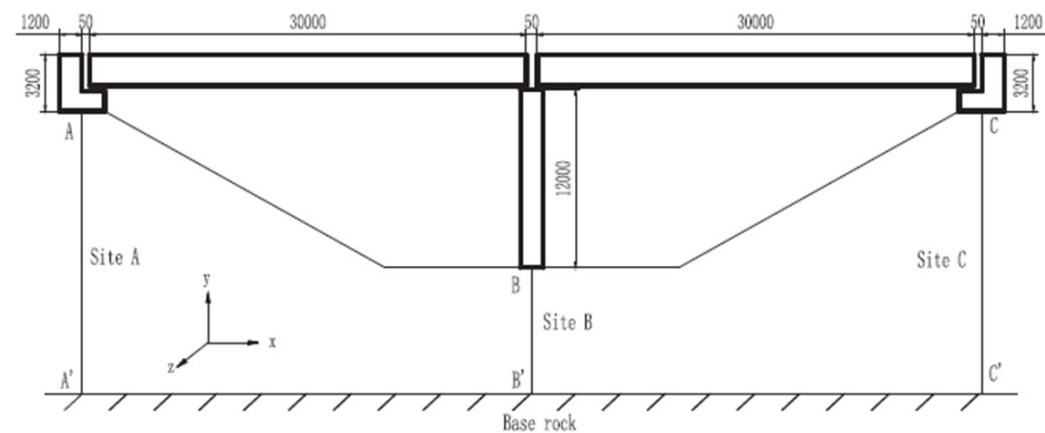
Water level	Longitudinal (Hz)			Transverse (Hz)		
	Test	Numerical	Error	Test	Numerical	Error
Empty	2.64	2.70	2.3%	4.73	3.95	-16.5%
Full	2.24	2.70	20.5%	3.59	3.04	-15.3%

单跨 40 m, 单跨水重 5717 ton, 共 17 跨, 总长 680 m

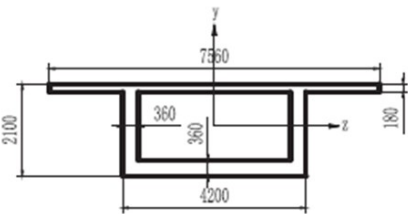
Housner 水模
开刊



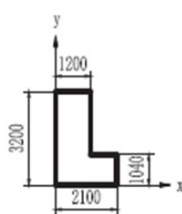
Wenyi Chen and H. Hao, 'Dynamic response analysis of large aqueduct to earthquake ground excitations', Proceedings of 13th WCEE, Vancouver, Canada, August 1-6 2004.



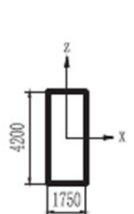
(a)



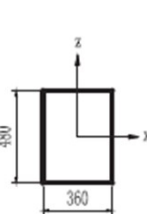
(b)



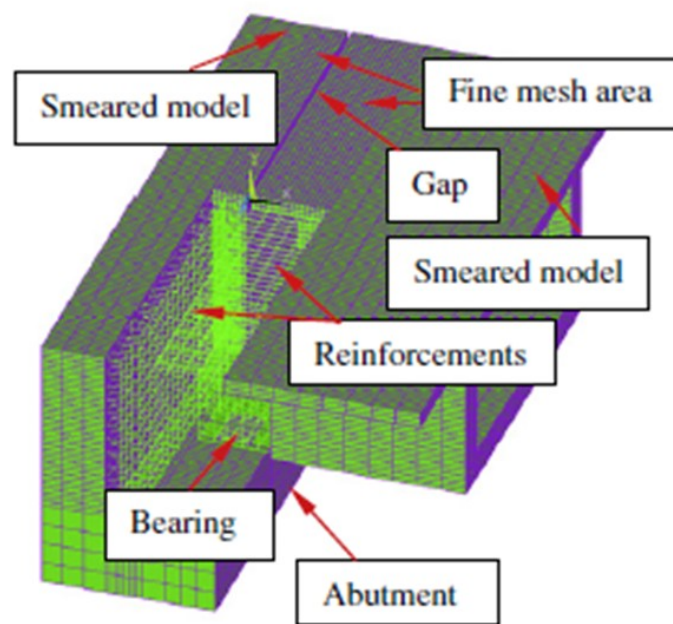
(c)



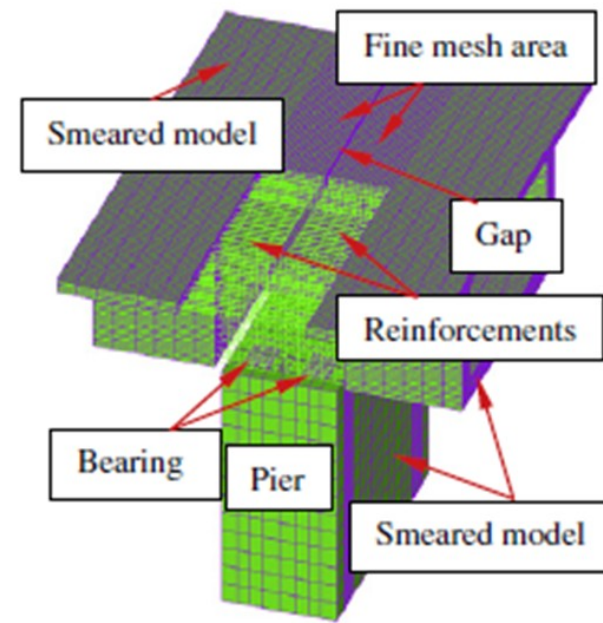
(d)



(e)



(a)



(b)

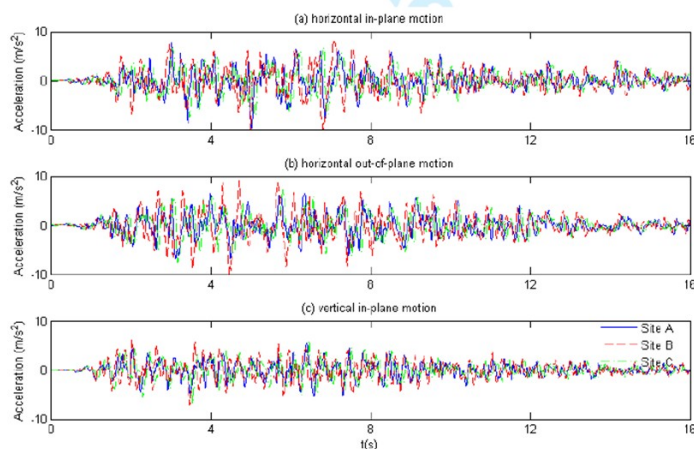


Figure 7. Simulated acceleration time histories with soft soil condition and intermediately correlated coherency loss

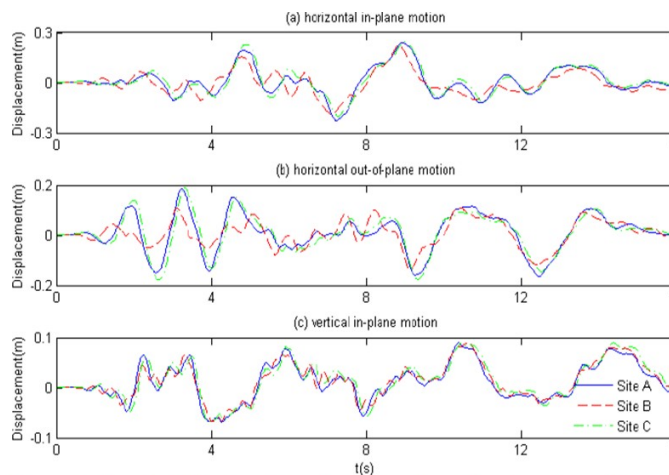


Figure 8. Simulated displacement time histories with soft soil condition and intermediately correlated coherency loss

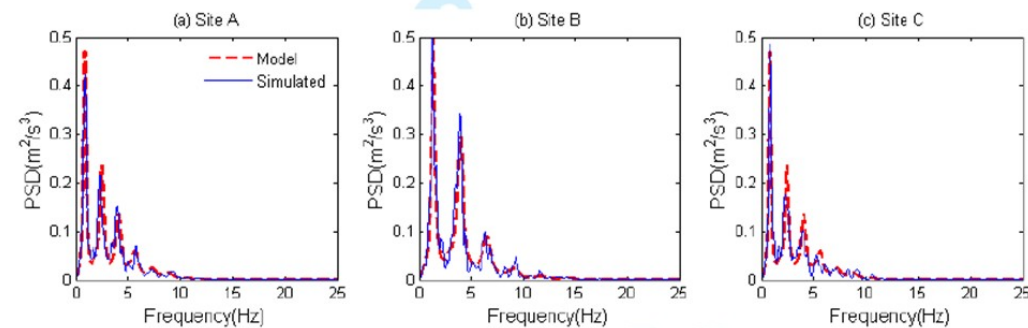
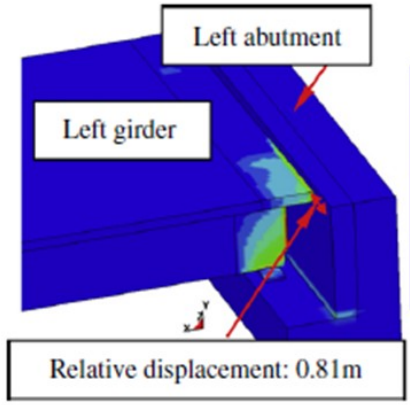
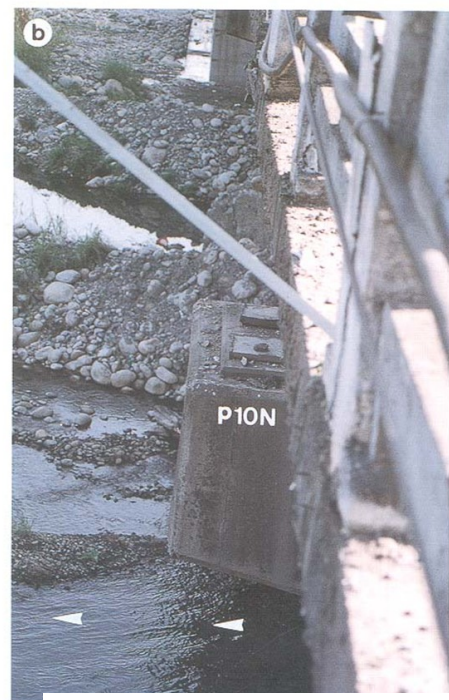
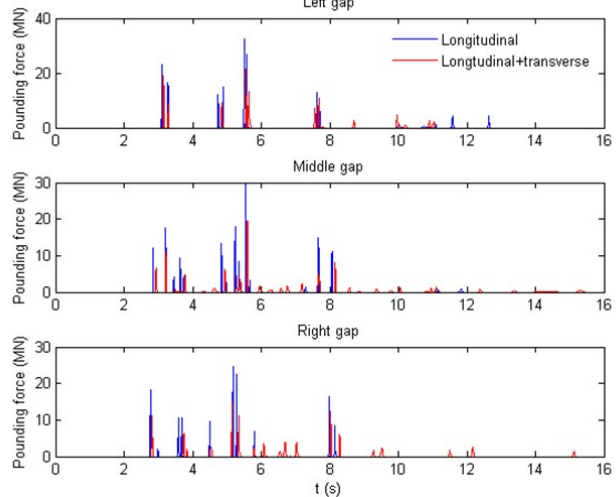
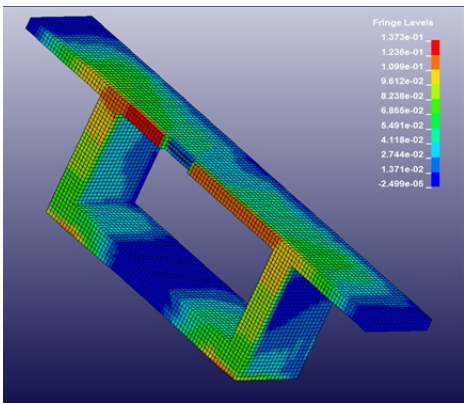
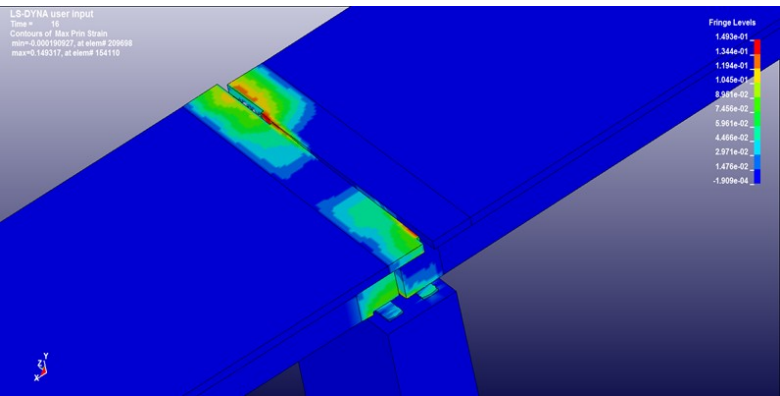
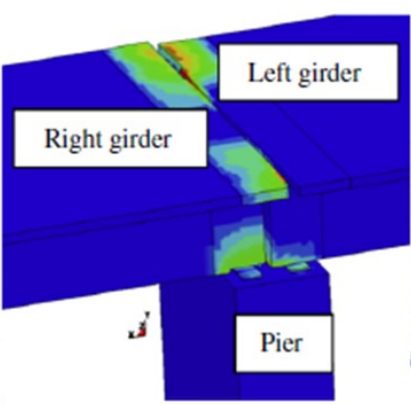


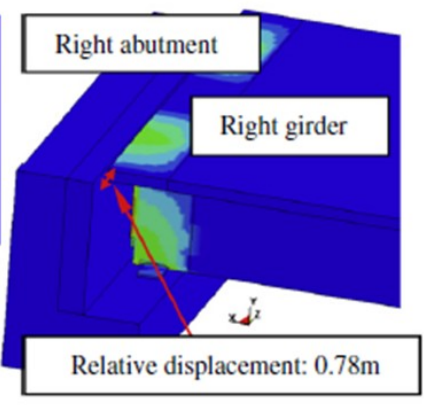
Figure 9. Comparison of PSDs between the generated horizontal in-plane soft soil motions on ground surface with the respective theoretical model values



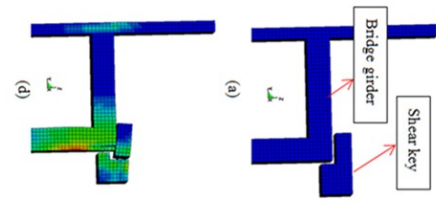
(a)



(b)



(c)



(d)



(e)



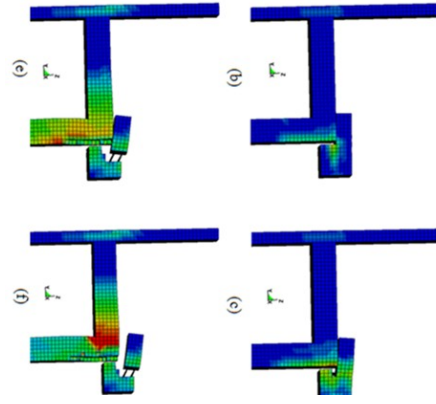
(a)

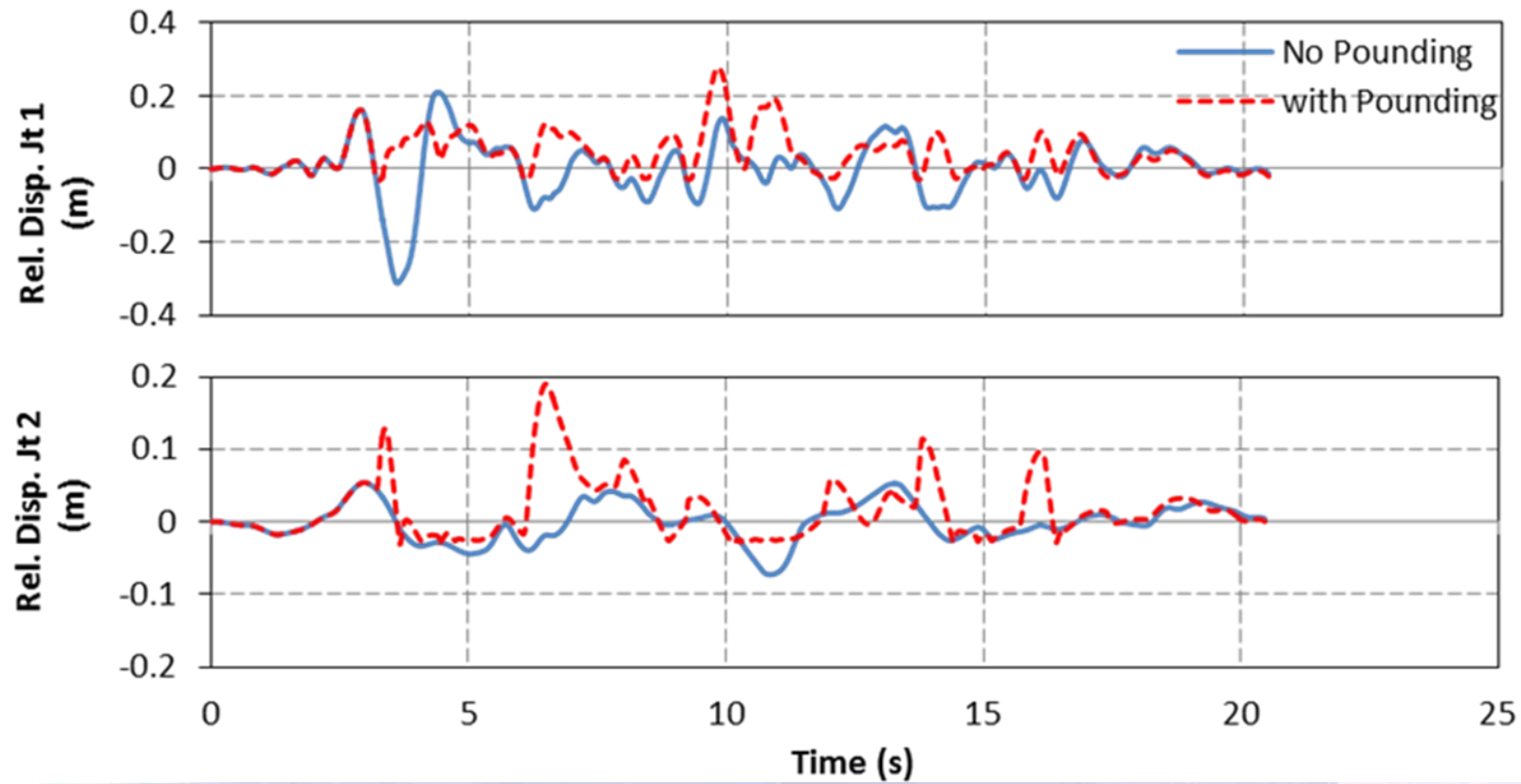


(b)



(c)





Kaiming Bi, Hong Hao. Numerical simulation of pounding damage to bridge structures under spatially varying ground motions. *Engineering Structures* 2013; 46: 62-76.

Design Analysis: EURO8 & Italian Code

- 1) Use standard design response spectrum for uniform input, calculate dynamic response M_d .
- 2) From design spectrum, calculate the maximum ground displacement d_g :
- 3) Determine a set of in-phase ground displacement at multiple supports A .
- 4) Apply ground displacement set A at multiple bridge supports, calculate pseudo response M_A .
- 5) Determine a set of out-of-phase ground displacement at multiple supports B .
- 6) Apply ground displacement set B at multiple bridge supports, calculate pseudo response M_B .

7) Combine response responses

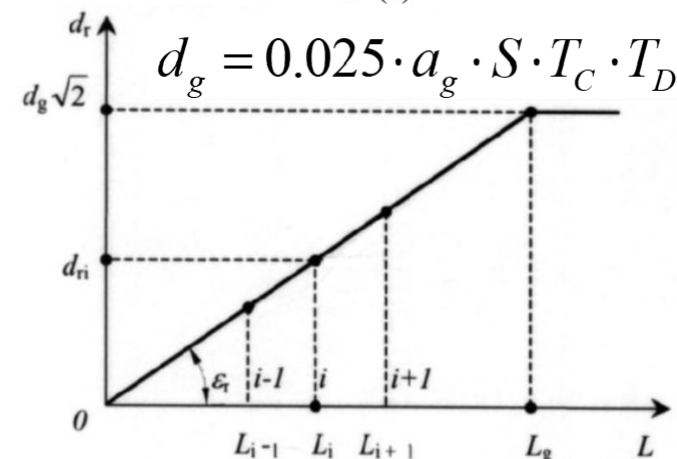
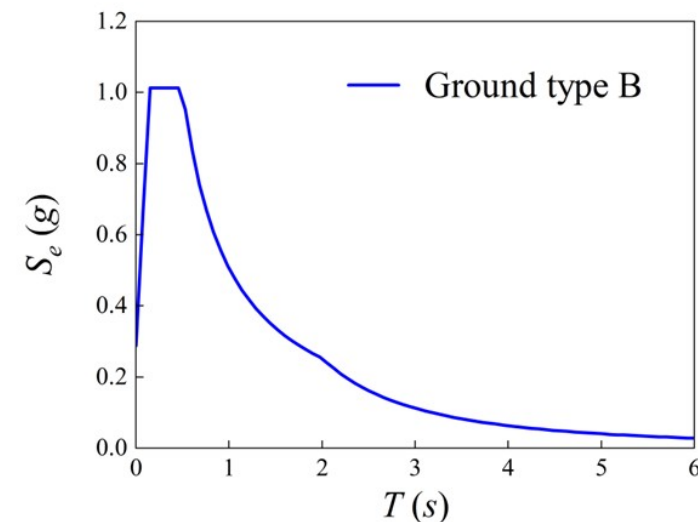
SRSS-A:

$$M_{Ati} = \sqrt{M_{di}^2 + M_{Ai}^2}$$

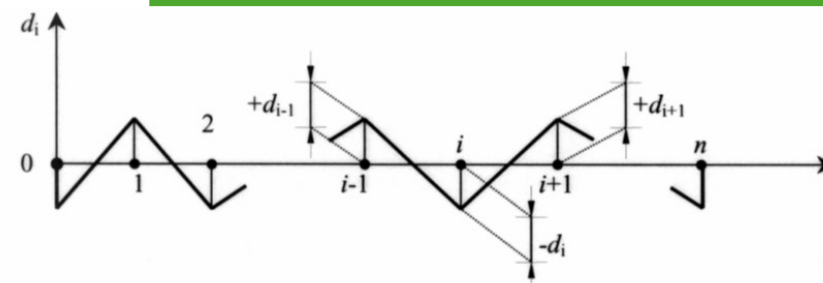
SRSS-B:

$$M_{Bti} = \sqrt{M_{di}^2 + M_{Bi}^2}$$

$$M_{ABti} = \sqrt{M_{di}^2 + M_{Ai}^2 + M_{Bi}^2}$$

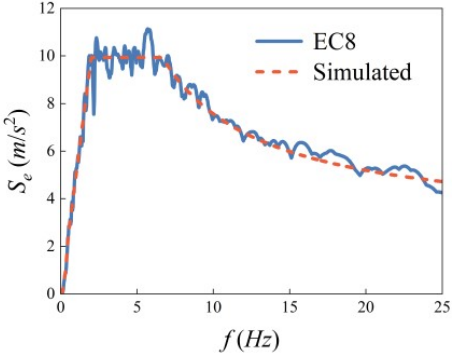
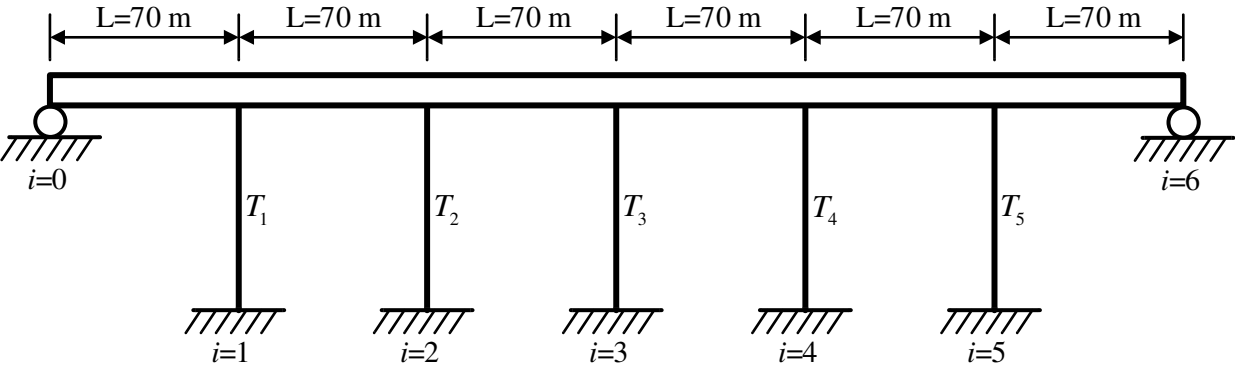


In-phase ground displacement set A

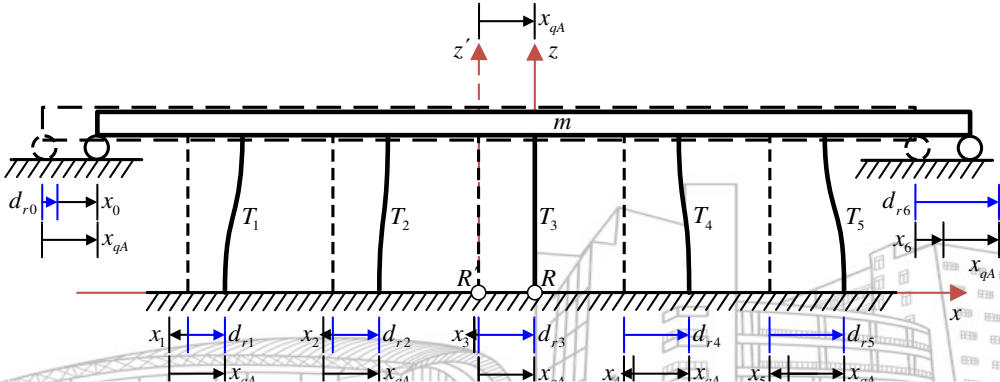


Out-of-phase ground displacement set B

Verification Analysis



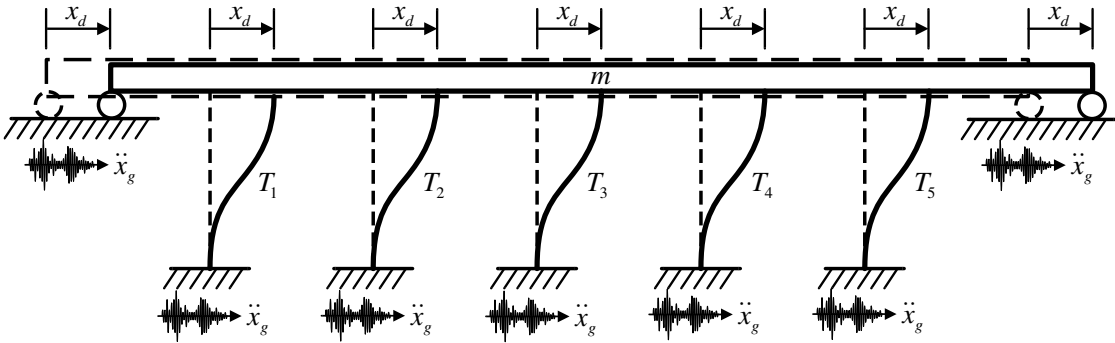
Simulate response spectrum compatible ground motions



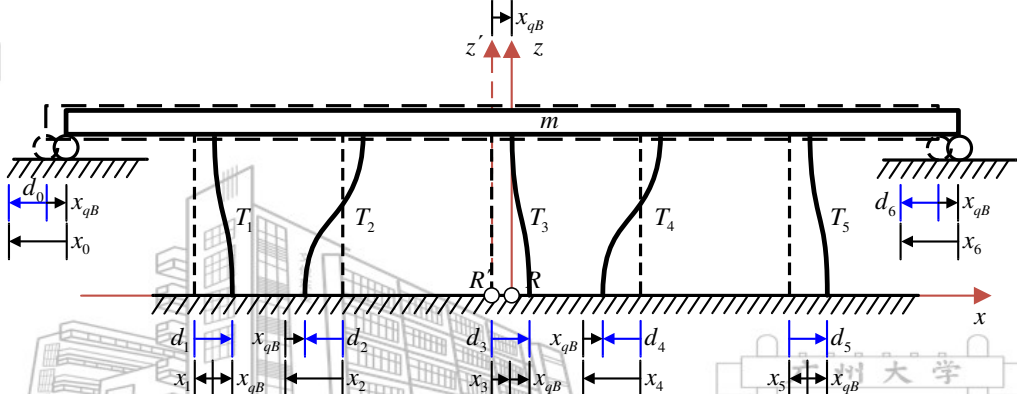
Bridge response under support movement set A

Bridge model Parameters

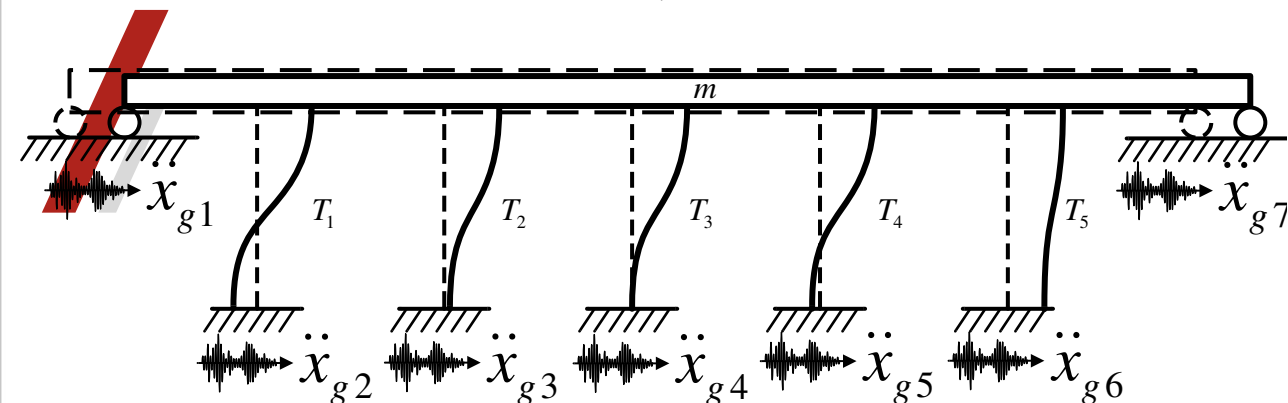
Parameter	Span length	Dead load	Height	Fundamental Period	Damping ratio
Value	70 m	200 kN/m	7.5 m	0.43 s	5%



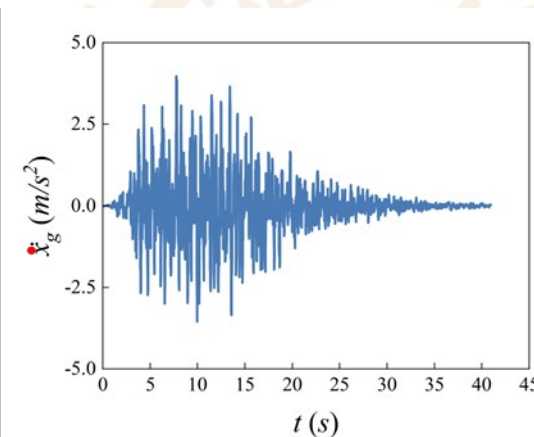
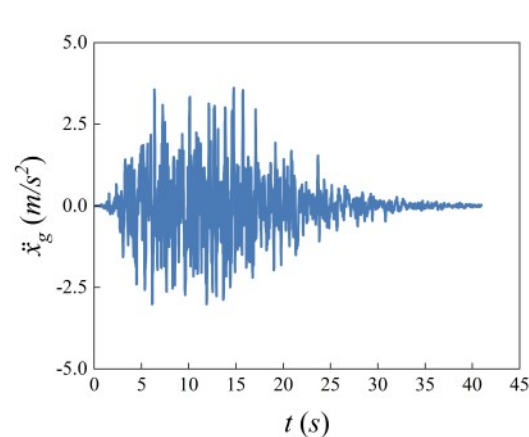
Dynamic response analysis subjected to uniform excitation



Bridge response under support movement set B



Response prediction of bridges subjected to spatially varying ground motions



Simulated spatially varying ground motions

Cases		η_{ABi}				
		Pier 1	Pier 2	Pier 3	Pier 4	Pier 5
Highly correlated	Case 1	-27.16%	0.83%	27.28%	6.39%	-25.33%
Inter. correlated	Case 2	500m/s	-29.25%	-10.02%	6.89%	-7.60%
Weakly correlated	Case 3		-23.14%	-19.17%	-21.71%	-30.48%
Highly correlated	Case 4	1000m/s	-5.64%	22.37%	76.98%	51.22%
Inter. correlated	Case 5		-11.66%	2.47%	14.70%	15.83%
Weakly correlated	Case 6		-27.24%	-19.96%	-13.84%	-15.98%
Highly correlated	Case 7	2000m/s	-4.63%	3.83%	13.41%	17.09%
Inter. correlated	Case 8		-12.40%	-7.48%	-0.63%	-3.23%
Weakly correlated	Case 9		-24.80%	-26.11%	-16.03%	-18.55%

Structural response to spatially varying ground motions

Dynamic response

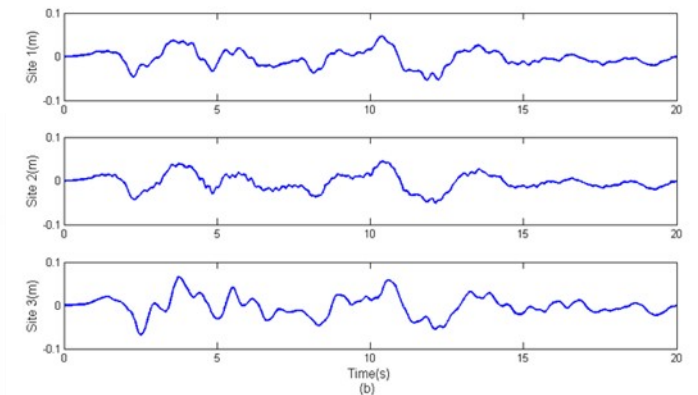
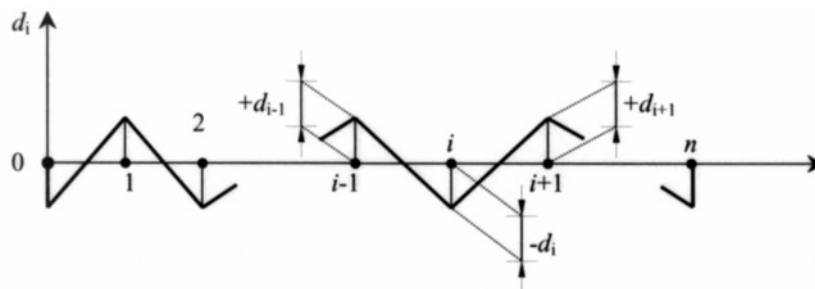
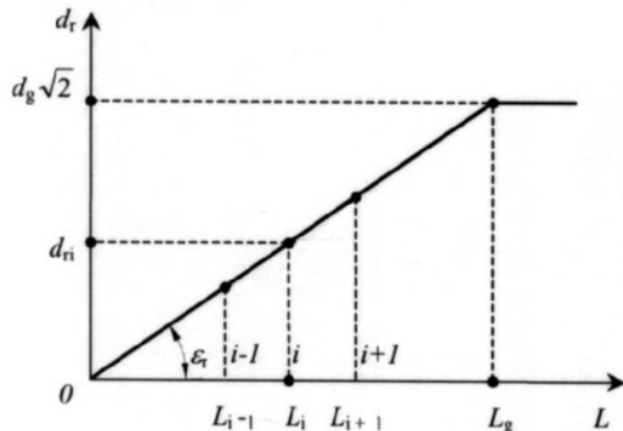
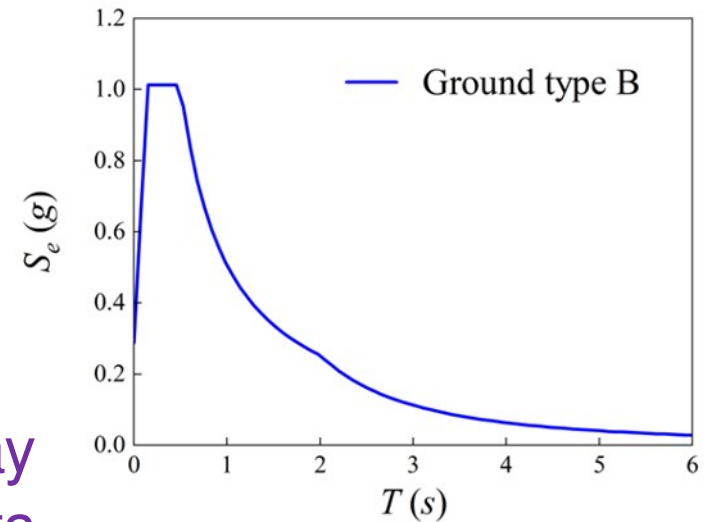
$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{K}_{ss}\mathbf{U} = \mathbf{M}\mathbf{K}_{ss}^{-1}\mathbf{K}_{sb}\ddot{\mathbf{V}}_g$$

Quasi-static response

$$u_{ps} = \mathbf{R}u_g \quad \mathbf{R} = -\mathbf{K}^{-1}\mathbf{K}_{sg}$$

Code method: uniform ground motion assumption for dynamic response analysis, which fails to capture the inertial, damping and elastic resistance of structures associated to nonuniform ground excitations

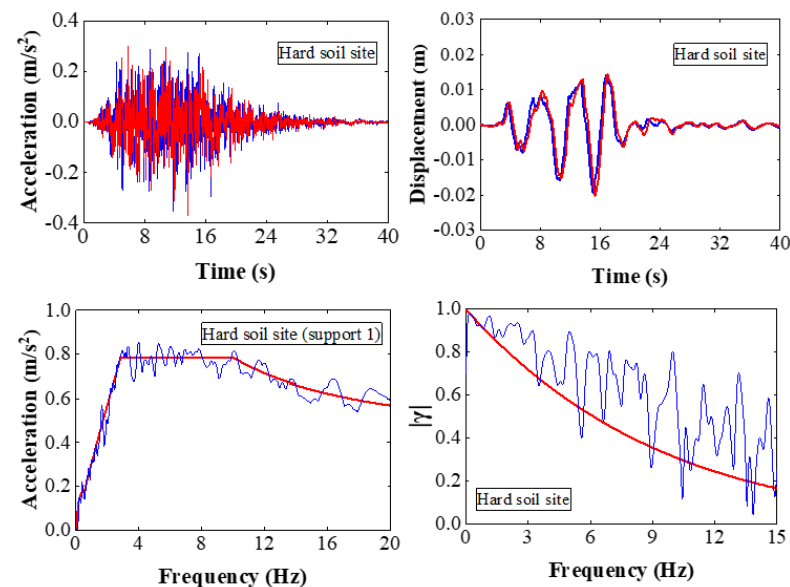
Assumed two sets of nonuniform support displacements may not represent the true spatially varying ground displacements



To improve the prediction accuracy for design analysis, we recently proposed a Multi Supports Response Spectrum (MSRS) method. The method keeps the simplicity as the normal design analysis methodology, but more realistically account for the effects of ground motion spatial variations on structural responses.

Step-1: Establishment of SVGEM Database

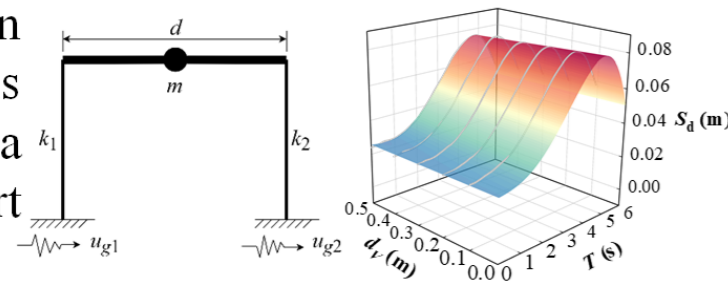
A large amount of ground motion time histories is simulated, accounting for the wave-passage and coherency loss effects.



Design-oriented MSRS procedure

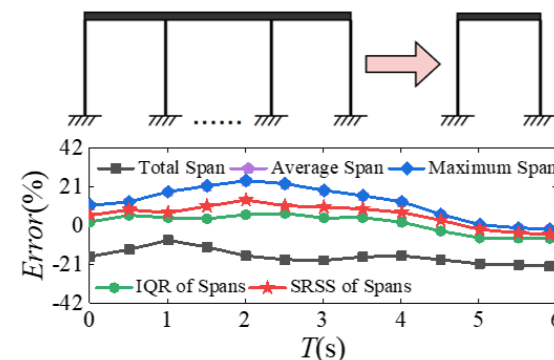
Step-2: Multi-support design response spectrum

Multi-support design response spectrum is developed using a one-span two-support SDOF system.

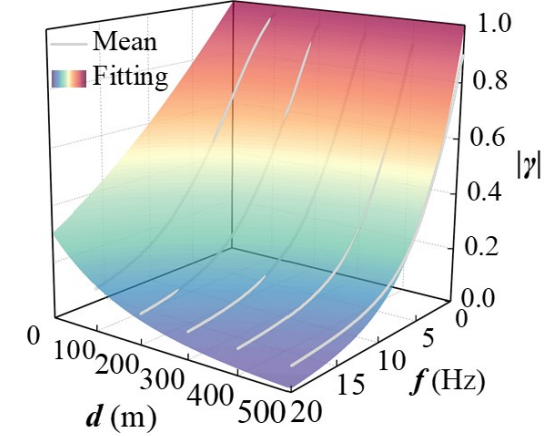
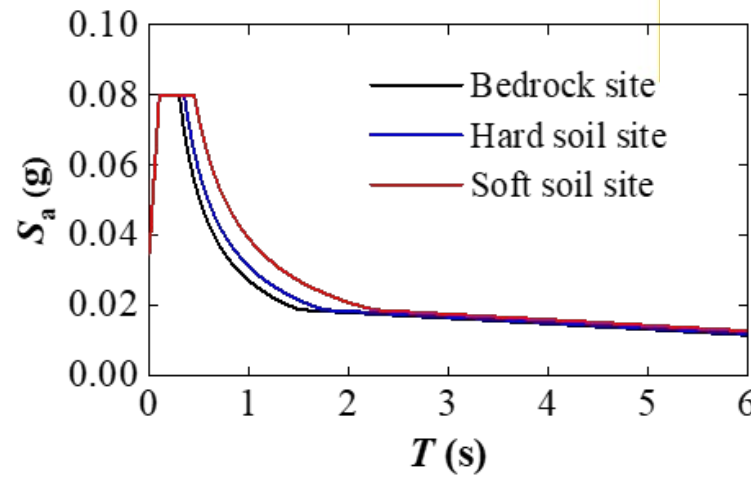
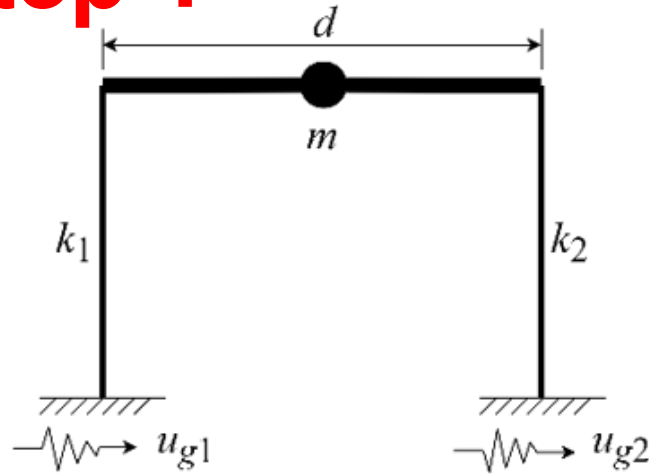


Step-3: Effective-span equivalence strategy

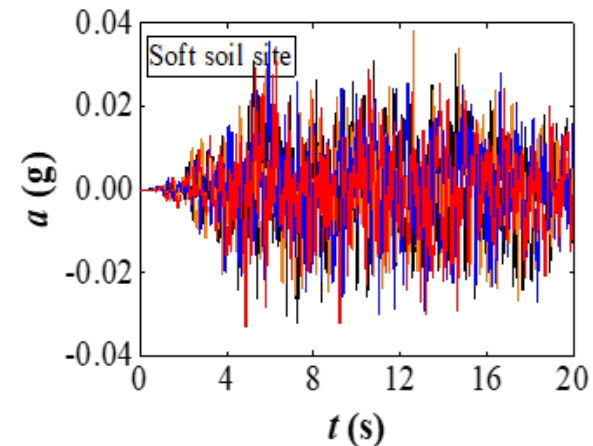
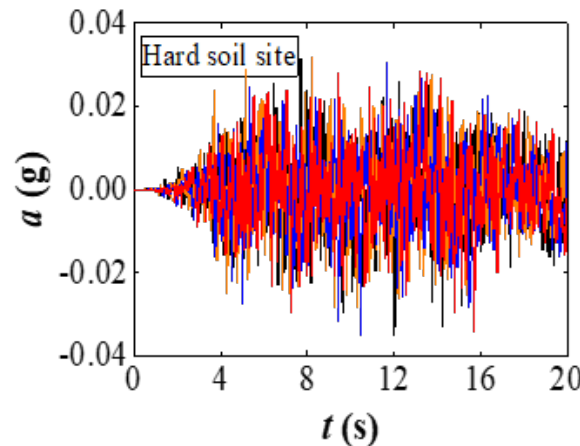
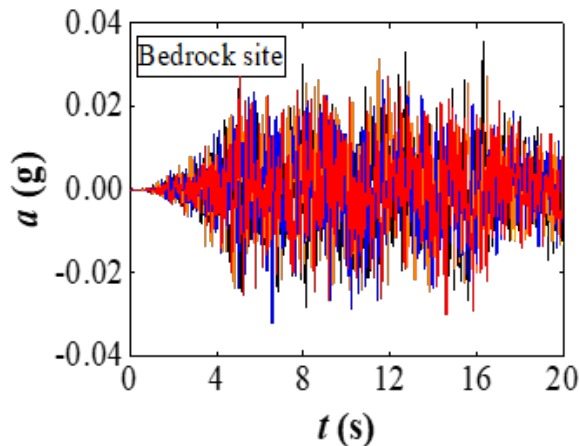
A methodology to equate a multi-span structure to a single-span structure is proposed, following an error validation.

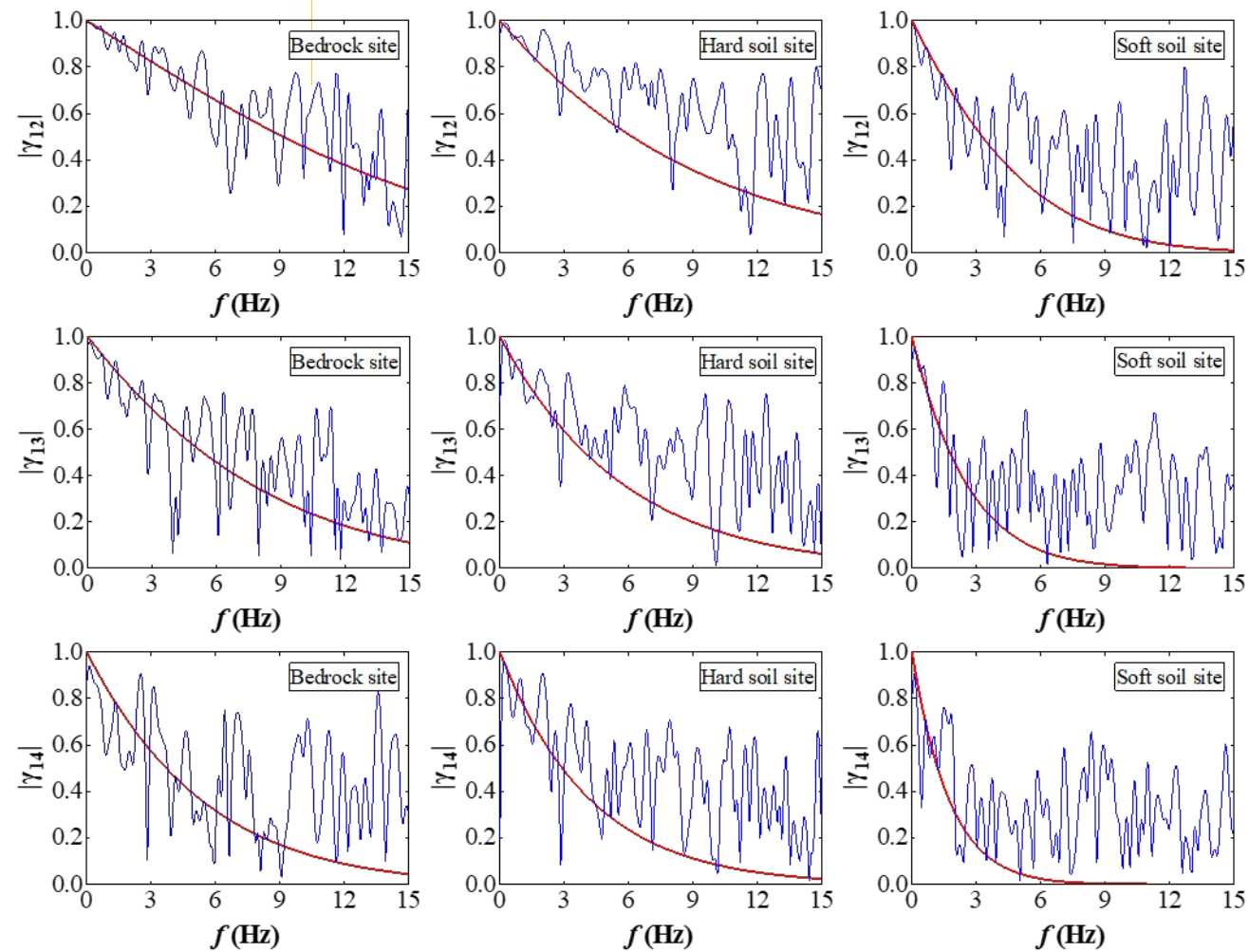
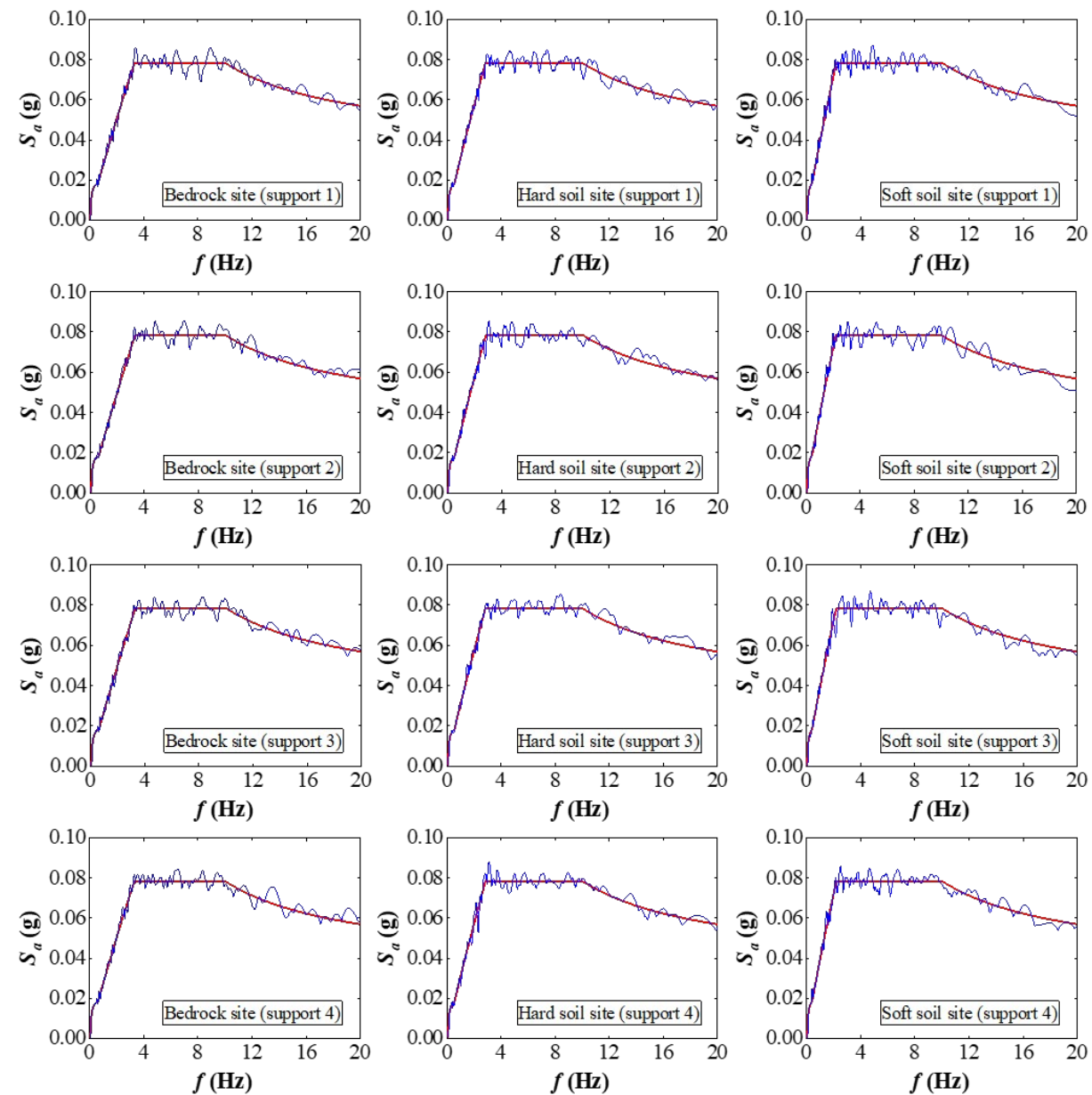


Step 1



Consider a SDOF system with two supports with separation distance d and vibration frequency f , the design response spectrum such as those defined in the Chinese seismic design code GB50011, the mean or mean plus/minus standard coherency loss function, and assume an apparent wave propagation velocity, simulate a set of spatially varying, response spectrum and coherency loss compatible ground motions

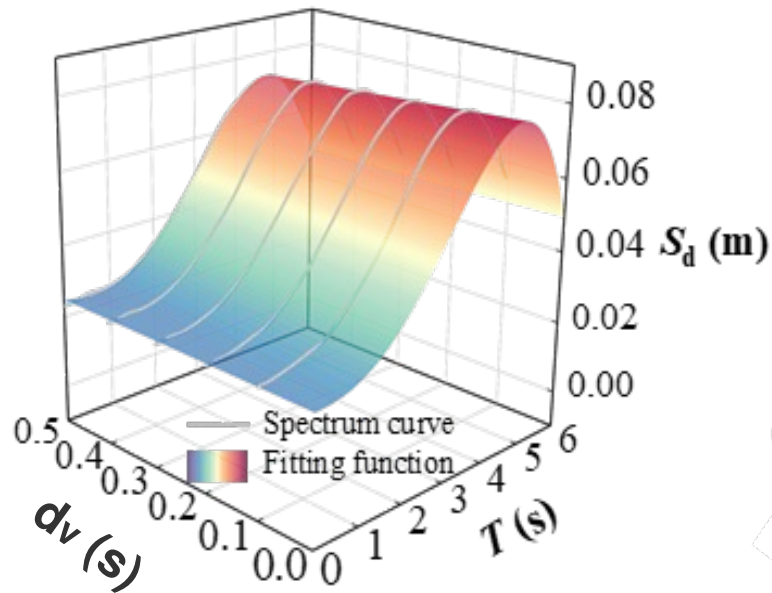




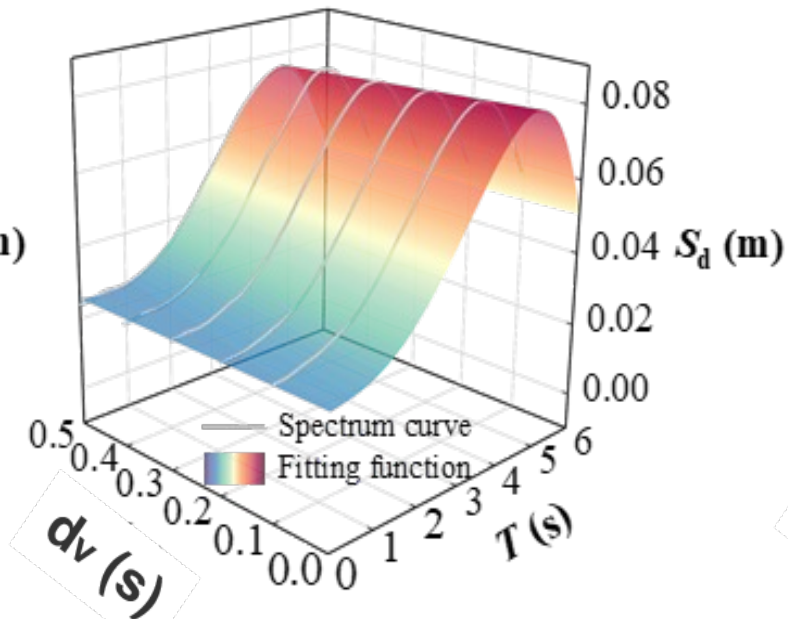
Design response spectrum and coherency loss function compatibility check

Step 2

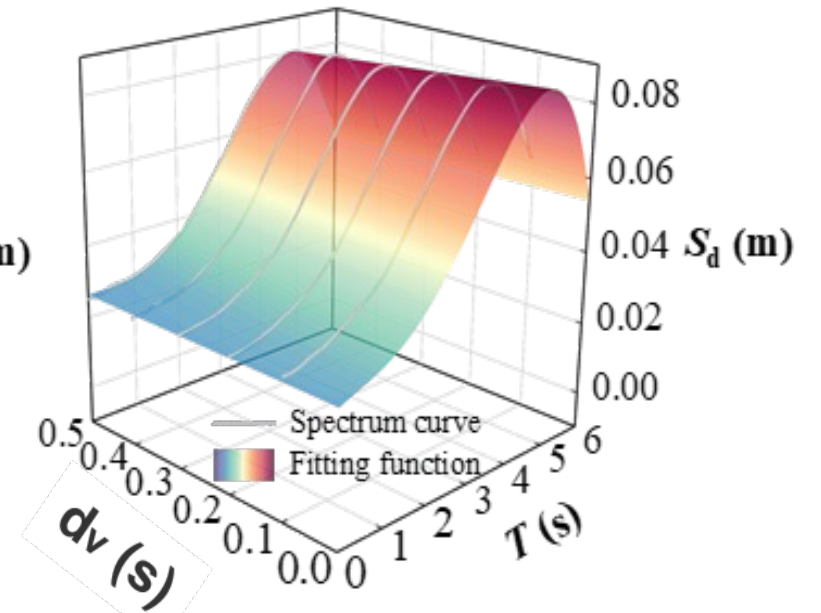
Varying the vibration period T and separation distance d , calculate the peak response of the SDOF system with two supports, construct the MSRS with respect to T and $d_v = d/V_{app}$.



(a) Bedrock site



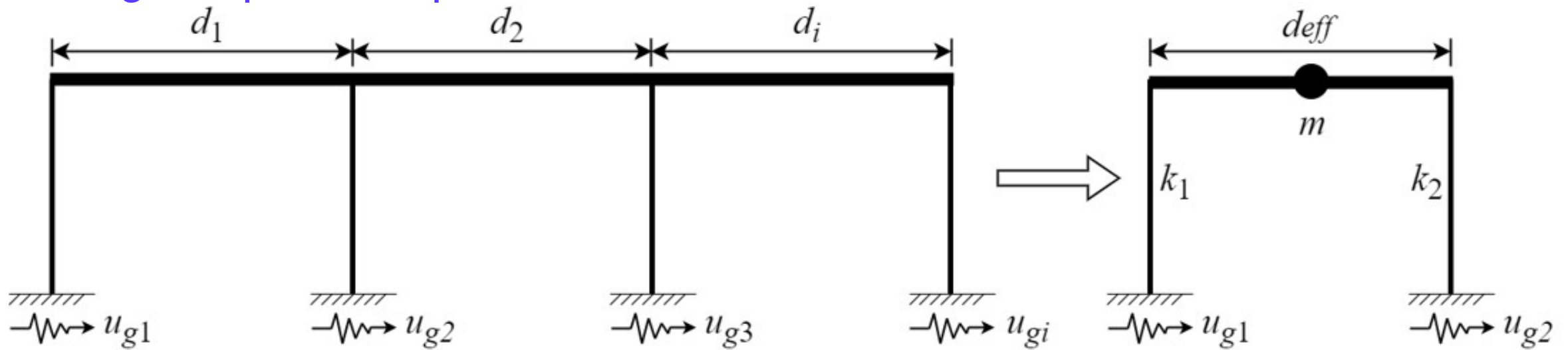
(b) Hard soil site



(c) Soft soil site

Step 3

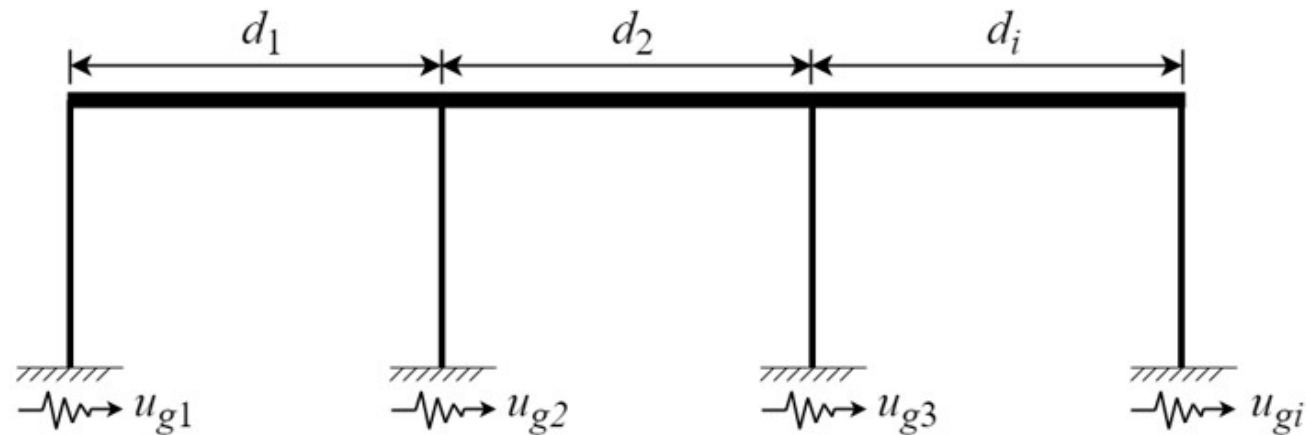
Estimating the equivalent span length of multiple span bridges for directly reading the peak responses from the MSRS



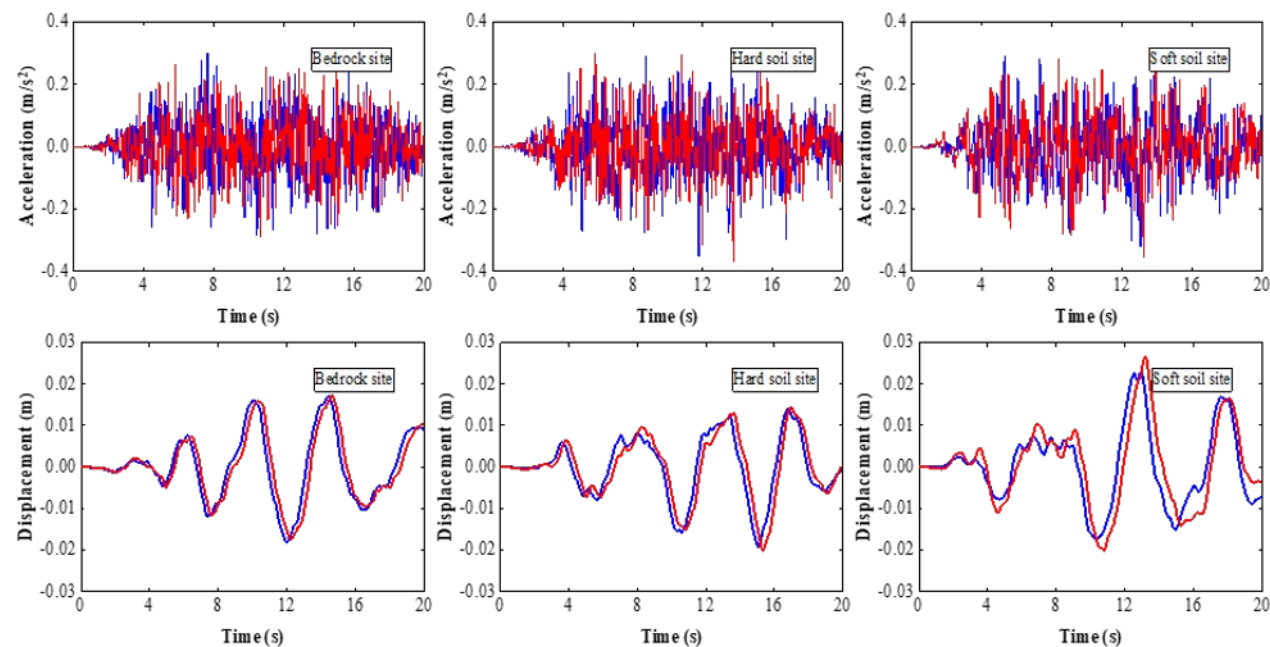
(i) total span length, (ii) average span length, (iii) maximum span length, (iv) the interquartile range (IQR) of span lengths, and (v) **the square root of the sum of squares (SRSS) of span lengths**

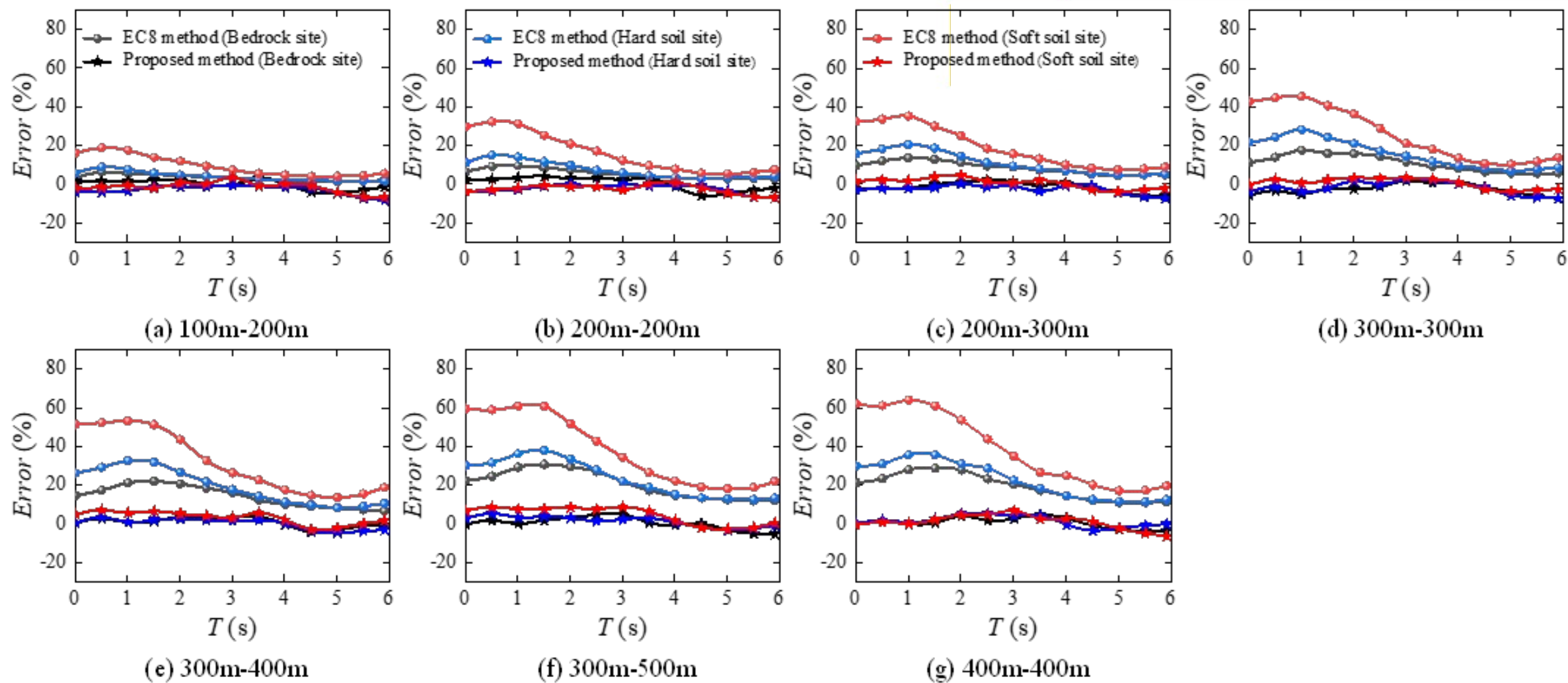
Verification and Comparison

Site Category↵	Span length↵
Bedrock site / Hard soil site / Soft soil site↵	100m-200m↵
	200m-200m↵
	200m-300m↵
	300m-300m↵
	300m-400m↵
	300m-500m↵
	400m-400m↵
	100m-100m-100m↵
	100m-200m-300m↵
	200m-200m-200m↵
	100m-200m-400m↵
	300m-100m-400m↵
	300m-200m-400m↵
	300m-300m-300m↵
	100m-100m-100m-100m↵
	200m-100m-100m-200m↵
	200m-200m-200m-200m↵
	300m-200m-100m-300m↵
	100m-200m-300m-400m↵
	300m-200m-100m-400m↵
	100m-100m-100m-100m-100m↵
	200m-100m-100m-100m-200m↵
	200m-200m-200m-200m-200m↵
	300m-200m-100m-100m-300m↵

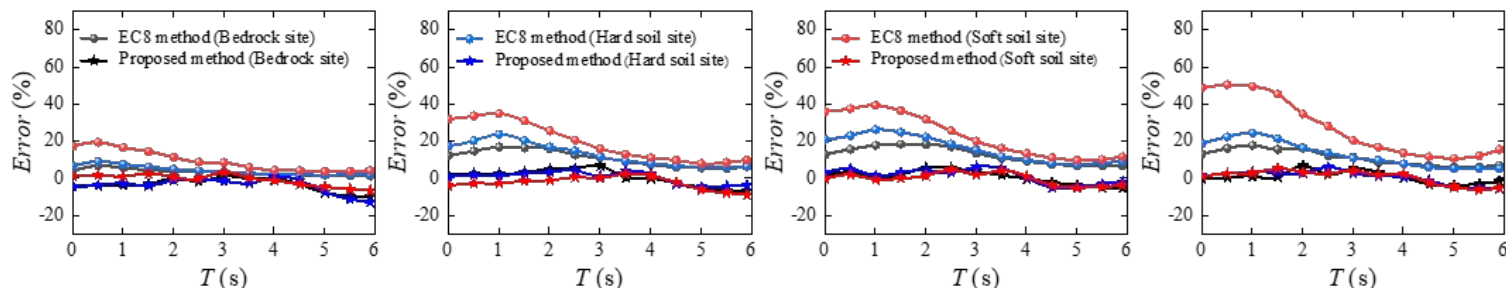


Two to five span structures, calculate responses using the proposed MSRS and EUROCODE 8-2 methods, compare them with time history analyses results, which are used as the benchmark responses of the bridge





Two span structure

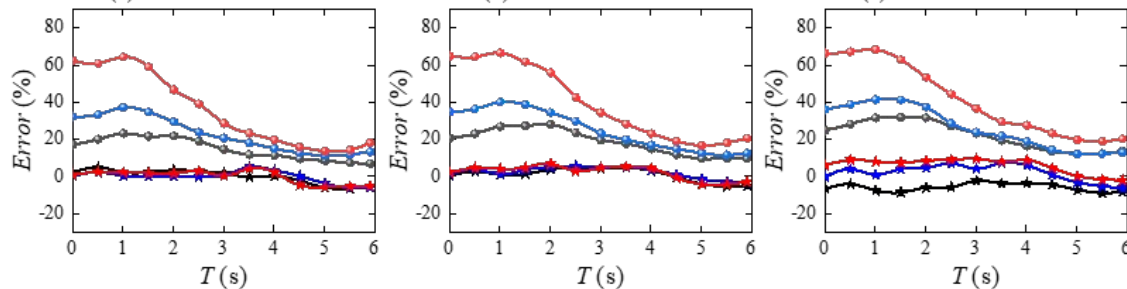


(a) 100m-100m-100m

(b) 100m-200m-300m

(c) 100m-200m-400m

(d) 200m-200m-200m

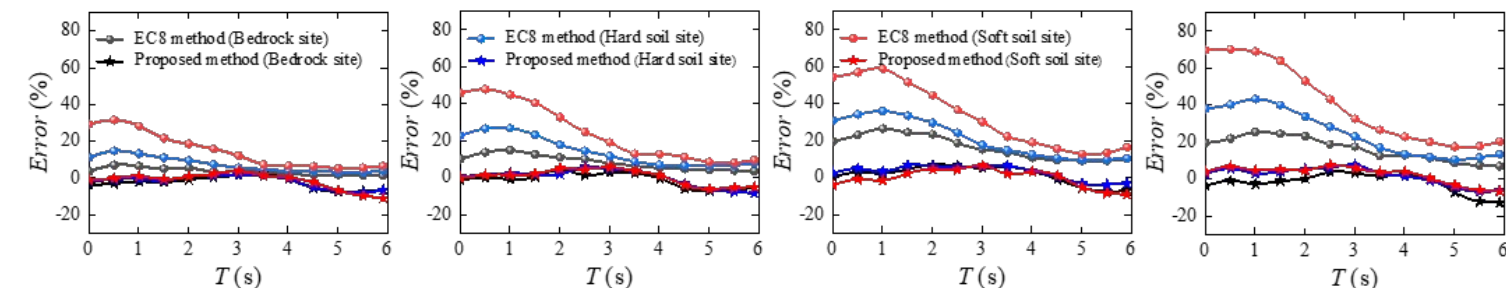


(e) 300m-100m-400m

(f) 300m-200m-400m

(g) 300m-300m-300m

Three spans

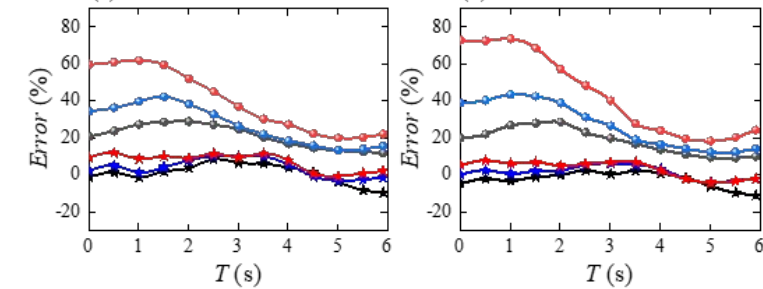


(a) 100m-100m-100m-100m

(b) 200m-100m-100m-200m

(c) 200m-200m-200m-200m

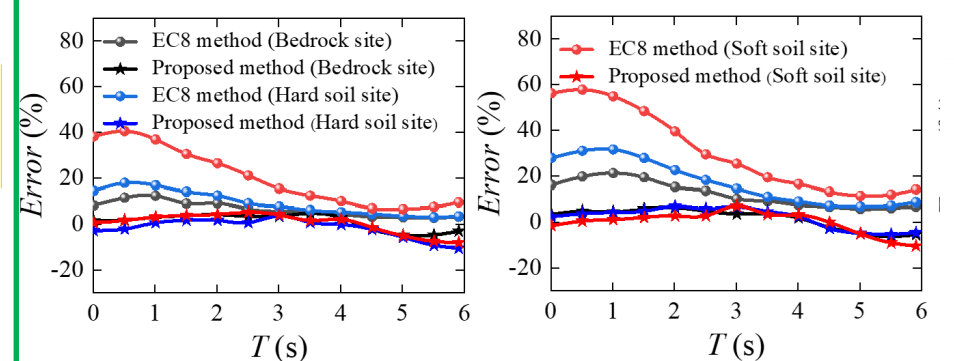
(d) 300m-200m-100m-300m



(e) 100m-200m-300m-400m

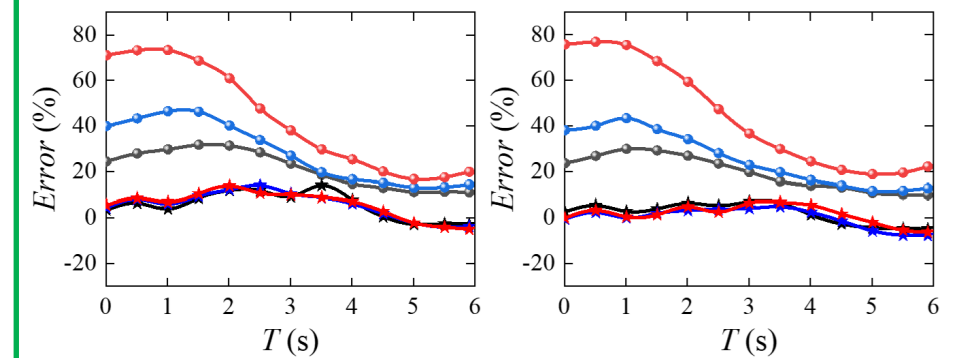
(f) 300m-200m-100m-400m

Four spans



(a) 100m-100m-100m-100m-100m

(b) 200m-100m-100m-100m-200m

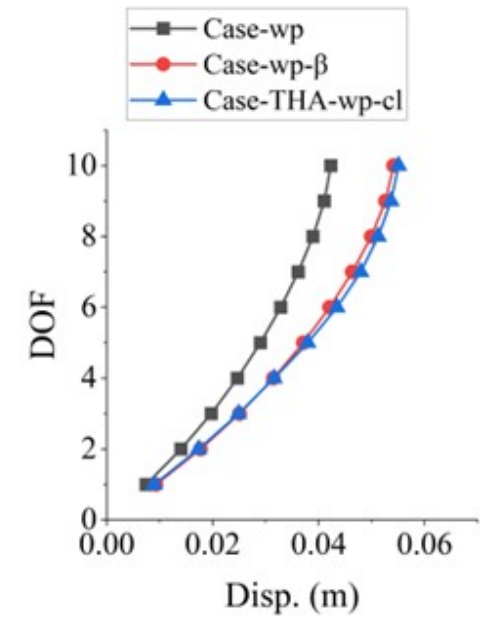
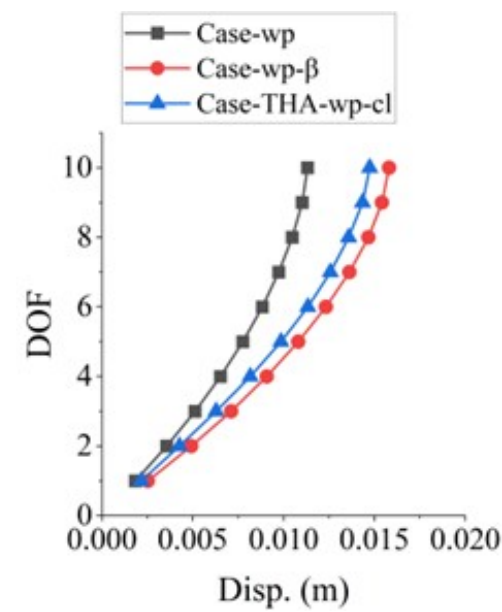
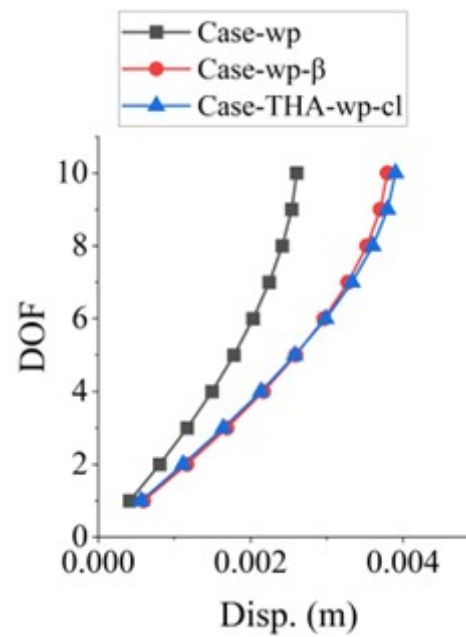
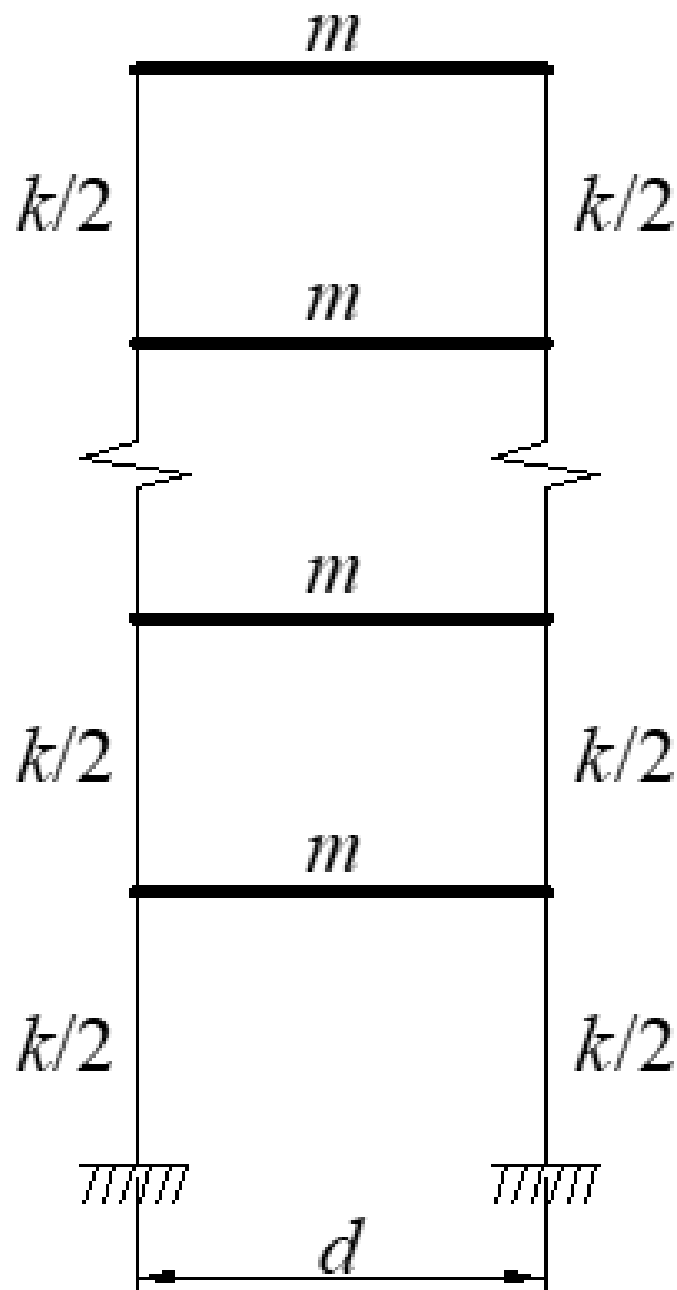


(c) 200m-200m-200m-200m-200m

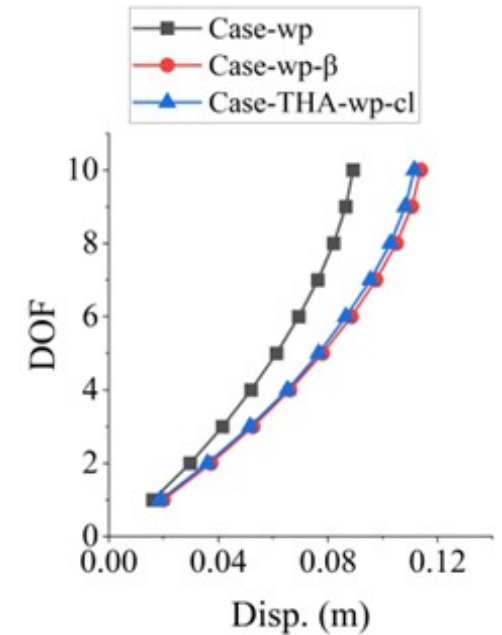
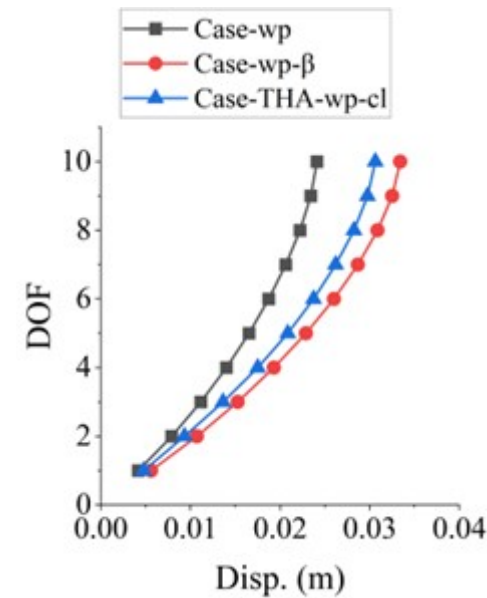
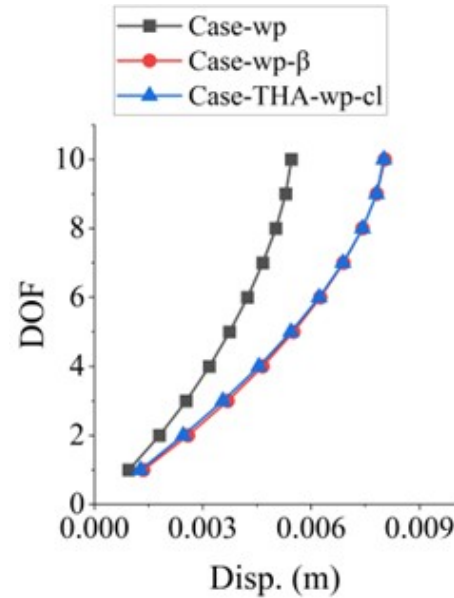
(d) 300m-200m-100m-100m-300m

Five spans

MSRS outperforms EUROCODE for all the considered cases, reducing the largest prediction errors from over 70% to less than 15%

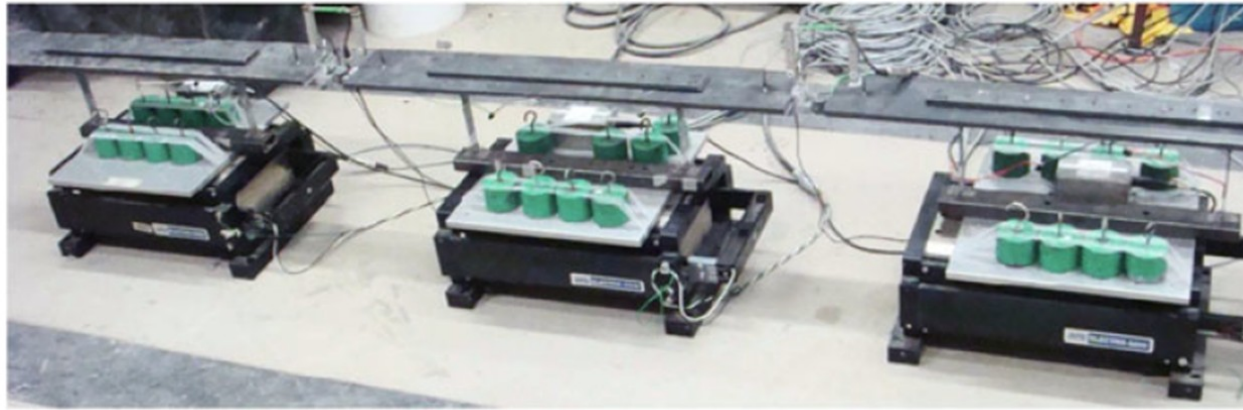


$d=200m$



Spectral analysis, mode superposition

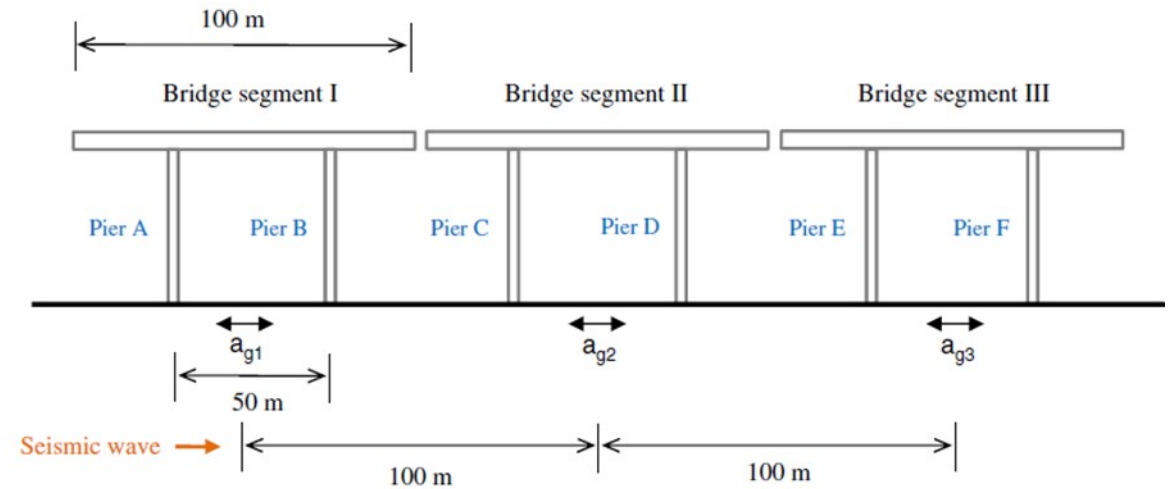
Lab tests



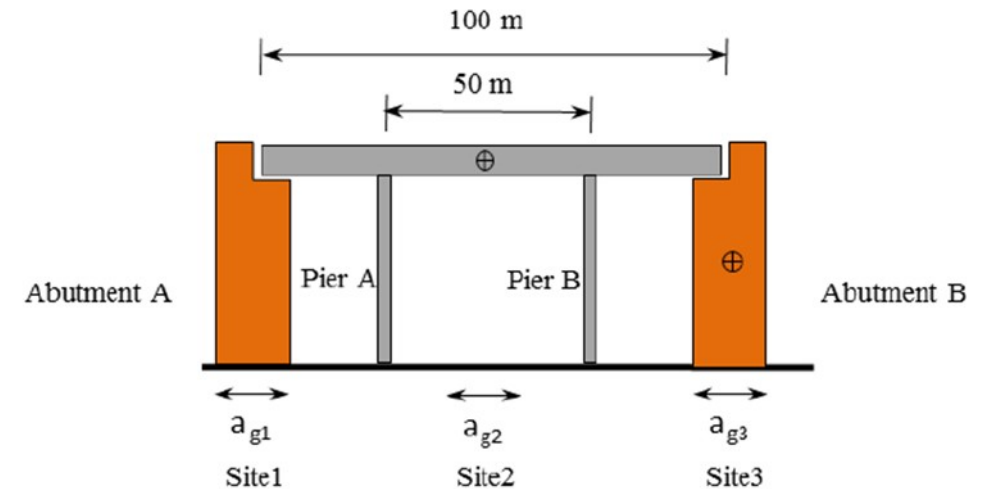
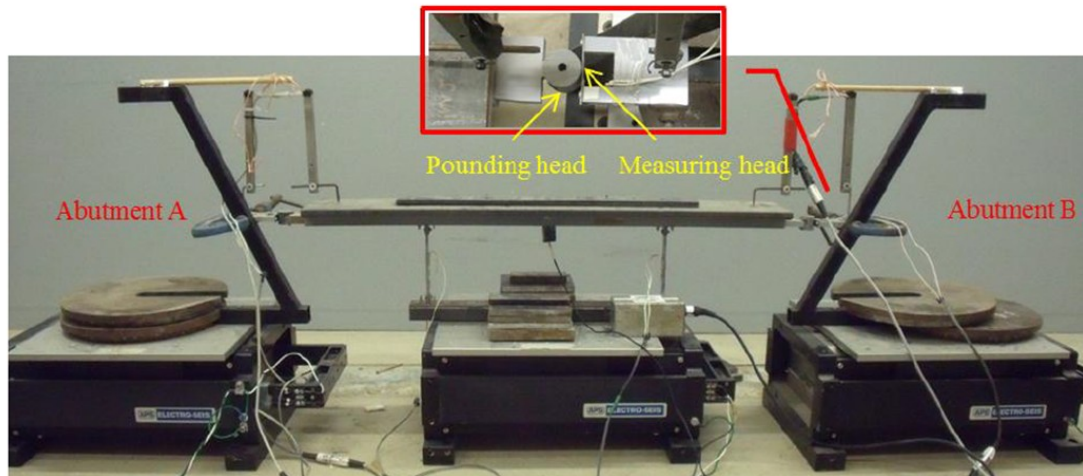
Segment I

Segment II

Segment III

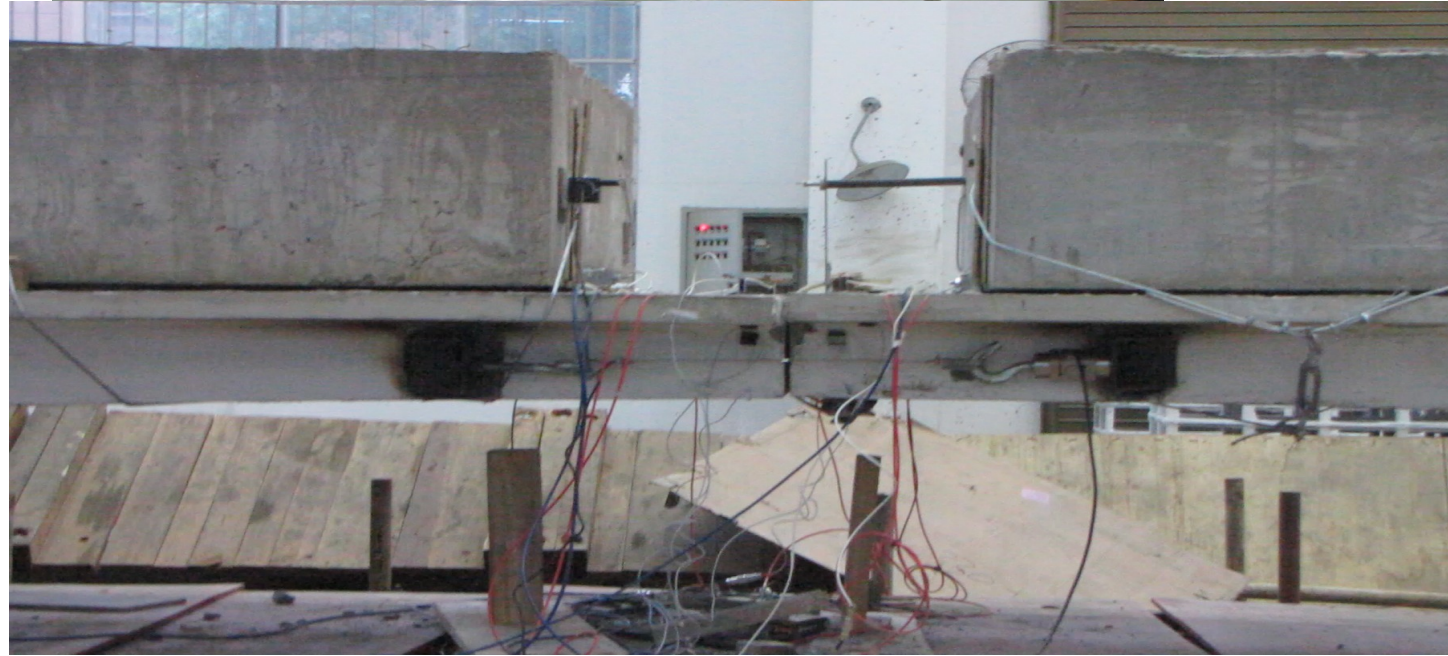


Bo Li, Kaiming Bi, Nawawi Chouw, John W Butterworth, Hong Hao. Experimental investigation of spatially varying effect of ground motions on bridge pounding. Earthquake Engineering and Structural Dynamics 2012; 41(14): 1959-1976



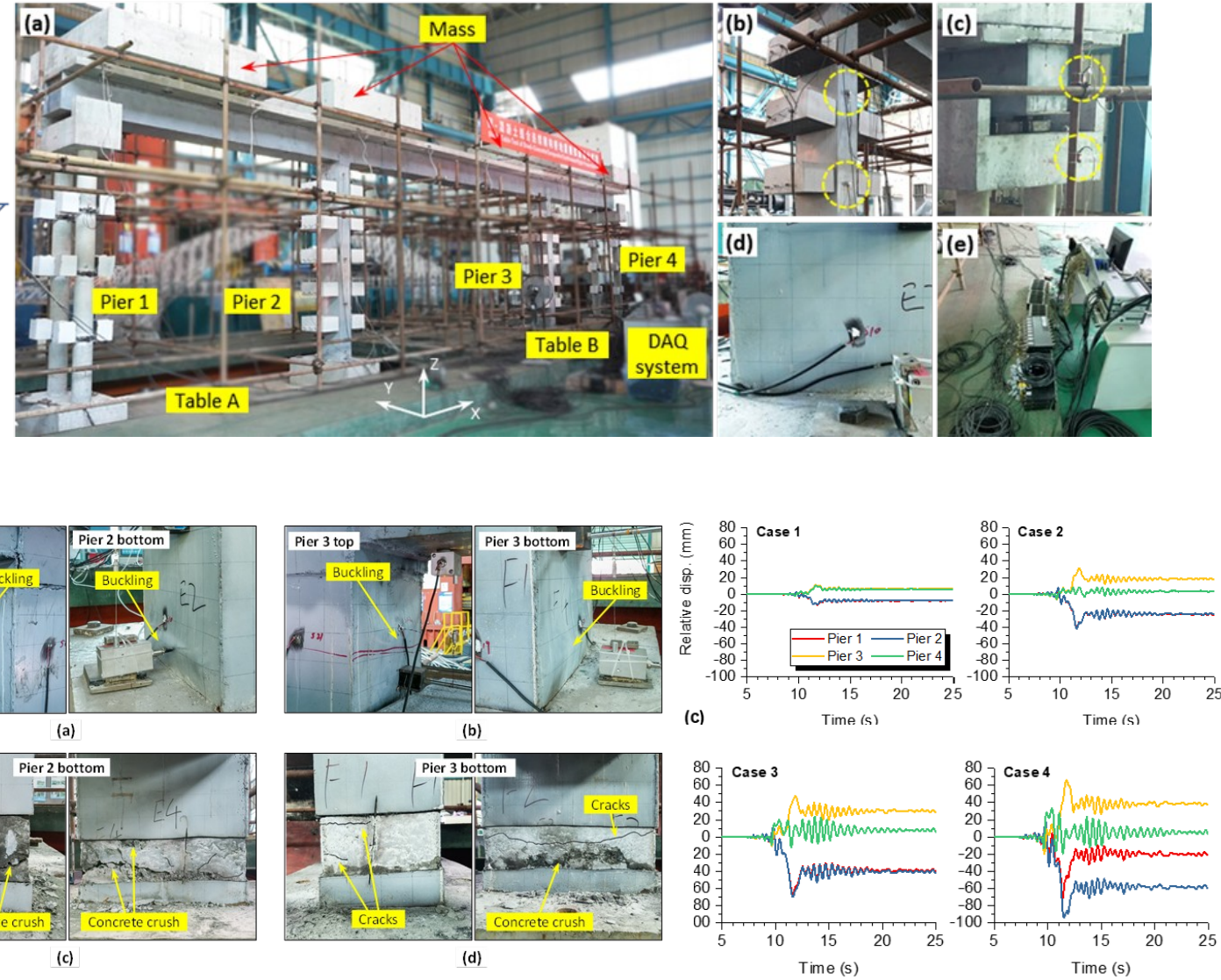
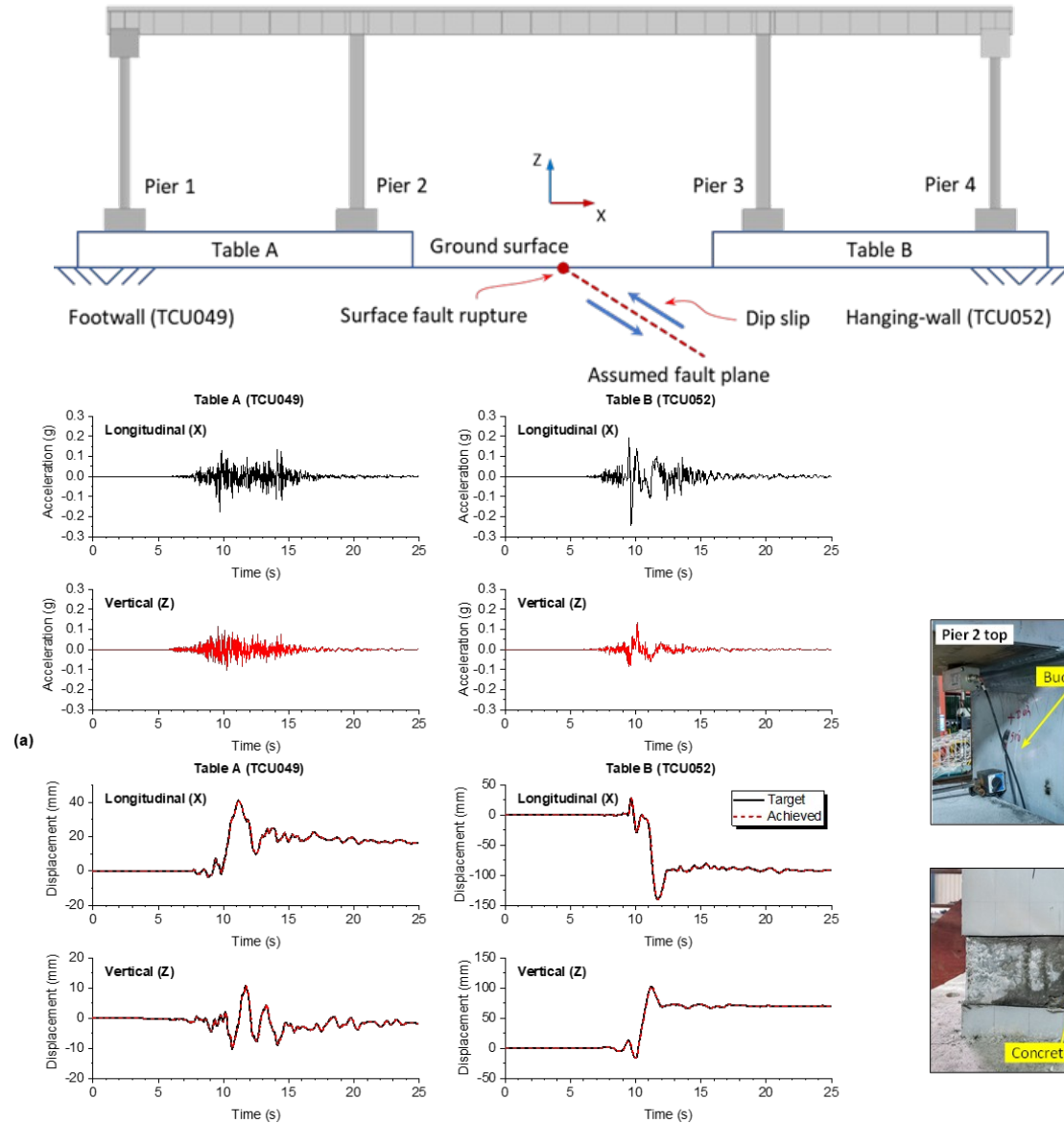
Bo Li, Kaiming Bi, Nawawi Chouw, John W Butterworth, Hong Hao. Effect of abutment excitation on bridge pounding. Engineering Structures, Vol. 54, 2013, pp57–68.

Tests in Central South University, two spans, 16m model : 1) fixed pier , 2) rocking pier

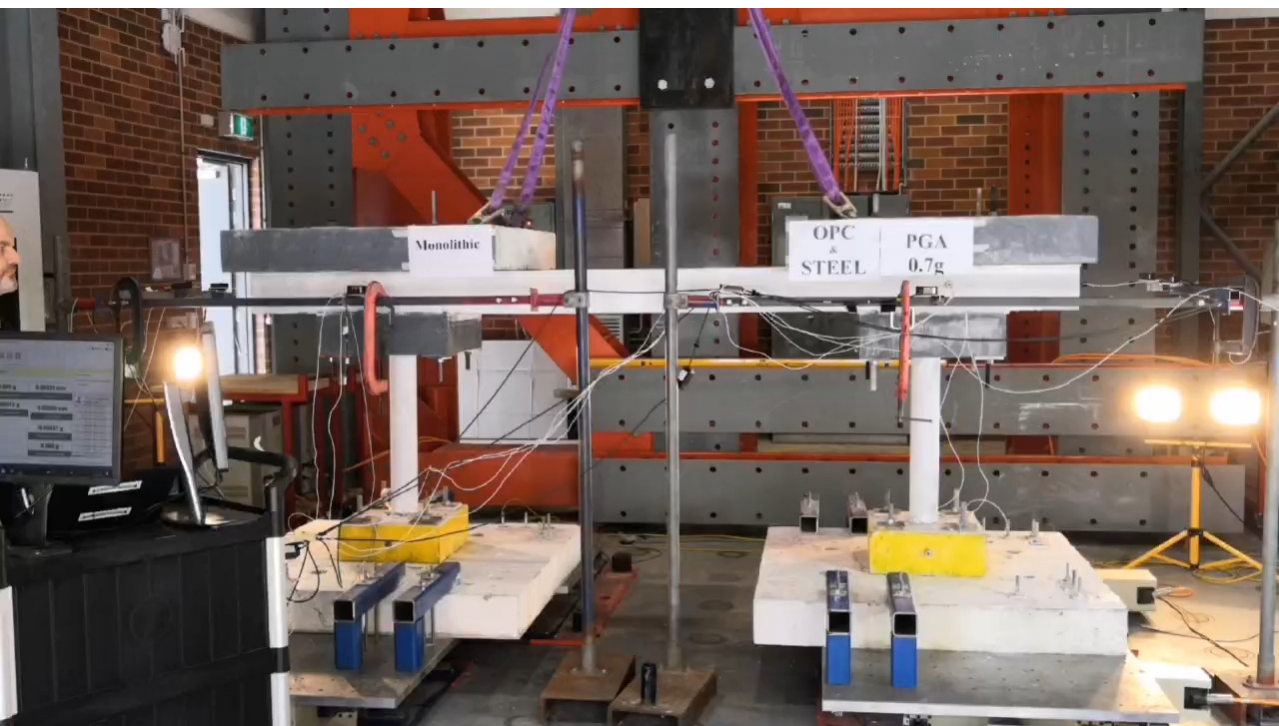


Bipin Shrestha, Li-Xiang He, Hong Hao, Kaiming Bi, Wei-Xin Ren, 'Experimental study on relative displacement responses of bridge frames subjected to spatially varying ground motion and its mitigation using superelastic SMA restrainers', Soil Dynamics and Earthquake Engineering, Vol. 109, 2018, pp76-88.

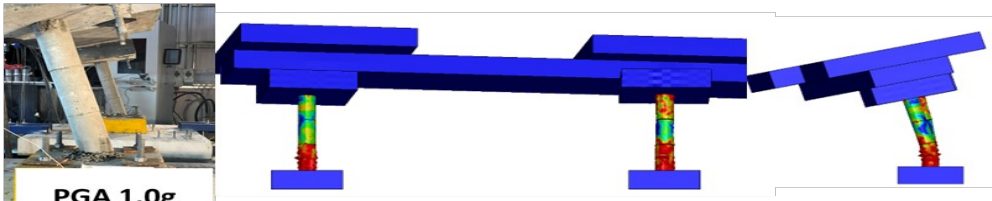
Tests in Chongqiang Transportation research institute



Yuanzheng Lin, Zhouhong Zong, Kaiming Bi, Hong Hao, Jin Lin and Yiyan Chen, 'Experimental and numerical studies of the seismic behavior of a steel-concrete composite rigid-frame bridge subjected to the surface rupture at a thrust fault', Engineering Structures, Volume 205, 2020, Article 110105.



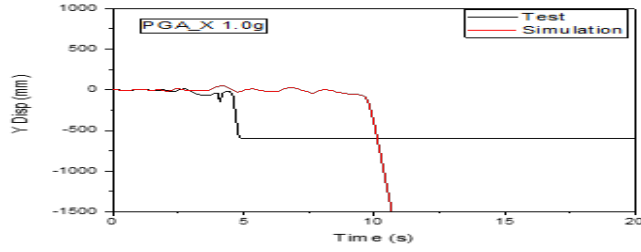
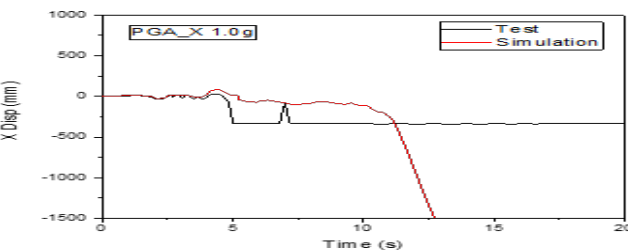
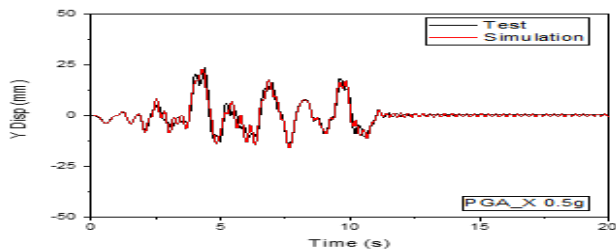
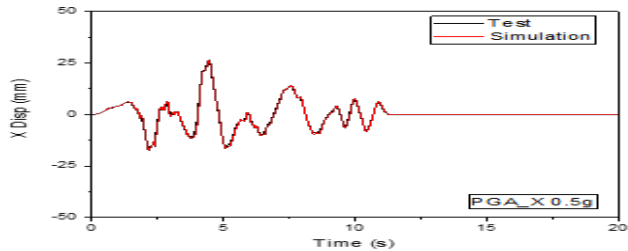
PGA_X 0.5g



PGA 1.0g

PGA_X 1.0g

Structure	Seismic Excitations	Collapsed PGA_X (g)
GPC-BFRP01	Uniform	1.0
OPC-steel	Uniform	1.0
GPC-BFRP02	Multi-support	0.9



Displacement time histories

长大结构多点地震反应分析和试验

技术标准

Technical standard for multi-point seismic response analysis and testing of large-scale structures

(征求意见稿)

20XX-XX-XX 发布

20XX-XX-XX 实施

中国土木工程学会 发布

本标准主编单位： 广州大学

本标准参编单位： 大连理工大学， 香港理工大学， 天津大学， 东南大学， 同济大学， 河海大学， 北京建筑大学， 福州大学， 中南大学， 天津城建大学， 香港科技大学， 哈尔滨工业大学， 重庆大学， 山东大学， 北京工业大学， 招商局重庆交科院
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本标准主要审查人： 周建庭， 李山有， 徐龙河， 徐赵东， 陆新征， 朱宏平， 任伟新

Provide guides on selecting ground motion spatial variation models. **For phase shift effect:** range of apparent wave velocity V_s to ∞ , structural response extreme values $fd/v_{app}=0.5, 1.5$ or 1.0 and 2.0
For coherency loss: $|\gamma_{jk}(i\omega)|=1.0$, $|\gamma_{jk}(i\omega)|\leq 0.3$ for perfectly correlated or uncorrelated, and mean, mean plus/minus standard deviation coherency loss function
Method for scaling apparent velocity $v_{app-m}=\lambda_l\lambda_f v_{app-p}$ **and coherency loss** $\alpha_m=\frac{1}{\lambda_l\lambda_f}\alpha_p$ for scaled model test

Concluding remarks

- 1) Earthquake ground motions at multiple structure supports inevitably vary.
- 2) Ground motion spatial variations excite some structural response modes that would not be excited by uniform ground motion inputs.
- 3) Most seismic design codes state the importance of considering ground motion spatial variations in the analysis and design of large dimensional structures, but do not give specific guides. Those provide design analysis method may not lead to reliable estimations of structural responses.
- 4) The ground motion spatial variation models, simulation methods, structural response analysis and response characteristics are summarized in this talk.
- 5) Some recent research results on statistical coherency loss model, Multi Support Response Spectrum method for design analysis are introduced
- 6) Some laboratory tests using multiple shake tables are also presented
- 7) A new Chinese Civil Engineering guides on analyses and tests of large dimensional structures subjected to spatially varying ground motions, which is under the final stage of evaluation before publication, is briefly introduced.

***Thank you for your
attention!***