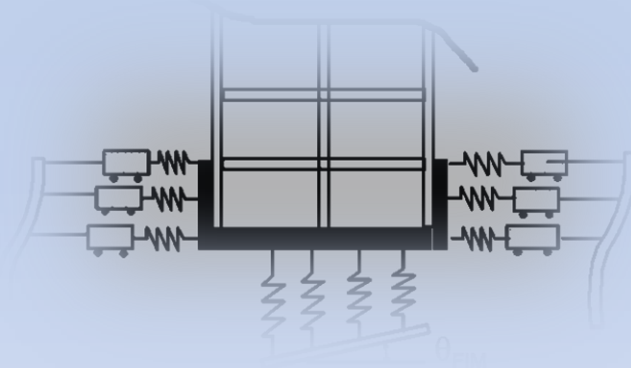


Soil-Structure Interaction

Soil-Structure Interaction of Piled Foundations for Buildings



A technical seminar about modeling soil-structure interaction in the analysis of buildings and bridges, and how the design of structures is affected by the incorporation of SSI effects.

Blair Gohl

AMEC Earth & Environmental, Burnaby, B.C.

Guoxi Wu

BC Hydro, Burnaby, B.C.

Presentation Outline

- Code-based commentary
- Mechanisms of soil – pile interaction
(superstructure loading & ground motion loading)
- Observations of typical pile damage during past earthquakes
- Dynamic sub-structuring versus coupled analysis procedures
- Single pile analysis procedures
- Simplified pile group (2-D) analysis procedures
- Coupled 3-D pile group analysis (G. Wu)

Code Commentaries (FEMA 450, 2003)

- Consideration of potential benefit of soil-structure (pile) interaction in reducing predicted earthquake motions (base shear and foundation moment) at base of superstructure relative to rigid base conditions
- Base shear & moment reductions due to foundation flexibility which lengthens fundamental period of superstructure and effects of increased foundation damping
- Increased foundation flexibility may increase superstructure lateral displacements and rocking
- Effective input motions at top of foundation are not necessarily the same as the free field surface motions due to kinematic interaction effects

Code Commentaries (FEMA 450, 2003)

- Pile foundations shall be designed and constructed to withstand maximum curvatures from earthquake ground motions (KI) and superstructure inertia loading
- The effects of increased lateral ground deformation due to soil liquefaction or slope effects on pile foundations shall be considered
- Ground improvement may be required to reduce seismic ground movements and effects on piles
- Pile group interaction effects shall be considered
- Detailed analysis of forces and moments developed in batter pile groups and pile cap connection details needs to be carried out

Code Commentaries (FEMA 450, 2003)

- Pile foundations are to be designed as ductile structural elements but FEMA implies inelastic behavior is permissible as long as don't have plastic hinging
- Piles to remain functional following design earthquake in view of difficulties of repairing pile foundations
- High bending moments and shears at pile cap – pile interface due to inertial interaction and near surface soil liquefaction effects
- Potential for high moments and shears due to sharp transitions in lateral ground movement at larger depths, particularly at interfaces between stiff and soft soil layers

Code Commentaries (FEMA 450, 2003)

- Superstructure – pile foundation interaction may be modeled using nonlinear load-deformation or moment-rotation relationships
- Parametric studies required to examine sensitivity of superstructure and pile foundation response to nonlinear foundation stiffness parameters
- Soil moduli and strengths used in foundation stiffness calculations should be based on ground strain levels and pore water pressures induced under design levels of seismic shaking

NBCC, 2005 – Structural Commentary J

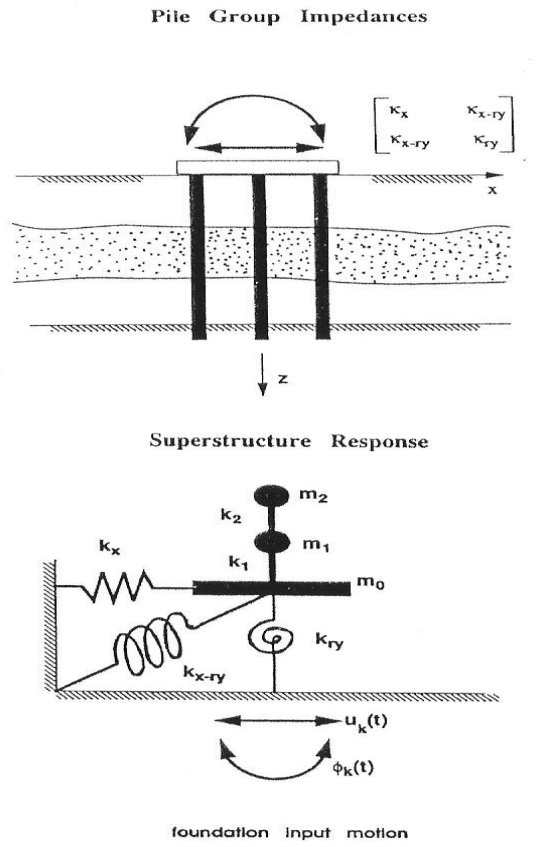
- Pile foundations required to resist the lateral load capacity of the Seismic Force Resisting System with suitable reductions in transmitted foundation load due to structure over-strength and ductility factors
- Design foundations to continue to behave elastically following design earthquake for low to moderate eq. shaking with no soil liquefaction effects – see comments below)
- Foundation design to take into account seismically induced ground strains and/or soil liquefaction effects without undergoing excessive foundation displacements – may require ground improvement to ensure this

NBCC, 2005 – Structural Commentary J

- In regions of high seismicity ($I_e F_a S_a (0.2) > 0.75$), pile foundation elements to be designed to accommodate cyclic inelastic behavior for high moments $> 0.75 M_{ult}$
- Use structural resistance factors for calculation of pile moment and shear capacity depending on material type
- For strong earthquake shaking and where liquefaction effects occur, inelastic bending response of the piles is permitted as long as they can continue to carry axial dead plus live load following the earthquake
- Piles must behave in a ductile fashion during strong eq. shaking and must not fail in shear

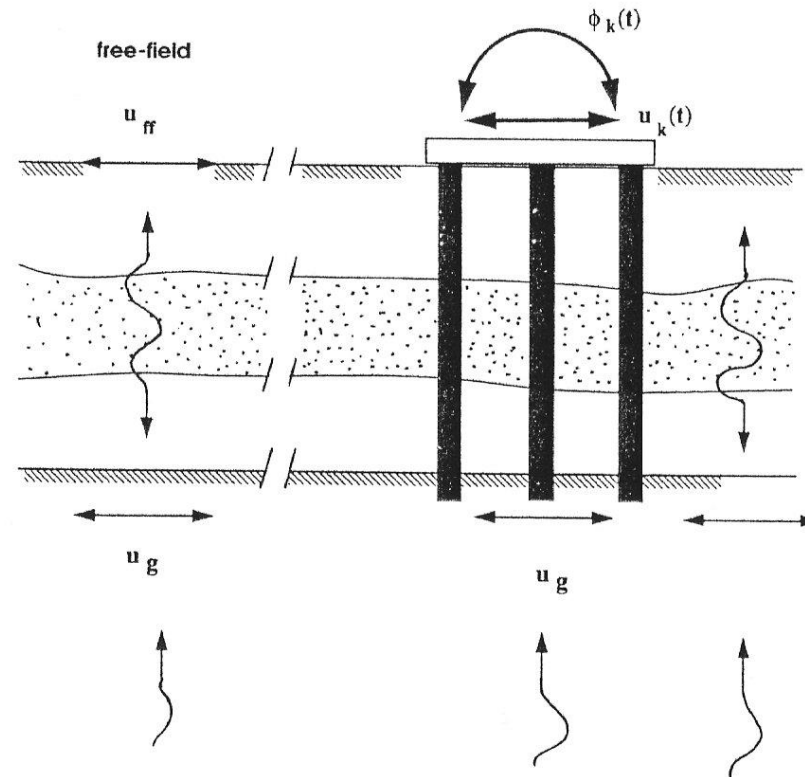
Soil-Pile Superstructure Interaction

- Superstructure inertia loading at tops of piles (generally dominates over depth of about 10-15 pile diameters)



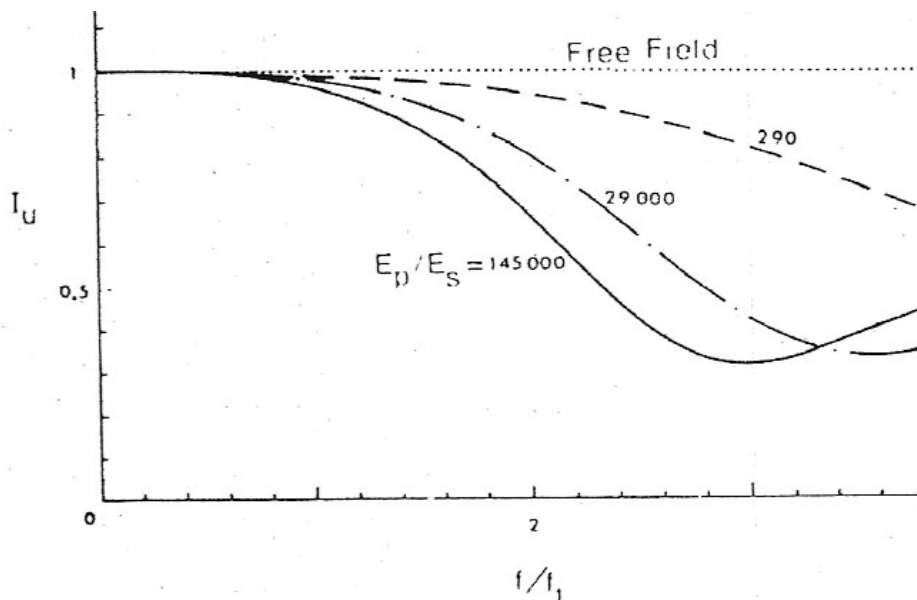
Soil-Pile Superstructure Interaction

- Ground motion loading along piles (kinematic interaction)



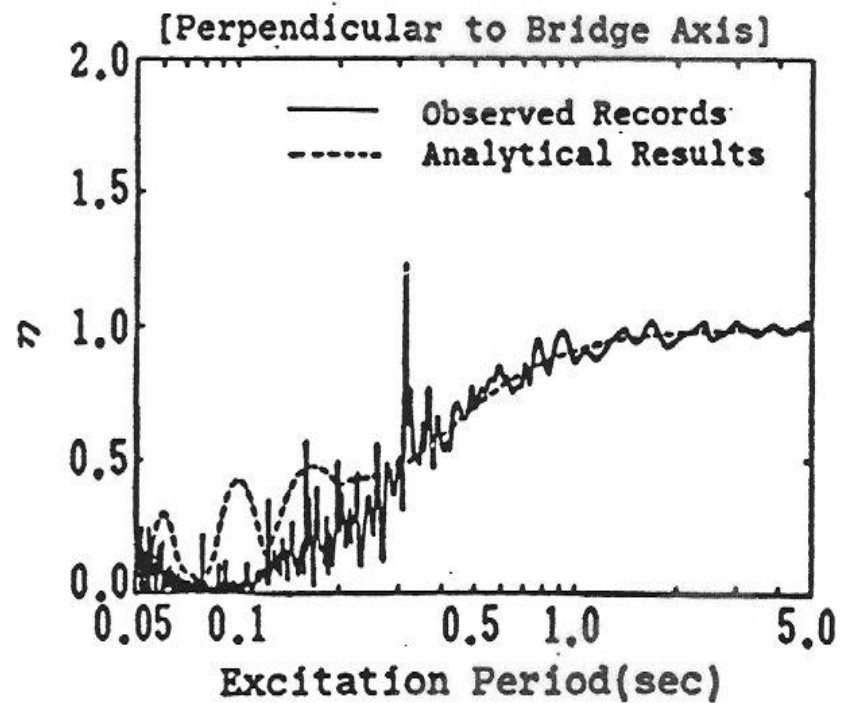
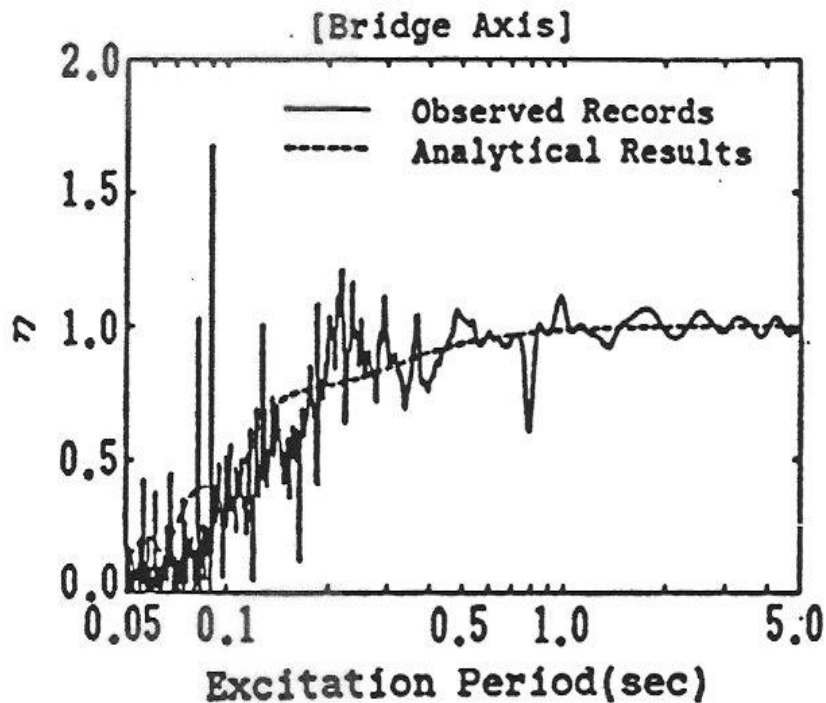
Theoretical KI Effects – G. Gazetas (1984)

- Single piles – elastic soil response
- I_u = pile head lateral movement/free field g.s. lateral displ.
- pile head motion generally less than free field motion at higher frequencies depending on pile-soil stiffness ratios, strength of shaking, and details of soil profile



Measured KI Effects – Tazoh et al (1988)

- Pile groups for a bridge foundation
- Micro-tremor measurements
- η = ratio of effective pile cap motions to f.f. surface motions



Considerations in Seismic Pile Design

- Effects of shaking on axial capacity and axial buckling (loss of shaft friction)
- Effects of shaking on post-seismic pile settlement
- Effects of pile head inertia forces (SS interaction) and imposed ground displacement (KI) on lateral bending and shear response
- Influence of pile foundations on combined stiffness and damping of superstructure – foundation system

Damage Reports During Strong Shaking

- Effects of near surface soil liquefaction results in loss of lateral soil resistance, increased lateral pile displacements, and possible pile breakage due to SS inertia forces
- Soil liquefaction effects along pile shafts leads to loss of axial capacity and increased post-seismic settlement – bearing capacity failures have been reported for soil liquefaction under pile bases
- Large lateral displacement gradients at transition between stiff and soft (liquefied??) soil layers results in damaging bending and shear in piles – check influence of lateral movements of stiff crust on top of liquefied layer

Damage Reports During Strong Shaking

- Effects of seismically induced slope or river bank movements on pile bending response and lateral pile movements (e.g. Niigata eq. bridge failures supported on piles in liquefied soil)
- If the soil does not fail, bending damage governed by pile ductility – maximum curvatures just below pile cap for fixed head piles or slightly deeper locations for pinned head piles (SS loads), and where rapid changes in soil modulus occur (KI effects)
- Use of batter pile groups may decrease seismic lateral pile cap movements but need to check pile cap punching shear and tendency for rotation of pile caps

General Guidelines for Pile Design

- Prevent soil liquefaction near pile head or else design for it
- Prevent large lateral ground movements acting on piles (slope effects, imbalanced fill loads) – ground improvement may be required
- Check effects of loss of shaft friction or pile tip capacity (loss of shaft friction due to free field pwp build-up and cyclic reversals of near field shear stresses)
- Use ductile piles (steel pipe, timber piles) where possible

General Procedures in Seismic SS – Pile Analysis

- Geotechnical site characterization
- Static pile foundation design to reduce building settlements
- Free field site response analysis (simplified or using wave propagation models)
- Is soil liquefaction an issue and is there potential for large lateral ground movement acting on piles?

General Procedures in Seismic SS – Pile Analysis

- Assessment of “effective” seismic motions acting at base of superstructure
- Conservative to use free field motions from 2005 NBCC for a particular site class, or site-specific seismic response analysis (SHAKE, DESRA, FLAC, etc.) for single piles
- KI effects may be more pronounced for pile groups than for single piles and use of FF motions may be unconservative for longer period motions (G. Wu)

General Procedures in Seismic SS – Pile Analysis

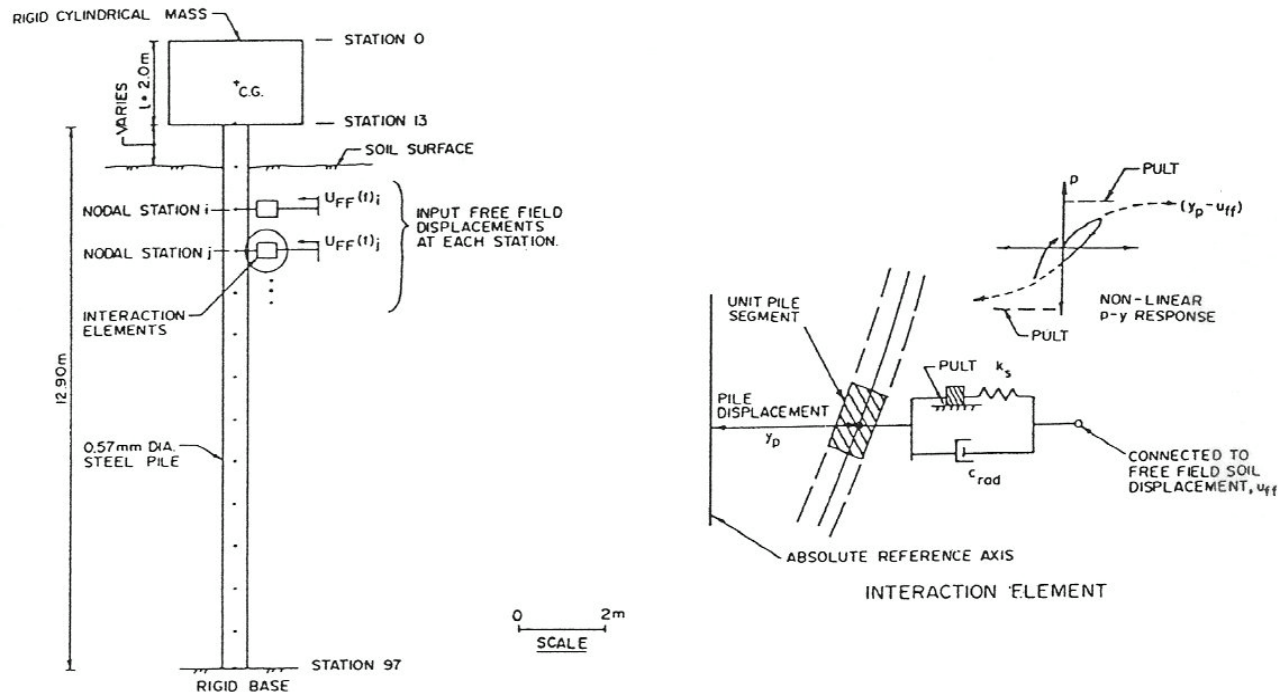
- Is a rigid base SS analysis to be performed using “effective” or “free field” seismic input motions?
- Is a flexible base SS analysis to be performed using an uncoupled analysis procedure and foundation compliances? Is this an equivalent linear modal analysis, or a nonlinear time step analysis?
- Is a fully coupled SS – pile foundation to be performed where seismic ground motions are propagated at depth up through the piles?

KI Analysis of Single Piles

- Most common practice is to use numerical finite element or finite difference models of a beam on a nonlinear Winkler foundation to represent near field soil-pile interaction
- Closed form analytic expressions for KI effects (e.g. Gazetas) are generally based on linear elastic soil response and have limited applicability for strong shaking
- Input free field, lateral ground displacement time histories $U_{ff}(z,t)$ at ends of springs where $U_{ff}(z,t)$ derived from nonlinear site response analysis (SHAKE, DESRA, FLAC, etc.)

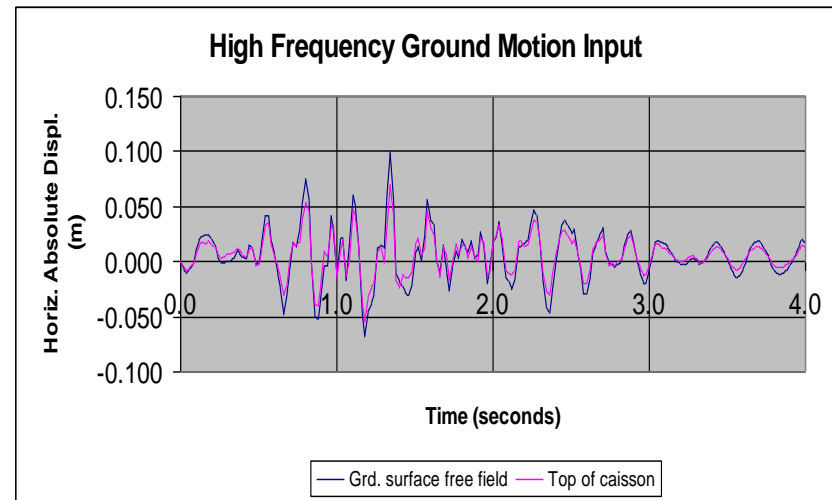
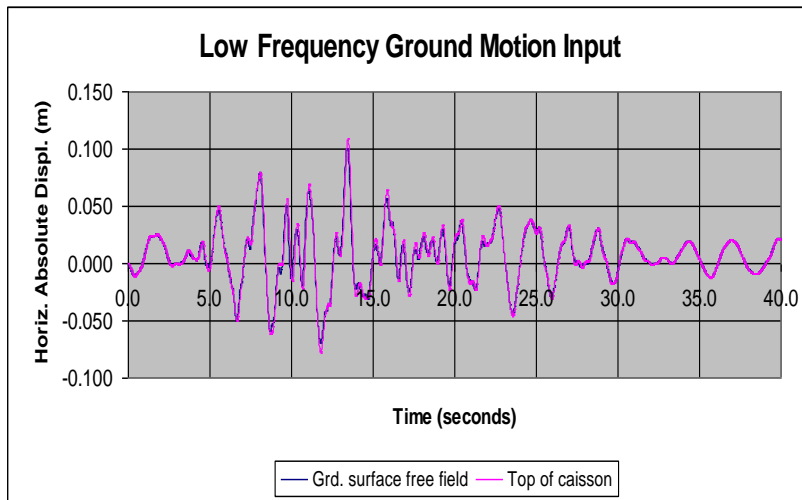
KI Analysis of Single Piles

- Single pile analysis with support motion model
- no pile head mass for KI analysis
- soil radiation damping parameters important
- near field soil-pile interaction described using p-y curves



KI Analysis of Single Piles

- stiff piles in softer soils attenuate higher frequency motions
- consider KI effects for larger diam. piles with $f > 1.5 f_1$ (Gazetas)

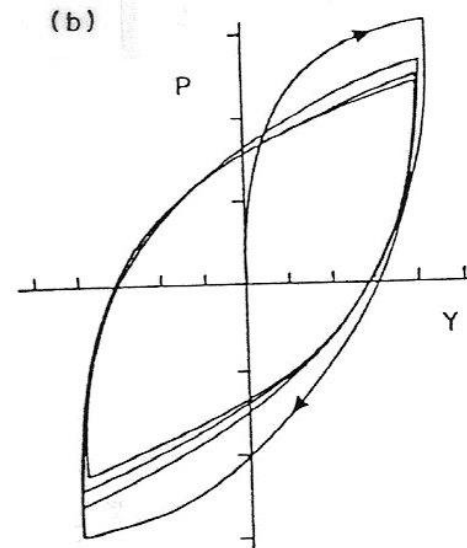
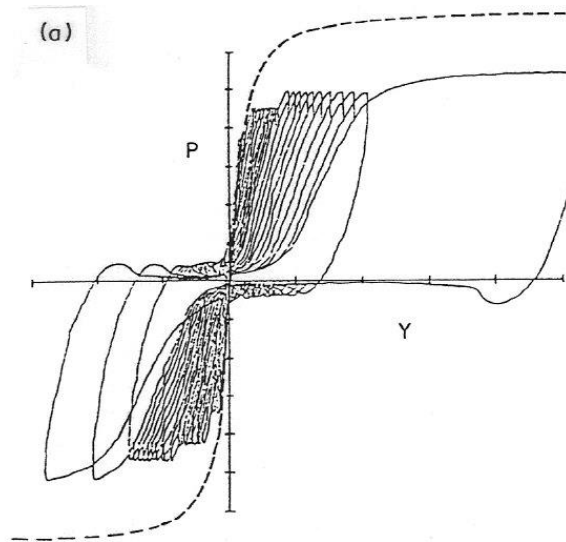


Near Field Soil-Pile Interaction (Lateral)

- P-y curves derived from static and cyclic pile head loading tests for various soil types (reference API) but not calibrated for seismic loading conditions at larger depths
- Backbone P-y curves at larger depths typically based on plasticity theory relating ultimate lateral soil capacity to a translating disc (pile) with initial slope of P-y curve derived from elasticity theory – hysteretic behavior simulated using various load-unload rules
- Complications in P-y curve modeling to account for soil-pile separation (near soil surface) and softening of P-y curve with time due to cyclic pore pressure generation
- Common to look at pre- and post-earthquake P-y backbone curves to look at sensitivity of response

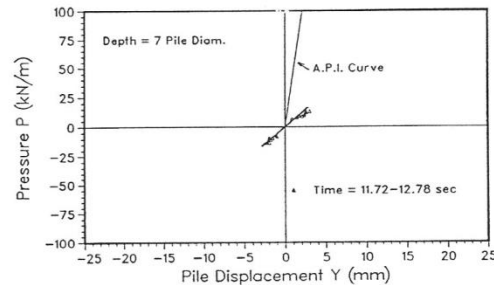
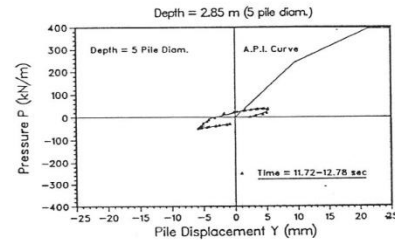
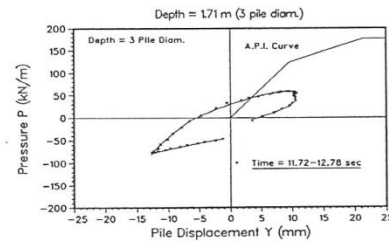
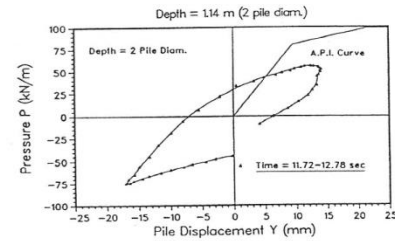
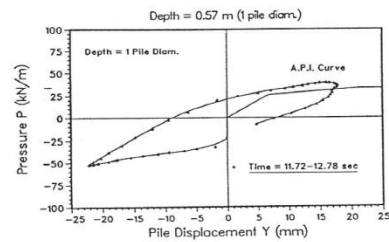
Near Field Soil-Pile Interaction (Lateral)

- Cyclic P-y curves in soft clay (a) zone of unconfined response and (b) zone of confined response (R. Bea, 1980)



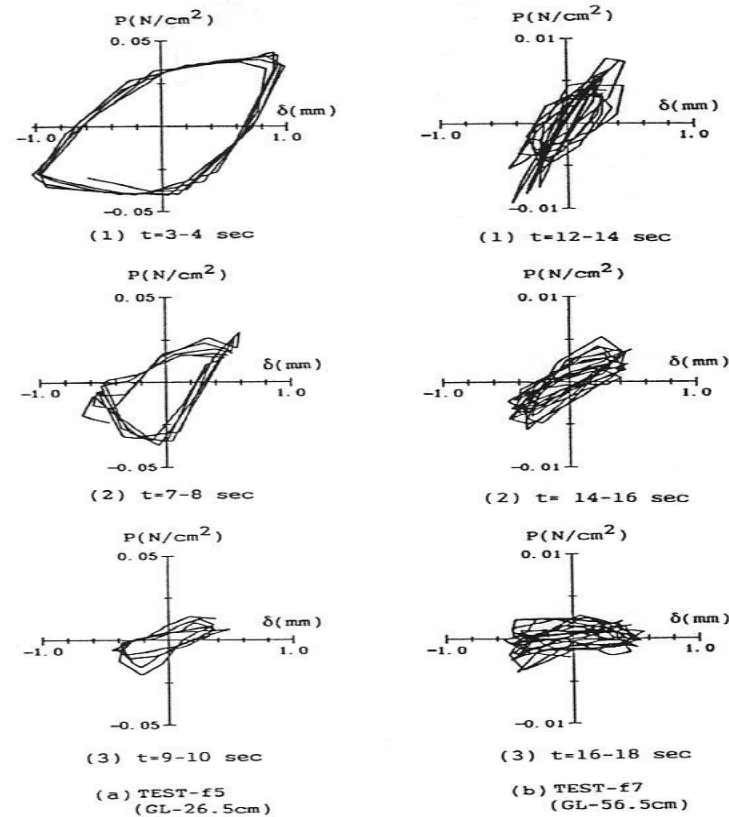
Near Field Soil-Pile Interaction (Lateral)

- Cyclic P-y curves in dry sand (Gohl, 1991)



Near Field Soil-Pile Interaction (Lateral)

- Cyclic P-y curves in loose saturated sand (Yao and Kobayashi, 1992)



Near Field Soil-Pile Interaction (Axial)

- T-z curves represent soil resistance along pile shaft versus relative axial pile deflection
- Backbone Q-z curves represent soil resistance at pile tip versus relative pile tip deflection
- Backbone T-z and Q-z curves derived from load test data and theory for cohesive and cohesionless soils (Reese et al, 2006)
- Account for cyclic axial behaviour using various load-unload rules
- Where cyclic pore pressure generation occurs along the pile during shaking need to account for this by cyclically degrading the backbone T-z or Q-z curves or else bound axial pile response using pre-earthquake and post-earthquake curves

Uncoupled Seismic Analysis of Single Piles

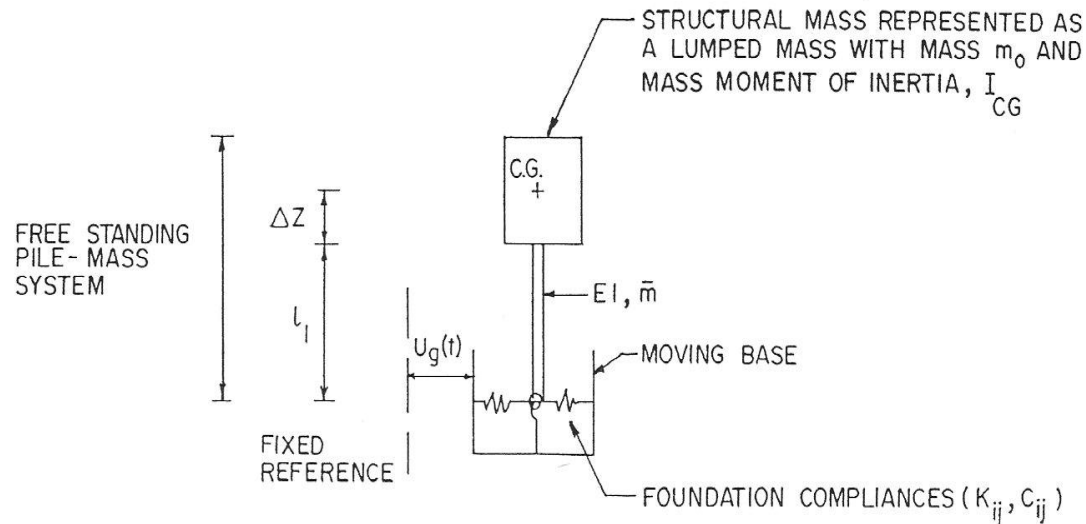
- Consider response of broadly spaced piles ($s/d > 6$)
- Objective is to determine pile stresses in response to SS inertia loads and imposed lateral ground displacements
- Use uncoupled SS analysis to determined pile head loads and influence of SS inertia forces on single piles
- Where effects of lateral soil displacement at depth on pile bending is important, need to do coupled SS – pile analysis considering both inertial and KI loading

Uncoupled Seismic Analysis of Single Piles

- For uncoupled superstructure analysis require pile head lateral, vertical and rotational stiffness parameters with nonlinear soil-pile interaction represented using P-y and T-z curves (programs LPILE and APILE developed by Ensoft)
- Compute nonlinear lateral, rotational and vertical load-deflection (moment-rotation) curves at pile head
- For uncoupled modal analysis use equivalent linear, secant stiffness at an effective pile head displacement level and iterate on solution
- For uncoupled time step analysis use nonlinear pile head force-displacement (moment-rotation) curves

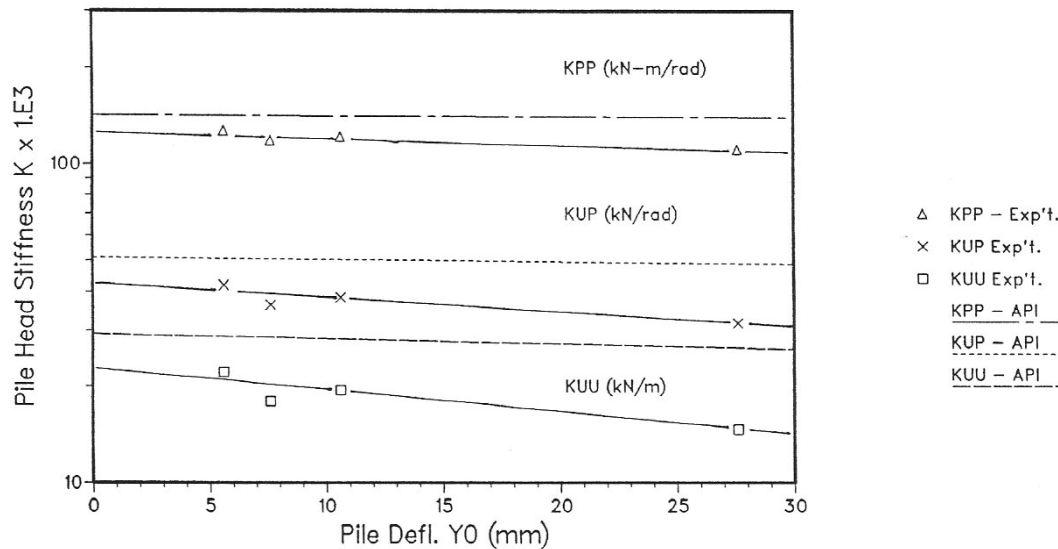
Uncoupled Seismic Analysis of Single Piles

- Uncoupled superstructure – pile model using nonlinear time step analysis
- Objective is to calculate pile head forces and moments at ground surface and do separate analysis of pile bending response due to SS inertia loads



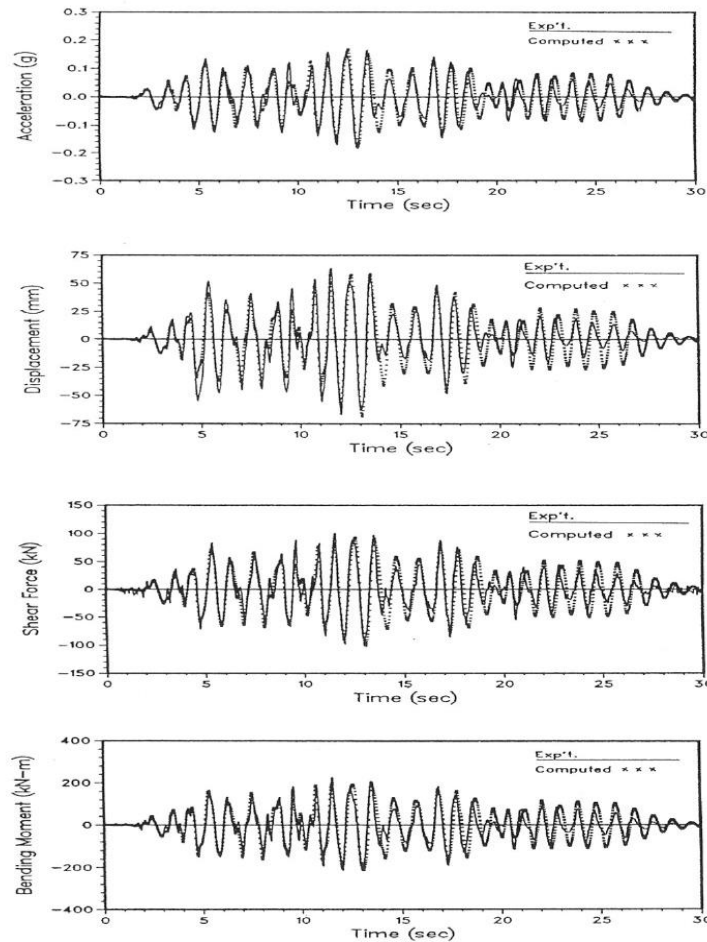
Uncoupled Seismic Analysis of Single Piles

- Consider variation in pile head stiffness (K_{uu} , K_{rr} and K_{ur}) with lateral pile head deflection relative to free field ground surface deflection
- Use pile head viscous dashpots to represent far field radiation damping and near field hysteretic damping



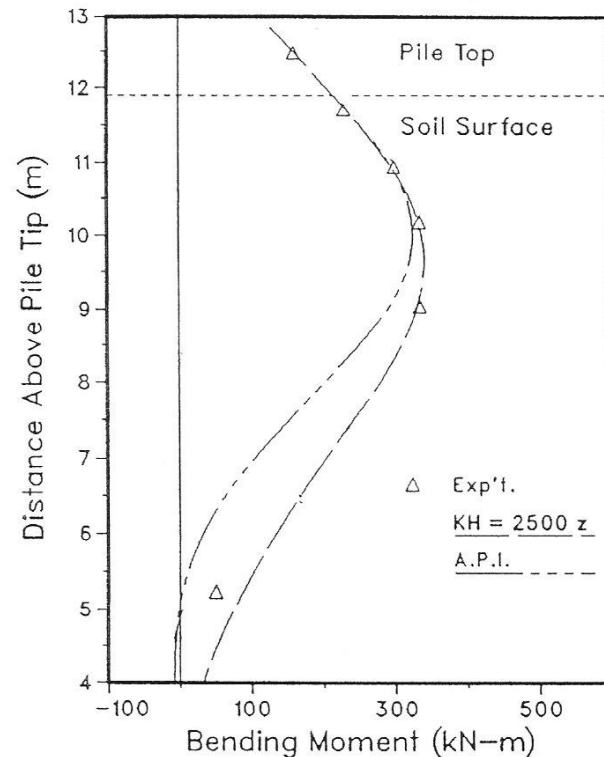
Uncoupled Seismic Analysis of Single Piles

- Results



Uncoupled Seismic Analysis of Single Piles

- From uncoupled SS-pile analysis compute pile head moments and shears and do lateral pile bending analysis, assuming dominated by SS inertia



Uncoupled Seismic Analysis of Single Piles

- Key input is effective ground motions transmitted to base of pile cap and pile head stiffness/damping parameters
- Require good prediction of natural frequency of superstructure – pile foundation system which varies during shaking due to nonlinear pile head compliances
- Use computed pile head moments and shears to assess pile bending response during shaking based solely on SS inertia loading
- Where lateral ground displacements impose significant loading on pile, perform coupled SSI analysis

Coupled Seismic Analysis of Single Piles

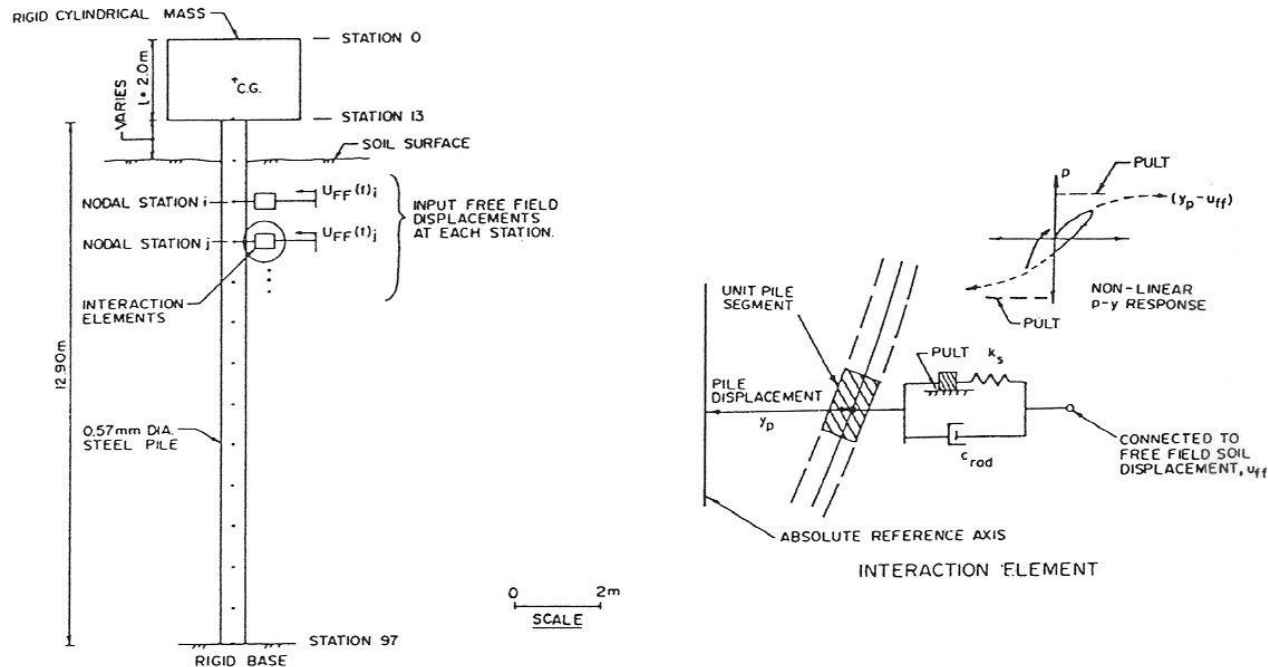
- Simplified pseudo-static analysis using LPILE or LATPILE where apply maximum pile head moments and shears concurrently with maximum $U_{ff}(z)$ at some time during or after shaking
- Maximum pile head loading may not occur at same time as maximum U_{ff}
- Neglect resistance due to pile inertia and soil damping – generally leads to over-prediction of pile bending response
- Preferable to do dynamic coupled analysis

Coupled Seismic Analysis of Single Piles

- Method 1 - Beam on nonlinear Winkler springs with soil radiation damping and imposed $U_{ff}(z,t)$ – key input is U_{ff} from separate FF site response analysis
- Method 2 – Beam (linear or nonlinear structural response) on nonlinear Winkler springs to represent near field response connected to FE or FD mesh representing nonlinear far field response
 - In Method 2 radiation damping automatically accounted for
 - In Method 2 generally employ 2-D plane strain model of far field soil response and need to employ 2-D beam elements with $I = t^3/12$ and $A = t \times 1.0$
 - Calibrate 2-D far field response to approximately match 1-D site response results where level ground conditions occur

Coupled Seismic Analysis of Single Piles

- Method 1 example (beam on nonlinear Winkler foundation in dry sand – centrifuge test data using SHAKE free field motions)

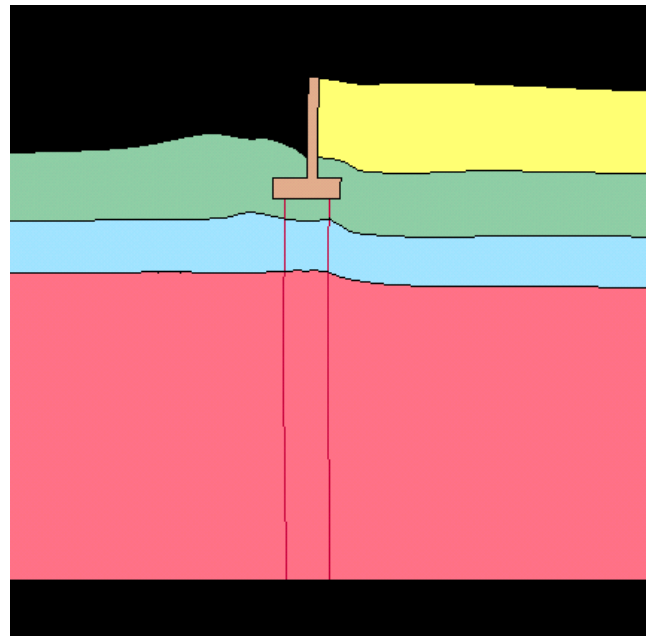


Coupled Seismic Analysis of Single Piles

- General experience with Method 1 approach
 - Results sensitive to input $U_{ff}(z,t)$ – use of equivalent linear SHAKE model generally under-predicts ground displacements and should use nonlinear site response models for strong shaking
 - Locations of maximum moment strongly depends on p-y curves used and resultant lateral soil stiffness versus depth
 - Frequency content of pile response (displacements, moments, shears) simulated well provided have accurate U_{ff} time history input

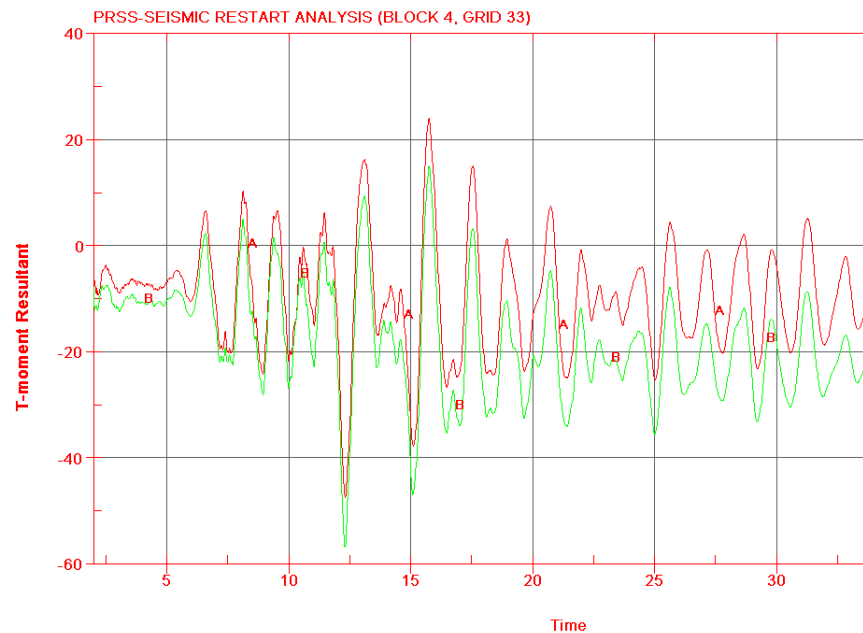
Coupled Seismic Analysis of Single Piles

- Method 2 example (imbalanced fill loads on existing timber piles in liquefiable fill over soft peat and clayey silt)
- Pile cap constrained laterally
- Neglect pile group interaction effects
- Large lateral ground displ. in liq. fill and peat



Coupled Seismic Analysis of Single Piles

- Method 2 results
 - Ground displacement loading dominates pile response
 - Highest bending moment and curvatures near underside of pile cap with $\phi \cong 0.06$ (near limit for timber piles)



Uncoupled Inertial Analysis of Pile Groups

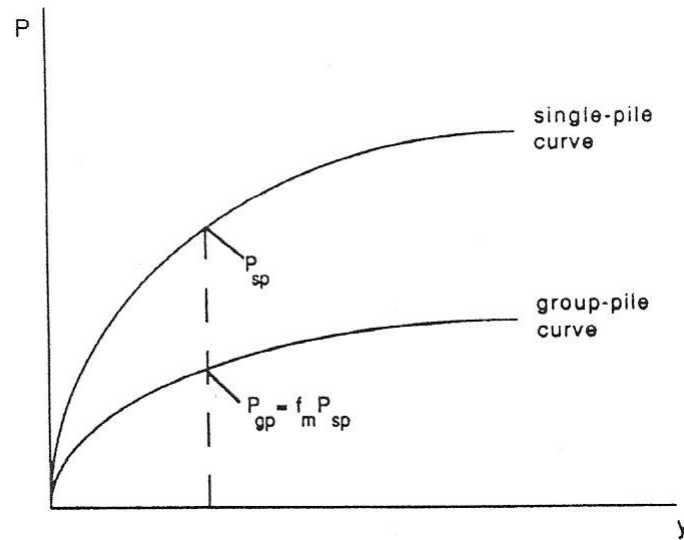
- Require equivalent linear or nonlinear force-displacement or moment rotation relations at pile cap level to be used in equivalent linear or nonlinear inertial interaction analysis
- Most commonly used program for nonlinear response (strong shaking) is GROUP (Ensoft)
- Program based on FE discretization of linear elastic beam elements on nonlinear Winkler springs (lateral and axial response) developed from P-y, T-Z and Q-Z curves
- Stress overlap effects between closely spaced piles (group interaction effects) approximately modeled using user-defined P-y and T-Z curve multipliers

General Commentary of Group Interaction

- Interaction strongest for inline loading of piles (trailing piles) and reduced interaction for offline loading
- Linear elastic models of group interaction have less applicability for moderate to strong shaking where plastic yield of soil around piles dominates
- Continue to use elastic models of GI for small strain dynamic excitation of piles (machine foundations) embedded in programs like DYNA5 – GI shows strong frequency effects due to out of phase motions of pile vibrations within a group

P-Y Curve Multipliers

- Typical multipliers suggested by Reese et al (2006), “Analysis and Design of Shallow and Deep Foundations” for inline and offline interaction
- Multipliers depend on pile spacing and locations of piles in group – should be selected by a qualified geotechnical engineer

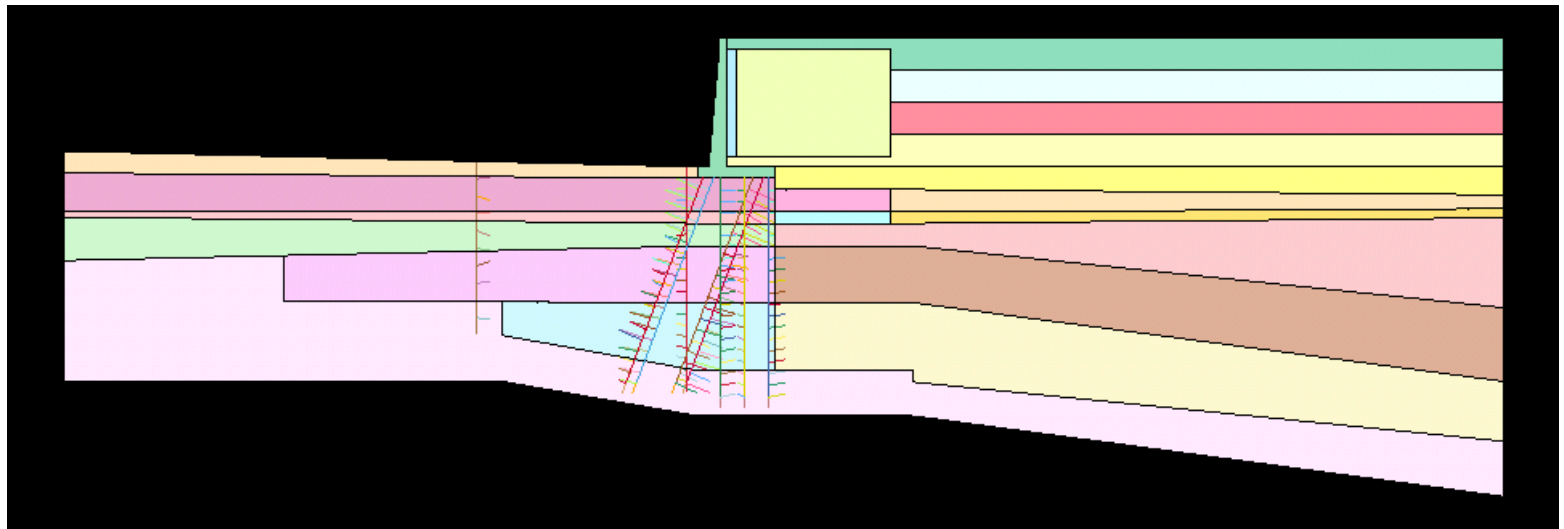


2-D Coupled Model of Pile Group Response

- In plane representation of a typical line of piles with accounting for out of plane spacing S of piles to get equivalent 2-D representation of pile structural properties
- Model near field soil-pile interaction using nonlinear lateral and axial springs, connected to adjacent soil nodes in FE or FD mesh
- Do parametric studies with and without GI effects on scaling of near field interaction springs
- Couple far field ground displacement response to pile group

2-D Coupled Model of Pile Group Response

- Example using program LSDYNA with following features:
 - Concrete batter pile group with nonlinear moment-curvature relation and pinned head connections
 - Nonlinear soil stress-strain response with softening due to pwp build-up & use of lightweight EPS fill
 - Near field nonlinear lateral and axial springs using GI multipliers on lateral springs

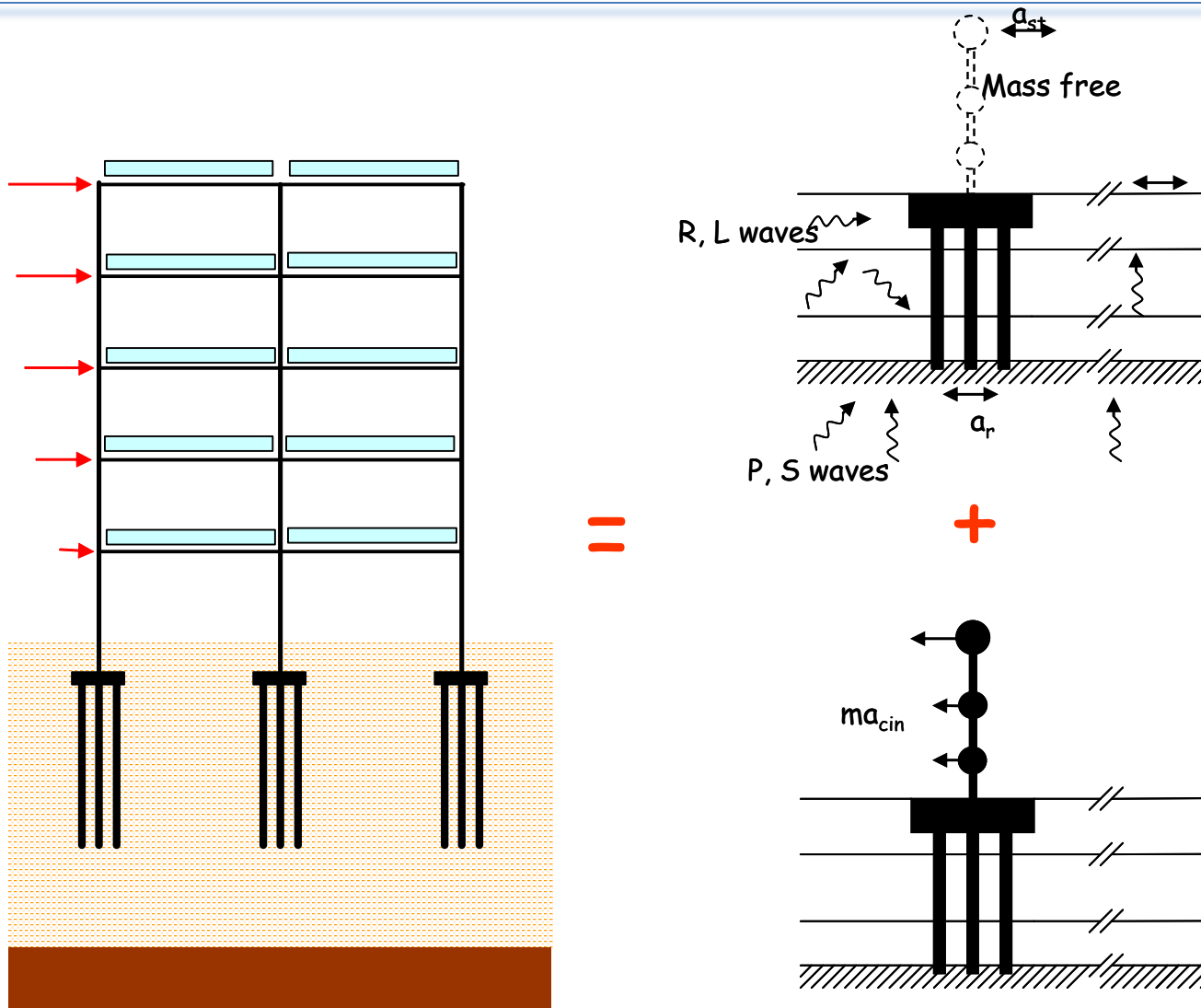


2-D Coupled Model of Pile Group Response

- Key conclusions:
 - Modeling nonlinear free field soil response with non-level ground effects, including pwp build-up, very important in assessing dominant loading on pile group
 - Modeling structural response of piles (nonlinear moment curvature, pinned head connections) very important to indicate zones where plastic yield occurs and whether axial buckling could occur
 - Near field soil-pile interaction, including GI, important in simulating maximum lateral pressures that can be applied to a pile in the group

Approximate 3-D Model of Pile Group Response

Response of Structures on Piled Foundations: Kinematic Interactions + Inertial Interaction



Kinematic Interactions:

- Foundation input motions
- Pile stresses (moments etc)
- Impedance values

Inertial Interaction on piles:

- Pile/cap stresses
- Pile/cap deflections

Acknowledgement to Dr. Rosa M.S. Maiorano

Response of Structures on Piled Foundations:

Current methods of analysis and design

Kinematic Interactions:

- Foundation input motions (FIM)
- Pile stresses (moments etc)
- Impedance values

Tools:

1. Analytical solutions, empirical methods
2. VERSAT-P3D finite element method

Often not completed

Inertial Interaction on piles:

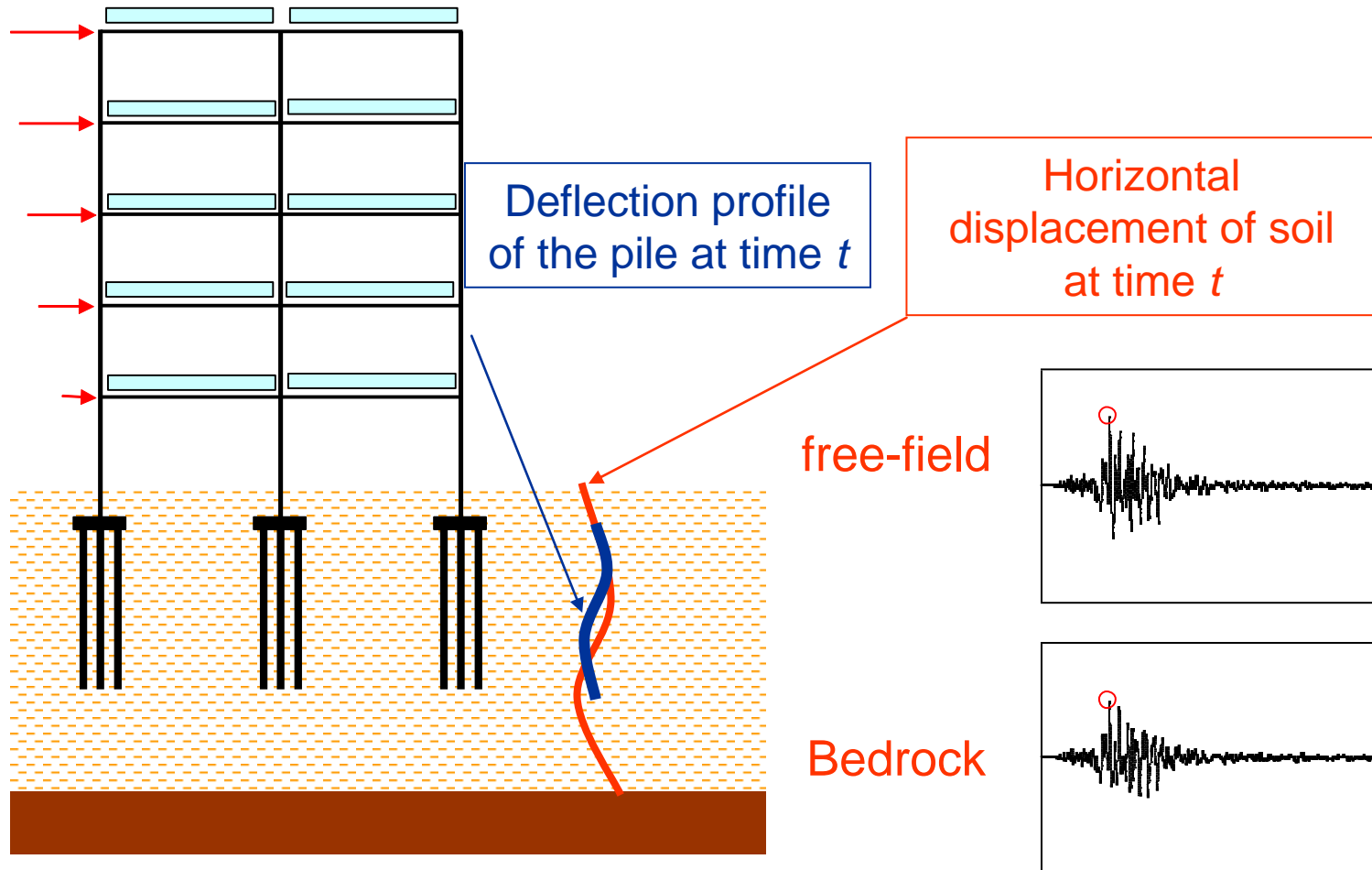
- Pile/cap stresses
- Pile/cap deflections

Tools:

1. FIM = free field motions (ProShake, VERSAT-2D, FLAC?)
2. Kinematic stresses ignored
3. Pile cap impedance, DYNA5 (interaction factors), VERSAT-P3D
4. Lateral single pile analyses LPILE, VERSAT-P3D
5. Pile Group Analysis (GROUP using p-multipliers, & VERSAT-P3D)

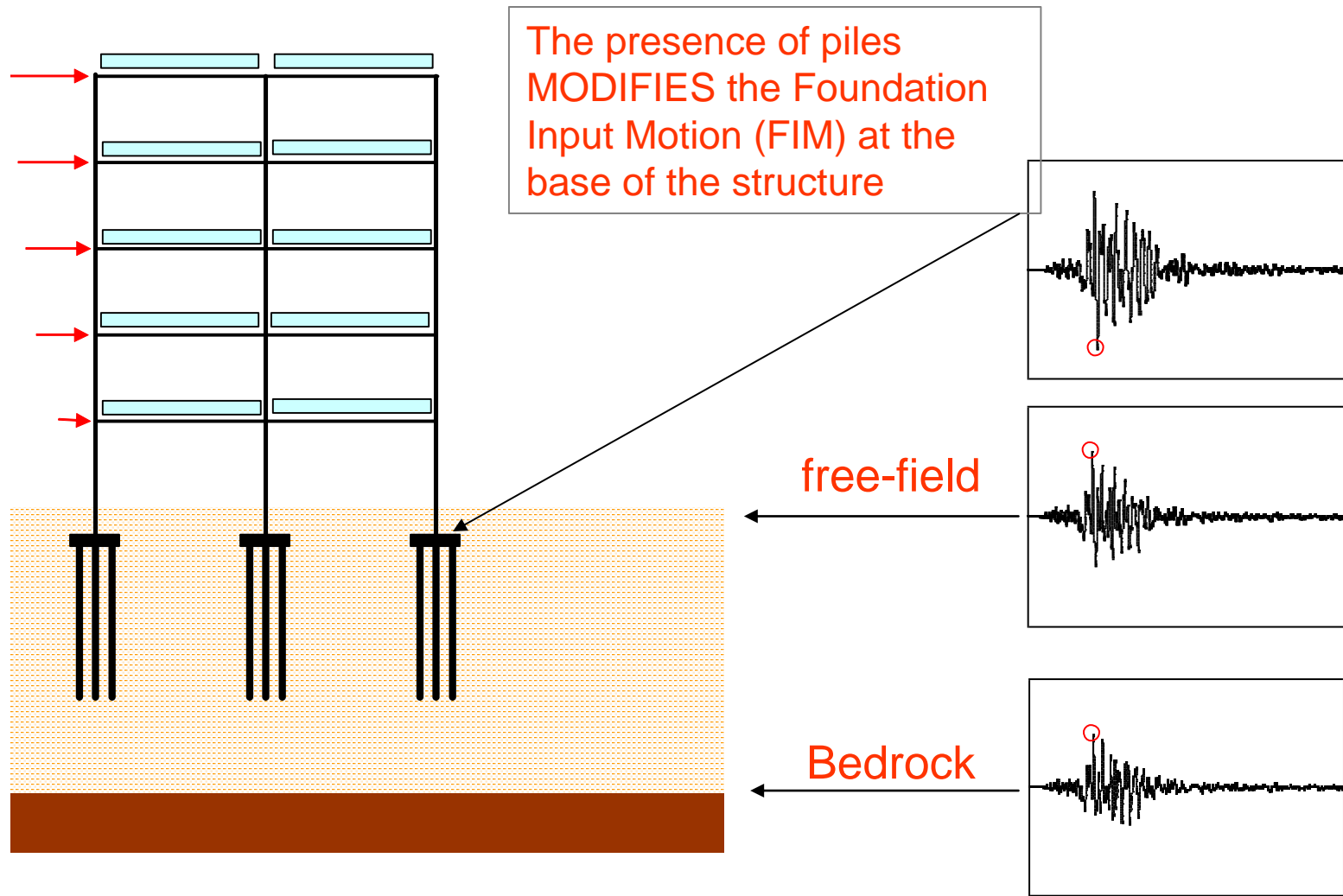
To be addressed, Eurocode 8 (2003), Italian building code (2008)

Kinematic Response of Pile Foundations: kinematic deflections & bending moments along the piles



Kinematic Response of Pile Foundations:

Seismic motions at the pile head / pile cap



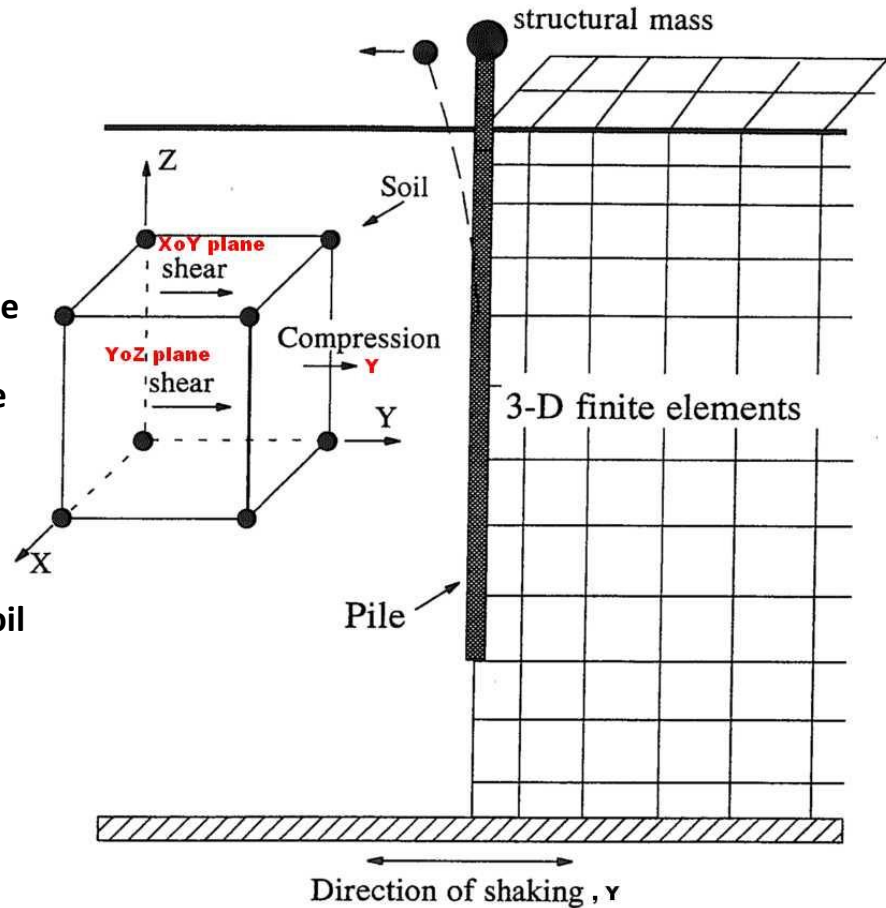
Kinematic Response: Quasi-3D Finite Element Method of Analysis

Equations of Motions (Wu and Finn 1997):

$$\rho_s \frac{\partial^2 v}{\partial t^2} = G^* \frac{\partial^2 v}{\partial x^2} + \theta G^* \frac{\partial^2 v}{\partial y^2} + G^* \frac{\partial^2 v}{\partial z^2}$$

Features:

1. Solutions in frequency domain for impedance (stiffness and damping) calculation
2. Solutions in time-domain for non-linear time history analyses of piled foundations under earthquake loadings
3. Superstructures are represented by a rigid pile cap with masses in translation and rotation.
4. Equivalent linear procedures for modeling soil nonlinearity.

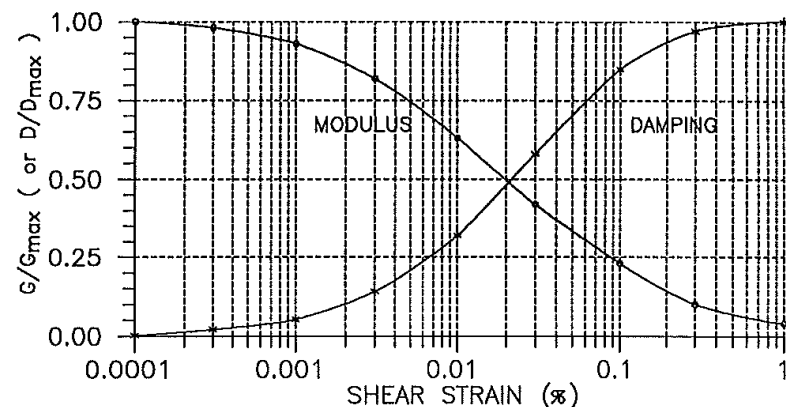


References:

- Wu, G., and Finn, W.D.L. 1997a. Dynamic elastic analysis of pile foundations using finite element method in the frequency domain. Canadian Geotechnical Journal, 34: 34-43
- Wu, G., and Finn, W.D.L. 1997b. Dynamic nonlinear analysis of pile foundations using finite element method in the time domain. Can Geot J, 34: 44-52

Nonlinear modeling:

- Seed's equivalent linear procedure modified for time-history analyses, i.e., G and λ vary with shear strain (γ) in the time domain.
- User defined curves for G/G_{\max} and λ (figures on the right as an example)



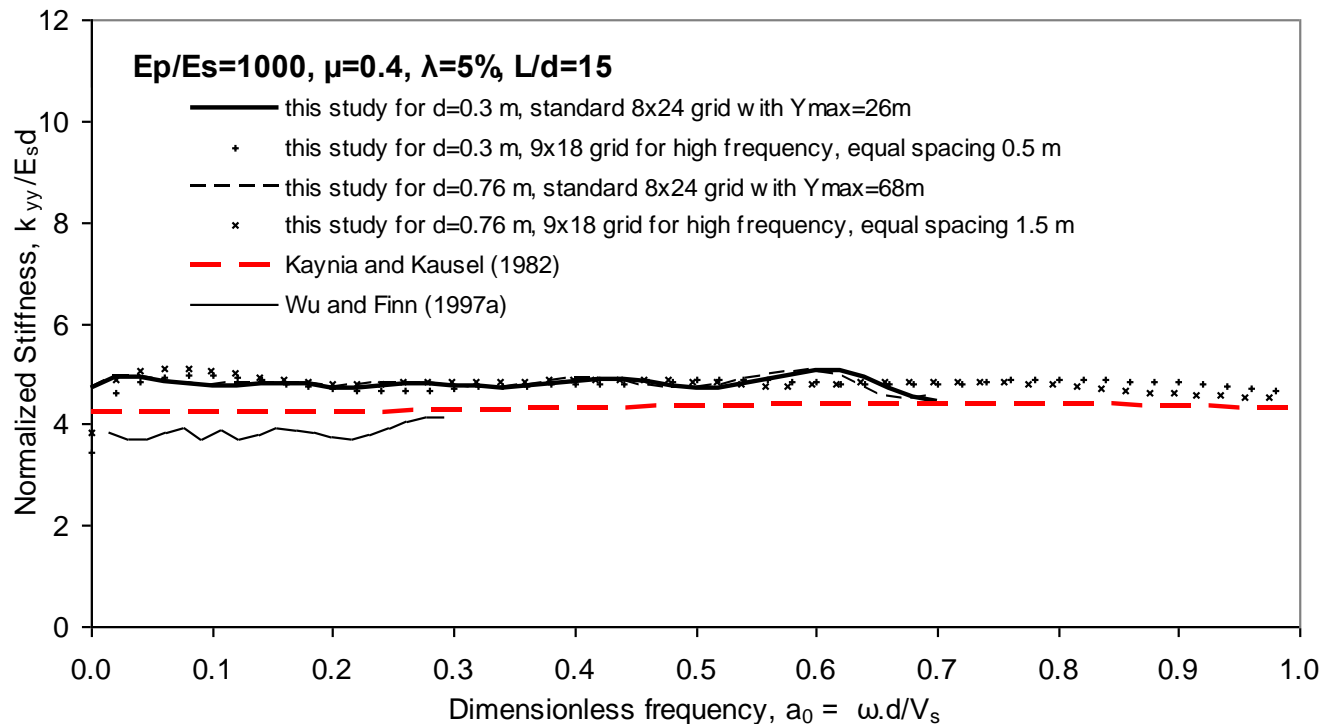
New enhancements

- 8-node pile element
- energy boundary for radiation damping at boundaries.
$$c_s = \int_A \rho V_s dA \quad \text{for shearing}$$
$$c_p = \int_A \rho V_p dA \quad \text{for compression}$$
- Incorporated in the program VERSAT-P3D (Wu 2006)

Quasi-3D Method: Verification with Solutions by Kaynia and Kausel

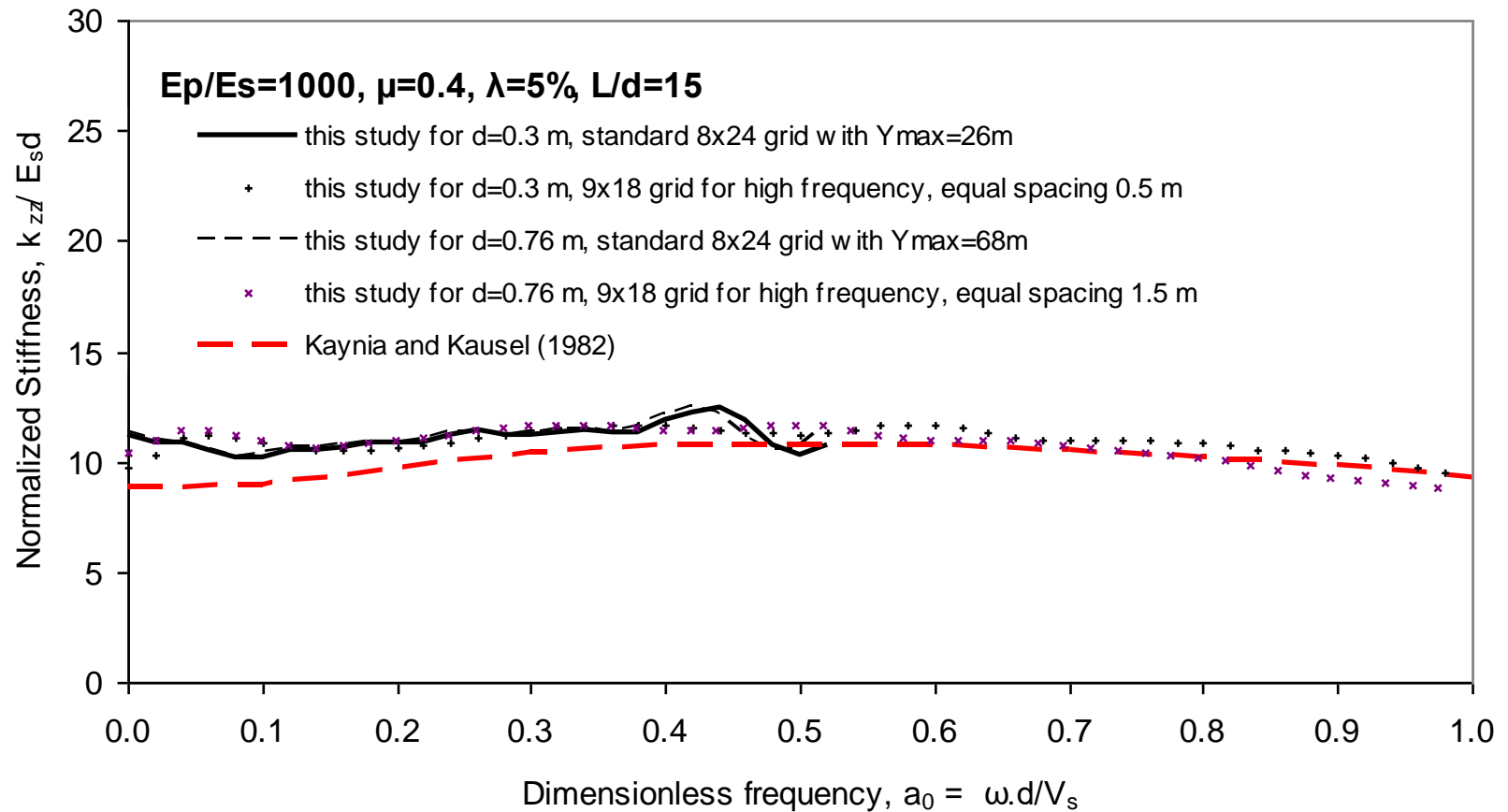
Normalized horizontal stiffness of single piles

(Source: Wu 2007 60th Can. Geot. Conference)



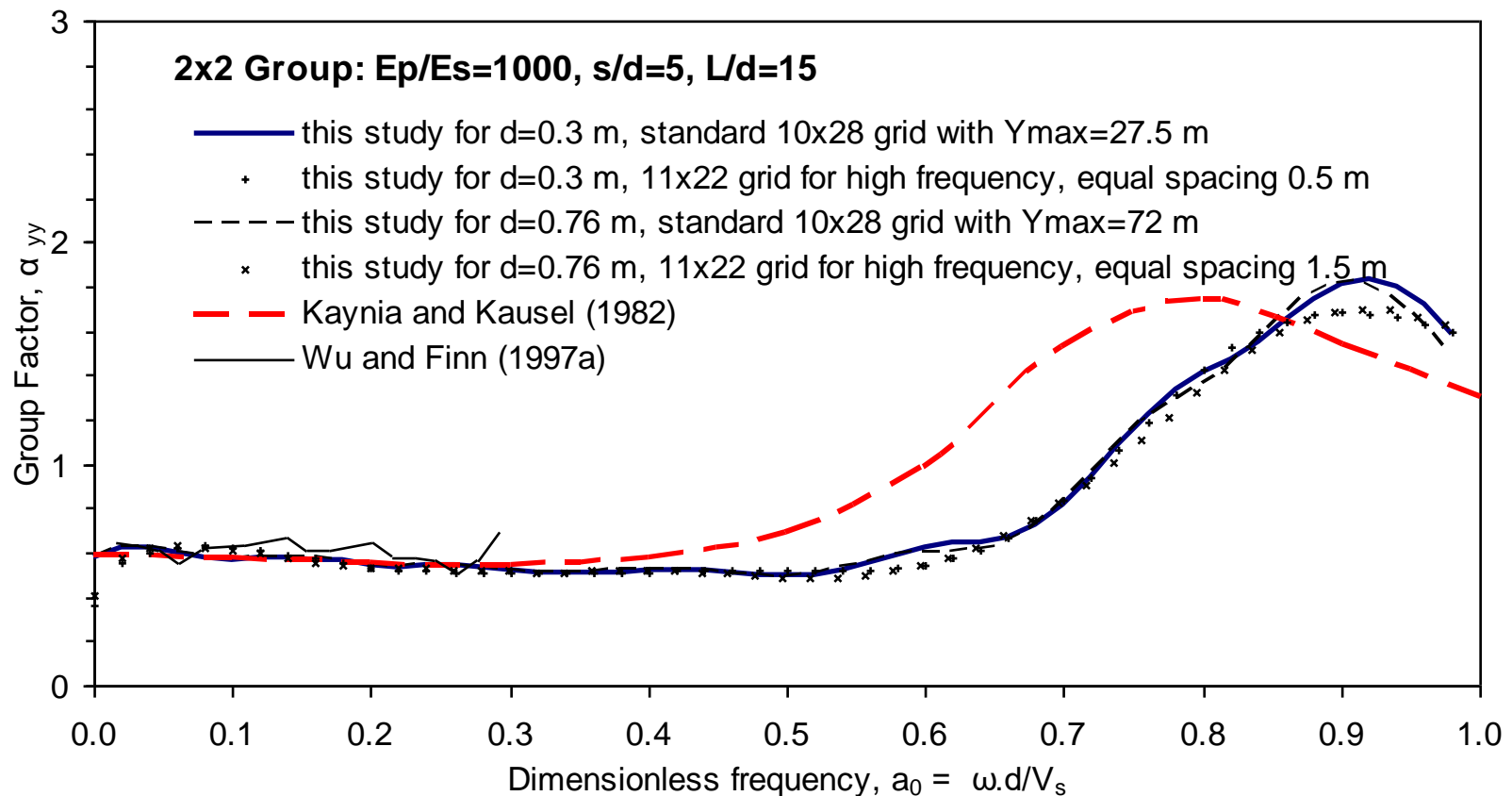
Quasi-3D Method: Verification with Solutions by Kaynia and Kausel

Normalized vertical stiffness of single piles



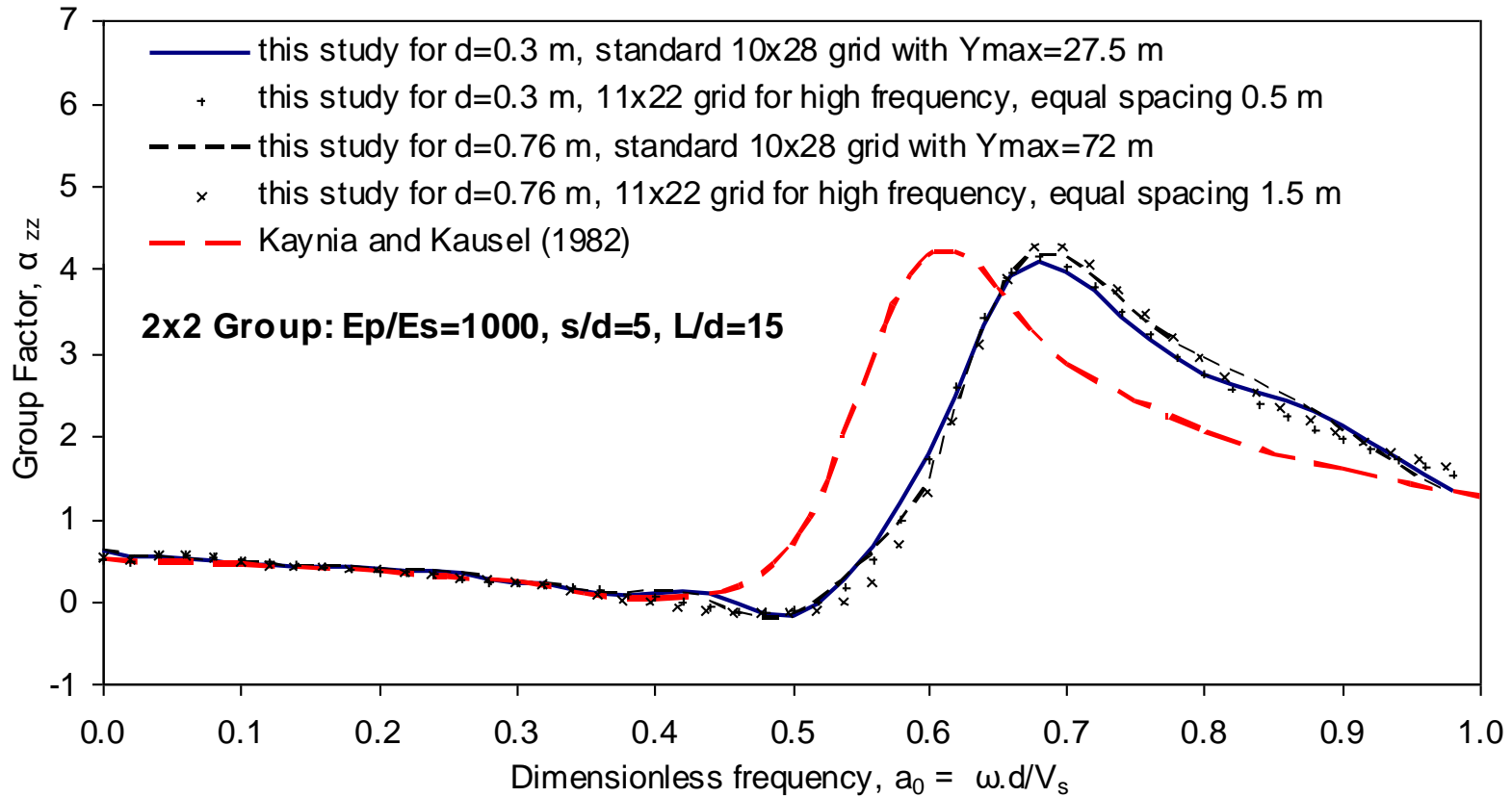
2x2 pile group: Horizontal stiffness

(Source: Wu 2007 60th Can. Geot. Conference)



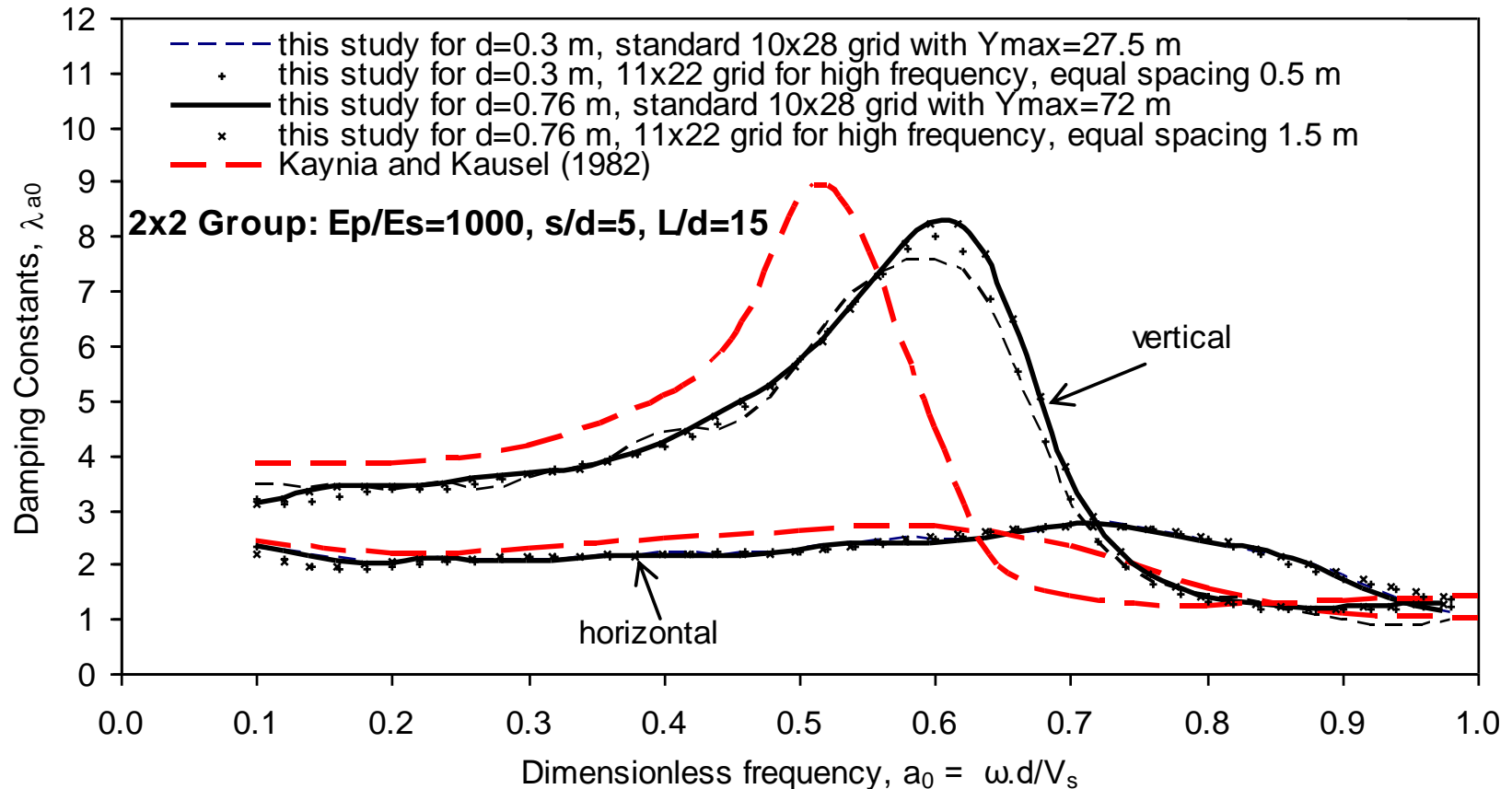
Quasi-3D Method: Verification with Solutions by Kaynia and Kausel

2x2 pile group: Vertical stiffness



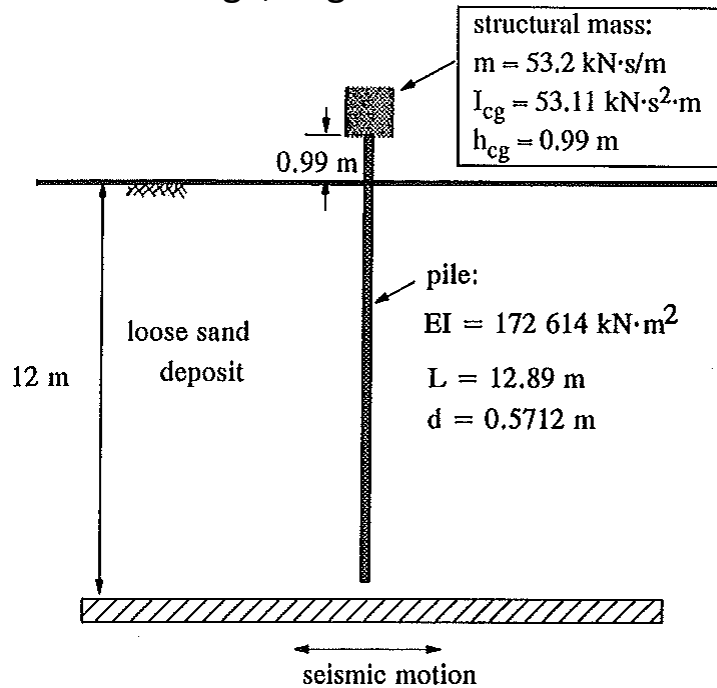
Quasi-3D Method: Verification with Solutions by Kaynia and Kausel

2x2 pile group: Horizontal and vertical damping

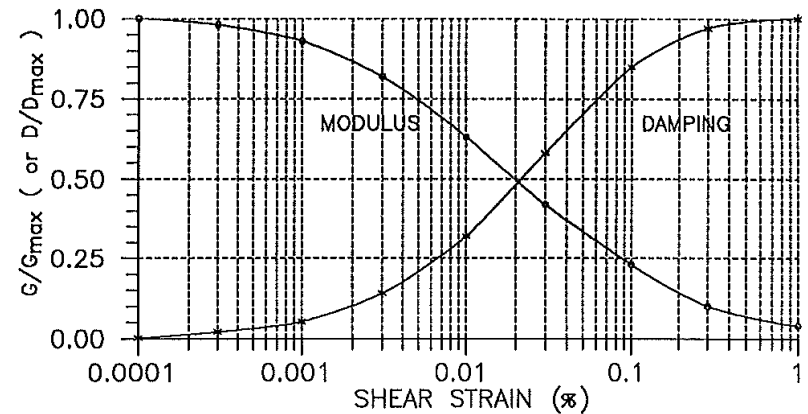


Quasi-3D Method: Verification with centrifuge tests of piles

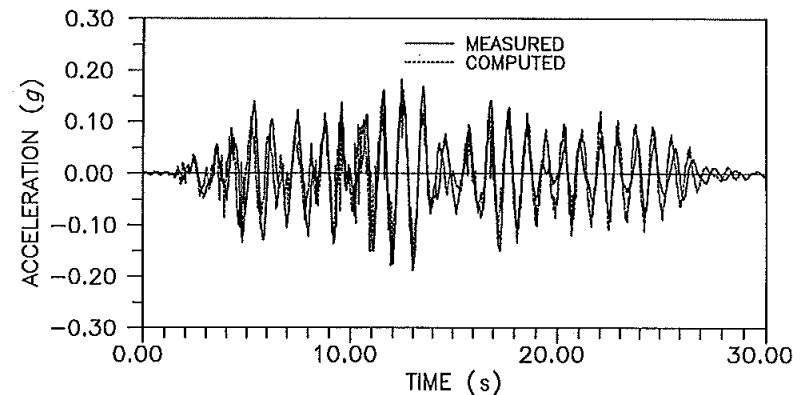
Caltech centrifuge, 60g



G/G_{\max} and $\lambda = f(\gamma)$



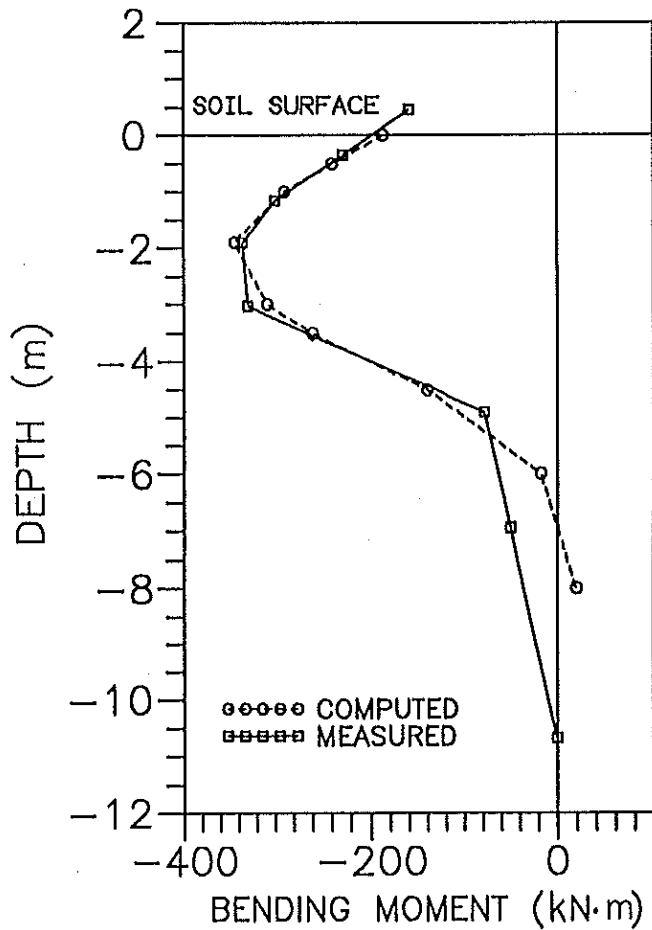
Pile head accelerations



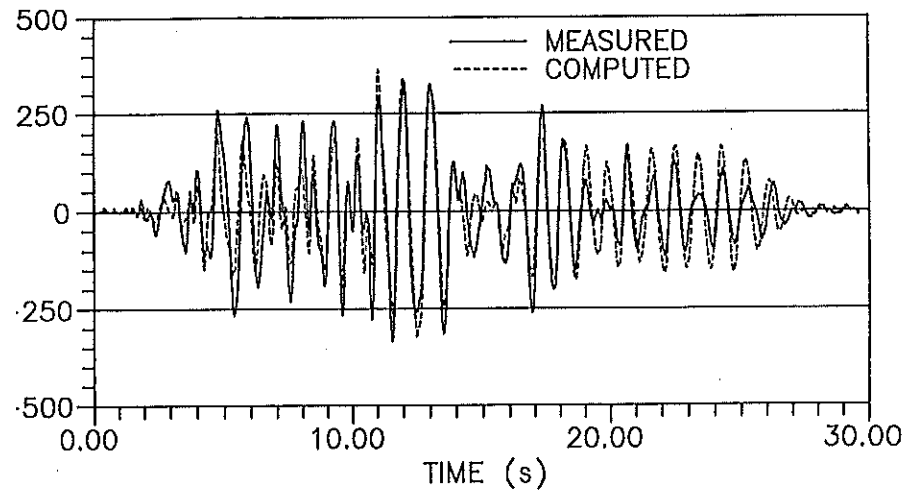
References:

- Finn, W.D.L., and Gohl, W.B. 1987. Centrifuge model studies of piles under simulated earthquake loading. Dynamic Response of Pile Foundations - Experiment, Analysis and Observation, ASCE Geotechnical Special Publication No. 11, pp. 21-38.
- Wu, G., and Finn, W.D.L. 1997b. Dynamic nonlinear analysis of pile foundations using finite element method in the time domain. Can Geot J, 34: 44-52

Quasi-3D Method: Verification with centrifuge tests of piles



Bending moment at 3 m depth



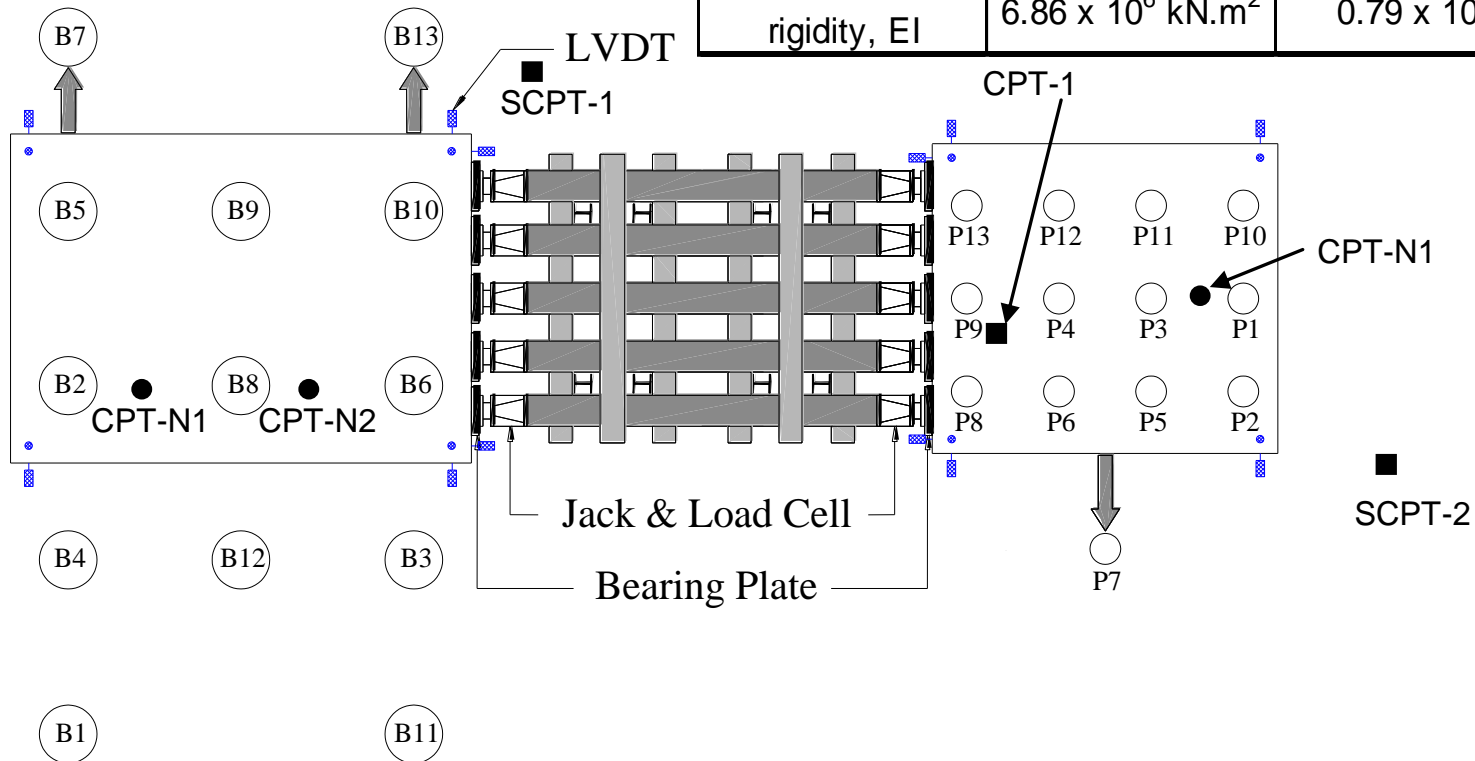
Note:

Similar quasi-3D analyses were also conducted for a centrifuge test on a 2 x 2 pile group (Wu and Finn 1997).

Quasi-3D Method: Verification with in-situ load tests of pile groups

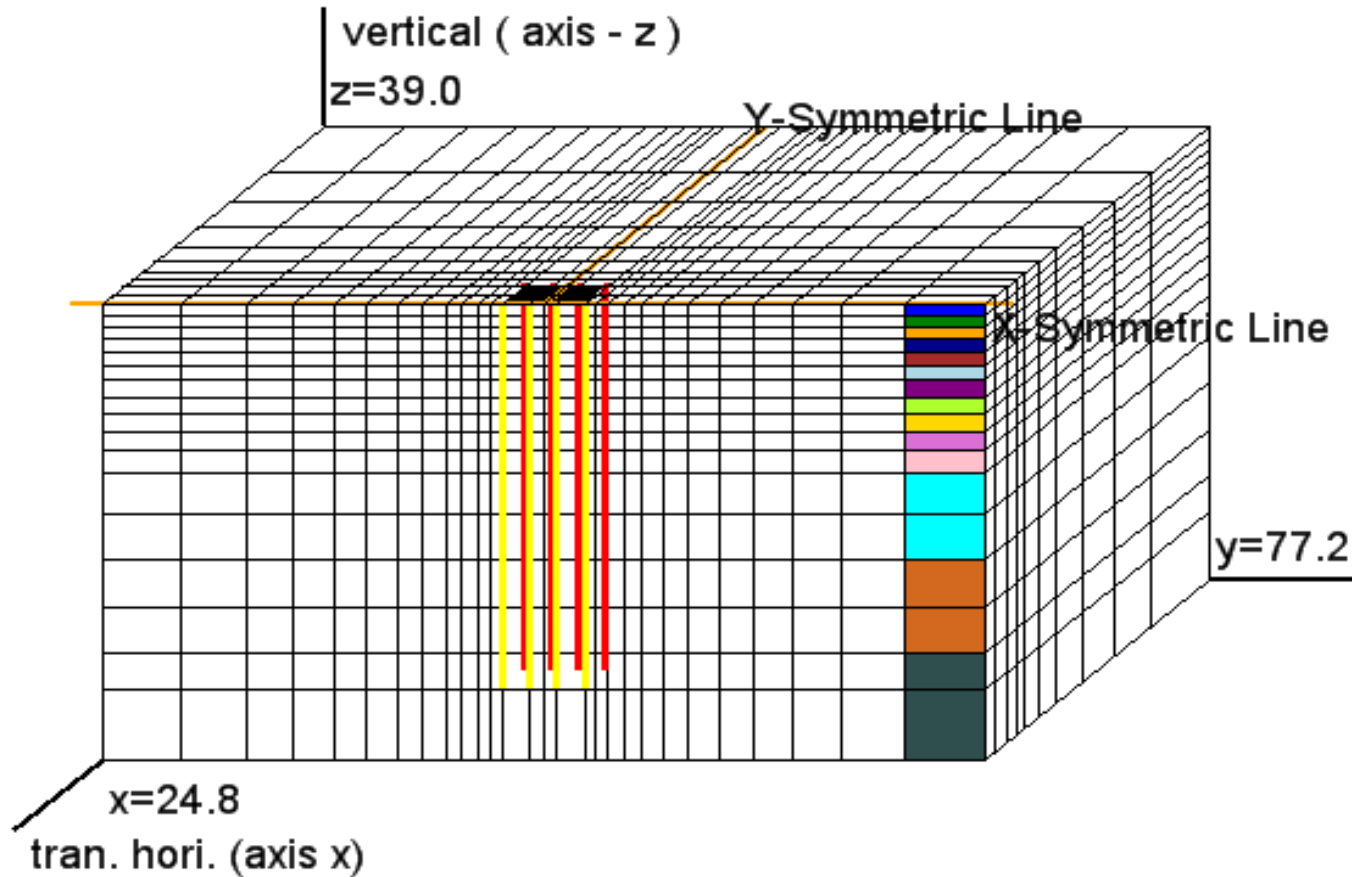
Test Piles and CPT Test Holes from Huang et al. (ASCE Geot J. 2001)

Item	Bored piles	PC piles
Diameter	1.5 m	0.8 m OD, 0.56 m ID With concrete infill
Length	34.9 m	34 m
Intact flexural rigidity, EI	$6.86 \times 10^6 \text{ kN.m}^2$	$0.79 \times 10^6 \text{ kN.m}^2$



Quasi-3D Method: Verification with in-situ load tests of pile groups

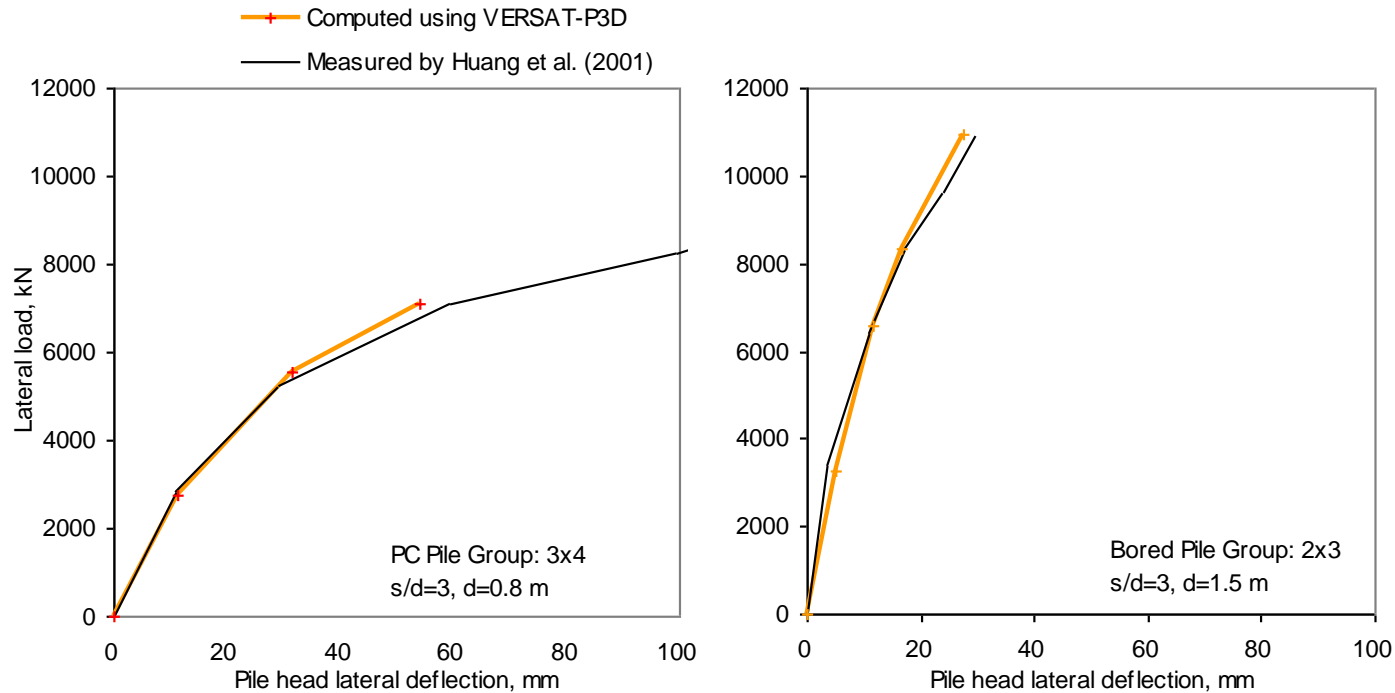
- Quasi-3D finite element model used for the analysis of a 3x4 PC pile group



Quasi-3D Method: Verification with in-situ load tests of pile groups

Lateral load versus pile head deflection for pile groups

(Source: Wu 2007 60th Can. Geot. Conference)



Conventional Method:

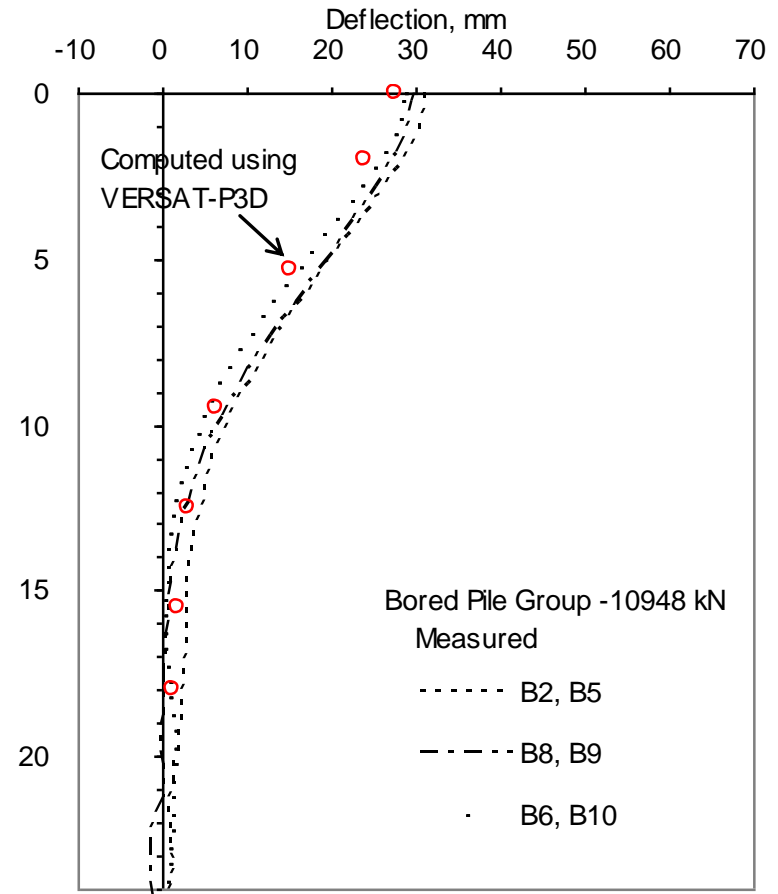
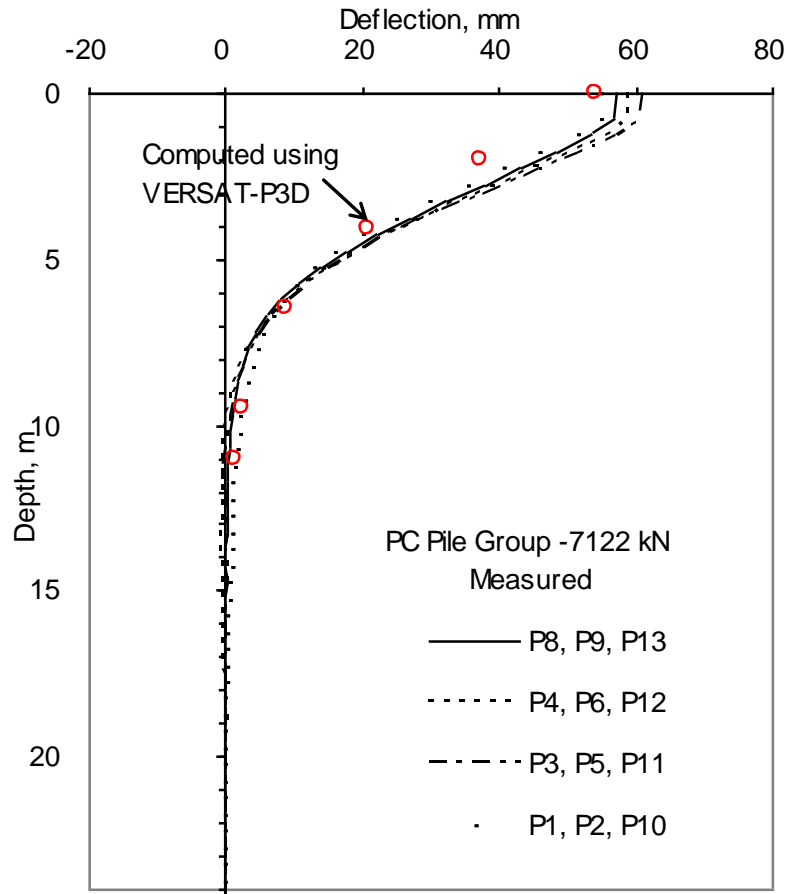
P-y curves and P-multipliers back-calculated by Huang et al. (ASCE Geot J 2001)

Single piles: $p_{multi} = 0.21(PC), 0.50(\text{bored})$

Group piles from DMT data: $p_{multi}(\text{average}) = 0.26(PC \text{ piles}), 0.37(\text{bored})$

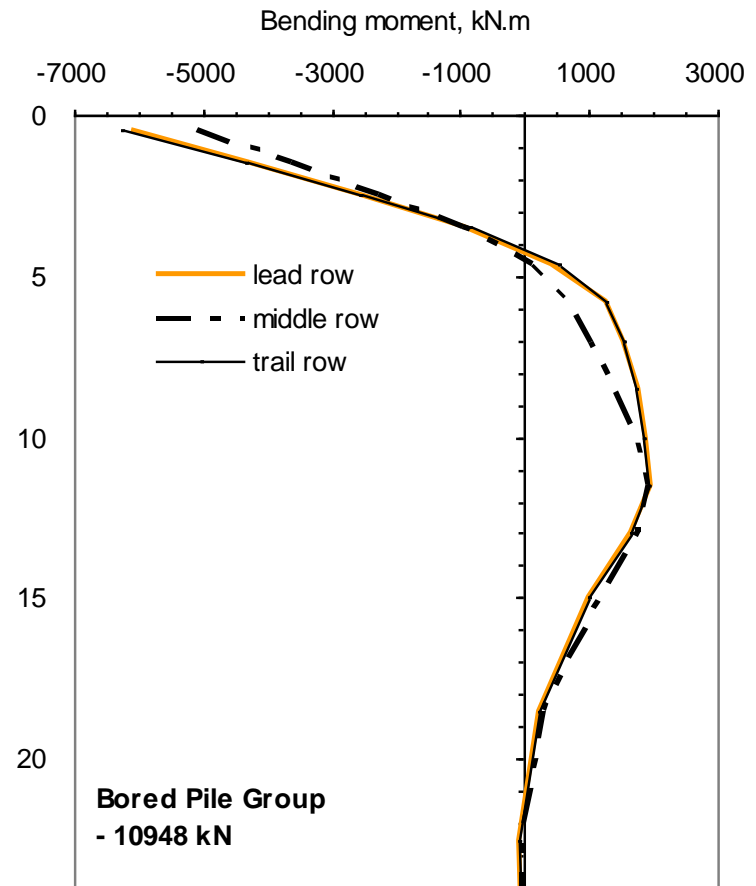
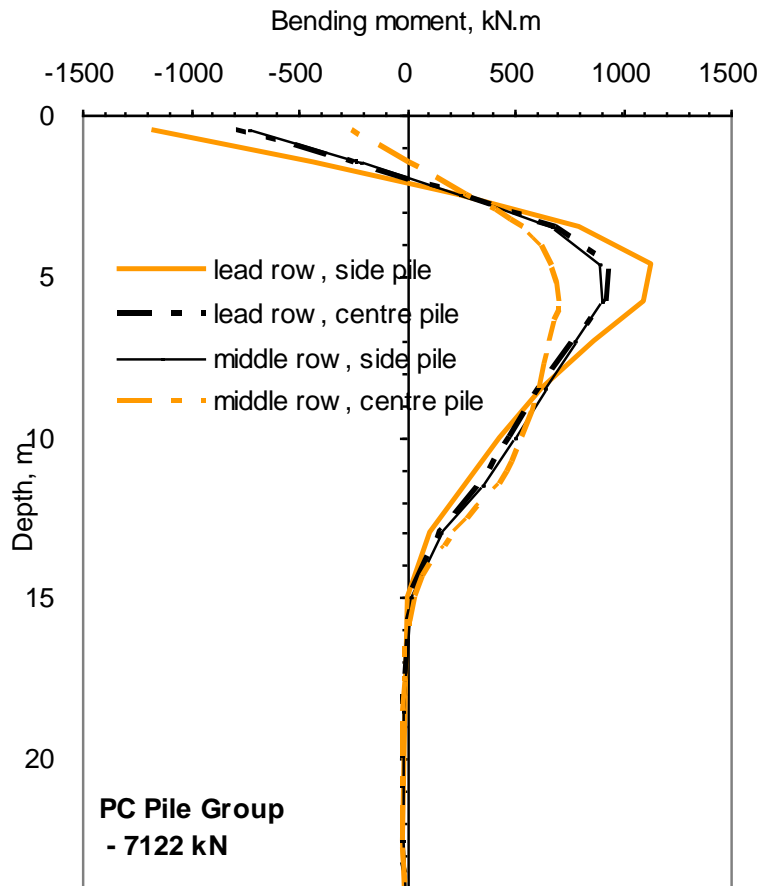
Group piles from load test data: $p_{multi}(\text{average}) = 1.22(PC \text{ piles}), 0.75(\text{bored})$

Quasi-3D Method: Verification with in-situ load tests of pile groups



Quasi-3D Method: Verification with in-situ load tests of pile groups

Computed bending moments of piles versus depth for pile groups



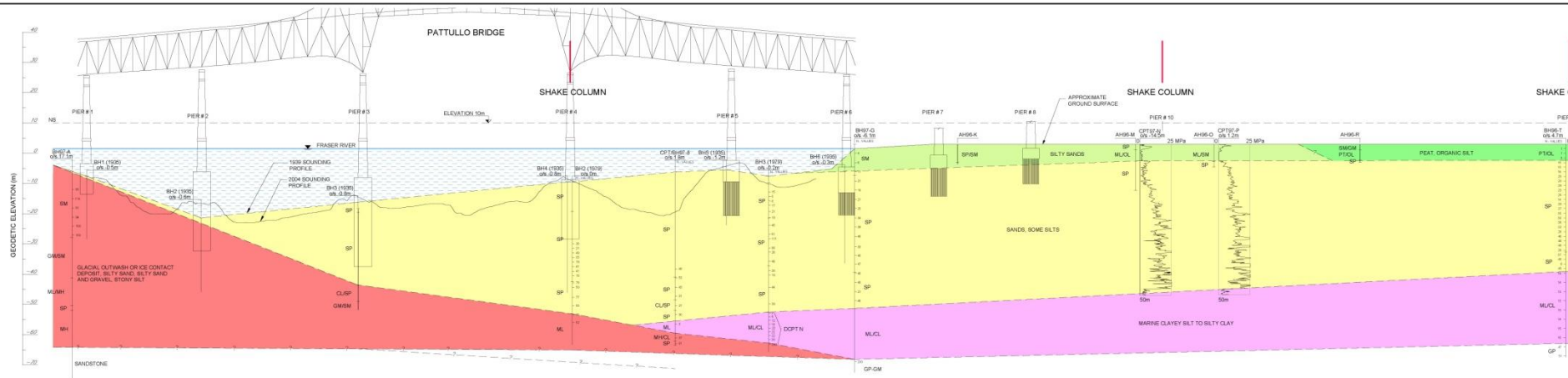
Quasi-3D Method: Application Case History on the Pattullo Bridge

Seismic Assessment for Pattullo Bridge, NW/Surrey, BC

Owner: Greater Vancouver Transportation Authority (Translink)

Consultants: Golder Associates, Sandwell and Hatch Mott MacDonald

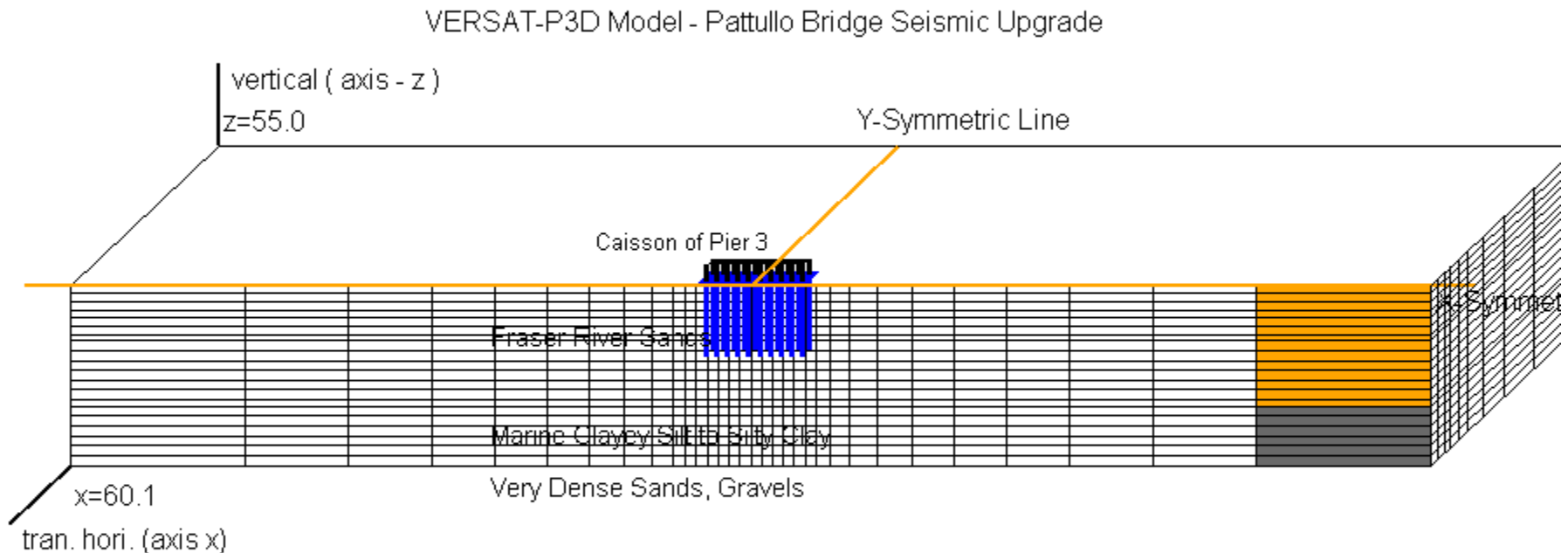
Pier 3 4 under the steel bridge section



1. Vashon Drift - Glacial outwash or ice contact
2. Capilano Sediments - Marine clayey silt to silty clay
3. Fraser River Channel Deposits

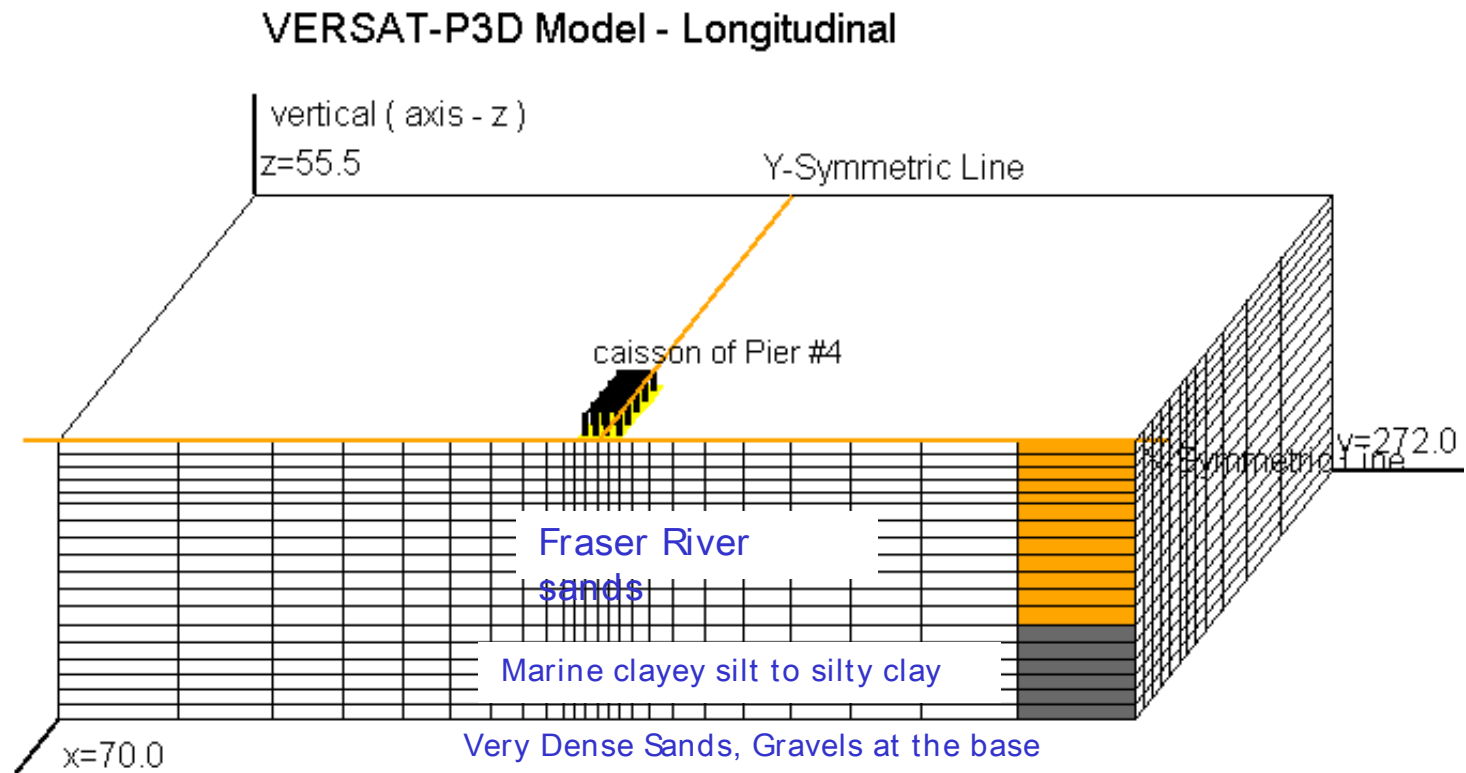
Quasi-3D Method: Application Case History on the Pattullo Bridge

VERSAT-P3D MODEL OF PIER 3 CAISSON (31.7 m by 10.7 m, 22 m deep) - TRANSVERSE TO BRIDGE ALIGNMENT



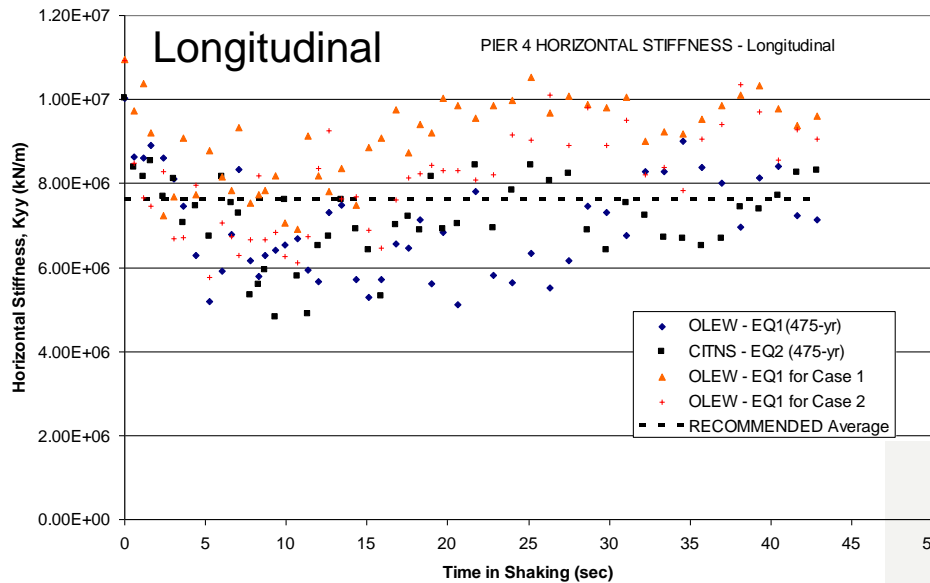
Quasi-3D Method: Application Case History on the Pattullo Bridge

VERSAT-P3D MODEL OF PIER 4 CAISSON (31.7 m by 10.7 m, 20 m deep) - Longitudinal to Bridge Alignment

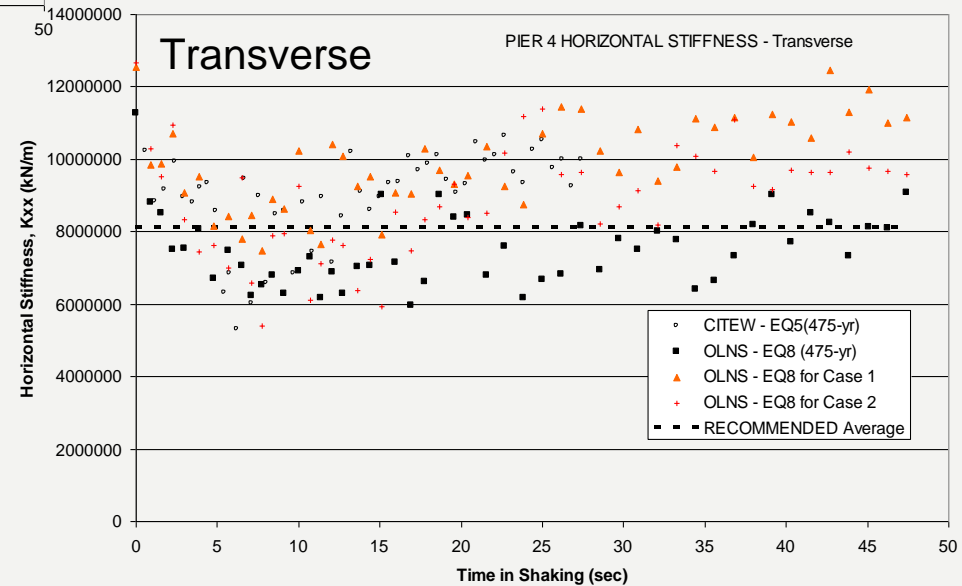


Quasi-3D Method: Application Case History on the Pattullo Bridge

Foundation Impedance Values (stiffness, damping)

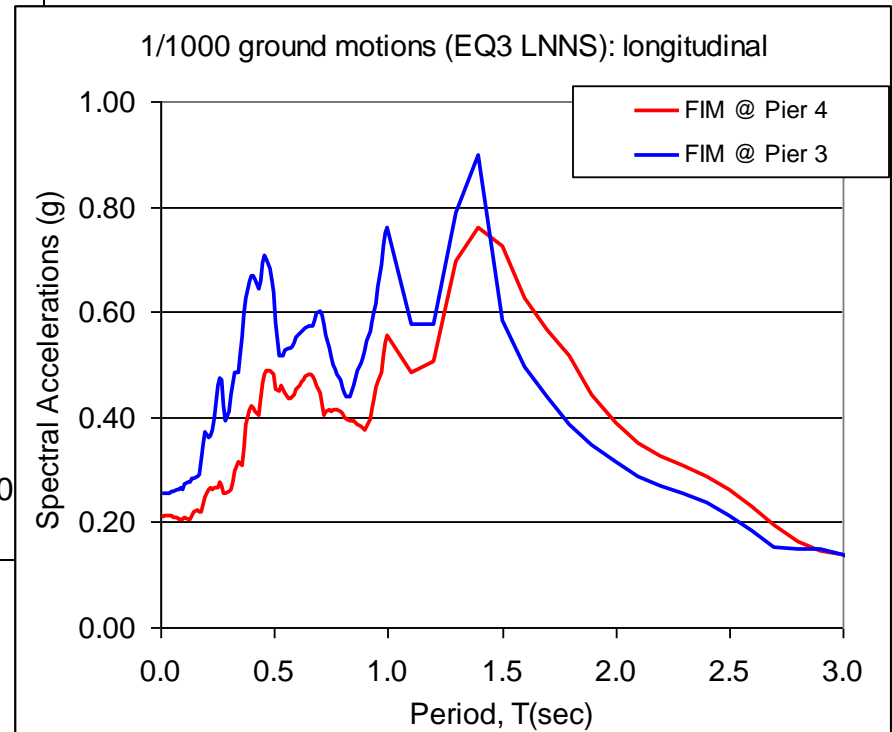
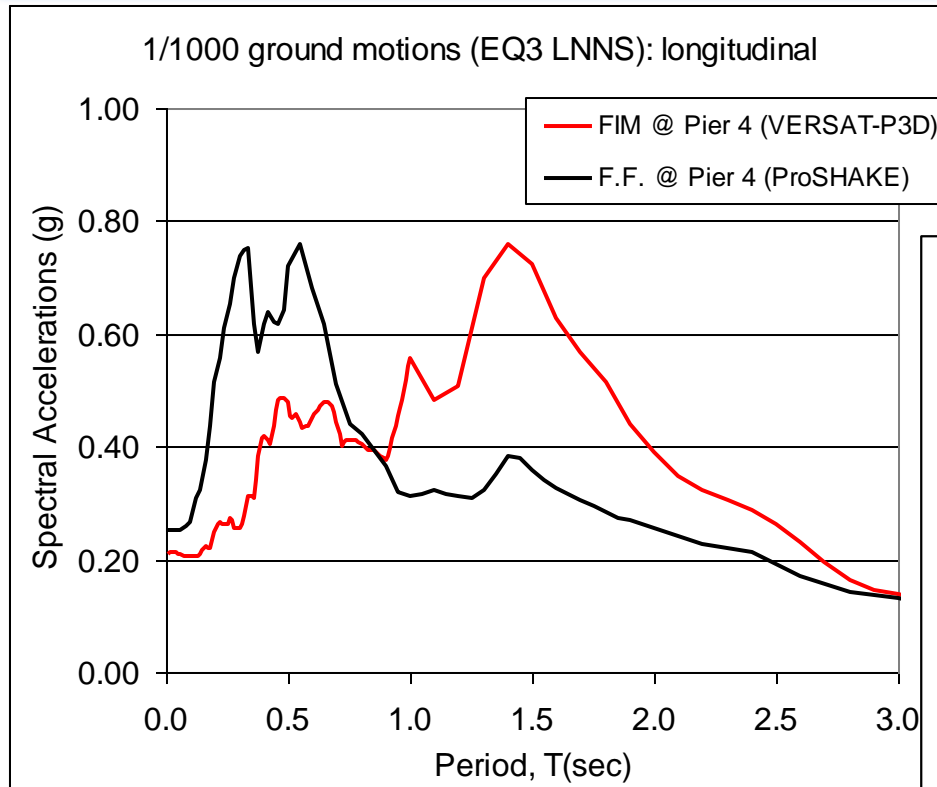


VERSAT-P3D Results: Horizontal Stiffness Time-history of Pier 4 Caisson



Quasi-3D Method: Application Case History on the Pattullo Bridge

Foundation Input Motions (FIM)



Quasi-3D Method:

Research case history for Eurocode 8 (2003) and Italian code (2008)

Eurocode 8: Design of structures for earthquake resistance by CEN/TC250 (European committee for standardization technical committee 250)

Italian code: Italian Building Code DM 14.01.2008

“kinematic effects have to be addressed for the performance evaluation of a piled foundation”

- Research group at the University of Napoli Parthenope, Naples, Italy, including Prof. S. Aversa, Dr. Rosa Maiorano, L. de Sanctis, & A. Mandolini
- In Canada: G. Wu
- Primary goal: kinematic bending moments between two layers & at pile head
- Primary tool: VERSAT-P3D (Wu 2006)

REFERENCES:

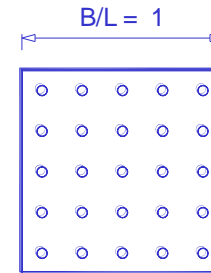
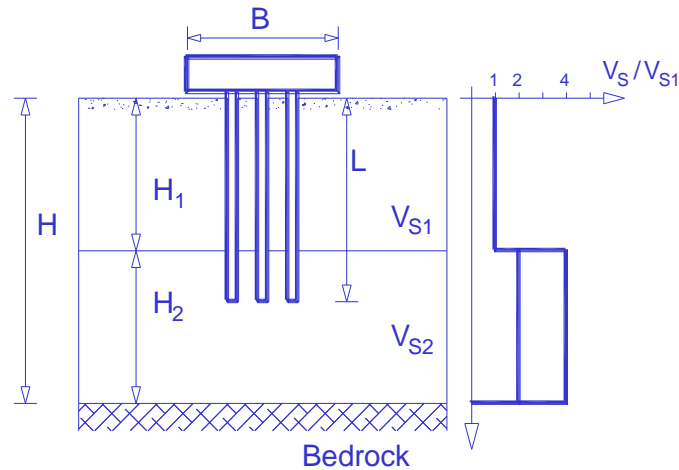
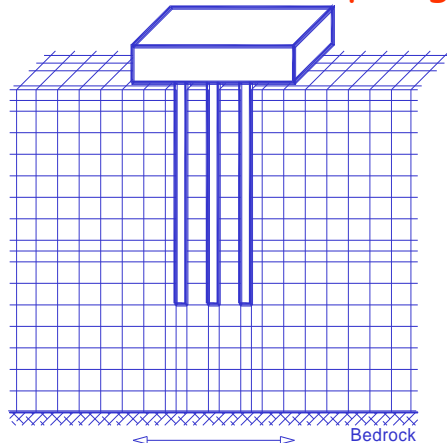
- de Sanctis, L., Maiorano, R.M.S., Aversa, S. A method for assessing kinematic bending moments at the pile head, Earthquake Engineering and Structural Dynamics, vol. 39; p. 375-397, ISSN 0098-8847.
- de Sanctis, L., Maiorano, R.M.S. Earthquake induced kinematic bending moments. International Conference on Performance based design in earthquake geotechnical engineering, IS-Tokyo, Tsukuba, Japan, 15-17 June 2009.
- Maiorano, R.M.S., de Sanctis, L., Aversa, S., Mandolini, A. 2008. Kinematic response analysis of piled foundations under seismic excitation. Canadian Geotechnical Journal 46: 571 - 584
- Maiorano, R.M.S., Aversa, S., and Wu, G. 2007. Effects of soil non-linearity on bending moments in piles due to seismic kinematic interaction. Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering, Paper No. 1574, Thessaloniki, Greece.

Quasi-3D Method:

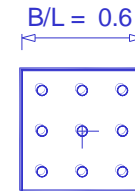
Research case history for Eurocode 8 (2003) and Italian code (2008)

PARAMETRIC STUDY

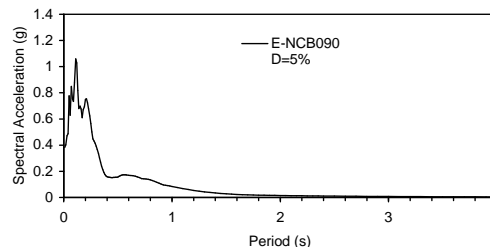
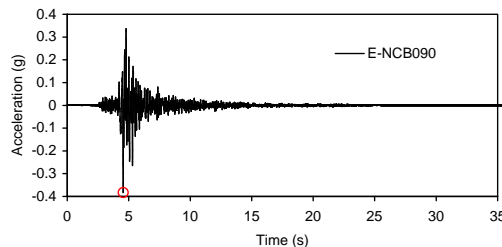
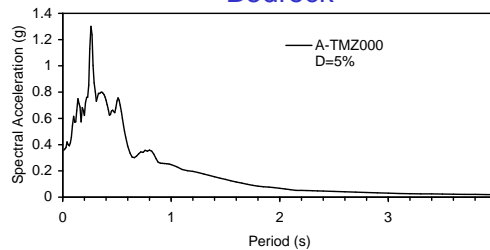
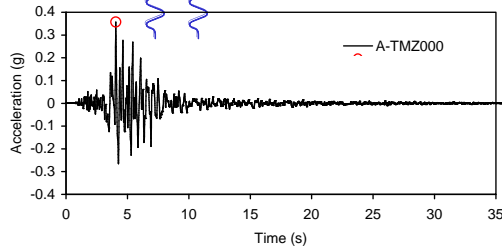
Kinematic interaction analysis performed for both single pile and pile group models. Interface



$n = 25$
 $L/d = 33.3$
 $s/d = 4$
 $H/L = 1.5$



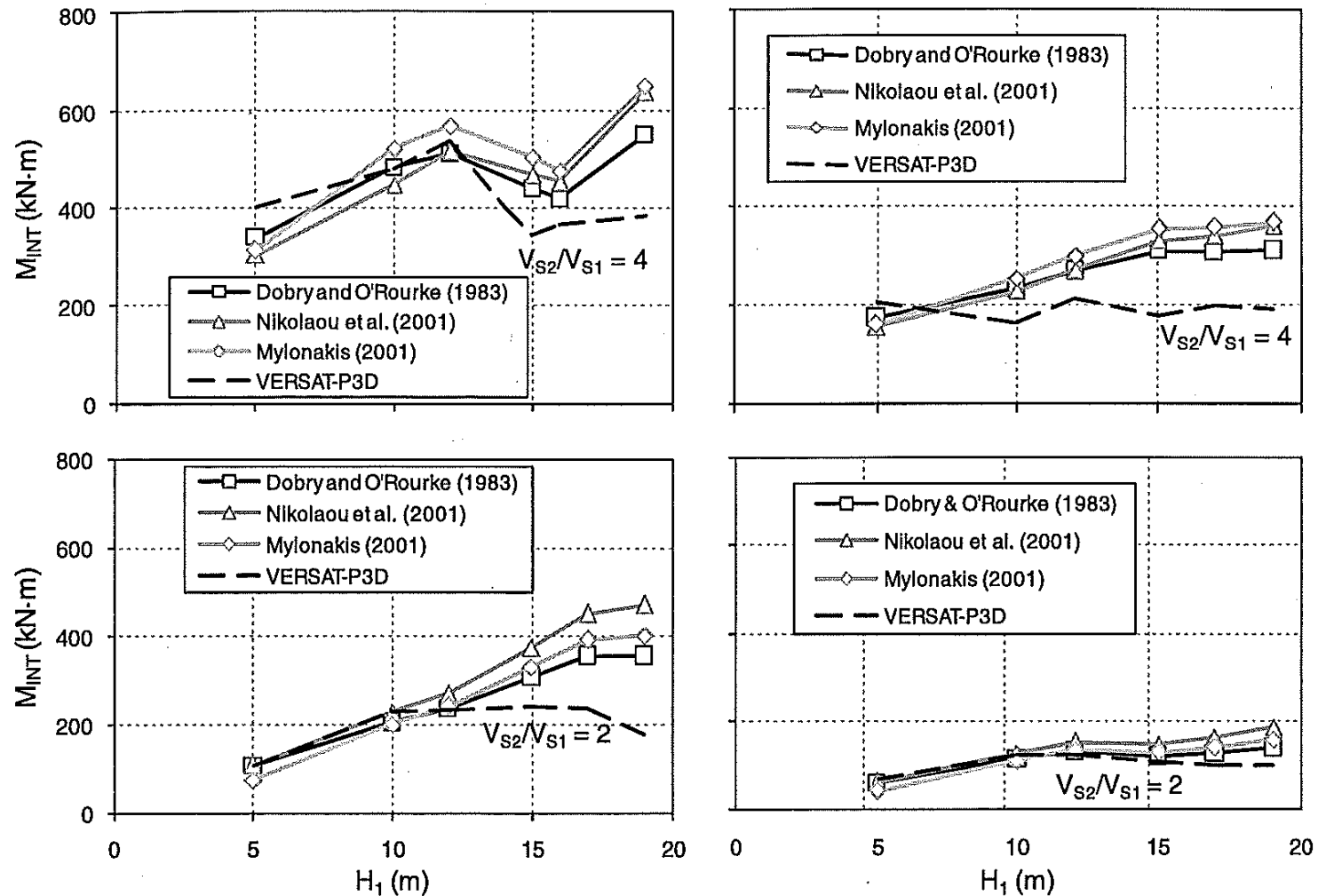
$n = 9$
 $L/d = 33.3$
 $s/d = 2.5, 4$
 $H/L = 1.5$



Quasi-3D Method:

Research case history for Eurocode 8 (2003) and Italian code (2008)

Bending moments of single piles (L=20 m) at the interface:

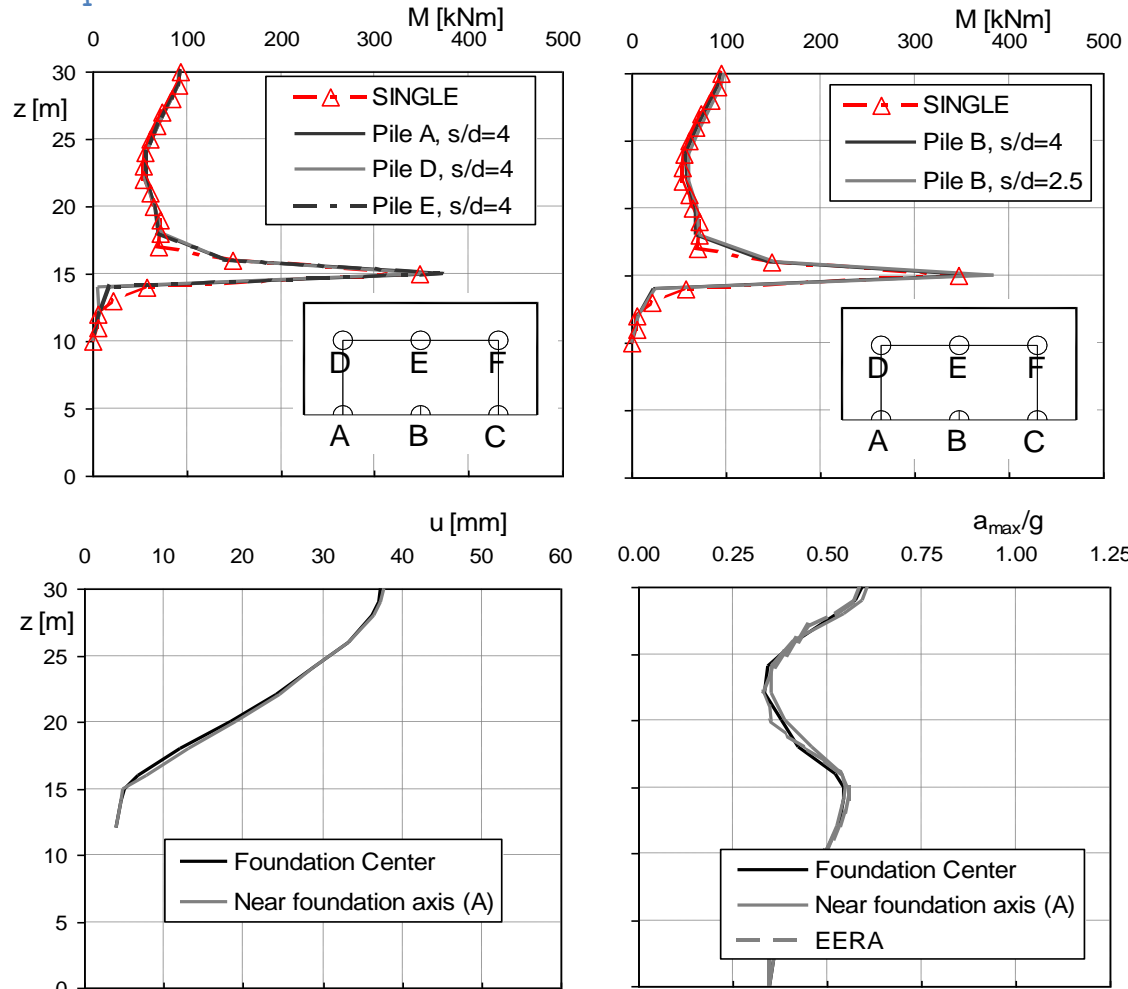


Quasi-3D Method:

Research case history for Eurocode 8 (2003) and Italian code (2008)

EXAMPLES OF RESULTS

A-TMZ000 earthquake, $V_{S2}/V_{S1}=4$, $V_{S1}=100\text{m/s}$, 3x3 pile group, and $H_1=15\text{m}$

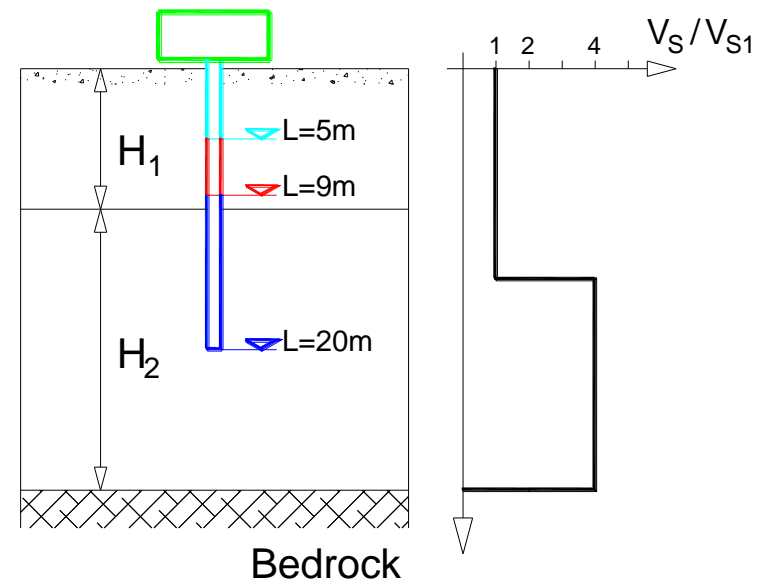
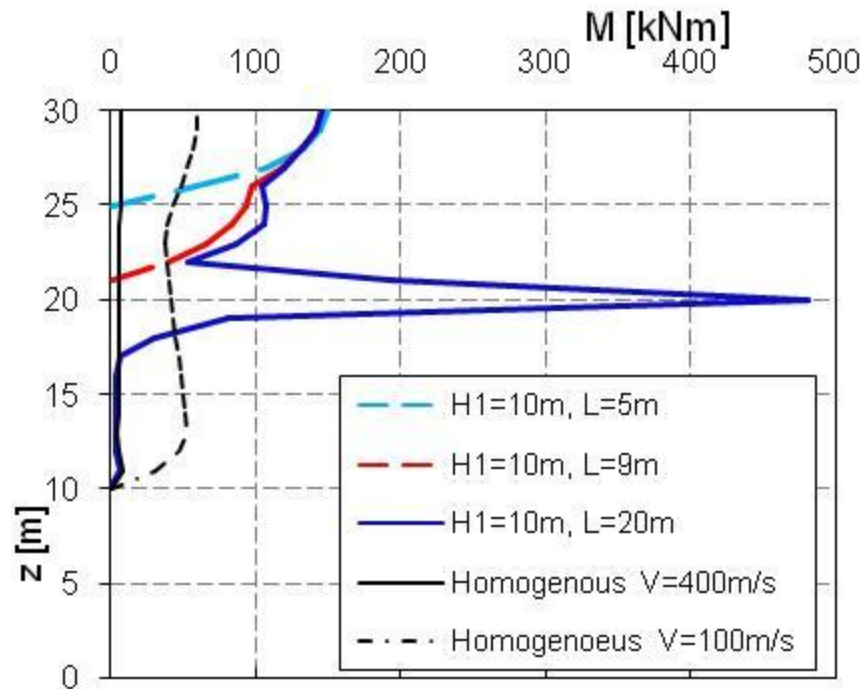


Quasi-3D Method:

Research case history for Eurocode 8 (2003) and Italian code (2008)

KINEMATIC BENDING EFFECT AT THE PILE HEAD

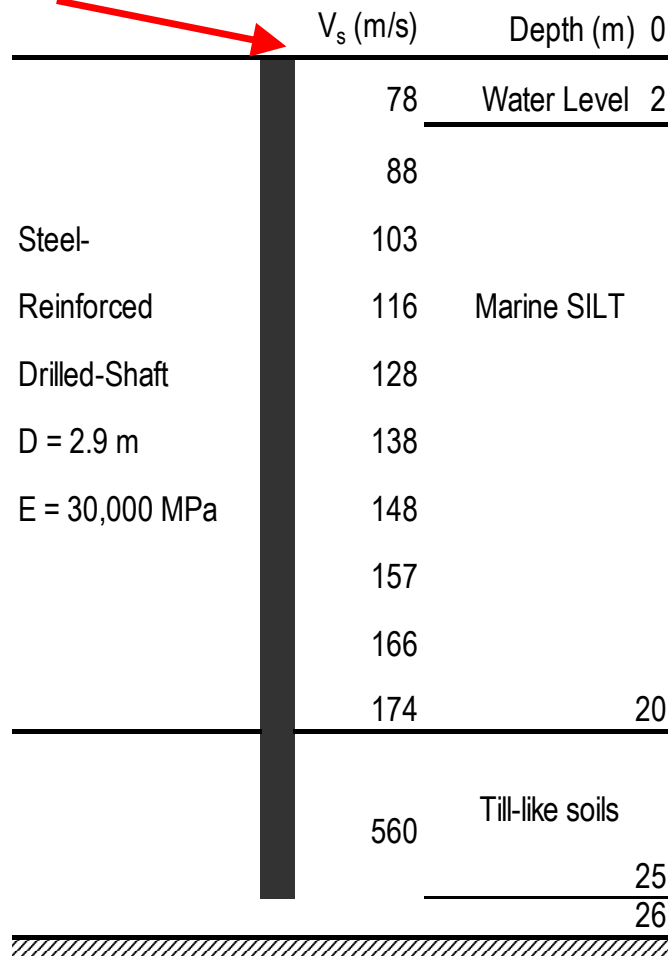
Influence of pile length on the kinematic bending moment at the pile head



Quasi-3D Method:

Kinematic bending of a large diameter cassion embedded in till-like soils

point of interest

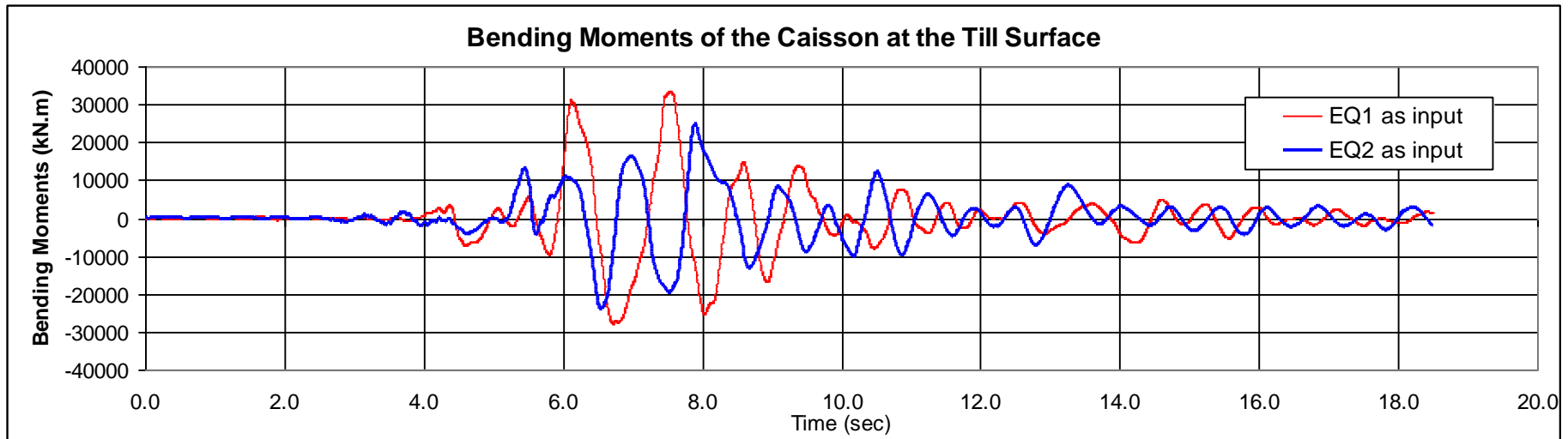


Quasi-3D Method:

Kinematic bending of a large diameter caisson embedded in till-like soils

VERSAT-P3D Results: Bending moments at the till surface

(source: 7th GeoChina Conference in Beijing)



Quasi-3D Method:

Summary Remarks

Quasi-3D Finite Element Method:

- A newly proposed method
- a more fundamental and straight forward approach in solving problems of pile-soil interactions, including kinematic effect
- This approach uses only the fundamental parameters of the pile-soil system such as the size and stiffness of the pile, and stiffness and strength of the surrounding soils.

Conventional beam-and-spring (p-y) method:

- Most commonly used method in current practice
- p-y curve and group effect multipliers are highly dependent on pile size, pile stiffness, soil nonlinearity, pile spacing, etc
- p-y curves and group effect multipliers largely empirically based – GI effects depend on strength of shaking and interaction via a nonlinear soil medium
- Kinematic effect included through complex interplay of structural pile stiffness, near field springs and far field soil response