ASSESSMENT OF BRIDGE DYNAMIC CHARACTERISTICS AND UNKNOWN FOUNDATIONS THROUGH LARGE-AMPLITUDE SHAKING

By

SHAREF FARRAG

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ABSTRACT OF THE DISSERTATION

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By Sharef Farrag

Dissertation Director:
Nenad Gucunski

A fixed-base is assumed in various dynamic response analyses and the design of bridges. However, soil-foundation flexibility and energy absorption and radiation by the soil system can alter the response of bridges to dynamic loads. This interaction between the structure, foundation, and soil, which in some cases may even change the dynamic load transmitted through the ground, is, in general, referred to as dynamic soil-structure interaction (DSSI). DSSI can either have detrimental or beneficial effects on a bridge response, particularly forces and displacements. In this research, the dynamic characteristics of actual bridges are inferred via an experimental program and numerical simulations. The research concentrated on the evaluation of the significance of DSSI effects under operational live load levels and exploration of unknown foundations. The bridge was shaken using T-Rex, a large-amplitude mobile shaker from the National Hazards Engineering Research Infrastructure (NHERI) facilities. Studies implementing Finite Element Modeling to develop time histories and eigenmodes were conducted in a forward-modeling problem
setup. Two models were created to assess the DSSI effects on the dynamic response of the bridge. One model included elements that incorporate DSSI effects, while the other had fixed-base boundary conditions. The response from the DSSI FEM model matched the field results better than the fixed-base model, in terms of the peak response amplitudes and identified natural frequencies and modes. The model incorporating DSSI effects led to a reduction in stress levels compared to the fixed-base model. The characterization of unknown bridge foundations by the means of dynamic testing for their potential reuse based on the estimation of their ultimate bearing capacity as a limit state was also studied. The applicability of using large-amplitude mobile shakers in tandem with finite element models for sub-structural identification is examined. By exploiting dynamic features of structures, such as dynamic amplification and knowledge of eigen-modes, significantly lower magnitudes of load can be implemented diagnostically as a global nondestructive evaluation (NDE) technique, while having notable response levels. This is accomplished by i) examining material properties extracted from dynamic testing of the foundation/soil system to provide empirical estimates of bearing capacity, or ii) evaluating the global dynamic response of the bridge-foundation system and relating it to failure mechanisms. The methodology and results from field testing and numerical simulations for foundation exploration are presented.
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# Table of Contents

ABSTRACT OF THE DISSERTATION ............................................................................. ii

Acknowledgements ............................................................................................... iv

Table of Contents ................................................................................................. v

List of Tables ......................................................................................................... viii

List of Figures ....................................................................................................... x

Acronyms ............................................................................................................... xxvi

CHAPTER 1: INTRODUCTION .............................................................................. 1

1.1. Structural Identification (St-Id) of Bridges .................................................. 1

1.2. Dynamic Soil-Structure Interaction ............................................................. 2

1.3. Reuse of Bridge Foundations ....................................................................... 3

1.4. Research Objectives .................................................................................... 4

1.5. Dissertation Overview .................................................................................. 7

CHAPTER 2: BACKGROUND INFORMATION ................................................. 10

2.1. Overview of Soil-Structure-Foundation Systems ....................................... 10

2.2. Factors Influencing Soil-Structure Interaction Effects .............................. 19

2.3. Dynamic Testing for Structural Identification ........................................... 24

2.4. T-Rex as a Mobile Shaker for Dynamic Bridge Testing ............................ 30

2.5. Overarching Research Aims ...................................................................... 35

CHAPTER 3: RESEARCH METHODOLOGY .................................................. 38
3.1 Experimental Program ........................................................................................................ 38
  3.1.1. Hobson Avenue Bridge, New Jersey ................................................................. 38
  3.1.2. Gate Creek Bridge, Oregon .................................................................................. 46
3.2. Numerical Simulation ......................................................................................................... 60
  3.2.1. Hobson Avenue Bridge, New Jersey .................................................................. 60
  3.2.2. Gate Creek Bridge, Oregon ................................................................................ 70

CHAPTER 4: RESULTS FROM THE EXPERIMENTAL PROGRAM .................. 75
  4.1 Bridges Response to T-Rex Shaking ............................................................................. 75
     4.1. Hobson Avenue Bridge, New Jersey ................................................................. 76
     4.2. Gate Creek Bridge, Oregon ................................................................................ 93
     4.3. T-Rex as a Global Evaluation NDE Tool ......................................................... 100

CHAPTER 5: NUMERICAL SIMULATIONS AND PARAMETRIC STUDIES OF
HOBSON AVENUE BRIDGE ......................................................................................... 106
  5.1 Numerical Simulation of Hobson Avenue Bridge and Model Validation .......... 106
     5.1.1. Eigenfrequency Analysis ................................................................................ 107
     5.1.2. Model Validation ............................................................................................ 112
     5.1.3. Analysis of Stresses ....................................................................................... 116
     5.2. Parametric Study .................................................................................................. 122
     5.3. Effect of Superstructure Rigidity on Structural Response ................................. 132
CHAPTER 6: EXPLORATION OF UNKNOWN FOUNDATIONS AND
ESTIMATION OF BEARING CAPACITY .................................................. 137

6.1. Approach ................................................................................................. 137

6.2. Exploration of Parameters Affecting the Response of Unknown Foundations 141

6.3. Estimation of Ultimate Bearing Capacity .................................................. 153

6.3.1. Determination of Bearing Capacity Using Empirical Relationships Based on
General Shear Failure .................................................................................. 154

6.3.2. Determination of Bearing Capacity Based on Serviceability Limits....... 156

CHAPTER 7: CONCLUSIONS AND FUTURE RESEARCH ......................... 160

REFERENCES ................................................................................................. 164

APPENDIX A: Supplementary Information for Preliminary Site Investigations...... 173

APPENDIX B: 2D FEM Model for Incorporating DSSI Effects ....................... 179

APPENDIX C: Hobson Avenue Bridge Response Evaluation for Determining Resonant
Frequencies ......................................................................................... 184

APPENDIX D: Supplementary Information from Numerical Simulations ............ 222
List of Tables

Table 2-1. Sample of Studies on Evaluation of DSSI................................................................. 26
Table 3-1. Description of T-Rex Test Runs on Hobson Avenue Bridge, NJ.................. 42
Table 3-2. Summary of T-Rex Shake Runs in the Experimental Program.................... 55
Table 3-3. Summary of SASW Runs Carried out to Determine Shear Wave Velocity.... 57
Table 3-4. Geometric and Material Parameters of the Foundation Included in the FE Models.................................................................................................................................................................................. 64
Table 3-5. Static Stiffness for Different Vibration Modes Included in the FE Models (Gazetas, 1991)........................................................................................................................................................................................................... 65
Table 3-6. Dynamic Radiation Damping for Different Vibration Modes Included in the FE Models (Gazetas, 1991)........................................................................................................................................................................................................... 66
Table 3-7. Summary of Parameters Included in the Numerical Simulation of Tested Bridge. ........................................................................................................................................................................................................... 72
Table 4-1. Summary of Response from Highest Load Level Scenarios in Various Configurations........................................................................................................................................................................................................... 81
Table 4-2. Summary of the EMA and Modal Parameters Obtained from Shaking Hobson Avenue Bridge. ........................................................................................................................................................................................................... 85
Table 5-1. Eigenfrequencies Obtained from Numerical Simulations of DSSI-incorporating and Fixed-base Models. ........................................................................................................................................................................................................... 107
Table 5-2. Participation Factors as Obtained from the DSSI-Incorporating Model for Translational Modes........................................................................................................................................................................................................... 109
Table 5-3. Calculation of MPF of each mode for Translation along X ......................... 110
Table 5-4. Participation Factors as Obtained from the DSSI-Incorporating Model for Rotational Modes. .......................................................................................................................... 111

Table 5-5. Calculation of MPF of each Mode for Rotation around the Y-axis (Rocking). ........................................................................................................................................ 111

Table 5-6. Summary of MPFs of each Mode for Translational and Rotational DOFs... 112

Table 5-7. Transverse Response due to Transverse Loading at Frequencies of Interest from Time Trace of Frequency. .................................................................................................. 114

Table 6-1. Summary of Parameters Swept in Numerical Models for Footing Identification ........................................................................................................................................... 142

Table 6-2. Comparison Between Estimated and Actual Footing Dimensions.................. 149

Table B-1. Parameters Included in the 2D FEM Model .................................................. 182

Table D-1. Calculation of MPF of each Mode for Translation along Y. ...................... 224

Table D-2. Calculation of MPF of each Mode for Translation along Z. ...................... 224

Table D-3. Calculation of MPF of each Mode for Rotation about the X-axis............ 224

Table D-4. Calculation of MPF of each Mode for Rotation about the Y-axis............ 224
List of Figures

Fig. 2-1. Typical finite element model components included in the direct approach, adopted from (NIST, 2012). .............................................................. 12

Fig. 2-2. Description of the kinematic and inertial effects in rigid and flexible foundations in the Sub-structuring method, adopted from (NIST, 2012). ................................................. 14

Fig. 2-3. Rigid foundation resting on an infinite homogeneous viscoelastic half-space under dynamic loads, adopted from (Gazetas, 1991). .............................................................. 15

Fig. 2-4. Typical cross-section at notional bridge piers, showing different components of a structure-foundation soil system. .............................................................. 17

Fig. 2-5. Description of soil-structure-soundation system in notional bridges at the pier section. ........................................................................................................... 17

Fig. 2-6. Coherence function obtained from tested bridge under operational traffic, adapted from (Davis and Sanayei, 2020). .............................................................. 29

Fig. 2-7. Comparison between conventional St-Id techniques with the proposed use of NHERI shakers. .......................................................................................... 32

Fig. 2-8. A photograph of T-Rex. ........................................................................... 33

Fig. 2-9. Theoretical force outputs of T-Rex. .......................................................... 33

Fig. 2-10. A closed photograph of the T-Rex. ......................................................... 34

Fig. 3-1. Hobson Avenue Bridge, Hamilton Township, New Jersey. ....................... 38

Fig. 3-2. (a) Deck dimensions, and (b) substructure dimensions of Hobson Avenue Bridge. ................................................................................................................. 39

Fig. 3-3. (a) T-Rex, and (b) control room to monitor response in real-time on Hobson Avenue Bridge. ............................................................................................ 40
Fig. 3-4. A reverse linear chirp signal used to excite the bridge from 15 Hz to 1 Hz ...... 40

Fig. 3-5. (a) Geophone arrays on the ground for MASW, (b) geophone and accelerometer on the deck, (c) and on the pier ........................................................................................................................................... 43

Fig. 3-6. (a) Overall sensor layout and T-rex positions and (b) geophone channel numbers (from Google Earth, 2017).................................................................................................................................................. 44

Fig. 3-7. Shear wave velocity profile of the test site back-calculated from the MASW test data .................................................................................................................................................................................. 46

Fig. 3-8. (a) Bridge location and (b) old and new parts of the bridge with different superstructures. ............................................................................................................................................. 47

Fig. 3-9. Typical geophone used for measuring response. ........................................................................................................................................... 48

Fig. 3-10. Geophone attached to the pier to measure response. ........................................................................................................................................... 49

Fig. 3-11. Overall sensor layout for the tested pier ............................................................................................................................................. 50

Fig. 3-12. Overall sensor layout for the tested pier ............................................................................................................................................. 51

Fig. 3-13. Overall sensor layout for the tested pier ............................................................................................................................................. 52

Fig. 3-14. T-Rex and control room on the bridge ready for shaking. ............................................................................................................................................. 53

Fig. 3-15. Left: operator monitoring response, and right: waveform generator controlling T-Rex shaking ............................................................................................................................................. 54

Fig. 3-16. Left: setting up geophones and adjusting spacing, right: generating acoustic surface wave ............................................................................................................................................. 58

Fig. 3-17. Phase shift diagrams of two trials with indicated spacing, forward measurement. ............................................................................................................................................. 59

Fig. 3-18. Experimental shear wave velocity profile of soil at tested bridge. ............................................................................................................................................. 59

Fig. 3-19. Dynamic stiffness and damping coefficients for various vibration modes. ..... 63
Fig. 3-20. Perspective view of the Hobson Avenue Bridge FE model. .......................... 67
Fig. 3-21. Overall 3D, side, and bottom view of the bridge model mesh. ...................... 68
Fig. 3-22. Bridge overview in 3D model. ........................................................................ 70
Fig. 3-23. Profile views indicating the embedded footing. ............................................ 71
Fig. 3-24. Intensity of gravity loads in the FEM model. .................................................. 73
Fig. 3-25. View of the Creek Gate Bridge model mesh. .................................................. 73
Fig. 4-1. Transverse response time histories from sensor locations mentioned in the legend to transverse shaking at 12 kips with T-Rex centered above the pier. ............... 77
Fig. 4-2. Transverse power spectra from sensor locations mentioned in the legend to transverse shaking at 12 kips with T-Rex centered above the pier. ........................... 77
Fig. 4-3. Average transverse response of several observation points due to horizontal shaking @21 k, with ground/foundation geophone signal amplified. ......................... 77
Fig. 4-4. (a) Transverse response due to transverse loading above the pier, (b) vertical response due to vertical loading, and (c) vertical power spectra due to transverse loading. (1 cm/s = 0.39 in/s) ................................................................. 79
Fig. 4-5. Stabilization diagram for FRFs from lateral response ................................. 84
Fig. 4-6. Stabilization diagram for FRFs from the vertical response ........................... 84
Fig. 4-7. Testing setup for determination of ground damping and attenuation. ........... 86
Fig. 4-8. Ground response to T-Rex shaking atop the bridge pier with as function of frequency and distance away from the source. ......................................................... 88
Fig. 4-9. Longitudinal cross-section of Fig. 4-8 at 7.5 Hz. ........................................ 88
Fig. 4-10. Determination of soil attenuation coefficient $\alpha$. ................................. 89
Fig. 4-11. $\zeta_g$ as estimated from the attenuation coefficient $\alpha$. ............................... 90
Fig. 4-12. Mechanical impedance of Hobson Avenue Bridge for the rocking mode, consisting of (a) real part (stiffness) and (b) imaginary (damping)................................. 91
Fig. 4-13. T-Rex loading one of the tested piers of the bridge........................................... 93
Fig. 4-14. Bridge response to chirp loading under various load levels at (a) the footing and (b) the deck level....................................................................................... 95
Fig. 4-15. Response of footing and deck to a 36 k chirp loading function (80-10 Hz) .... 95
Fig. 4-16. Response of footing and deck to a 36 k chirp loading function (15-3 Hz) ...... 97
Fig. 4-17. Power spectra of response due to a chirp loading function 10-80 Hz at (a) deck and (b) footing........................................................................................................ 97
Fig. 4-18. Power spectra of response due to a chirp loading function 3-15 Hz at (a) deck and (b) footing........................................................................................................ 97
Fig. 4-19. Footing response to steady state shaking at 26 Hz............................................. 98
Fig. 4-20. Mechanical impedance of Gate Creek Bridge for the vertical mode, consisting of (a) real part (stiffness) and (b) imaginary (damping) for the 3-15 Hz range, and of (c) real part (stiffness) and (c) imaginary (damping) for the 10-80 Hz range....................... 99
Fig. 4-21. Transfer functions and phase angles of vertical response between east and west side of the deck above the pier due to vertical load @ (a) 13.3 kN [3 k]. (b) 26.7 [6 k]. (c) 40 kN [9 k]. (d) 53.4 kN [12 k]................................................................................................................. 101
Fig. 4-22. Coherence functions of transverse response to transverse shaking with T-Rex the above pier at the (a) deck, (b) pier, (c) midspan, and (d) foundation. ....................... 103
Fig. 4-23. Coherence functions of vertical response to vertical shaking with T-Rex the above pier at the (a) deck, (b) pier, (c) midspan, and (d) foundation. ....................... 104
Fig. 5-1. Modes shapes obtained from FEM of DSSI-incorporating and fixed-base models.
.............................................................................................................................................. 108

Fig. 5-2. Time history of east geophone from the test, fixed-base model, and DSSI model
due to 93.4 kN (21 k) transverse load .................................................................................... 113

Fig. 5-3. Lateral displacement response (Fourier spectrum) to lateral shaking @93.4 kN
(21 k) .......................................................................................................................................... 115

Fig. 5-4. Results from frequency-domain models compared to experimental results for the
lateral response of the deck to lateral shaking at 21 k (93.4 kN) ............................................. 116

Fig. 5-5. Results from frequency-domain models compared to experimental results for the
vertical response of the midspan to lateral shaking at 21 k (93.4 kN) ........................................ 116

Fig. 5-6. Time history of maximum shear stress in the deck near the top rebar level in both
models due to 93.4 kN load ........................................................................................................... 117

Fig. 5-7. Peak shear stresses from both models due to a 93.4 kN transverse load at the
resonant frequency ........................................................................................................................... 118

Fig. 5-8. Shear stress distribution (in MPa) from the (a) DSSI-incorporating model and (b)
fixed-base model ............................................................................................................................... 119

Fig. 5-9. von Mises stress comparison between the fixed-base and DSSI-incorporating
model due to lateral shaking at 93.4 (kN), at the (a) deck, (b) bent, (c) girders, (d) columns,
(e) braces, (f) footing-column connection, (g) column-bent connection, and (h) girder-bent
cap connection. ............................................................................................................................. 120

Fig. 5-10. Comparison of von Mises stress at the top and bottom surfaces of the deck
between the DSSI-incorporating and fixed-base model ............................................................. 121

Fig. 5-11. DAF of deck lateral displacement due to lateral shaking from both models. 122
Fig. 5-12. Effect of varying $H_{tot}$ on lateral deck response to lateral shaking. (Shown for $V_s = 200$ m/s, $B = 5.7$ m, $D_f = 2$ m). [1 in/s = 2.54 cm/s]. ......................................................... 123

Fig. 5-13. Effect of varying $V_s$ (m/s) on lateral deck response to lateral shaking. (Shown for $H_{tot} = 5$ m, $B = 5.7$ m, $D_f = 2$ m). [1 in/s = 2.54 cm/s]. ......................................................... 124

Fig. 5-14. Effect of varying $D_f$ (m) on lateral deck response to lateral shaking. (Shown for $H_{tot} = 5$ m, $B = 5.7$ m, $V_s = 200$ m/s). [1 in/s = 2.54 cm/s]. ......................................................... 125

Fig. 5-15. Effect of varying $D_f$ (m) on lateral footing response to lateral shaking. (Shown for $H_{tot} = 5$ m, $B = 5.7$ m, $V_s = 200$ m/s). [1 in/s = 2.54 cm/s]. ......................................................... 126

Fig. 5-16. Effect of varying $B$ (m) on lateral footing response to lateral shaking. (Shown for $H_{tot} = 5$ m, $D_f = 2$ m, $V_s = 200$ m/s). [1 in/s = 2.54 cm/s]. ......................................................... 126

Fig. 5-17. The extent of $(D/B)$ ratio on altering the DSSI effect based on (a) peak amplitude and (b) peak frequency, with other parameters held constant. ........................................ 128

Fig. 5-18. Effect of varying $h$ on altering DSSI effects relative to a fixed base based on the peak lateral amplitude ratio of the deck due to lateral shaking. Shown for $V_s = 200$ m/s, $B = 5.7$ m, and $D_f = 2$ m. ......................................................... 129

Fig. 5-19. Extent of $h$ on structural softening due to DSSI effects relative to a fixed base. Shown for $V_s = 200$ m/s, $B = 5.7$ m, and $D_f = 2$ m. ......................................................... 130

Fig. 5-20. Extent of $s$ alteration of lateral peak amplitude caused by lateral shaking due to DSSI effects relative to a fixed base. Shown for $B = 5.7$ m, $D_f = 2$ m, and varying $h_{tot}$ and $V_s$. ......................................................... 131

Fig. 5-21. Effect of varying $s$ on altering DSSI effects relative to a fixed-base based on the peak lateral amplitude ratio of the deck due to lateral shaking. Shown for $B = 5.7$ m, $D_f = 2$ m, and varying $V_s$ and $h_{tot}$. ......................................................... 131
Fig. 5-22. First bending mode of the Hobson Avenue Bridge for the existing (left) and hypothetical stiffer girders (right). ................................................................. 133

Fig. 5-23. Dynamic vertical displacement amplitudes of the Hobson Avenue Bridge under harmonic loading at 3.1 Hz for the existing (left) and 5.1 Hz for the hypothetical stiffer girders (right) ................................................................. 133

Fig. 5-24. Longitudinal normal (top) and Von Mises stress (bottom) at the top of the deck of the Hobson Avenue Bridge under harmonic loading at 3.1 Hz for the existing (left) and 5.1 Hz for the hypothetical stiffer girders (right) ................................................................. 134

Fig. 5-25. Longitudinal normal at the top of the deck of the Hobson Avenue Bridge under harmonic loading (97 kN) at 3.1 Hz for the existing (left) and 5.1 Hz for the hypothetical stiffer girders (right) ................................................................. 135

Fig. 6-1. Flowchart describing the process of exploring unknown foundations ........ 140

Fig. 6-2. Force input used in the numerical model .................................................. 142

Fig. 6-3. Deformed shape of the bridge during one time instant of shaking. .......... 143

Fig. 6-4. Effect of footing depth (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $V_s = 400$ m/s, $2B=2.75$ m, and $2L = 4.5$ m. [1 in/s = 2.54 cm/s] ................................................................. 143

Fig. 6-5. Error estimates for the results of varying footing depth shown in Fig. 6-4 on peak amplitude and time history fit ................................................................. 144

Fig. 6-6. Effect of footing width (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $V_s = 200$ m/s, $D_f=2.4$ m, and $2L = 4.5$ m ................................................................. 145
Fig. 6-7. Error estimate for the results of varying footing width shown in Fig. 6-6 on peak amplitude and time history fit ................................................................. 145

Fig. 6-8. Effect of footing length (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $V_s = 200 \text{ m/s}, D_t = 2.4 \text{ m},$ and $2B = 2.75 \text{ m}$................................................................. 146

Fig. 6-9. Error estimate for the results of varying footing length shown in Fig. 6-8 on peak amplitude and time history fit ................................................................. 146

Fig. 6-10. Effect of shear wave velocity (in m/s) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $2L = 4.5 \text{ m}, D_t = 2.4 \text{ m},$ and $2B = 2.75 \text{ m}$ ................................................................. 147

Fig. 6-11. Error estimate for the results of varying shear wave velocity shown in Fig. 6-10 on peak amplitude and time history fit ................................................................. 148

Fig. 6-12. The best-match model obtained from the parametric sweep of several footing geometries and soil properties, representing vertical footing response to a 24 k steady-state vertical force. Shown for $V_s = 425 \text{ m/s}, 2L = 4.75 \text{ m}, 2B = 2.67 \text{ m},$ and $D_t = 2.75 \text{ m}$... 148

Fig. 6-13. Steady-state displacement obtained by integration of velocity @ 26 Hz........ 150

Fig. 6-14. Steady-state (a) force and (b) displacement at the geophone near footing due to T-Rex shaking.................................................. 151

Fig. 6-15. Experimental impedance curve resulting from dynamic testing .......... 152

Fig. 6-16. Typical normalized soil modulus degradation and material damping curves (Darendeli, 2001). .................................................................................. 157

Fig. 6-17. Bearing pressure-settlement plot from FEM simulation of the tested bridge pier. ........................................................................................................ 158
Fig. A-1. Site layout documenting the location of single station horizontal-to-vertical (H/V) spectral ratio noise measurements made in the free-field (i.e., well-separated from the structural system) and the 23-m long Multichannel Analysis of Surface Waves (MASW) linear array. Each single station measurement location is denoted relative to the Hobson Avenue Bridge over Interstate 195 (i.e., NW, NE, SW, and SE).

Fig. A-2. Shown for each inversion parameterization are the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southeast (SE) station location; (c) Vs profiles constrained solely by the experimental dispersion data; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\ln V_s}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].

Fig. A-3. Shown for each inversion parameterization are the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southeast (SE) station location; (c) Vs constrained by the experimental dispersion and the H/V data from the SE station location; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\ln V_s}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].
Fig. A-4. Shown for each inversion parameterization is the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southwest (SW) station location; (c) Vs constrained by the experimental dispersion and the H/V data from the SW station location; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\ln Vs}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].

Fig. B-1. Dynamic swaying and rocking coefficients of a surface rectangular footing.

Fig. B-2. 2D model representing typical bridge pier frames.

Fig. B-3. Displacement [m] from 2D FEM model at variable rigidity (R, Vs); a) [2,200], b) [3,300], c) [4, 400], d) [fixed base].

Fig. B-4. Rotation [rad] from 2D FEM model at variable rigidity (R, Vs); a) [2,200], b) [3,300], c) [4, 400], d) [fixed base].

Fig. C-1. T-Rex centered above the pier to shake the bridge transversely and corresponding transverse response in Fig. C-2 to Fig. C-6.

Fig. C-2. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.

Fig. C-3. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum,
(c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them. .......................................................... 188

Fig. C-4. Transverse response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them. .................................................................................................. 189

Fig. C-5. Transverse response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them. .................................................................................................. 190

Fig. C-6. Transverse response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them. .................................................................................................. 191

Fig. C-7. T-Rex centered above the pier to shake the bridge transversely and corresponding vertical response in Fig. C-8 to Fig. C-12 .................................................................................................. 192

Fig. C-8. Vertical response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them. .................................................................................................. 193

Fig. C-9. Vertical response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum,
(c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them. ................................................................. 194

Fig. C-10. Vertical response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them. ........................................................................................................................................ 195

Fig. C-11. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them. ........................................................................................................................................ 196

Fig. C-12. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them. ........................................................................................................................................ 197

Fig. C-13. T-Rex centered above the pier to shake the bridge vertically and the corresponding vertical response in Fig. C-14 to Fig. C-18................................................................. 198

Fig. C-14. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them. ........................................................................................................................................ 199

Fig. C-15. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum,
(c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them. ................................................................. 200

Fig. C-16. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them. ................................................................. 201

Fig. C-17. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the ground sensor at the same side, and (d) the phase shift between them. ................................................................. 202

Fig. C-18. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the ground sensor at the other side, and (d) the phase shift between them. ................................................................. 203

Fig. C-19. T-Rex centered above the mid-span to shake the bridge transversely and corresponding transverse response in Fig. C-20 to Fig. C-24................................. 204

Fig. C-20. Transverse response to transverse shaking @15 k with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them. ......................................................................................... 205

Fig. C-21. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power
Fig. C-22. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them. 

Fig. C-23. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.

Fig. C-24. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them.

Fig. C-25. T-Rex centered above the mid-span to shake the bridge transversely and corresponding vertical response in Fig. C-26 to Fig. C-30.

Fig. C-26. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.

Fig. C-27. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the mid-span, and (d) the phase shift between them.
spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them. ................................................................. 212

Fig. C-28. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them. ........................................................................................................... 213

Fig. C-29. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them. ........................................................................................................... 214

Fig. C-30. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them. ........................................................................................................... 215

Fig. C-31. T-Rex centered above the mid-span to shake the bridge vertically and corresponding vertical response in Fig. C-32 to Fig. C-36. ................................................................. 216

Fig. C-32. Vertical response to vertical shaking @10.8 kips with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with other side of the deck, and (d) the phase shift between them. ........................................................................................................... 217

Fig. C-33. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum,
(c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them. .................................................................................................................................................................................. 218

Fig. C-34. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them. .................................................................................................................................................................................. 219

Fig. C-35. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them. .................................................................................................................................................................................. 220

Fig. C-36. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them. .................................................................................................................................................................................. 221
## Acronyms

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEM</td>
<td>Boundary Element Model</td>
</tr>
<tr>
<td>DOF</td>
<td>Degree of Freedom</td>
</tr>
<tr>
<td>DSSI</td>
<td>Dynamic Soil-Structure Interaction</td>
</tr>
<tr>
<td>EMA</td>
<td>Experimental Modal Analysis</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Model</td>
</tr>
<tr>
<td>FFT</td>
<td>Fast Fourier Transform</td>
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<tr>
<td>FIM</td>
<td>Foundation Input Motion</td>
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<tr>
<td>MASW</td>
<td>Multi-Channel Analysis of Surface Waves</td>
</tr>
<tr>
<td>MCF</td>
<td>Modal Complexity Factor</td>
</tr>
<tr>
<td>NDE</td>
<td>Non-Destructive Evaluation</td>
</tr>
<tr>
<td>NHERI</td>
<td>Natural Hazards Engineering Research Infrastructure</td>
</tr>
<tr>
<td>FRF</td>
<td>Frequency Response Function</td>
</tr>
<tr>
<td>SASW</td>
<td>Single-Channel Analysis of Surface Waves</td>
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<tr>
<td>SFS</td>
<td>Soil-Foundation-Structure</td>
</tr>
<tr>
<td>St-Id</td>
<td>Structural Identification</td>
</tr>
<tr>
<td>TF</td>
<td>Transfer Function</td>
</tr>
</tbody>
</table>
CHAPTER 1: INTRODUCTION

In recognition of the safety risks of the aging transportation infrastructure systems under natural and anthropogenic hazards, there has been significant attention paid to the development of reliable safety assessment approaches to support their management, adaptation, and reuse. The development of more robust structural health monitoring (SHM) and nondestructive evaluation (NDE) techniques is crucial for the preservation of infrastructure, and the assessment of its condition/performance. The introduction of innovative, cost-effective, and safe approaches is required to quantitatively evaluate the condition/performance of existing infrastructure systems and to provide a basis for the design of future infrastructure. Dynamic testing and behavior of bridges is introduced in this chapter.

1.1. Structural Identification (St-Id) of Bridges

Many of the existing bridge testing approaches are overly complex, time-consuming, and expensive, and the level of certainty of obtained results in some cases is insufficient. Structural Identification (St-Id), a subset of SHM, is an emerging field that employs a wide variety of state-of-the-art sensing technologies. It involves various SHM methods, such as static/quasi-static and vibration-based testing. Static/quasi-static SHM involves assessing the static response (displacement and rotations) of structures, namely bridges. However, considerable load levels are required to generate a noticeable response. On the other hand, a vibration based SHM can help capture global characteristics related to the load carrying/transfer from the super- and substructure of bridges onto the foundation and surrounding soil. Vibration-based testing can be more beneficial as more information regarding the condition of foundations can be extracted as well. In addition, highly-refined Finite Element
Models (FEM) can be developed and utilized in tandem with experimental results to better understand the dynamic behavior of bridges, and/or to confirm the design assumptions/parameters and expected behavior (Catbas and Kijewski-Correa, 2013). This synergistic application of model- and vibration-based St-Id promotes SHM as a tool to assess actual response to dynamic loads and behavior rather than to detect damage in various parts of a bridge locally.

1.2. Dynamic Soil-Structure Interaction

Boundary conditions of a structure affect its response to dynamic excitations. In most highway bridge designs, the dynamic soil-structure interaction is not considered, with an underlying assumption that bridge piers have fixed ends. Foundation flexibility, and more importantly radiation damping from the foundation, whether it is a shallow or deep foundation, can significantly influence the response of the substructure/superstructure system. This may lead to deviations of the actual response compared to the design assumptions, depending on soil properties and geometrical and structural characteristics of the bridge. Namely, dynamic soil-structure interaction (DSSI) can significantly alter the dynamic response. The effect of DSSI on the response depends on several factors such as the rigidity ratio (ratio of the stiffness of the structure to the same of the soil-foundation system), slenderness ratio (height of the structure to the base width ratio), type of the foundation, and the mass of the structure relative to the mass of the engaged soil-foundation system. Therefore, developing dynamic testing methods that can capture DSSI effects is required, which necessitates the efforts in the current research. This involves the use of large-amplitude shakers for dynamic testing of bridges.
1.3. Reuse of Bridge Foundations

Foundations of existing bridges can be considered as assets of substantial operational/economic value, which entices their reuse in the rehabilitation/reconstruction of bridges. Subsequently, a direct benefit of quantifying the effects of DSSI and exploitation of dynamic testing using large-amplitude mobile shakers is the potential to use this method for the reusability of bridge foundations. Numerous studies are focusing on the local characterization of foundation integrity parameters, such as the evaluation of concrete strength of footings or resistivity measurements for corrosion potential (Agrawal et al., 2018). Moreover, other studies involving dynamic testing were geared towards damage identification in the superstructure. Nevertheless, there is a lack of studies on evaluating the capacity of the existing bridge substructure by the means of dynamic testing. It shows that dynamic testing can be used to assess damage in sub/superstructure elements of bridges. This is done by evaluating the loss of vertical stiffness of foundations due to scour or earthquake damage. Furthermore, the evaluation of existing bridge foundations can vary in complexity based on the available information about the substructure. Hence, there is a variety of scenarios that can be encountered with different bridges, including the lack of as-built drawings or geotechnical site exploration/boring logs. This is the motivation of the proposed approach, which enables a fully nondestructive and rapid assessment of bridges and their components using large mobile shakers as a dynamic excitation source and fast deployable sensor arrays to capture the response. Combined with Structural Identification (St-Id), the proposed evaluations provide information about the global bridge condition, assess the need for system adaption, and in the case of a superstructure and/or substructure replacement, assess the feasibility of the foundation reuse. The ultimate bearing capacity
can be used to judge the substructure reusability, based on the response to dynamic shaking. FEM models calibrated versus actual results can be developed to capture the effect of various parameters on the response and resulting bearing capacity. A large library of datasets can be produced to feed a machine learning algorithm, to develop a model-free tool that can estimate bearing capacity from the bridge's response to shaking.

1.4. **Research Objectives**

While cost-effective, existing methodologies, such as ambient vibration, noise or impact hammers suffer from a reliance on low-amplitude shaking, which is unable to overcome intermittent stick-slip mechanisms or induce appreciable responses within the substructure-foundation system. Therefore, to overcome these low-level mechanisms in a controlled manner and improve the reliability of St-Id, large-amplitude shaking is proposed. In addition to analytical and numerical simulation tools, the use of large-amplitude mobile shakers that are available through the National Science Foundation Natural Hazards Engineering Research Infrastructure (NSF NHERI) Program is considered. Such shakers open opportunities for pushing the structural-foundation systems beyond their low-level response to reveal performance characteristics that are more representative of the expected behavior under low-strain levels.

The first primary objective of the research reported herein is to study the effects of DSSI on the dynamic response of actual bridges, assessed through experimental and numerical evaluations. T-Rex, a large-amplitude mobile shaker from the NHERI facility at the University of Texas, Austin, was used to shake two bridges. To assess the DSSI effects, 3D FEM simulations of the bridges are conducted, and the results of a fixed-base model are compared to those from a model incorporating DSSI effects. Subsequently, another
primary objective of this research is overcoming the limitations of existing dynamic testing methodologies for structural and foundation systems. Estimating the bearing capacity of foundations plays another significant role in the assessment of foundations. Therefore, the second primary objective of this research is to estimate the bearing capacity under two different scenarios based on available information:

- Known foundation geometry and soil properties, and
- Unknown foundation and soil properties.

The achievement of the two objectives is pursued by utilizing T-Rex to induce bridge vibrations, monitoring the bridge response near the foundation, and matching it with 3D FEM results. The scenario with unknown foundation and soil properties can then be used as a basis for parametric studies through finite element simulations, through which the foundation can be explored. Further details on the modeling, experimental setup, and methods used to evaluate the bearing capacity are presented in the following chapters.

In addition to the primary objectives stated above, there are several secondary objectives included in this research. Firstly, the feasibility of using large-amplitude shakers for dynamic testing was evaluated and compared to conventional methods. This comparison was based on the quality of obtained results, at varying load levels, where low load levels represent expected levels from conventional methods. In addition, various loading schemes and placement of the shaker on the bridge deck were included. This was done to provide a framework for future testing based on findings in the current study. Secondly, the effect of DSSI on the dynamic response is to be evaluated. This includes examining bridge structural response and expected behavior at various load levels. It is expected that several sub- and super-structural factors can play a role in altering the
dynamic response, which is described further in the Background section.

To address the objectives stated in this study, the current research is directed toward answering the following questions:

- How best can large-amplitude mobile shakers be used to identify and quantify the effects of DSSI from bridge shaking?
- How does the use of large-amplitude mobile shakers lead to an improved evaluation of the dynamic characteristics of bridges?
- Can that application be extended as a method to explore unknown foundations, and estimate their ultimate bearing capacity?

To answer those questions, the following tasks and corresponding goals should be achieved:

1. Develop an experimental framework for dynamic testing of bridges using large-amplitude mobile shakers.
2. Utilize current computing capabilities to build highly refined FEM models, validated by experimental results, to numerically study dynamic testing.
3. Exploit validated numerical models in running a parametric study to quantify the effect of several super- and substructural aspects on the global response of a bridge to shaking.

Task 1 pertains to the planning of the experimental program in terms of the placement of required sensors to measure the response, in addition to determining loading locations, direction, and amplitude. Results from the experimental program are the basis of model validation, inferring DSSI effects on bridge response, and estimation of ultimate bearing capacity. Task 2 involves the development of 2D and 3D models to evaluate the response
from numerical simulations and validate them with the data from the experimental program. The primary criterion to evaluate the effects of DSSI on response is the ability of the FEM models to capture accurately modal parameters, such as resonant frequencies, response spectra, and mode shapes, when compared to the experimental results. Hence, it is paramount to develop models that incorporate DSSI effects and compare them with models that include a fixed-base condition. This includes both a priori and posterior models. Once satisfactory models were achieved, by matching the modal parameters and dynamic response, a series of parametric studies were initiated. The purpose of such parametric sweeps was to examine the effect of varying site/bridge parameters as part of Task 3. This included the type of bridge, sub- and superstructure dimensions, dynamic properties of soil, or foundation features.

The ultimate bearing capacity of foundations was determined by assessing the dynamic response of the tested bridge and extracting impedance functions. In turn, those can be used to inform several foundation characteristics. Furthermore, the ability of calibrated 3D FEM models to estimate the ultimate bearing capacity of explored foundations using small-strain data obtained from experimental results is examined. Combined with experimental results, soil modulus degradation curves were implemented to ascertain failure, hence ultimate bearing capacity.

1.5. Dissertation Overview

The dissertation is organized as follows:

Chapter 2 presents background information about DSSI and a literature review of factors affecting the dynamic response of bridges. It also includes a review of published
research pertinent to dynamic testing of bridges for exploration of unknown foundations.

Chapter 3 details the research methodology including the experimental program adopted in this research to deploy mobile large-amplitude mobile shakers to determine the dynamic characteristics of bridges, including DSSI and exploration of unknown foundations. In addition, it describes the numerical models built to simulate the behavior of bridges under the dynamic loading from large-amplitude mobile shakers. FEM models are then validated against the experimental results in the following chapter.

Chapter 4 presents the results from the experimental study, which includes time histories, transfer functions, and spectra obtained from accelerometers and geophones. In addition, estimation of modal parameters from Experimental Modal Analysis (EMA) is presented. This includes damping, resonant frequencies, and impedance functions of the tested bridges.

Chapter 5 presents the results from 3D FEM simulations conducted through COMSOL Multiphysics to match the experimental results. The simulations comprise of time history, eigenfrequency, and frequency domain analyses. Furthermore, this chapter includes results from parametric sweeps to assess the response of bridges with varying super- and structural geometric and material parameters.

Chapter 6 provides the proof-of-concept showing that results from the use large-amplitude mobile shakers combined with highly refined 3D models can be used in identifying unknown bridge foundation. A case study in which the geometry of the
foundation and the soil properties are unknown is illustrated. Consequently, examples and methods of bearing capacity estimation are presented.

Chapter 7 presents the primary outcomes of this research and provides ideas for future research.
CHAPTER 2: BACKGROUND INFORMATION

This chapter presents the literature review and state-of-the-art knowledge about Dynamic Soil-Structure Interaction. This includes concepts pertaining to DSSI and how it alters the overall response of the structure-foundation-soil (SFS) system. Furthermore, this chapter discusses current practices to evaluate and assess how various geometric and material parameters of the structure and the foundation affect the overall response. Such methods include testing and numerical simulations. It also shows how the current practice and knowledge about DSSI can be implemented to explore unknown foundations.

2.1. Overview of Soil-Structure-Foundation Systems

A fixed base, typically at grade level, is assumed in various dynamic response analyses and designs of bridges. However, soil-foundation flexibility, energy absorption, and radiation by the soil system can alter the response of bridges to dynamic loads (Antonellis and Panagiotou, 2014; Nikolaos et al., 2017). This interaction between the structure, foundation, and soil, which in some cases may even change the dynamic load transmitted through the ground is, in general, referred to as Dynamic Soil-Structure Interaction (DSSI). To quantify the effects of DSSI on structural response, it is customary practice to compare the analysis results of a structure incorporating DSSI effects with one that has a fixed base. According to the commentary section of FEMA P-2082-1 NEHRP Recommended Seismic Provisions For New Buildings And Other Structures (2020), DSSI effects are categorized as:
• **Inertial effects:** A massive structure vibrating at a certain frequency generates base shear, moment, and axial loads. As a result, vertical and horizontal displacements in addition to rotations are produced at the soil-foundation interface since the foundation is flexible. This flexibility softens the overall structure and decreases natural frequencies. Furthermore, the displacements and rotations at the soil-foundation interface lead to energy dissipation of the applied load into the soil volume through radiation and hysteretic damping, which also affects the overall effective structural damping.

• **Kinematic effects:** Free-field motion (response of the ground to dynamic loads in the lack of structure and foundation) is altered once a foundation is placed on the ground surface. This is due to the stiffness of the foundation which displaces incoherently (spatially or temporally variable from the free field). Therefore, it is crucial to accurately assess the foundation input motion (FIM) that accounts for this variation. This is primarily conducted by applying transfer functions of known free-field ground motion at the surface to the FIM at the foundation level, which leads to a reduction in FIM since ground motion decreases with depth. In larger base structures, for example, those with large mat foundations, this effect becomes more pronounced such that ground motions transmitted through the foundation can be significantly variable spatially.

• **Foundation deformations:** These effects are more important when evaluating pile foundations in which shear, flexural, and axial
deformation in the foundation due to the forces applied by the superstructure and soil volume can alter the dynamic response.

There are two methods to incorporate DSSI effects in dynamic analyses: the (i) Direct Method, and (ii) the Sub-structuring method. In the Direct Method, the structure, the foundation, and soil are modeled as a complete system. This is either done by establishing 2D axisymmetric studies or full 3D finite or boundary element models. Low absorbing boundaries are assigned on the soil far-field perimeter boundaries with a fixed boundary at bedrock. While the direct approach can more accurately capture all DSSI effects, it is a challenging task to assign spatially varying foundation input motions at the foundation-soil interface, especially if it is done in three dimensions. Furthermore, such complex modeling is computationally expensive. Fig. 2-1 illustrates a typical analysis employing the Direct Method.

Fig. 2-1. Typical finite element model components included in the direct approach, adopted from (NIST, 2012).
On the other hand, the Sub-structuring method offers a more straightforward alternative to include DSSI effects in dynamic analyses. In this method, the kinematic and inertial effects are determined separately, then the total response becomes the superposition of both. In a complete Substructure procedure, foundation input motions should be evaluated and impedance functions representing spring and dashpots at the soil foundation interface are included. Impedance functions can be described as frequency-dependent stiffness and damping of the soil-foundation system. Fig. 2-2 presents the breakdown of the Sub-structuring method. The FIM is imparted onto the spring ends of the soil-foundation interface to excite the structure, whether it was a flexible or rigid foundation, which is the difference between (i) and (ii) in Fig. 2-2.

It is important to note the following definitions when describing DSSI inertial effects:

- **Rigid Base:** This represents soil with infinite stiffness (no springs or dashpots)
- **Rigid Foundation:** The foundation can undergo displacements and rotations but not deformation (foundation with springs and dashpots)
- **Fixed Base:** A rigid foundation on a rigid base
- **Flexible Base:** The foundation can undergo displacements, rotations, and deformation.

Knowing the forces that are acting on a foundation resting on a homogeneous half-space and resulting displacements/rotations at varying frequencies enables the determination of impedance functions. The soil medium in this case exhibits a viscoelastic behavior, and material damping is primarily hysteretic (absorption), while geometric damping from the foundation occurs due to scattering (radiation) (Gazetas, 1981; Zatar et al., 2021).
Both contribute to the overall energy dissipation of dynamic loads by the soil-foundation system, but the hysteretic damping is frequency independent. Fig. 2-3 depicts an overview of a rigid foundation resting on a viscoelastic homogeneous half-space for which impedance functions are determined.
While the actual structure-soil-foundation vibration is a 3D problem, it has been established that it is analogous to a single degree-of-freedom (DOF) oscillator (Gazetas, 1983). A spring-dashpot-mass system would oscillate about its equilibrium position in a translational motion. The equation of motion of this system is:

$$m\ddot{u} + c\dot{u} + ku = F(t)$$  \hspace{1cm} \text{Eq. 2-1}$$

where $m$ is the mass, $c$ is the dashpot’s damping coefficient, $k$ is the spring stiffness, $F$ is the harmonic excitation force, $u$ is displacement, and the dot as accent denotes derivative with respect to time.

The undamped natural frequency is $\omega_n = \sqrt{\frac{k}{m}}$ in rad/s. The solution to this differential equation is well-known ignoring the transient response, the steady-state response from Eq. 2-1 can be represented as:

$$u(t) = u_0 \sin(\omega t + \phi)$$  \hspace{1cm} \text{Eq. 2-2}$$
where the response amplitude $u_0 = \frac{F}{k} \frac{1}{\sqrt{[1-(r)^2]^2 + [2\xi r]^2}}$ in which $r$ is the ratio of the driving excitation frequency ($\omega$)/natural frequency ($\omega_n$), $\xi = \frac{c}{2\sqrt{km}}$ is the damping ratio, and $\phi = \tan^{-1} \frac{-2\xi r}{1-r^2}$ is the phase shift.

To establish the analogy of 1-DOF oscillators to bridges, it is essential to identify the different elements involved. Fig. 2-4 shows a typical cross-section at the pier of notional bridges, describing the load transfer mechanism from the superstructure onto the soil in a structure-foundation-soil system. Dynamic excitation occurs naturally if an earthquake takes place and shakes the foundation from the bottom (the foundation input motion drives the bridge). However, in forced vibration experiments of buildings, it is common to shake the structure on the roof or on a certain floor. In the case of bridges, the shaking takes place on the deck. Girders in Fig. 2-4 can be either structural steel or reinforced concrete beams. Pier caps can be single-pier or multi-pier caps, and the piers typically rest on a single connected spread footing. Deep foundations can be used as the substructure, but this research focuses on shallow foundations. A lumped mass system can represent this section as depicted in Fig. 2-5. In this case, two additional degrees of freedom (DoFs) are introduced to the system, which are the translation and rotation of the foundation. Those lead to swaying and rocking responses of the system. The equation of motion of the system shown in Fig. 2-5 is presented in Eq. 2-3. Due to the additional compliance at the footing compared to the fixed-base oscillator, it is always the case that the inclusion of DSSI effects would soften the structure (Veletsos and Meek, 1974).
Fig. 2-4. Typical cross-section at notional bridge piers, showing different components of a structure-foundation soil system.

Fig. 2-5. Description of soil-structure-foundation system in notional bridges at the pier section.
\[ M\ddot{U} + C\dot{U} + Ku = F \]  

Eq. 2-3

where 

\[ M = \begin{bmatrix} m_f + m_{st} & m_f h_f + m_{st} h & m_{st} \\ m_f h_f + m_{st} h & I_f + m_f h_f^2 + m_{st} h^2 & m_{st} \\ m_{st} & m_{st} h & m_{st} \end{bmatrix}, \]

\[ K = \begin{bmatrix} k_x & k_{xy} & 0 \\ k_{yx} & k_{ry} & 0 \\ 0 & 0 & k_{st} \end{bmatrix}, \]

\[ C = \begin{bmatrix} c_x & c_{xy} & 0 \\ c_{yx} & c_{ry} & 0 \\ 0 & 0 & c_{st} \end{bmatrix}, \]

\[ U = [u_x \quad \theta \quad u_s]^T, \text{ and } F = [F_{st} \quad hFs \quad F_{st}]^T. \]

This equation is based on the solution presented by Crouse and Maguire (2001) but slightly modified to fit the system shown in Fig. 2-5. Tileylioglu et al. (2011) resolved Eq. 2-3 to present stiffness as a function of measured response at the roof and foundation levels, which simplify to Eq. 2-4 and Eq. 2-5. \( K_x \) and \( K_{ry} \) are the complex-valued frequency-dependent swaying and rocking impedance functions of the footing, respectively.

\[ K_x = \frac{F_{st} + \omega^2 m_{st}(u_{tot}) + \omega^2 m_f(u_x + h\theta)}{u_x} \]  

Eq. 2-4

\[ K_{ry} = \frac{hF_{st} + \omega^2 m_{st} h(u_{tot}) + \omega^2 m_f h_f(u_x + h\theta) + \omega^2 I_f \theta}{\theta} \]  

Eq. 2-5

where \( u_{tot} = u_x + h\theta + u_s \) is the measured roof response, \( I_f = \frac{2m_f(h_f^2 + B_f^2)}{12} \) is the mass moment inertia of the foundation, and \( h = h_s + 2h_f \) is the total structural height.

It is important to note that the off-diagonal coupling terms of swaying-rocking in Eq. 2-3 are neglected for simplification, which is true for surface foundations. However, this introduces an underestimation of stiffness if embedded foundations are considered. Nevertheless, this provides a simple derivation for impedance functions from measured response directly. The underestimation of stiffness could be then accounted for by knowing typical stiffness and damping magnification ratios at varying frequencies. Another advantage of this formulation is that it eliminates the need to estimate the stiffness of the superstructure and it intrinsically becomes a part of the solution/measured response. In
addition, the displacement response \( u \) can be determined from Fourier transforms of measured velocity or acceleration records, by dividing them by \( -\omega \) or \( -\omega^2 \) respectively. By inspecting the solution, \( \text{re}(K_i) \) is the foundation stiffness and \( \text{Im}(K_i)/\omega \) is its damping, expressed as \( K_j = k_j + i\omega c_j \), where \( j \) is an index for the \( j^{th} \) degree of freedom. This can also be expressed as Eq. 2-6:

\[
K_j = k_j(1 + 2i\beta_j) \tag{Eq. 2-6}
\]

where \( \beta_j = \frac{\omega c_j}{2k_j} \) can be thought of as a percentage of critical damping. Expressing impedance in this manner enables the estimation of \( \beta_j \) from the force/response transfer function in which the loss angle \( \phi_j \), or the measured phase difference between the peak harmonic force and peak harmonic response, hence \( \phi_j = \tan^{-1}(\beta_j) \) (Wolf, 1985). This can be computed at multiple frequencies to estimate modal damping.

2.2. Factors Influencing Soil-Structure Interaction Effects

When addressing the effects the inclusion of DSSI introduces to a structure, it is imperative to understand that those imply the extent to which DSSI alters dynamic response and characteristics when compared to the same structure with a fixed base. Such factors that control this extent are discussed separately in this subsection when all other factors are held constant. In general, these factors affect the structural response, resonant frequencies, damping, and in some cases the overall dynamic behavior of a structure-foundation-soil system. In addition, to standardize the comparison between various geometric and material aspects, sub- and superstructural configurations, and different dynamic behavior structures
can exhibit at different frequencies, the dimensionless frequency $a_0$ is introduced in Eq. 2-7.

$$a_0 = \frac{\omega B}{V_s} \quad \text{Eq. 2-7}$$

where $\omega$ is the excitation frequency, $B$ is the half-width of a footing (or radius if circular), and $V_s$ is the effective shear wave velocity of the underlying soil.

**Structure-to-soil Stiffness Ratio (Rigidity Ratio):**

This ratio is defined as $\bar{s} = \frac{h}{V_s T}$, where $h$ is the total structural height, and $T$ is the fundamental period of the 1st mode. Increasing $\bar{s}$ (or resting on softer soils) leads to an immediate drastic reduction in resonant frequencies when compared to a fixed base. At $\bar{s} = 1$, a reduction in natural frequency of up to 60% is expected. On the other hand, increasing $\bar{s}$ significantly increases effective or equivalent damping. Eq. 2-8 (Wolf, 1985) shows that effective damping is proportional to $s^3$

$$\xi = \frac{\bar{\omega}^2}{\omega^2} \xi + \left( 1 - \frac{\bar{\omega}^2}{\omega^2} \right) \xi_g + \frac{\bar{\omega}^3 \bar{s}^3 m}{\bar{h}} \left[ 0.0036 \frac{s - v}{\bar{h}^2} + 0.028(1 - v) \right] \quad \text{Eq. 2-8}$$

where $\xi$ is the effective damping, $\bar{\omega}$ is the equivalent SDOF natural frequency, $\omega_s$ is the fixed-base natural frequency, $\xi$ and $\xi_g$ are the hysteretic damping ratios of the structure and soil respectively, and $\bar{h}$ and $\bar{m}$ are the Structure-to-Soil Slenderness and Structure-to-Soil Mass ratios as described in the following subsections respectively. The effect of increasing $\bar{s}$ on the maximum structural response (displacement or forces) can or increase or decrease as a function of $a_0$ when compared to a fixed base. Overall, increasing $\bar{s}$ leads to increasing SSI effects to varying extents depending on whether vertical, rocking, or swaying motions are the predominant mode of vibration (Kakhki et al., 2022; Sextos et al., 2003; Wolf, 1985). This parameter is the most influential in controlling the SSI effects extent. In general, a foundation can be considered rigid if $\bar{s}$ is $> 10$ (Gucunski, 1996).
**Structure-to-Soil Slenderness Ratio:**

This ratio is defined as \( \tilde{h} = \frac{h}{a} \) where \( a \) is the characteristic dimension of the footing (radius if circular, or half-width if rectangular). Roof displacements significantly increase as the slenderness ratio increases. The amplification of roof motions due to SSI effects relative to a fixed-base structure is caused by the dynamic behavior becoming governed by rocking motion. However, taller structures at the same rigidity ratio become less prone to period elongation (structural softening) effects from SSI. This means that taller structures do not necessarily experience more SSI effects, especially since they typically possess low rigidity ratios. Moreover, overall damping significantly decreases with decreasing slenderness ratio at the same rigidity ratio. These effects are reversed between tall and short structures. The extent to which (whether amplification or reduction) SSI effects alter the structural response (forces and displacements) of the structure-soil-foundation system depends on how far the resonant frequencies are changed relative to the fixed-base structure (Bazaios et al., 2022; Ganjavi and Hao, 2012; NIST, 2012; Wolf, 1985).

**Structure-to-Soil Mass Ratio:**

This ratio is defined as \( \bar{m} = \frac{m}{\rho V} \) where \( m \) is the effective modal mass supported by the foundation, \( \rho \) is the mass density of the underlying soil, and \( V \) is the volume of soil excited around the foundation equivalent to the volume of the foundation. Smaller values of the mass ratio lead to a lesser extent of SSI effects on a structure relative to one with a fixed base. Increasing the mass ratio significantly reduces structural peak dynamic response relative to a fixed base. This effect is even more pronounced in embedded foundations, especially at higher frequencies \( (a_0 > 1.5) \) (Chen et al., 2019). In general, increasing the mass ratio leads to an increase in impedance, owing to a more meaningful
increase in the real part (stiffness) than the imaginary part (damping) (Chen and Hou, 2009; Gucunski and Peek, 1993). This is expected since hysteretic (material) damping is frequency-independent at low strains since it is controlled by the frictional forces between the soil and foundation at the interface. The dependence of the imaginary part of impedance on frequency would be driven by radiation damping, which can vary in complexity. It can be neglected in some sites, while other sites may exhibit significant viscous damping (Wolf, 1985).

**Embedment Ratio:**

This ratio is defined as $\frac{D}{a}$ where D is the foundation embedment depth. The current discussion is for perfect contact between the sidewall of the soil and the foundation throughout dynamic loading. Increasing the embedment ratio leads to a significant increase in impedance in all vibration modes. This can affect the softening of a structure, hence the extent of SSI effects, relative to a fixed-base scenario. The imaginary part of impedance (damping) significantly increases when compared to a surface foundation, across all vibration modes. On the other hand, the increase in dynamic stiffness is more pronounced for rocking than vertical or horizontal modes. In addition, the trench height (the height of the sidewall embedded) shows a similar trend of increasing impedance as the embedment ratio increases. In other words, this increase is 0 for a surface foundation and increases gradually as the foundation is more embedded and passes by the minimum embedment ratio, where the foundation is just embedded below the ground surface (Gazetas, 1983). This effect is true for various types of foundations, including strip footings and piles (Jahankhah and Farashahi, 2017). As a result, embedment typically leads to an increase in resonant frequencies relative to a surface foundation, while peak response is reduced
primarily due to damping. Furthermore, the kinematic interaction is altered depending on
the location of the foundation relative to the free field. Therefore, appropriate estimation
of FIMs should be considered when conducting a complete DSSI analysis, since they can
alter the response. In general, increasing embedment can lead to lower structural demands
(P-2082-1, 2020).

**Other Factors:**

Apart from the aforementioned factors, other parameters can alter the response of
a structure-foundation-soil system, one of which is the configuration of the underlying soil.
The above factors are discussed presuming the foundation lies on a homogeneous half-
space, which can vary by site. Some sites may have layered stratum over half-space or
bedrock. In this case, the extent to which the dynamic properties would be altered depends
on the thickness of the layers and distance to bedrock/half-space (Cai et al., 2009; Gazetas,
1983). Furthermore, factors affecting the soil rigidity can affect the response amplitudes,
due to altering static stiffness. These include the angle of internal friction, Poisson’s ratio,
void ratio, degree of saturation, confining pressure, and grain characteristics (Gucunski,
1995; Richart et al., 1970). The extent to which these factors influence the response
depends on the strain level the soil is undergoing under a given load scenario.

To summarize, several past research efforts related to DSSI concentrated on the
development of either implicit or closed-form solutions that incorporate soil flexibility and
effects of DSSI, i.e. consideration of rocking/sliding of foundations including the factors
mentioned in this subsection (Anastasopoulos et al., 2012; Antonellis and Panagiotou,
2014; Carbonari et al., 2018; Jian and Nicos, 2002; Karatzetzou and Pitilakis, 2018; Santisi
d'Avila and Lopez-Caballero, 2018; Ülker-Kaustell et al., 2010).
2.3. Dynamic Testing for Structural Identification

Dynamic structural testing is garnering more attention for better preservation of infrastructure, specifically bridges, and gaining deeper insights into theoretical concepts. Evaluation of their dynamic characteristics and development of fundamental understanding of properties affecting them is required. Some research efforts utilize the ever-increasing computing capabilities to facilitate modeling and visualization of problems of DSSI through computer simulations (Jian and Nicos, 2002; Lu et al., 2011; Martínez-De la Concha et al., 2018; Sextos et al., 2016; Wang et al., 2014). Furthermore, soil nonlinearity, local site effects, near- and far-fault effects are among some considerations that need to be considered, which are essential for earthquake analyses. The research presented herein, however, is focused on operational-level loads. An example of a successful implementation of 3D models is an extensive study on a complex structure to study the effects of near and far-fault effects with the consideration of DSSI, compared to a fixed-base solution (Güllü and Karabekmez, 2017). More research efforts are being directed towards St-Id, and they can vary in complexity and applicability. The main goals of those research efforts are to accurately assess and describe dynamic structural behavior, infer DSSI phenomena, and obtain modal properties.

Table 2-1 summarizes studies on the effects of DSSI on the response of bridges and buildings. The evaluation of DSSI effects on the dynamic response in these studies is conducted through numerical simulations, by comparing results from the fixed-base models to those from models incorporating DSSI. The comparisons were made in several ways. Some efforts incorporating DSSI relied on the direct method approach, in which both the structure and soil are modeled, and low absorbing boundaries are assigned to the soil
perimeter boundaries. Other studies coupled the Finite Element Method (FEM) and Boundary Element Method (BEM), which eliminates the need to model the soil layer(s) as discussed in the previous subsection (2.1). The dynamic stiffness matrices at the interface represent in those cases the soil’s response/behavior. More commonly, sub-structuring approximate methods, involving foundation impedance functions, a system of dashpots and springs with frequency-dependent properties, are used to simplify the DSSI problem definition (Gazetas, 1991; Pais and Kausel, 1988). Sample of Studies on Evaluation

The first method to experimentally investigate the effects of DSSI is through scaled laboratory shake table tests (Li et al., 2015; Mao et al., 2021; Sextos et al., 2016). A major drawback of this method is that scaling coefficients to represent a full scale can be challenging and may lead to extrapolation. Nevertheless, they can provide a basic understanding of structural performance and behavior during a dynamic event such as earthquakes (Goktepe et al., 2019; Segaline et al., 2022; Tochaei et al., 2020).

As illustrated in Table 2-1, the experimental validation of DSSI problems through field measurements on actual bridges or buildings has not been implemented extensively. One of the reasons is that such validations necessitate large-amplitude excitation to capture the DSSI effects as a part of the Structural Identification (St-Id). Nevertheless, there are some research efforts implementing various forced/controlled vibrations on serviceable structures for dynamic characterization of super- and substructures (Star et al., 2019).

<table>
<thead>
<tr>
<th>Study</th>
<th>Structure Type</th>
<th>Analysis Performed</th>
<th>Source of Excitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Doménech et al., 2016)</td>
<td>Railway bridge on shallow foundation</td>
<td>Numerical simulations (FEM + BEM)</td>
<td>Simulated Railway traffic (sequence of moving loads)</td>
</tr>
<tr>
<td>Reference</td>
<td>Type of Structure</td>
<td>Analysis Method</td>
<td>Ground Motion Source</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>------------------------------------------</td>
<td>----------------------------------</td>
<td>---------------------------------------</td>
</tr>
<tr>
<td>Zanganeh et al. (2018)</td>
<td>Overpass bridge on soil columns</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Anastasopoulos et al., 2015)</td>
<td>Overpass bridge on a spread footing</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Luo et al., 2016)</td>
<td>Multistory building on piles</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Mallick and Raychowdhury, 2015)</td>
<td>Overpass bridge on inclined piles</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Martínez-Rodrigo et al., 2018)</td>
<td>Railway bridge on shallow foundation</td>
<td>Numerical simulations (FEM + BEM)</td>
<td>Simulated Railway traffic (moving train)</td>
</tr>
<tr>
<td>(Hassani et al., 2018)</td>
<td>Idealized structure (SDOF)</td>
<td>Analytical/Statistical Models</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Mallick and Raychowdhury, 2015)</td>
<td>Overpass bridge on piles</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Van Nguyen et al., 2017)</td>
<td>Multistory building on piles</td>
<td>Numerical simulations (FEM)</td>
<td>Historic earthquake records</td>
</tr>
<tr>
<td>(Manos et al., 2015)</td>
<td>Multistory building and bridge bent on a spread footing</td>
<td>Numerical simulations (FEM) + Field measurements</td>
<td>Controlled explosion</td>
</tr>
<tr>
<td>(Güllü and Özel, 2020)</td>
<td>2-span masonry arch bridge on a limestone layer</td>
<td>Numerical simulations (FEM) + Field measurements</td>
<td>Ambient Vibration (Microtremor)</td>
</tr>
</tbody>
</table>

Sample of Studies on Evaluation of DSSI.

Zanganeh et al. (2018) deployed a hydraulic exciter that can be attached to the bottom soffit of a bridge deck and shake it vertically. The exciter can deliver a load of 50 kN; however, the force amplitude was limited to 15 kN. The tested bridge was a reinforced concrete rigid short-span portal frame that supports a single-track railway. The foundation of the bridge is laid on a 0.5-1 m thick layer of crushed rock, which directly lies on bedrock. In addition, the authors employed the direct approach to model SSI inertial interactions. The instrumentation plan shows that accelerometers were placed on the superstructure. Results from their study indicate that neglecting SSI effects led to a significant deviation from the actual experimental response of the bridge. The authors suggested that increasing the central load of the exciter from 1 kN to 15 kN did not lead to considerable changes in
the observed natural frequency of the determined vibration mode nor in the damping ratio. The highest acceleration achieved at the closest sensor was 0.5g at the highest load magnitude, which exceeded serviceability limits. While such a smaller load could be utilized to fully engage the studied bridge, it could be attributed to the relatively small size and mass of the bridge, which has a span of 7 m and a width of 7.1 m. It is noteworthy to mention that the exciter was placed at 2 separate locations, which enabled capturing additional vibration modes (torsional) when the load was applied at the edge of the deck.

Olson (2005) carried out an experimental and numerical investigation of evaluating the dynamic response of sub- and superstructures. Multiple bridges with various sub- and superstructure configurations were assessed. A Vibroseis truck capable of delivering up to 445kN was used to excite the bridge; however, a linear behavior of the bridges was observed up to 178kN. Vertical loading of the bridge was conducted directly above the piers. One of the goals of that study was to examine the ability to determine damage in the substructure (foundations) through the means of controlled forced vibration. This is not to be conflated with the condition attributes of the foundation, e.g., rebar corrosion or cracking which are typically assessed through nondestructive evaluation techniques. Olson (2005) showed that it is possible to detect damage in foundations through dynamic testing because of the associated drop in vertical stiffness as a result of damage when compared to a sound footing. This phenomenon would be accompanied by a decrease in natural frequencies and an increase in flexibility. Another objective of the study was to investigate the ability of modal dynamic testing to provide discernable differences to determine if the tested foundation is shallow or deep. The results from two bridges with similar superstructures, one supported on a spread footing and the other on piles with a pile cap,
show that their resonant frequencies were different. Nevertheless, the author concluded that this is insufficient evidence without further testing of identical bridges in similar soils.

Ma et al. (2021) conducted various falling weight impact tests to determine the vertical dynamic stiffness of foundations under five different multi-span railway bridges with hammerhead and multi-column piers. The foundations for those bridges were caissons, grouped piles, and spread footings. A total of 60 foundations were evaluated, 20 of each type. The impactor can deliver a force of up to 200kN, which was deemed sufficient to excite the substructure dynamically while maintaining that the test is nondestructive. The impacts were imparted directly onto the foundations and the sensors were placed near the impact locations, exciting frequencies in the range of 10-30 Hz. In addition, the authors used pass-by trains having a maximum axle of 30t to measure the lateral response of the bridge. The authors presented that the bearing capacity of the footings, specifically spread footings, cannot be estimated from lateral displacements. However, the vertical dynamic stiffness of the footing can be determined by the means of dynamic testing. Their efforts show that the dynamic footing response can be effective if the excitation can dynamically excite the structure. The authors also indicated that it is possible to discern the type of foundation from the patterns of dynamic vertical impedances of each type, which agrees with the results from (Maser et al., 1998).

Davis and Sanayei (2020) conducted foundation identification using strain and acceleration measurements under operational traffic loads. A 3-span continuous steel bridge supported by drilled shafts in stiff clay was instrumented with sensors during its construction, and it carries some truck traffic. The system was set up to measure significant loads (loads higher than noise) and record the response. Fig. 2-6 shows a sample coherence
function (as described later in this dissertation) which can be used as an indicator of correlation between load and response. It can be observed that a significant part of the frequency spectrum data (below 0.9) cannot be used to uniquely describe the dynamic behavior; only the circled regions in Fig 2-6 can be used to describe it. In other words, noise from several sources contaminates the measurements and contributes to the response. Therefore, it is imperative to ensure that the dynamic excitation force is at a level that can inform the dynamic behavior of the SFS system. Nevertheless, the study by the authors shows promise foundation characteristics can be deduced from the structural response.

Fig. 2-6. Coherence function obtained from tested bridge under operational traffic, adapted from (Davis and Sanayei, 2020).
2.4. T-Rex as a Mobile Shaker for Dynamic Bridge Testing

As discussed in the previous sub-section, the use of ambient vibrations, wind, or temperature changes to experimentally carry out the St-Id of bridges led to response levels that were insufficient to provide information about the structure-foundation-soil systems (Bao et al., 2012; Brownjohn et al., 1994; Yarnold and Moon, 2015). Moreover, the participation of unreliable mechanisms under low-load levels (e.g., unintended composite action, engagement of nonstructural elements, frozen bearings, etc.) poses significant challenges when the identified model is used to estimate the force effects associated with much higher load levels (Farrag et al., 2018; Moon and Aktan, 2006). On the other hand, other research efforts included experimental evaluation of DSSI on scaled bridge models or individual bridge elements, such as bents, in a controlled environment/laboratory (Deng et al., 2012; Manos et al., 2015). Such studies can be beneficial to evaluate various limit states of bridges, especially for inelastic behavior during earthquakes. Ambient vibrations or operational demands are insufficient to mobilize DSSI mechanisms. Moreover, at small-scales shakers have been successful at mobilizing and permitting the observation of DSSI mechanisms. However, excitation devices should be scaled up to observe DSSI on a full-scale structure.

The Natural Hazards Engineering Research Infrastructure (NHERI) large mobile shakers can be employed to overcome limitations stemming from low-amplitude loading conventional methods of St-Id, by shaking actual bridges at higher levels and in a fully controlled manner. Fig. 2-7 illustrates the proposed shaking as part of the SHM/NDE of bridges, as opposed to the conventional low-level methods. T-Rex, a large-amplitude
mobile shaker from the Natural Hazards Engineering Research Infrastructure (NHERI) experimental facility at the University of Texas at Austin was employed on bridges in this research effort. T-Rex, shown in Fig. 2-8, can generate large dynamic forces in any of three directions (vertical, horizontal in-line, and horizontal cross-line). To change from one shaking direction to another, the operator simply pushes a button in the driver’s cab. The shaking system is housed on an off-road, all-wheel-drive vehicle. The theoretical force outputs of T-Rex in the vertical and two horizontal directions are shown in Fig. 2-9. The maximum force output is about 267 kN in the vertical mode and about 134 kN in each horizontal mode. The maximum force output is limited by the hold-down weight of the T-Rex truck. Shaking at a higher force output can cause the shaker to decouple (jump) from the ground in the vertical mode or slide on the ground in the horizontal mode. In the lower frequency range, the force output is limited by the stroke of the reaction mass. In the higher frequency range, the force output is limited by the speed of the hydraulic servo valve. The
actual force output depends on the ground condition. The force output can be measured by load cells, but it requires stiff ground to support the load cell.

Fig. 2-7. Comparison between conventional St-Id techniques with the proposed use of NHERI shakers.
For most shaker operations, the ground surface varies from soft soil to concrete pavement, so using load cells is not a practical option. The current method uses accelerometers
mounted on the reaction mass and base plate of the shaker from which the force output can be calculated.

Fig. 2-10 shows a close-up photograph of the T-Rex shaker. As shown in the figure, airbags are used to isolate the shaker from the truck. The airbags function as a low pass filter and transfer only static force. If one takes a free body of the T-Rex shaker and ignores the hydraulic system, the only external dynamic force is the dynamic ground force which is also the dynamic force output of T-Rex. The dynamic force output, $F_d$, can be determined as in Eq 2-8:

$$F_d = m_{RM} \cdot a_{RM} + m_{BP} \cdot a_{BP}$$  \hspace{1cm} \text{Eq. 2-9}

where: $m_{RM}$ is the mass of the reaction mass, $a_{RM}$ is the reaction-mass acceleration, $m_{BP}$ is the mass of the base plate, and $a_{BP}$ is the base-plate acceleration. The T-Rex onboard Pelton controller was modified to provide an external control option. With the external control option, T-Rex can output an arbitrary waveform generated by an analog waveform generator.

Fig. 2-10. A closed photograph of the T-Rex.
The amplitude of the force output is proportional to the amplitude of the arbitrary waveform, and the maximum force output is set at 5 V of the arbitrary waveform. Both chirp and stepped sine functions were used to drive T-Rex for tests on the bridges. In the chirp function, the frequency varies linearly from the start frequency to the end frequency at a given time length. In the stepped sine function, the frequency varies from the start frequency to the end frequency in a fixed number of steps. The stepped function is ideal for steady state measurements at small amplitude. The chirp function is a better option at a larger amplitude to limit the number of loading cycles (Menq et al., 2008; Stokoe et al., 2017).

2.5. Overarching Research Aims

The research reported herein aims to study the effects of DSSI on the dynamic response of an actual bridge, assessed through experimental and numerical evaluations. The technique presented herein introduces a novel nondestructive evaluation (NDE) tool that can adequately and directly capture global dynamic features of bridges, including the significance of DSSI. Determination of DSSI effects related to response under typical operational, live loads, while providing sufficient loading to inform DSSI effects and overcome stick-slip mechanism attributed to low-level methods is emphasized. T-Rex was used to shake the bridges included in this research effort. 3D FEM simulations evaluated bridges were conducted to assess the DSSI effects, and the results of a fixed-base model compared to those from a model incorporating DSSI effects. Results of the response in time and frequency domains, and the Eigen-modes, were compared to the experimental results to evaluate both models.
Furthermore, the evaluation of existing bridge foundations can vary in complexity based on the available information about the substructure. Hence, there is a variety of scenarios that can be encountered with different bridges, including the lack of as-built drawings or geotechnical site exploration/boring logs. The feature of the proposed approach enables a fully nondestructive and rapid assessment of bridges and their components using large mobile shakers as a dynamic excitation source and fast deployable sensor arrays to capture the response. Combined with Structural Identification (St-Id), the proposed evaluations provide information about the global bridge condition, assess the need for system adaption, and in the case of a superstructure and/or substructure replacement, assess the feasibility of the foundation reuse. To assess the potential reuse of the substructure, the ultimate bearing capacity is the basis of judging the substructure reusability herein. The bearing capacity can be fundamentally defined as the average contact pressure between the load-bearing soil and the bottom surface of the foundation/substructure under a given limit state. From a structural/mechanical point of view, there are primarily three limit states: i) critical state (when a failure mechanism takes place), ii) serviceability (excessive settlements), and iii) extreme events such as earthquakes (liquefaction).

Furthermore, since this study serves as a proof of concept, the simpler case of shallow foundations is sought after rather than pile foundations. In addition, a relatively rigid structure in cohesionless soil was selected for testing to avoid more complicated/indirect phenomena that can affect response such as consolidation or volume change. Therefore, the objective of the study is to estimate the bearing capacity under two different scenarios based on available information:
• Known foundation geometry and soil properties.
• Unknown geometry and foundation soil properties.

The condition with known foundation and soil properties is used as a basis for parametric studies through finite element simulations to investigate the condition with unknown foundation or property. More details on the modeling, experimental setup, and methods used to evaluate bearing capacity are presented in the following chapters.
CHAPTER 3: RESEARCH METHODOLOGY

The experimental program conducted to achieve the objectives of this research is presented in this chapter. Two different bridges were dynamically excited using T-Rex large mobile shaker: Hobson Avenue Bridge in Hamilton, New Jersey, and Gate Creek Bridge in Oregon. Details about the bridges, testing sequences, instrumentation, sensor layouts, and site characterization are presented. In addition, details about the FEM model setup for both bridges are provided.

3.1 Experimental Program

3.1.1. Hobson Avenue Bridge, New Jersey

This experiment aimed to conduct large-amplitude shaking of a bridge to capture and quantify the significance of the DSSI effects on its dynamic response. Hobson Avenue Bridge, a bridge over Interstate 195 in Hamilton Township, New Jersey, was selected for the study. It is a 67.4 m [221 ft] two-span continuous steel multi-girder bridge with rocker end bearings supported by a three-column bent on a shallow continuous reinforced concrete (RC) footing. Fig. 3-1 provides a side view of the bridge. Fig. 3-2 depicts various views of the bridge and the dimensions of the super- and substructure.

Fig. 3-1. Hobson Avenue Bridge, Hamilton Township, New Jersey.
Fig. 3-2. (a) Deck dimensions, and (b) substructure dimensions of Hobson Avenue Bridge.

T-Rex was employed to shake the bridge. The T-Rex, shown in Fig. 3-3(a), can induce large-amplitude vibrations of the bridge which can be monitored in real-time in a control room, as shown in Fig. 3-3(b). The control room is moved off the bridge before testing. The maximum force output is limited by the hold-down weight of the T-Rex truck. To measure the force, T-Rex uses accelerometers mounted on the reaction mass and base plate of the shaker from which the force output can be calculated. However, for this testing,
the excitation amplitude was capped at 94 kN transversely and 48 kN vertically to limit the bridge response to about 2.54 cm/s (1 in/s).

![Fig. 3-3. (a) T-Rex, and (b) control room to monitor response in real-time on Hobson Avenue Bridge.](image)

A linear chirp function, shown in Fig 3-4, was used to drive the T-Rex shaker for tests on the bridge. In the linear chirp function, the frequency of the load varies linearly from the start to end frequency during a given time. The chirp function is a better option to limit the number of loadings cycles but might not always lead to the full attainment of a steady-state condition. A linear chirp from 15 Hz to 1 Hz and 80 to 10 Hz, with a total duration of 32 s and 16 s, respectively, were implemented.

![Fig. 3-4. A reverse linear chirp signal used to excite the bridge from 15 Hz to 1 Hz](image)

The load was defined at a sampling rate of 200 Hz (or time increments of 0.005 s), which satisfies the Nyquist frequency condition. The loading was applied at multiple
magnitude levels and directions, as shown in Table 3-1. The directions of loading were vertical, transverse, and longitudinal. The driving voltage was increased incrementally until the maximum response reached 1 in/s (2.54 cm/s). Except for Run 1, all runs were repeated to ensure data quality and to estimate the coherence function. Geophone arrays were used to capture the response of the bridge deck, bent, abutment, and the ground response up to 23 m [80 ft.] away from the bridge, due to the T-Rex shaking of the bridge. The arrays are shown in Fig. 3-5. Although accelerometers were deployed in the experimental program, data obtained from them is not reported herein.
Table 3-1. Description of T-Rex Test Runs on Hobson Avenue Bridge, NJ.

<table>
<thead>
<tr>
<th>Run</th>
<th>Mode</th>
<th>Start Frequency (Hz.)</th>
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*1 volt ~ 6000 lb ~ 26.7 kN
Fig. 3-5. (a) Geophone arrays on the ground for MASW, (b) geophone and accelerometer on the deck, (c) and on the pier.

The overall sensor layout, with corresponding channel numbers and locations, used to measure the bridge response, is shown in Fig. 3-6. A total of 45 geophones were
employed. Results from the ground geophones were used to evaluate which vibrations were measurable above ambient vibrations. Furthermore, they were also used to estimate the volume of soil engaged in the bridge motion during different vibration levels and modes. This is crucial to quantify energy dissipation and damping characteristics of the soil by examining wave attenuation. In addition, four single stations for horizontal-to-vertical (H/V) spectral ratio noise measurements in the free field were placed on the bridge perimeter to evaluate site effects and identifying the fundamental resonant frequency of the soil deposit, as shown in Fig. 3-6(b). Each single station measurement location is denoted relative to the Hobson Avenue Bridge (i.e., NW, NE, SW, and SE).

Fig. 3-6. (a) Overall sensor layout and T-rex positions and (b) geophone channel numbers (from Google Earth, 2017).
Before the bridge testing, the site’s shear wave velocity ($V_s$) profile was obtained through Multichannel Analysis of Surface Waves (MASW) testing. The site layout indicating the location of the 23-m long MASW linear array is also shown in Fig. 3-6(b). To obtain the $V_s$ profile of the site, various inversion parameterizations for the theoretical fundamental mode Rayleigh wave dispersion, considering several layering ratios, were evaluated to match the experimental data. Fig. 3-7 illustrates the median $V_s$ profiles obtained for the site derived from the 1000 lowest misfit $V_s$ profiles for each inversion parameterization. The layering ratio ($\Xi$), representing each inversion parameterization, is shown in the legend. The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the H/V curve used to constrain the Rayleigh wave ellipticity peak in the inversion (i.e., SE or SW). The resolution depth ($d_{res}$), which corresponds to the theoretical resolution limit of the experimental dispersion data (i.e., $d_{res} = \lambda_{max}/2$), is indicated in the figure. An average shear wave velocity of 200 m/s was deemed appropriate for the depth down to about 15 m. More details about the MASW results are presented in Appendix A.
Fig. 3-7. Shear wave velocity profile of the test site back-calculated from the MASW test data

3.1.2. **Gate Creek Bridge, Oregon**

The objective of evaluating another bridge with certain requirements was to utilize lessons learned from the Hobson Avenue Bridge testing to examine the applicability of large amplitude shakers for the exploration of unknown foundations. This would be carried out from evaluation of vertical impedance functions, and the testing scheme followed as such. Gate Creek overpass on McKenzie Highway in Vida, Lane County, Oregon was selected in coordination with Oregon DOT which fits the desired bridge features; a bridge with shallow foundations placed in sandy soil. Fig. 3-8(a) shows the location of the bridge; the solid lines represent the extent of the bridge. The structure consists of two parts because the bridge widening was done in 1986. Both parts are the multi-span reinforced-concrete bridge on spread footings. However, while the older part is a ribbed deck supported by a
portal frame, the newer part is supported by a single hammerhead pier. Fig. 3-8(b) displays the underside of the bridge with different superstructures.

Fig. 3-8. (a) Bridge location and (b) old and new parts of the bridge with different superstructures.

For this experiment, only the newer part of the bridge was evaluated. Since the objective of this study was to explore unknown foundations and subsequently evaluate the bearing capacity of the underlying footings, only vertical shaking was conducted on the bridge. The exploration of unknown foundations in this research follows an iterative process through numerical models and parametric studies validated by results from the experimental program, namely vertical impedance functions. A local traffic control company was hired to assist with handling traffic and flagging. The testing of the bridge
was conducted over 2 days. On the first day, a chirp loading function at different loading levels was conducted to identify the natural frequencies of the bridge system as a whole. Once the natural frequencies were identified, both the steady-state and ramp-up loading functions were carried out the following day at various frequencies to assess the dynamic amplification of the response. A total of 10 geophones (velocity sensors) were deployed to measure the response of the bridge to shaking. The shaking was conducted on 3 different piers of the new section of the bridge. Therefore, the geophones were moved and laid out to capture the response of the excited span and respective pier. Fig. 3-9 shows a geophone and connecting cable that was used for measuring bridge response on the bridge deck above the pier. As for the response of the pier, geophones were attached to the pier 2’ (60 cm) above ground with a bolted bracket, as illustrated in Fig. 3-10. An overall geophone layout for the piers evaluation is depicted in Fig. 3-11, Fig. 3-12, and Fig.3-13.

Fig. 3-9. Typical geophone used for measuring response.
Fig. 3-10. Geophone attached to the pier to measure response.
Fig. 3-11. Overall sensor layout for the tested pier.

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Fig. 3-12. Overall sensor layout for the tested pier.
Fig. 3-13. Overall sensor layout for the tested pier.
Fig. 3-14 shows T-Rex and the control room on the bridge ready for shaking. The operator in the control room is initiating the loading sequence and monitors the bridge response. Fig. 3-15 shows the operator monitoring and controlling response in real-time, and the waveform generator initiating the T-Rex shaking. Throughout all shaking trials, a maximum response of 1 in/s was set as a vibration level limit. This was to ensure that there is no damage imparted into the bridge within the given frequency range while maximizing the force output. Table 3-2 presents a summary of T-Rex shake runs conducted above all piers. The sampling frequency was 1536 Hz, which by far exceeds the Nyquist frequency requirement for the test range.
Fig. 3-15. Left: operator monitoring response, and right: waveform generator controlling T-Rex shaking.
Table 3.2. Summary of T-Rex Shake Runs in the Experimental Program.

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*1 volt ~ 6000 lb ~ 26.7 kN

There were no boring logs/site exploration reports provided by ODOT for the bridge. However, there were results from two test bores (done at the bridge abutments), reporting a Standard Penetration Test (SPT) value of 40, and classifying the soil as very dense sand, with gravel and cobbles, which can be also observed from Fig. 3-8(b). While this information provides some insight into expected soil mechanical properties, it was just used as a qualitative measure to estimate soil density and shear wave velocity range. Instead, a Spectral Analysis of Surface Waves (SASW) was conducted at the bridge site to
estimate the soil shear wave velocity \((V_s)\) profile, which is correlated to the response and mechanical properties. From the knowledge of shear wave velocity, the shear modulus \((G)\) can be determined by:

\[
G = \rho (V_s)^2.
\]

Eq. 3-1

where \(\rho\) is the mass density of the soil.

In the SASW method, the stiffness of a layered medium is determined through the measurement of the velocity of Rayleigh (surface) waves. In this test, two or more sensors (geophones) are placed on the ground, and a sledgehammer is used to generate a wave on the surface by striking the ground and recording the response at various spacings. Shorter receiver spacing is used to examine shallower layers, while longer spacing is used for deeper layers. The test is performed in two directions to cover the effects of dipping layers and internal phase shifts due to receivers and instrumentation (Nazarian et al., 1983). The source and receiver signals are recorded by a signal analyzer. The cross-power spectrum of signals at two receiver positions is used to determine the Rayleigh wave velocity (phase velocity) with the knowledge of the time or phase delay between receivers as a function of frequency as:

\[
t(f) = \phi_{XY}(f)/360^{\circ}*f
\]

Eq. 3-2

\[
V_r(f) = D/t(f)
\]

Eq. 3-3

\[
\lambda_r(f) = V_r(f)/f
\]

Eq. 3-4

where \(t(f)\) is the time delay between two receivers as a function of frequency, \(\phi_{XY}\) is the phase shift obtained from the cross-power spectrum, \(f\) is frequency, \(V_r(f)\) is Rayleigh wave velocity, \(D\) is receiver spacing, and \(\lambda_r(f)\) is the wavelength as a function of frequency.

The plot of Rayleigh wave velocity vs. wavelength is referred to as the dispersion curve, which is used, in turn, in the inversion process (Gucunski and Woods, 1992). The
shear-wave velocity profile is determined by matching the experimental results (Rayleigh-wave velocity) with the one for an assumed profile (forward model) until convergence (best fit) is achieved, which is a numerically intensive procedure. Alternatively, an approximation of shear wave velocity from the Rayleigh wave velocity can be done in uniform soils with regular stratification (velocity increases with depth), which was done in the current research. The approximation is that $V_r$ is 10% lower than the shear wave velocity. Fig. 3-16 shows the SASW test conducted at the bridge site, and Table 3-3 provides a summary of SASW runs conducted at the site (locations are relative to the sensor 2 location).

Table 3-3. Summary of SASW Runs Carried out to Determine Shear Wave Velocity.

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<td>F</td>
<td>-15</td>
<td>0</td>
<td>30</td>
<td>-30</td>
</tr>
<tr>
<td>13</td>
<td>R</td>
<td>-30</td>
<td>0</td>
<td>15</td>
<td>30</td>
</tr>
</tbody>
</table>
The SASW test results were analyzed to produce a representative shear wave velocity of the site. Fig. 3-17 shows a sample phase shift between two receivers obtained from the cross-power spectrum of two tests. The receiver spacing is indicated in the legend of each figure. This was done for the tests indicated in Table 3-3. Results from all tests were then used to produce an overall shear wave velocity profile along the depth of the soil, which is shown in Fig. 3-18. It can be observed that the determined range of velocities is in agreement with typical values for dense sand. The higher velocities at deeper portions of the profile are indicative of higher confining pressures and the presence of gravel/cobbles, which was evident in Fig. 3-8(b), and which validates the profile. Based on the plot in Fig. 3-17, the shear wave velocity in the zone around the footing was approximated as 400 m/s. This is the reference value for the estimation of mechanical properties and matching of the measured response with the one from numerical simulations.
Fig. 3-17. Phase shift diagrams of two trials with indicated spacing, forward measurement.

Fig. 3-18. Experimental shear wave velocity profile of soil at tested bridge.
3.2. Numerical Simulation

3.2.1. Hobson Avenue Bridge, New Jersey

Results from established 3D FE models can be indicative of the expected dynamic behavior of bridges if the boundary conditions and material properties are reasonably well-defined and reflect the physics of the problem (Sextos et al., 2016). Some FEM programs enable the incorporation of DSSI effects in the evaluation of the dynamic response of structures. COMSOL Multiphysics software was used in this research to produce 3D FEM simulations of the Hobson Avenue Bridge and the response due to a chirp-type dynamic loading. To determine the significance of DSSI effects on the dynamic response of the structure, a 2D model was first developed. Incorporating the effects of DSSI on the soil-foundation-structure system was introduced as a system of translational and rotational frequency-dependent springs and dashpots, representing the impedance functions of an equivalent circular surface foundation. Details about this study can be found in Appendix B.

To investigate the effect of DSSI on the actual bridge response, two 3D models were developed; one is incorporating DSSI effects and another with a fixed base. In both the fixed-base and DSSI-incorporating FEM models, linear elastic material properties were assigned, since the response was in the elastic range, due to the controlled vibration levels. Another model, not presented herein, was built that included solid soil elements surrounding the footing under the same load level. Results from that model showed that maximum strain values are in the order of $5 \times 10^{-5}$. Therefore, soil properties in this study were retained constant as low-strain moduli and damping, confirming the linear elastic material model assumption.
To incorporate the DSSI effects in the model, impedance functions of a sub-structured soil-foundation system (SFS) were modeled as a system of frequency-dependent translational and rotational springs and viscous dampers. The boundary conditions at abutments do not incorporate DSSI effects in the current research, and the bridge has rocker bearings at its ends. This is due to the experimental setup conducted in the study in which loading was applied directly above the pier and at mid-span. Hence, the response was primarily controlled by the pier footing boundary conditions. These springs and dashpots of frequency-dependent properties were placed at the bottom of the bent footing. Those represent the total mechanical impedance in the $S(\omega) = K(\omega) + i\omega C(\omega)$ form as discussed in Chapter 2, in which the real part represents the stiffness, while the imaginary part represents the damping. The impedance function shown in Eq. 2-6 can also be written in the form: $K_i = K_{is}[k(\omega)+ia_0c(\omega)]$, where $K_i$ is the complex impedance for the $i^{th}$ degree of freedom, $K_{is}$ is the static stiffness of the same degree of freedom, $k(\omega)$ is the stiffness coefficient, $c(\omega)$ is the damping coefficient and $a_0 = \omega B/V_s$ is the dimensionless frequency. The dimensionless frequency is a function of driving frequency $\omega$, the half-width (or radius) of the footing $B$, and the shear wave velocity of soil $V_s$. Gazetas (1991) has summarized simplified closed-form solutions for foundations on an elastic half-space, including the effects of foundation shape, embedment, and soil uniformity. Based on different scenarios, static stiffness is calculated by knowing the shear modulus of soil ($G$) and the geometry of the footing. The dynamic stiffness of the footing is then obtained by multiplying the static stiffness by the dynamic coefficients corresponding to the dimensionless frequency of the driving excitation.
Formulae and parameters for embedded rectangular foundations in a homogenous half-space were used. The footing vibration modes considered include vertical, transverse and longitudinal swaying, and transverse and longitudinal rocking. Torsional vibrations and coupled modes were excluded from the impedance functions modeling study. Table 3-4 summarizes the parameters used for simulating the bridge. Tables 3-5 and 3-6 summarize the foundation stiffness and damping properties, respectively, used in the current study. Fig 3-19 presents the frequency-dependent impedance functions for the appropriate footing aspect ratio embedment ratio used in this study. Those were obtained from (Gazetas, 1991) with the use of cubic splines to interpolate between curve points (appearing as solid dots in the graphs). The plots where dots do not appear were generated analytically. The value of $c_x$ is constant with frequency and it is $\sim 1$, hence it is not plotted.
Fig. 3-19. Dynamic stiffness and damping coefficients for various vibration modes.
Table 3-4. Geometric and Material Parameters of the Foundation Included in the FE Models.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Expression[units]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \mu )</td>
<td>Poisson Ratio</td>
<td>0.33333</td>
</tr>
<tr>
<td>( V_{la} )</td>
<td>Lysmer Wave Speed</td>
<td>( 3.4V_s/((\pi(1-\nu)) )</td>
</tr>
<tr>
<td>( \rho_s )</td>
<td>Soil Density</td>
<td>( 2000[kg/m^3] )</td>
</tr>
<tr>
<td>( V_s )</td>
<td>Shear Wave Velocity</td>
<td>( 200[m/s] )</td>
</tr>
<tr>
<td>( B )</td>
<td>Footing Half-Width</td>
<td>( 1.75[m] )</td>
</tr>
<tr>
<td>( H )</td>
<td>Footing Half-Height</td>
<td>( 0.6[m] )</td>
</tr>
<tr>
<td>( L )</td>
<td>Footing Half-Length</td>
<td>( 5.8[m] )</td>
</tr>
<tr>
<td>( d )</td>
<td>Distance From Surface to Top of Footing</td>
<td>( 0.6[m] )</td>
</tr>
<tr>
<td>( D_f )</td>
<td>Footing Depth</td>
<td>( d+2H )</td>
</tr>
<tr>
<td>( h_f )</td>
<td>Distance From Surface to Center of Footing</td>
<td>( H+d )</td>
</tr>
<tr>
<td>( d_{emb} )</td>
<td>Embedded Height</td>
<td>( D_f )</td>
</tr>
<tr>
<td>( A_b )</td>
<td>Bearing Area</td>
<td>( 4BL )</td>
</tr>
<tr>
<td>( A_w )</td>
<td>Footing Side Contact Area</td>
<td>( 8H(B+L) )</td>
</tr>
<tr>
<td>( \chi )</td>
<td>Skin Ratio</td>
<td>( A_b/4L^2 )</td>
</tr>
<tr>
<td>( I_{by} )</td>
<td>Moment Of Inertia Around Y</td>
<td>( 4BH^3/3 )</td>
</tr>
<tr>
<td>( I_{bx} )</td>
<td>Moment Of Inertia Around X</td>
<td>( 4BL^3/3 )</td>
</tr>
<tr>
<td>( \eta_{rx} )</td>
<td>Frequency-Dependent Embedment Contact Effect - Along X</td>
<td>[ 0.25 + 0.65\sqrt{\frac{a_0}{D_f}} (\frac{d_{emb}}{D_f})^{-\frac{2}{3}} (\frac{D_f}{B})^{-\frac{1}{3}} ]</td>
</tr>
<tr>
<td>( \eta_{ry} )</td>
<td>Frequency-Dependent Embedment Contact Effect - Along Y</td>
<td>[ 0.25 + 0.65\sqrt{\frac{a_0}{D_f}} (\frac{d_{emb}}{D_f})^{-\frac{2}{3}} (\frac{D_f}{L})^{-\frac{1}{3}} ]</td>
</tr>
</tbody>
</table>

The expression for total mechanical impedance becomes \( S(\omega) = K_ik(\omega) + io[C_1(\omega) + \frac{2Kik(\omega)}{\omega}\beta] \) where \( i \) indicates the vibration mode, \( C_1 \) is the embedded footing radiation damping for the \( i \)th mode of vibration, and \( \beta \) is the soil hysteretic frequency-independent damping. The hysteretic damping term can be neglected at small strains which is the case for the model of the Hobson Avenue Bridge, but it becomes crucial when considering earthquake-level loads. The total mechanical impedance is assigned to the bottom of the footing in the FE model.
Table 3-5. Static Stiffness for Different Vibration Modes Included in the FE Models (Gazetas, 1991)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Expression</th>
<th>Value [units]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_z$</td>
<td>Surface Static Vertical Stiffness</td>
<td>$\frac{2GL}{1-\mu} \left(0.73 + 1.54\chi^{0.75}\right)$</td>
<td>$1.7676 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_y$</td>
<td>Surface Static Lateral Stiffness</td>
<td>$\frac{2GL}{2-\mu} \left(2 + 2.5\chi^{0.85}\right)$</td>
<td>$1.518 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_x$</td>
<td>Surface Static Longitudinal Stiffness</td>
<td>$K_y - \left[\frac{0.2GL}{0.75-\mu} \left(1 - \frac{H}{L}\right)\right]$</td>
<td>$1.3675 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_{rx}$</td>
<td>Surface Static Rocking around x-axis Stiffness</td>
<td>$\frac{GL_{bx}}{1-\mu} \left[\frac{L}{B}\right]^{0.25} \left[2.4 + 0.5 \left(\frac{B}{L}\right)\right]$</td>
<td>$8.8458 \times 10^8$ [N.m/rad]</td>
</tr>
<tr>
<td>$K_{ry}$</td>
<td>Surface Static Rocking around y-axis Stiffness</td>
<td>$\frac{GL_{by}}{1-\mu} \left[\frac{L}{B}\right]\left[3 \left(\frac{L}{B}\right)^{0.15}\right]$</td>
<td>$3.6289 \times 10^8$ [N.m/rad]</td>
</tr>
<tr>
<td>$K_{zemb}$</td>
<td>Embedded Static Vertical Stiffness</td>
<td>$K_z \left[1 + \left(\frac{D_f}{21B}\right)\right] \left[1 + 0.2 \left(\frac{A_w}{A_b}\right)^{2/3}\right]$</td>
<td>$2.2104 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_{yemb}$</td>
<td>Embedded Static Lateral Stiffness</td>
<td>$K_y \left[1 + 0.15 \left(\frac{D_f}{21B}\right)^{0.5}\right] \left[1 + 0.52 \left{ \left(\frac{D_f - H}{B}\right) \left(\frac{A_w}{L^2}\right)^{0.4}\right}\right]$</td>
<td>$1.9262 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_{xemb}$</td>
<td>Embedded Static Longitudinal Stiffness</td>
<td>$K_x \left(\frac{K_{yemb}}{K_y}\right)$</td>
<td>$1.7353 \times 10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_{rxemb}$</td>
<td>Embedded Static Rocking around x-axis Stiffness</td>
<td>$K_{rx} \left{1 + 1.26 \left(\frac{D_f - H}{B}\right) \left[1 + \left(\frac{D_f - H}{D}\right)^{0.2} \left(\frac{B}{L}\right)^{0.5}\right]\right}$</td>
<td>$2.8324 \times 10^9$ [N.m/rad]</td>
</tr>
<tr>
<td>$K_{ryemb}$</td>
<td>Embedded Static Rocking around y-axis Stiffness</td>
<td>$K_{ry} \left{1 + 0.92 \left(\frac{D_f - H}{L}\right)^{0.6} \left[1.5 + \left(\frac{D_f - H}{L}\right)^{1.9} \left(\frac{D_f - H}{L}\right)^{-0.6}\right]\right}$</td>
<td>$1.3986 \times 10^9$ [N.m/rad]</td>
</tr>
</tbody>
</table>
Table 3-6. Dynamic Radiation Damping for Different Vibration Modes Included in the FE Models (Gazetas, 1991)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_z$</td>
<td>Surface Vertical Radiation Damping</td>
<td>$\rho_s V_{la} A_b * c_z$</td>
</tr>
<tr>
<td>$C_y$</td>
<td>Surface Lateral Radiation Damping</td>
<td>$\rho_s V_{s} A_b * c_y$</td>
</tr>
<tr>
<td>$C_x$</td>
<td>Surface Longitudinal Radiation Damping</td>
<td>$\rho_s V_{s} A_b$</td>
</tr>
<tr>
<td>$C_{rx}$</td>
<td>Surface Rocking around x-axis Radiation Damping</td>
<td>$\rho_s V_{la} I_{nx} * c_{rx}$</td>
</tr>
<tr>
<td>$C_{ry}$</td>
<td>Surface Rocking around y-axis Radiation Damping</td>
<td>$\rho_s V_{la} I_{ny} * c_{ry}$</td>
</tr>
<tr>
<td>$C_{x,emb}$</td>
<td>Embedded Vertical Radiation Damping</td>
<td>$C_x + (\rho_s V_{s} A_b)$</td>
</tr>
<tr>
<td>$C_{x,emb}$</td>
<td>Embedded Lateral Radiation Damping</td>
<td>$C_y + (4\rho_s V_{s} B d_{emb}) + (4 \rho_s V_{la} L d_{emb})$</td>
</tr>
<tr>
<td>$C_{y,emb}$</td>
<td>Embedded Longitudinal Radiation Damping</td>
<td>$C_x + (4\rho_s V_{la} B d_{emb}) + (4 \rho_s V_{s} L d_{emb})$</td>
</tr>
<tr>
<td>$C_{rx,emb}$</td>
<td>Embedded Rocking around x-axis Radiation Damping</td>
<td>$C_{rx} + \rho_s I_{bx} \frac{d_{emb}}{B} \eta_{rx} [V_{la}(\frac{d_{emb}}{B})^2 + 3 V_{s} + V_{l}(\frac{L}{B})(1+(\frac{L_{emb}}{B})^2)]$</td>
</tr>
<tr>
<td>$C_{ry,emb}$</td>
<td>Embedded Rocking around y-axis Radiation Damping</td>
<td>$C_{ry} + \rho_s I_{by} \frac{d_{emb}}{L} \eta_{ry} [V_{la}(\frac{d_{emb}}{L})^2 + 3 V_{s} + V_{l}(\frac{L}{B})(1+(\frac{L_{emb}}{L})^2)]$</td>
</tr>
</tbody>
</table>
Fig. 3-20 presents the main elements of the 3D model developed in COMSOL. It matches the geometry and dimensions of the actual bridge. Rayleigh damping was used to describe the damping characteristics of the bridge superstructure. Rayleigh damping is defined as viscous damping and is proportional to a linear combination of mass and stiffness. It is expressed in terms of mass and stiffness as $\zeta = M\alpha_{dM} + K\beta_{dK}$, where $\alpha_{dM}$ is the mass damping parameter, and $\beta_{dK}$ is the stiffness damping parameter. A value of $\beta_{dK} = 0.0065$ was selected to represent the structural damping. $\alpha_{dM} = 0$ was set in the study, since inertial effects are low in low-frequency ranges, and the behavior is more stiffness controlled. In addition, $\beta_{dK}$ was kept constant in both time and frequency domains to maintain compatibility and modeling simplicity. It is a theoretical limitation that the damping loss factor cannot be set as a function of frequency in time domain studies. However, multiple Rayleigh damping ratios at multiple frequencies can lead to more accurate results in the frequency domain. Free tetrahedral elements with adaptive sizing were used to mesh the entire domain. Fig 3-21 illustrates the overall meshed model, in which concrete elements had coarser meshes, while steel elements (girders and bracing) were fine-meshed. This meshing resulted in 1,007,262 degrees of freedom (DoFs).
Fig. 3-21. Overall 3D, side, and bottom view of the bridge model mesh.
Before the chirp loading response analysis, Eigen-mode studies were conducted to estimate the resonant frequencies (mode shapes) of the bridge. The eigenfrequencies are obtained by resolving the displacement field \( u \) which is presented in Eq. 3-5.

\[-\rho \omega^2 u = \nabla \cdot S\]  

Eq. 3-5

where \( S \) is the stress tensor.

The eigenvectors were then scaled with respect to the mass matrix to obtain the directional mass participation factors and expected peak responses.

Following the experimental testing, the model was adjusted to include the obtained shear wave velocity of soil and to conduct time-history and frequency-domain studies. In the time domain study, the displacement field \( u \) with components \( (u,v,w) \) is resolved in the total LaGrangian formulation as shown in Eq. 3-6:

\[
\rho \frac{\delta^2 u}{\delta t^2} + \alpha_{dM} \rho \frac{\delta u}{\delta t} = \nabla \cdot [(FS)^T + \beta_{dK} \frac{\delta (FS)^T}{\delta t}] + F_v
\]

Eq. 3-6

where \( F \) is the deformation gradient tensor, and \( F_v \) is the volume force vector. In the frequency domain the displacement field \( u \) with components \( (u,v,w) \) is resolved in the total LaGrangian formulation, as shown in Eq. 3-7:

\[-\rho \omega^2 u + i \omega \alpha_{dM} \rho u = \nabla \cdot (+i \omega \beta_{dK} S) + F_v e^{i\phi}\]  

Eq. 3-7

where \( \phi \) is the phase shift. To match the experimental program, the time step in the time domain analysis was set to 0.005 s, and the frequency step in the frequency domain analysis was set to 0.0025 Hz. The model is fully parameterized to facilitate parametric sweeps for evaluating various aspects of the super- and substructure on the structural response of the bridge.
3.2.2. *Gate Creek Bridge, Oregon*

Like the Hobson Avenue Bridge, COMSOL Multiphysics was used in this study to produce 3D FEM simulations of one of the tested bridge piers, the supported span, and the surrounding soil volume. The model solves the displacement fields as an output, in a fully dynamic manner. The studies conducted in the simulations were time-dependent studies to match the response from measurements of the steady-state loading and compare them with the results. The dimensions of the model closely match those of the actual bridge, and all inputs for the model were parametrized, allowing for any combination of sweeps as deemed necessary. Solid elements were used in the model, and linear elastic material properties were assigned since the response was in the elastic range due to the controlled vibration levels. Fig. 3-22 shows an isometric overview of the 3D model developed in the current study, while Fig. 3-23 shows the bridge sections, illustrating the embedded footing. The block points on the figures are the observation points (either for loading or response measurements).

![Fig. 3-22. Bridge overview in 3D model.](image)
Table 3-7 provides a summary of the parameters defined in the study. The mass density of soil was selected to be 1900 kg/m$^3$. The shear wave velocity value reported in the table is close to the actual value, as described earlier in this chapter. It can be observed that the errors in the density used are far less consequential on the shear modulus, hence response, compared to the errors in estimating the shear wave velocity since it is a squared quantity. Low-reflecting boundaries were assigned to the side edges of the soil volume to cause the absorption of propagating elastic waves. The bottom edge of the soil was assigned a fixed condition, which represents bedrock, while the top surface was a free edge (can exhibit displacement). As for the deck, the N-S edges (in the longitudinal directions) were assigned conditions representing a simply supported span. The soil was assigned a
Poisson’s ratio of 0.33. For the concrete deck, a modulus of elasticity, density, and Poisson’s ratio of 25 GPa, 2,300 kg/m$^3$, and 0.2 were assigned, respectively. The loading was imposed above the center of the pier, which can be viewed in Fig. 3-22. To represent the mass of the T-Rex, a lumped mass was added to the loading point as well. In addition, gravity loads (self-weight) were included in the study. Fig. 3-24 shows the intensity of gravity loads. T-Rex weighs 64,000 lbs (29 tons).

Table 3-7. Summary of Parameters Included in the Numerical Simulation of Tested Bridge.

<table>
<thead>
<tr>
<th>Label</th>
<th>Expression</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_w$</td>
<td>9 [ft]</td>
<td>2.7432 m</td>
<td>Pier Width</td>
</tr>
<tr>
<td>$P_h$</td>
<td>20 [ft]</td>
<td>6.096 m</td>
<td>Pier Height</td>
</tr>
<tr>
<td>$P_l$</td>
<td>2.5 [ft]</td>
<td>0.762 m</td>
<td>Pier Length</td>
</tr>
<tr>
<td>$F_w$</td>
<td>9 [ft]</td>
<td>2.7432 m</td>
<td>Footing Width</td>
</tr>
<tr>
<td>$F_h$</td>
<td>5 [ft]</td>
<td>1.524 m</td>
<td>Footing Thickness</td>
</tr>
<tr>
<td>$F_l$</td>
<td>15 [ft]</td>
<td>4.572 m</td>
<td>Footing Length</td>
</tr>
<tr>
<td>$P_{cw}$</td>
<td>36 [ft]</td>
<td>10.973 m</td>
<td>Pier Cap Width</td>
</tr>
<tr>
<td>$P_{ch}$</td>
<td>4 [ft]</td>
<td>1.2192 m</td>
<td>Pier Cap Thickness</td>
</tr>
<tr>
<td>$P_{cl}$</td>
<td>2.5 [ft]</td>
<td>0.762 m</td>
<td>Pier Cap Length</td>
</tr>
<tr>
<td>$D_w$</td>
<td>36 [ft]</td>
<td>10.973 m</td>
<td>Deck Width</td>
</tr>
<tr>
<td>$D_h$</td>
<td>8 [in]</td>
<td>0.2032 m</td>
<td>Deck Thickness</td>
</tr>
<tr>
<td>$D_l$</td>
<td>80 [ft]</td>
<td>24.384 m</td>
<td>Deck Length</td>
</tr>
<tr>
<td>Soil$B$</td>
<td>60 [ft]</td>
<td>18.288 m</td>
<td>Soil Width</td>
</tr>
<tr>
<td>Soil$L$</td>
<td>60 [ft]</td>
<td>18.288 m</td>
<td>Soil Length</td>
</tr>
<tr>
<td>Soil$D$</td>
<td>30 [ft]</td>
<td>9.144 m</td>
<td>Soil Height</td>
</tr>
<tr>
<td>$D_f$</td>
<td>8 [ft]</td>
<td>2.4384 m</td>
<td>Footing Depth</td>
</tr>
<tr>
<td>$V_s$</td>
<td>400 [m/s]</td>
<td>400 m/s</td>
<td>Shear Wave Velocity</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>120 [lb/ft$^3$]</td>
<td>1922.2 kg/m$^3$</td>
<td>Soil Density</td>
</tr>
<tr>
<td>$G_{max}$</td>
<td>$\rho_s*(V_s^2)$</td>
<td>1.73E8 Pa</td>
<td>Shear Modulus</td>
</tr>
<tr>
<td>$n_{load}$</td>
<td>1</td>
<td>1</td>
<td>Load Magnitude Modifier</td>
</tr>
</tbody>
</table>
Rayleigh damping was used to describe the damping characteristics of the bridge superstructure. A value of $\beta_{dk} = 0.001$ was selected to represent the structural damping. $\alpha_{dk} = 0$ was set in the study. In addition, $\beta_{dk}$ was kept constant to maintain compatibility and modelling simplicity. Free tetrahedral elements with adaptive sizing were used to mesh the entire domain. The time-domain studies included a time-step of 0.001 s, with a study span from 0-1.4 s. Fig 3-25 shows the meshed Gate Creek FE bridge model when the footing is placed at the surface.
In summary, Chapter 3 provides an overview of the experimental program and numerical simulations performed to achieve the objectives of this research. Sensor layouts and shaking sequence were defined in this chapter for both the Hobson Ave Bridge in NJ and Gate Creek Bridge in Oregon. Furthermore, a description of the 3D FE models created for both bridges was provided, including material properties, geometric parameters, and numerical domains.
CHAPTER 4: RESULTS FROM THE EXPERIMENTAL PROGRAM

Various bridge response results were obtained from the response captured by geophones on several locations on the bridge, at varying load levels, and in varying shaking directions. The results include time histories, transfer functions, and phase angles at multiple locations. The T-Rex loading, and the corresponding bridge response results are presented in this chapter in order to assess dynamic characteristics of the bridge including damping ratios, resonant frequencies, and estimated impedance functions.

4.1 Bridges Response to T-Rex Shaking

To determine the dynamic response following a Single Input Multiple Outputs (SIMO) scheme obtained from bridge shaking, the following signals are defined per Tables 3-1 and 3-2. The signal recorded \(X_x\) is the time history of \(T_{\text{span}} = 32 \text{s}\) at an increment \(dt = 0.005 \text{s}\) (corresponding to a sampling frequency of 200 Hz) from the \(x^{th}\) channel (from sensors). Then, for spectrum processing, \(X_x\) is multiplied with \(W_X\) (window function), from which the complex spectrum \((S_x)\) is obtained. After that, Equations 4-1 and 4-2 were used to evaluate the spectra:

\[
S_{xx} = |S_x|^2 = (\text{FFT} |X_x \cdot W_x|)^2
\]

Eq. 4-1

where \(S_{xx}\) is the auto (power) spectrum from channel \(x\).

\[
S_{xy} = S_x \cdot S_y^*
\]

Eq. 4-2

where \(S_{xy}\) is the complex cross-spectrum between inputs from channel \(x\) and channel \(y\), and the asterisk denotes the complex conjugate.

Subsequently, the auto (power) spectrum of input \(X_x\) \((G_{xx})\), which is the ensemble-average of \(S_{xx}\), and the (complex) cross-spectrum between inputs \(X_x\) and \(X_y\) \((G_{xy})\), which is the
ensemble-average of \( S_{xy} \) are computed. Finally, the transfer functions and coherence are computed as shown in Equations 4-3 and 4-4 respectively:

\[
H_{xy} = \frac{G_{xy}}{G_{xx}} \quad \text{Eq. 4-3}
\]
where \( H_{xy} \) is the (complex) transfer function between input \( X_x \) and output \( X_y \).

\[
C_{xy} = \frac{|G_{xy}|^2}{G_{xx} \times G_{yy}} \quad \text{Eq. 4-4}
\]
where \( C_{xy} \) is the ordinary coherence function between inputs \( X_x \) and \( X_y \) which is computed from the average of multiple signals.

### 4.1. Hobson Avenue Bridge, New Jersey

The primary goal of shaking the Hobson Avenue Bridge was to perform structural identification (St-Id) to obtain resonant frequencies and modal damping. Experimental Modal Analysis (EMA) was performed from signals measured in the time domain and processed to produce various outputs as defined in the introduction of this chapter. The testing was conducted following the program defined in Chapter 3. Fig. 4-1 shows a typical time history obtained for bridge response from various signals, with the caption providing information about loading direction and response measured. Response from one side of the bridge is shown as an example to avoid a cluttered representation. Fig. 4-2 shows the corresponding power spectra (\( G_{xx} \)) for the time histories shown in Fig. 4-1. Power spectra are helpful to show at which frequencies the energy of a time-varying signal is concentrated. Like Fig. 4-2, Fig. 4-3 shows the response of the bridge due to horizontal shaking at several observation points to illustrate the overall response to the load, and with the ground geophone signal amplified x10 times. It can be observed that the load started to diverge from the 21 kips loading level when the frequency was reduced, starting from \(~3.5\) Hz down to \(~1\) Hz. This led to a slight loss of testing accuracy in the same range.
Fig. 4-1. Transverse response time histories from sensor locations mentioned in the legend to transverse shaking at 12 kips with T-Rex centered above the pier.

Fig. 4-2. Transverse power spectra from sensor locations mentioned in the legend to transverse shaking at 12 kips with T-Rex centered above the pier.

Fig. 4-3. Average transverse response of several observation points due to horizontal shaking @21 k, with ground/foundation geophone signal amplified.

4.1.1. Identification of Resonant Frequencies

To identify resonant frequencies of the soil-foundation-bridge system, the spatial variation of the response was examined under vertical and transverse highest loading
scenarios when T-Rex was placed above the bent and at the mid-span. The above-bent geophones were crucial for identifying the transverse and rocking mode shapes. This is because the bent is the most compliant region of the bridge for rocking/swaying motion. Peak picking was used to extract natural frequencies. Fig. 4-4 depicts the power spectra obtained at various locations on the deck, above bent and at the mid-span, and on the bent cap, under vertical and transverse loading. The testing employed a reversed sweep, i.e., the load frequency was decreasing during the test. Seven distinct natural frequencies of the bridge were identified. The modes can be easily identified from the far-apart peaks, as shown in Fig 4-4(b). However, the complex motion of the bridge occurs in the 4-5 Hz range due to closely spaced peaks. As shown in Fig. 4-4(a) and (b), the first peak encountered occurs at 4.37 Hz. A peak in the vertical response occurs at 4.49 Hz, as highlighted in Fig 4-4(c), which is driven by the high energy vertical vibration of one of the bridge modes even though the shaking is horizontal, revealing a vertical mode of vibration. This also corresponds to the valley in Fig. 4-3(b). As the bridge vibrates and goes through this resonance, another peak of the dominant horizontal motion occurs at 4.37 Hz until the other natural frequency is attained at 4.61 Hz. This is deduced from the time history trace of the excitation and matching the time with T-Rex frequency.
Fig. 4-4. (a) Transverse response due to transverse loading above the pier, (b) vertical response due to vertical loading, and (c) vertical power spectra due to transverse loading. (1 cm/s = 0.39 in/s)
The results of the bridge response are plotted in and provided in Appendix C for the highest load scenarios as illustrated in Fig. C-1 through Fig. C-32. Table 4-1 provides a summary of the main parameters from the various plots in Appendix C. It describes the overall dynamic behavior of the bridge through phase information, response amplitudes from power spectra, and Transfer Function (TF) ratios. TF ratios in Table 4-1 are ratios of the Frequency Response Functions (FRF) at a certain geophone (estimated from Eq 4.3) to the FRF of the reference geophone (which selected as the west deck geophone). The forced vibration above the bent was beneficial in identifying the lateral modes, which were the focus of this research because they highlight DSSI effects more profoundly. The placement of T-Rex above the middle of one of the spans during horizontal shaking, on the other hand, confirmed the vertical mode that exhibited response amplitudes higher at the deck level compared to other sensor locations. From the configuration pairs (1+6), (2+7) and (3+8) shown in Table 4-1, which describe the motion of the deck, midspan, and pier respectively, it is evident that the bridge is undergoing rocking since the transverse sensors are in phase, while the vertical sensors are out-of-phase for the entire frequency range. In addition, the TF ratio between sensors in (1+6) is ~1 which means it is a symmetrical motion/rotation of the bridge deck, while some asymmetry was observed in (2+7) and (3+8). This can also describe the motion of the bridge for the configuration pairs (16+21) and (18+23), where T-Rex is placed above the midspan.
Table 4-1. Summary of Response from Highest Load Level Scenarios in Various Configurations.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Loading Location</th>
<th>Shaking Direction</th>
<th>Load level [k]</th>
<th>Response</th>
<th>Peak frequency(s) [Hz]</th>
<th>Peak Amplitude(s) [in/s]^2</th>
<th>TF Ratio</th>
<th>Phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Above Pier</td>
<td>Transverse</td>
<td>21</td>
<td>Deck - Transverse</td>
<td>4.37, 4.61</td>
<td>0.048, 0.051</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>2</td>
<td>Deck</td>
<td>Transverse</td>
<td></td>
<td>Midspan - Transverse</td>
<td>4.37, 4.64</td>
<td>0.039, 0.04</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>3</td>
<td>Pier</td>
<td>Transverse</td>
<td></td>
<td>Pier - Transverse</td>
<td>4.37, 4.64</td>
<td>0.04, 0.045</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>4</td>
<td>Foundation</td>
<td>Transverse</td>
<td></td>
<td>Foundation - Transverse</td>
<td>4.34, 4.64</td>
<td>0.0035, 0.004</td>
<td>Varies</td>
<td>Minor phase shifts</td>
</tr>
<tr>
<td>5</td>
<td>Foundation*</td>
<td>Transverse</td>
<td></td>
<td>Foundation* - Transverse</td>
<td>4.54</td>
<td>0.0017</td>
<td>Varies</td>
<td>Minor phase shifts</td>
</tr>
<tr>
<td>6</td>
<td>Deck</td>
<td>Vertical</td>
<td></td>
<td>Deck - Vertical</td>
<td>4.47</td>
<td>0.013</td>
<td>~1</td>
<td>Completely out-of-phase</td>
</tr>
<tr>
<td>7</td>
<td>Midspan</td>
<td>Vertical</td>
<td></td>
<td>Midspan - Vertical</td>
<td>2.71, 3.34, 4.49</td>
<td>0.0183, 0.025, 0.112</td>
<td>Varies</td>
<td>Significantly out-of-phase</td>
</tr>
<tr>
<td>8</td>
<td>Pier</td>
<td>Vertical</td>
<td></td>
<td>Pier - Vertical</td>
<td>4.27, 4.49</td>
<td>0.002, 0.007</td>
<td>Varies</td>
<td>Significantly out-of-phase</td>
</tr>
<tr>
<td>9</td>
<td>Foundation</td>
<td>Vertical</td>
<td></td>
<td>Foundation - Vertical</td>
<td>4.37, 4.71</td>
<td>0.006, 0.002</td>
<td>~2</td>
<td>In-phase</td>
</tr>
<tr>
<td>10</td>
<td>Foundation*</td>
<td>Vertical</td>
<td></td>
<td>Foundation* - Vertical</td>
<td>4.44, 4.71</td>
<td>0.006, 0.002</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>11</td>
<td>Deck</td>
<td>Vertical</td>
<td>12</td>
<td>Deck - Vertical</td>
<td>4.17</td>
<td>0.001</td>
<td>Varies</td>
<td>In-phase</td>
</tr>
<tr>
<td>12</td>
<td>Midspan</td>
<td>Vertical</td>
<td></td>
<td>Midspan - Vertical</td>
<td>2.75, 3.39, 4.2, 8.27, 8.96</td>
<td>0.007, 0.002, 0.007, 0.006, 0.013</td>
<td>Varies</td>
<td>Significantly out-of-phase</td>
</tr>
<tr>
<td>13</td>
<td>Pier</td>
<td>Vertical</td>
<td></td>
<td>Pier - Vertical</td>
<td>2.76, 4.17</td>
<td>0.0001, 0.001</td>
<td>~1 except at resonance</td>
<td>Minor phase shifts</td>
</tr>
<tr>
<td>14</td>
<td>Foundation</td>
<td>Vertical</td>
<td></td>
<td>Foundation - Vertical</td>
<td>4.17</td>
<td>0.0007</td>
<td>Varies</td>
<td>In-phase</td>
</tr>
<tr>
<td>15</td>
<td>Foundation*</td>
<td>Vertical</td>
<td></td>
<td>Foundation* - Vertical</td>
<td>4.17</td>
<td>0.001</td>
<td>~0.7</td>
<td>In-phase</td>
</tr>
<tr>
<td>Configuration</td>
<td>Loading Location</td>
<td>Shaking Direction</td>
<td>Load level [k]</td>
<td>Response</td>
<td>Peak frequency(s) [Hz]</td>
<td>Peak Amplitude(s) [in/s^2]</td>
<td>TF Ratio</td>
<td>Phase</td>
</tr>
<tr>
<td>---------------</td>
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<td>------------------</td>
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<td>----------</td>
<td>------------------------</td>
<td>-----------------------------</td>
<td>----------</td>
<td>---------------------</td>
</tr>
<tr>
<td>16</td>
<td>Transverse</td>
<td>Above Midspan</td>
<td>15</td>
<td>Deck - Transverse</td>
<td>4.39, 4.76</td>
<td>0.039, 0.027</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>17</td>
<td>Midspan - Transverse</td>
<td></td>
<td></td>
<td>Midspan - Transverse</td>
<td>4.39, 4.76</td>
<td>0.033, 0.021</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>18</td>
<td>Pier - Transverse</td>
<td></td>
<td></td>
<td>Pier - Transverse</td>
<td>4.39, 4.76</td>
<td>0.032, 0.024</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>19</td>
<td>Foundation - Transverse</td>
<td></td>
<td></td>
<td>Foundation - Transverse</td>
<td>4.39, 4.76</td>
<td>0.0025, 0.002</td>
<td>Varies</td>
<td>In-phase</td>
</tr>
<tr>
<td>20</td>
<td>Foundation - Transverse</td>
<td></td>
<td></td>
<td>Foundation - Transverse</td>
<td>4.39, 4.76</td>
<td>0.0025, 0.002</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>21</td>
<td>Deck - Vertical</td>
<td></td>
<td></td>
<td>Deck - Vertical</td>
<td>3.42, 4.47</td>
<td>0.0025, 0.012</td>
<td>~1</td>
<td>Completely out-of-phase</td>
</tr>
<tr>
<td>22</td>
<td>Midspan - Vertical</td>
<td></td>
<td></td>
<td>Midspan - Vertical</td>
<td>3.34, 4.49</td>
<td>0.044, 0.078</td>
<td>~0.5</td>
<td>Minor phase shifts</td>
</tr>
<tr>
<td>23</td>
<td>Pier - Vertical</td>
<td></td>
<td></td>
<td>Pier - Vertical</td>
<td>4.15, 4.51, 4.61</td>
<td>0.001, 0.004, 0.0036</td>
<td>Varies</td>
<td>Significantly out-of-phase</td>
</tr>
<tr>
<td>24</td>
<td>Foundation - Vertical</td>
<td></td>
<td></td>
<td>Foundation - Vertical</td>
<td>4.47</td>
<td>0.007</td>
<td>~1.5</td>
<td>In-phase</td>
</tr>
<tr>
<td>25</td>
<td>Foundation* - Vertical</td>
<td></td>
<td></td>
<td>Foundation* - Vertical</td>
<td>4.44</td>
<td>0.007</td>
<td>~1</td>
<td>Completely out-of-phase</td>
</tr>
<tr>
<td>26</td>
<td>Deck - Vertical</td>
<td></td>
<td></td>
<td>Deck - Vertical</td>
<td>2.51, 3.13, 4.13, 4.49, 8.81</td>
<td>0.0005, 0.0008, 0.004, 0.0014, 0.0088</td>
<td>~0.25</td>
<td>Minor phase shifts</td>
</tr>
<tr>
<td>27</td>
<td>Midspan - Vertical</td>
<td></td>
<td></td>
<td>Midspan - Vertical</td>
<td>NA</td>
<td>NA</td>
<td>Varies</td>
<td>Completely out-of-phase</td>
</tr>
<tr>
<td>28</td>
<td>Pier - Vertical</td>
<td></td>
<td></td>
<td>Pier - Vertical</td>
<td>2.49, 4.12, 8.81</td>
<td>0.0004, 0.005, 0.005</td>
<td>~1</td>
<td>In-phase</td>
</tr>
<tr>
<td>29</td>
<td>Foundation - Vertical</td>
<td></td>
<td></td>
<td>Foundation - Vertical</td>
<td>2.49, 4.12, 8.81</td>
<td>0.0004, 0.005, 0.004</td>
<td>Varies</td>
<td>In-phase</td>
</tr>
<tr>
<td>30</td>
<td>Foundation* - Vertical</td>
<td></td>
<td></td>
<td>Foundation* - Vertical</td>
<td>2.49, 4.12, 8.81</td>
<td>0.0003, 0.003, 0.002</td>
<td>~1</td>
<td>In-phase</td>
</tr>
</tbody>
</table>

All TF ratios are calculated with respect to the deck. Fields denoted with * are calculated with respect to the other ground/foundation sensor.
As illustrated in Table 4-1, multiple peaks were occurring at slightly varying frequencies. For instance, one of the resonant frequencies observed from configuration 1 is 4.37 Hz, while the same mode is resonating at 4.39 Hz from configuration 16. Another instance is where the second peak from configuration 1 is at 4.61 Hz, while it is at 4.76 Hz from configuration 16. Eigenmodes are a function of intrinsic properties of the structure (mass, stiffness, and damping), and force amplitude or location should not affect their determination. In addition, there are three closely-spaced modes (nominally 4.35 Hz, 4.51 Hz, and 4.61 Hz) that inevitably would influence each other, so peak picking can be inaccurate. For sparse modes, peak picking and damping estimation using the half-power method can be used to determine resonant frequencies and damping, respectively. Therefore, an additional step was conducted to provide a better determination of the resonant frequencies and damping. The Rational Fraction Polynomial (RFP) Method was utilized to extract the mode shapes. This was done by curve fitting the Frequency Response Functions (FRFs) of the time histories in the modal analysis software Artemis Modal Pro. A dynamic model is fit through the data within a given order, and the order is increased until an adequate fit is achieved (Omar et al., 2010). However, increasing the model order can introduce noise modes since bridges are continuous systems that have infinite modes shapes; but those could be identified and eliminated since they are trivial modes. Fig. 4-5 shows the stabilization diagram and detection of two stable modes when the lateral responses are considered. The boxed portions are artificial modes arising from over-fitting. Similarly, Fig. 4-6 shows the stabilization diagram for vertical response and the detection of the additional middle-frequency mode. The mode in Fig 4-6 occurring at 4.69 Hz is a repeated mode.
In addition, a Modal Complexity Factor (MCF) given by Eq. 4-5 was estimated for the three closely spaced mode shapes. The MCF values are between 0-1, with values closer to 1 indicating a real-valued eigenmode, while values closer to 0 indicate a complex mode, which can suggest the presence of nonclassical damping (Greś et al., 2021).
\[ MCF(\varphi) = \frac{(L_{xx} - L_{xx})^2 + 4L_{xx}^2}{(L_{xx} + L_{xx})^2} \]  

Eq. 4-5

where \( L_{xx} = \text{Re}(\varphi)^T \times \text{Re}(\varphi) \), \( L_{xy} = \text{Im}(\varphi)^T \times \text{Im}(\varphi) \), and \( L_{xy} = \text{Re}(\varphi)^T \times \text{Im}(\varphi) \) are scalar products from the mode shape \( \varphi \).

4.1.2. Estimation of Damping and Impedance Functions

The estimation of damping is a subsequent step after determining the resonant frequencies in EMA. Table 4-2 summarizes the findings from the EMA performed on Hobson Avenue Bridge. The resonant frequencies obtained experimentally will be the basis for comparison with eigenmodes from the numerical simulations presented in the following chapters. The complexity of modes 3-5 is a result of them being closely spaced and an indication of non-proportional damping in the soil-foundation-structure system. In addition, the obtained damping in Table 4-2 represents the structural damping of the deck. Other peaks in Table 4-1 represent Operational Deflected Shapes (ODS) which can vary with force amplitude, location, and direction. It is noteworthy to mention that the bridge exhibited higher vertical rigidity than lateral rigidity as observed from the non-proportionally lower vertical responses relative to the load levels applied. The MCFs of modes 1, 2, 6, and 7 were neglected since those modes are of lesser importance.

Table 4-2. Summary of the EMA and Modal Parameters Obtained from Shaking Hobson Avenue Bridge.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Resonant Frequency (Hz)</th>
<th>Determination Method</th>
<th>Damping ( \xi_\varphi ) (%)</th>
<th>Determination Method</th>
<th>MCF (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.75</td>
<td>Peak Picking</td>
<td>1.74</td>
<td>Half-power</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>3.39</td>
<td>Peak Picking</td>
<td>1.18</td>
<td>Half-power</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>4.39</td>
<td>RFP Model</td>
<td>3.71</td>
<td>RFP Model</td>
<td>35.909</td>
</tr>
<tr>
<td>4</td>
<td>4.44</td>
<td>RFP Model</td>
<td>3.73</td>
<td>RFP Model</td>
<td>22.375</td>
</tr>
<tr>
<td>5</td>
<td>4.69</td>
<td>RFP Model</td>
<td>3.67</td>
<td>RFP Model</td>
<td>20.249</td>
</tr>
<tr>
<td>6</td>
<td>8.27</td>
<td>Peak Picking</td>
<td>1.49</td>
<td>Half-power</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>8.81</td>
<td>Peak Picking</td>
<td>2.41</td>
<td>Half-power</td>
<td>-</td>
</tr>
</tbody>
</table>
Damping of the soil-foundation-structure system at a given mode, whether it was rotational or translational be described in the classical manner as composite damping, consisting of structural, soil radiation, and soil material damping since the response measured is a total response considering DSSI effects, which is described in Eq. 4-6.

\[ \xi_{\text{composite}} = \xi_{\text{rad}} + \xi_{\text{g}} + \xi_{\text{s}} \]  

Eq. 4-6

where \( \xi_{\text{rad}} \) is radiation (geometric) damping, \( \xi_{\text{g}} \) is soil (material) hysteretic damping, and \( \xi_{\text{s}} \) is structural (hysteretic) damping.

The structural damping has been determined and estimated in Table 4-2, so the ground and radiation damping are next to be determined. Like the SASW method, T-Rex was used as a continuous excitation generator for surface waves when it was shaking the bridge above the pier, which were captured with the ground geophone array described in Chapter 3. Fig 4-7 illustrates the testing setup, with T-Rex applying the reverse linear chirp from 15-1 Hz on top of the bridge deck above the pier.

![Testing setup for determination of ground damping and attenuation.](image)

For the determination of ground damping the Bornitz equation, given in Eq. 4-5 is utilized first to estimate the attenuation coefficient.
\[ A_2 = A_1 \left( \frac{r_1}{r_2} \right)^n e^{-\alpha (r_1-r_2)} \]  

Eq. 4-7

where \( A_{1,2} \) is the amplitude at receiver 1 and 2 respectively (in/s), \( r_{1,2} \) distances (ft) from the source to receivers 1 and 2 respectively, \( \alpha \) is the coefficient of attenuation (1/ft), and \( n \) is an exponent depending on the type of wave (\( n=0.5 \) for surface waves, \( n=2 \) for body waves).

As depicted in Fig. 4-7, the vertical projection of the foundation edge at the ground level was used as the source of the wave generation for simplicity. In addition, since surface (Rayleigh) waves are the main waves traveling in such setups (similar to SASW), the exponent \( n \) takes a value of 0.5 (square root relationship). The Bornitz equation provides geometric/radiation damping (carried by the square root term) and material damping (carried by the exponential term). Therefore, as the distance between the receivers decreases, the damping becomes predominantly influenced by the radiation term. More details about the spacing, testing setup and typical instrumentation can be found in (Nazarian et al., 1983). The acceptable separation between sensors when it is equal to the distance to the source is typically \( 0.33\lambda_{ph} < X < 2\lambda_{ph} \), where \( \lambda_{ph} \) the phase wavelength is approximately \( 0.9V_s/f \) for a soil layer of constant shear wave velocity. For the frequency range 1-15 Hz, the setup does not satisfy this condition below 5 Hz for the shear wave velocity \( V_s = 200 \) m/s, as determined in Chapter 3. The 5’ and 10’ spacings are rejected for the entire frequency range, the 20’ spacing is acceptable for 10-15 Hz, and the 40’ is acceptable for 5-15 Hz. Fig. 4-8 shows the general mode shape of the wave propagation as measured from the ground geophone array. It shows that the wave attenuates with distance as expected, across the testing frequency range. Fig 4-9 shows a typical longitudinal cross-section of Fig 4-8. The exponential decay can be observed in Fig 4-9 more clearly.
Fig. 4-8. Ground response to T-Rex shaking atop the bridge pier with as function of frequency and distance away from the source.

Fig. 4-9. Longitudinal cross-section of Fig. 4-8 at 7.5 Hz.
Since there is an overlap between the frequency ranges (between 10-15 Hz) for the 20’ and 40’ spacings, the minimum value of $\alpha$ was selected. Fig. 4-10 shows the attenuation coefficient $\alpha$ as a function of frequency, which shows an increasing trend, which is in agreement with typical findings in the literature in addition to reported values in this frequency range. Another way $\alpha$ can be obtained is through curve-fitting an exponential function through Fig 4-9. However, this is only applicable if the measurements are far enough from the source so that near-field effects do not influence the response. Therefore, both values will not match in the current test setup. With the knowledge of $\alpha$, the soil hysteretic damping $\xi_g$ can be determined from the relationship shown in Eq. 4-8, for which the graph is plotted in Fig. 4-11. The damping values ranged between 1%-25%, with an average value of 10%. The average value was selected to represent the soil hysteretic damping throughout the frequency range. since it is closer to the expected values for soils with this shear wave velocity.

$$\xi_g = \frac{0.9aV_s}{2\pi f}$$  \hspace{1cm} Eq. 4-8

![Fig. 4-10. Determination of soil attenuation coefficient $\alpha$.](image)
To determine radiation damping $\xi_{rad}$, inertial DSSI effects are used when considering a particular mode of shaking. For this experiment, the rocking behavior of the soil-foundation-structure system was emphasized and examined. Radiation damping can be estimated from the total response of the system, from which composite damping is determined, then Eq. 4-6 can be rearranged to determine $\xi_{rad}$. It is an underlying assumption that material damping (for both structure and soil) is constant with frequency and contributes equally to the total response for all modes of vibration.

The complex-valued mechanical impedance in the form $S(\omega) = K(\omega) + i\omega C(\omega)$ is the inverse of the flexibility TF (FRF) defined in Eq. 4-3. Since the mobility TFs were measured in this experiment, they were converted to flexibility by dividing them by $-\omega$, assuming a linear system. The most representative geophones to be used for this purpose would be the ground geophones above the footing. Nevertheless, they were excluded from the evaluation when the measurements were inspected. This is mainly due to their free placement next to the pier rather than being attached/bolted to it. Therefore, future
experiments should consider this preparation step, or if possible, excavate and attach the sensors to the foundation. However, the rocking response is less sensitive than swaying since rotations are the same for the entire system. This is because the soil-foundation-structure system is continuous and symmetric, and the experiment is in the linear range, the next closest sensors can be used, which were from the pier. Fig. 4-12 shows the impedance functions for pier rocking throughout the tested frequency range.

Fig. 4-12. Mechanical impedance of Hobson Avenue Bridge for the rocking mode, consisting of (a) real part (stiffness) and (b) imaginary (damping).

As expected, the velocity response is maximum at resonance. Therefore, damping forces within the system significantly increase. As shown from the modal analysis, there is a
presence of nonproportional damping in the system. Hence, estimating the composite damping ratio ($\xi_{\text{composite}}$) by treating the bridge as an SDOF oscillator will be erroneous (the system becomes overdamped), even more so when the foundation is embedded. Alternatively, the damping ratio can be estimated from the mechanical impedance using its modulus and argument as shown in Eq 4-9.

$$\xi_{\text{composite}} = \frac{C/\omega}{\sqrt{K^2 + (\frac{C}{\omega})^2}}$$  

Eq. 4-9

Evaluating damping for the lateral modes occurring at 4.39 Hz and 4.69 Hz, yields $\xi_{\text{composite}}$ 0.59 and 0.33. In turn, those yield $\xi_{\text{rad}}$ of 0.45 and 0.19 respectively, which are higher than typical values reported for embedded foundations (Gazetas, 1981, 1983, 1991). This can be attributed to significant coupling with lateral-swaying and vertical modes. Nevertheless, the mode at 4.69 Hz showed a more pronounced rocking motion than the mode at 4.39 Hz. This resulted in the value of 0.19 that is closer to typical damping reported for rocking vibrations.

The St-Id of Hobson Avenue Bridge showed that there were 3 distinct closely-spaced natural frequencies determined as 4.39 Hz, 4.44 Hz, and 4.69 Hz, respectively. An examination of those modes revealed that they are complex valued through the evaluation of MCFs. In addition, the rocking behavior of the bridge was captured through evaluating the bridge response obtained from several sensors and the bridge. Phase information, peak amplitude, and TF ratios facilitated the description of the dynamic bridge motion. Lastly, the rock behavior was emphasized by evaluating the rotational mechanical impedance functions. This enabled the evaluation of composite damping, along with structural, radiation, and soil damping.
4.2. Gate Creek Bridge, Oregon

Testing conducted on the Gate Creek Bridge was mainly focused towards obtaining vertical impedance functions of the soil-foundation-structure system. However, $\xi_{\text{composite}}$ cannot be used to estimate $\xi_{\text{rad}}$ since there was no geophone array on the ground. Various bridge response results were obtained from the geophone records. The results include time histories, transfer functions, and phase angles at various locations. The T-Rex loading, and the bridge response results are presented in this section. T-Rex was placed above the centerline of the piers in all tests, to eliminate potential effects of loading eccentricity. Fig. 4-13 shows T-Rex with the baseplate lowered right above the center of one of the tested piers. Results herein show are for one of the tested piers (Pier 1).

![T-Rex loading one of the tested piers of the bridge.](image)

The first set of shakes were done in with a linear chirp loading function, in which the frequency was changing at a constant rate over a span of 16 seconds in a frequency range of 10-80 Hz. This was done to identify the natural frequencies of the bridge. Fig. 4-14 shows the response of a geophone mounted on a pier near the footing and on deck (#19 and #6 respectively in Fig. 3-11) to chirp loading under various load levels. The response of the deck is orders-of-magnitude higher than that of the footing at lower load levels, but
this gap diminishes as the load increases. This is illustrated in Fig. 4-4, which shows the response for both locations when the load was increased to 36 k [160 kN], which was the highest load applied in the chirp loading. The maximum response at the deck level was ~1.2 in/s [30.48 mm/s], compared to ~0.3 in/sec [7.62 mm/s] at the footing level. This means that there is a better load transfer mechanism and adequate engagement of the super- and substructure as opposed to lower magnitude loads. This is a significant advantage of using large-amplitude mobile shakers for exploring dynamic features of soil/structure systems as opposed to other low-level load conventional methods, such as traveling vehicles. To determine the natural frequencies, the power spectra of the time histories were obtained through Fast Fourier Transforms (FFTs). The records of the highest load magnitude were used in this step, which is presented in Fig. 4-15.
Fig. 4-14. Bridge response to chirp loading under various load levels at (a) the footing and (b) the deck level.

Fig. 4-15. Response of footing and deck to a 36-k chirp loading function (80-10 Hz)
4.2.1. Identification of Resonant Frequencies

Additional sweeps, as indicated in Table 3-2, were carried out to identify resonances in lower frequency ranges (3-15 Hz), as shown in Fig. 4-16. Fig. 4-17 shows the power spectra at (a) the deck and (b) footing for the 10-80 Hz range, while Fig. 4-18 shows those for the 3-15 Hz range. From Fig. 4-17 and 4-18, the peaks in the plots represent resonances, or natural frequencies, of the soil/structure system. They are at 9.1 Hz, 14.8 Hz, and 26.2 Hz, respectively. However, the velocity magnitude around the 26.2 Hz resonance is the greatest. It is established that this frequency is the primary mode of vibration (or 3rd vertical mode). Hence, Table 3-2 shows the selected frequency to capture the response of the bridge around the primary resonance by approximating the system as a single degree of freedom (SDOF) system. This is crucial for establishing a site response analysis for displacement, velocity, and acceleration. In such a manner, the maximum response of each system will occur at different frequencies around the resonance.

In the following set of tests, a steady-state shaking was conducted as defined in Table 3-2 which describes the loading sequence and force magnitude. The load magnitude for this set of tests was 24 k. Fig. 4-19 shows the steady-state response at the footing level of the bridge shaking at 26 Hz. This was the highest velocity response across all frequencies. Therefore, this record was used as a benchmark to validate the FEM model and confirm numerical results, as will be discussed in the next section.
Fig. 4-16. Response of footing and deck to a 36 k chirp loading function (15-3 Hz)

Fig. 4-17. Power spectra of response due to a chirp loading function 10-80 Hz at (a) deck and (b) footing.

Fig. 4-18. Power spectra of response due to a chirp loading function 3-15 Hz at (a) deck and (b) footing.
4.2.2. Estimation of Vertical Impedance Functions

The vertical impedance functions were obtained for the Gate Creek Bridge in the 3-15 Hz and 10-80 Hz frequency ranges, which are depicted in Fig. 4-20. In contrast to the Hobson Avenue Bridge, the damping for the Gate Creek Bridge was estimated based on SDOF representation of the bridge as described in Chapter 2, since only vertical motion is taking place. The composite vertical damping ($\xi_{\text{composite}}$) estimated at the resonant frequencies 9.1 Hz, 14.8 Hz, and 26.2 Hz as 0.363, 0.75, and 0.175. The damping ratios are within expected values except the 0.75 which is higher than the other ratios unpredictably.
Fig. 4-20. Mechanical impedance of Gate Creek Bridge for the vertical mode, consisting of (a) real part (stiffness) and (b) imaginary (damping) for the 3-15 Hz range, and of (c) real part (stiffness) and (d) imaginary (damping) for the 10-80 Hz range.
4.3. T-Rex as a Global Evaluation NDE Tool

An important aspect of the experimental program was to assess T-Rex as an NDE tool for the global dynamic characterization of bridges. Several attributes were examined to ensure the applicability of field deployment of T-Rex for that purpose. The peak response amplitude (at resonance) of shaking, regardless of direction, in both Hobson Avenue and Gate Creel bridges was limited to 1 in/s (2.54 cm/s). This was enforced to achieve meaningful response levels that can inform lead to representative DSSI behavior while maintaining NDT requirements. Qualitatively, this response was easily noticeable while the testing crew was on the bridge, but not severe or troublesome.

TF ratios representing the ratio of the FRF evaluated at a particular geophone to the FRF of the reference geophone were evaluated to assess the effect of changing the load magnitude. Fig. 4-21 illustrates the transfer function ratios between the east and west sides of the bridge above the pier at several load levels of vertical shaking. As the power input increased, the TF clearly increased, evidenced by the reduction in undulations in both the amplitudes and phase angles, due to a higher signal-to-noise ratio. This allows for clearer identification of resonant frequencies, promoting the use of large mobile shakers, as opposed to conventional methods that rely on small shakers, ambient vibrations, wind, or temperature changes.
Fig. 4-21. Transfer functions and phase angles of vertical response between east and west side of the deck above the pier due to vertical load @ (a) 13.3 kN [3 k], (b) 26.7 [6 k], (c) 40 kN [9 k], (d) 53.4 kN [12 k].
Furthermore, coherence functions were calculated for each shaking configuration at varying load levels. Fig. 4-23 and 4-24 show coherence functions estimated at various locations on the bridge, for transverse response to transverse shaking, and vertical response to vertical shaking, respectively. The plots represent the lowest, mid, and highest-level loading scenarios conducted during the experimental program. The lowest level (3 kips) led to a significant loss of coherence throughout the entire frequency range. This is even more pronounced for foundation response. Increasing the driving force led to increasing coherence; hence, a better correlation between force and response. Despite being arbitrarily defined, a value of 0.9 is commonly accepted as a good correlation, while values below indicate improper instrumentation or noise contamination. This is crucial for qualifying excitation methods described in the literature for the St-Id of bridges. A large-amplitude hydraulic shaker such as T-Rex imparting 3 k did not generate acceptable load levels to inform DSSI or dynamically characterize the bridge. Subsequently, methods that are expected to generate even lower levels in a more transient manner are to be scrutinized. Such methods including ambient vibration and passing traffic are expected to exhibit substantially lower response levels than what is presented in the current study. Hence, those methods inherently would incorporate considerable extrapolation and noise that would affect inferring dynamic characteristics accurately.
Fig. 4-22. Coherence functions of transverse response to transverse shaking with T-Rex the above pier at the (a) deck, (b) pier, (c) midspan, and (d) foundation.
Fig. 4-23. Coherence functions of vertical response to vertical shaking with T-Rex the above pier at the (a) deck, (b) pier, (c) midspan, and (d) foundation.
To conclude, Chapter 4 provides a concise summary of the dynamic response to large amplitude shaking. Several outputs were evaluated at multiple locations on both bridges. The results enabled the assessment of dynamic characteristics and modal parameters, including overall dynamic response, resonant frequencies, damping, and impedance functions. The use of T-Rex at varying load levels revealed quantitatively what are required load levels to excite bridges sufficiently to produce adequate response that is correlated to the imparted loads. This was done through evaluation of coherence functions and TF ratio evaluations. This highlighted the need for such testing methods to examine the global bridge dynamic response and infer its dynamic features clearly and unambiguously.
CHAPTER 5: NUMERICAL SIMULATIONS AND PARAMETRIC STUDIES OF HOBSON AVENUE BRIDGE

FEM models were developed to capture the effects of DSSI on dynamic response. COMSOL Multiphysics software was used to produce both 2D and 3D FEM simulations of the bridge response due to a chirp signal dynamic loading. The purpose of the models was to examine the effects of DSSI on the stress levels in bridge components. Particularly, the bridge deck and the possible impacts on its durability was of interest. In both FEM models, linear elastic material properties were selected since the load was in the elastic range. To incorporate the effect of DSSI, the foundation-soil system was modeled as a system of translational and rotational frequency-dependent springs and dashpots, the impedance functions. In addition, parametric sweeps of super- and substructural features of the soil-foundation-bridge (SFS) system were conducted to assess the difference between various SFS configurations.

5.1 Numerical Simulation of Hobson Avenue Bridge and Model Validation

To evaluate the effects of DSSI on bridge response, two FEM models were developed. The first model incorporated DSSI through the inclusion of impedance functions on the foundation level, while the second was a fixed base bridge model.
5.1.1. **Eigenfrequency Analysis**

An eigenfrequency analysis was conducted to identify the Eigenvalues (natural frequencies) and Eigenvectors (mode shapes) of the bridge from the DSSI-incorporating model, and to compare those with the modes at frequencies in Table 4-2. Fig. 5-1 shows the identical mode shapes extracted from both numerical models, which were normalized with respect to the mass matrix.

Table 5 illustrates the comparison of the eigenfrequencies from the bridge shaking with those from the DSSI-incorporating model and the fixed-base model. It can be observed that dashpot/damping assigned to the footings resulted in complex-valued eigenmodes, which show that the modes shapes exhibit some complexity. This is the case because whenever damping is included in a FEM model, eigenmodes become complex. On the other hand, the fixed-base model exhibited real mode shapes.

Table 5-1. Eigenfrequencies Obtained from Numerical Simulations of DSSI-incorporating and Fixed-base Models.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Resonant Frequency – Experimental (Hz)</th>
<th>Resonant Frequency* – DSSI (Hz)</th>
<th>Resonant Frequency ** – Fixed-base (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.75</td>
<td>2.9785 (+8%)</td>
<td>3.1267 (+13%)</td>
</tr>
<tr>
<td>2</td>
<td>3.39</td>
<td>3.2700+5.0747E-5i (-4%)</td>
<td>3.2804 (-3%)</td>
</tr>
<tr>
<td>3</td>
<td>4.39</td>
<td>4.1779+3.2040E-4i (-5%)</td>
<td>4.2034 (-4%)</td>
</tr>
<tr>
<td>4</td>
<td>4.44</td>
<td>4.3734+0.0020710i (-2%)</td>
<td>4.3913 (-1%)</td>
</tr>
<tr>
<td>5</td>
<td>4.69</td>
<td>4.7502+0.0037410i (+1%)</td>
<td>4.8082 (+2%)</td>
</tr>
<tr>
<td>6</td>
<td>8.27</td>
<td>7.9983 (-3%)</td>
<td>7.9998 (-3%)</td>
</tr>
<tr>
<td>7</td>
<td>8.81</td>
<td>8.5763+4.5570E-4i (-3%)</td>
<td>8.5871 (-3%)</td>
</tr>
</tbody>
</table>

*,** error relative to experimental values in brackets
Fig. 5-1. Modes shapes obtained from FEM of DSSI-incorporating and fixed-base models.

The imaginary part is indicative of energy loss at the foundation level due to damping and energy decay rate for each cycle. As expected, the resonant frequencies for each eigenmode
are higher for the fixed base model compared to the DSSI-incorporating model. It was also observed that higher energy losses are incurred as frequency increases in both models when isotropic damping is considered. Both models achieved adequate accuracy, with a slight accuracy advantage over the DSSI-incorporating model. To assess the contribution of each mode (or mobilization of effective dynamic mass) to the total response, the Modal Participation Factors (MPF) were determined for each eigenmode from the DSSI-incorporating model. This was done for translational and rotational DOFs. For a certain mode shape, the participation factor ($\gamma_i$) can be defined through Eq. 5-1.

$$\gamma_i = \{\phi_i\}^T [M] [D]$$

where $\{\phi_i\}$ is the $i$th mode shape vector, $[M]$ is the mass matrix, and $[D]$ is a unit displacement/rotation vector in the direction of excitation for global Cartesian coordinates and rotations about their axes. The effective mass $M_{eff,i}$ is defined as $\gamma_i^2$. Subsequently, the ratio of effective mass to total mass indicates the contribution of the $i$th mode to the dynamic response of the bridge (MPF). The estimated total mass of the bridge is 1,029,400 kg, including the weight of the T-Rex. Table 5-2 provides the participation factors for translational modes.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency – DSSI (Hz)</th>
<th>$\gamma_x$ (kg$^{0.5}$)</th>
<th>$\gamma_y$ (kg$^{0.5}$)</th>
<th>$\gamma_z$ (kg$^{0.5}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.9785</td>
<td>2.2226</td>
<td>62.363</td>
<td>0.26141</td>
</tr>
<tr>
<td>2</td>
<td>3.2700+5.0747E-5i</td>
<td>3.5704</td>
<td>1.9152</td>
<td>0.47685</td>
</tr>
<tr>
<td>3</td>
<td>4.1779+3.2040E-4i</td>
<td>671.67</td>
<td>1.147</td>
<td>49.9</td>
</tr>
<tr>
<td>4</td>
<td>4.3734+0.0020710i</td>
<td>60.155</td>
<td>0.033763</td>
<td>815.28</td>
</tr>
<tr>
<td>5</td>
<td>4.7502+0.0037410i</td>
<td>561.24</td>
<td>0.66752</td>
<td>26.375</td>
</tr>
<tr>
<td>6</td>
<td>7.9983</td>
<td>0.26853</td>
<td>0.97864</td>
<td>0.11772</td>
</tr>
<tr>
<td>7</td>
<td>8.5763+4.5570E-4i</td>
<td>0.16803</td>
<td>0.23584</td>
<td>18.097</td>
</tr>
</tbody>
</table>

From the determined translational participation, the contribution of each mode in a particular direction/DOF (X direction or lateral swaying) was determined as presented in
Table 5-3, which shows an example for translation along X. It is crucial to note that the participation factors are complex-values since the eigenmodes are complex-valued. Nevertheless, the real part is used for simplicity. The MPF is the ratio of $M_{\text{eff},i}$ to the total mass. The sum MPFs, $\sim 0.75$, indicate that the obtained modes account for 75% of the response. In particular, Modes #3 and #5 contribute as high as $\sim 0.44$ and $\sim 0.31$ respectively. This indicates that lateral swaying is the main driver of response for those modes. To increase the $\sim 0.75$, more modes shapes should be examined; however, 0.75 is deemed sufficient to describe the dynamic behavior of the bridge.

<table>
<thead>
<tr>
<th>Mode</th>
<th>$y_x$ (kg$^{0.5}$)</th>
<th>$M_{\text{eff},i}$ (kg)</th>
<th>MPFx</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.2226</td>
<td>4.939951</td>
<td>4.79886E-06</td>
</tr>
<tr>
<td>2</td>
<td>3.5704</td>
<td>12.74776</td>
<td>1.23837E-05</td>
</tr>
<tr>
<td>3</td>
<td>671.67</td>
<td>451140.6</td>
<td>0.438255866</td>
</tr>
<tr>
<td>4</td>
<td>60.155</td>
<td>3618.624</td>
<td>0.003515275</td>
</tr>
<tr>
<td>5</td>
<td>561.24</td>
<td>314990.3</td>
<td>0.305994111</td>
</tr>
<tr>
<td>6</td>
<td>0.26853</td>
<td>0.072108</td>
<td>7.00489E-08</td>
</tr>
<tr>
<td>7</td>
<td>0.16803</td>
<td>0.028234</td>
<td>2.74277E-08</td>
</tr>
<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>769767.3</td>
<td>0.747782532</td>
</tr>
</tbody>
</table>

The mass moment of inertia is used instead of the mass to calculate MPFs for rotational DOFs. This is not as straightforward as determining translational MPFs, since this calculation requires knowledge of the center of mass of individual components of the SFS system. However, as an approximation, Eq. 5-2 provides an estimate for the rotational MPF of typical/notional bridges.

$$MPF_{i,r} = \frac{nM_{\text{eff},i,r}}{J_{i,r}}$$

Eq. 5-2

where, $J_{i,r}$ is the mass moment of inertia about the $r$th axis for the $i$th (e.g $J_{1,x} = m[Y^2+Z^2]$), $(X, Y, Z)$ is the center of mass of the SFS system, and $n$ is modifier between 3-5 to account for the overall off-center estimation in lieu of individual bridge components estimation. An
average value of n = 4 was used, and the center of mass of the SFS system was found to be at (6.05,33.7, -1.19) m from the origin indicated in Fig. 5-1. Like the translational mode shapes, Table 5-4 shows the participation factors for rotational modes, and Table 5-5 shows the MPFs determined for rocking (rotation around the y-axis).

Table 5-4. Participation Factors as Obtained from the DSSI-Incorporating Model for Rotational Modes.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency – DSSI (Hz)</th>
<th>$\gamma_{x,x}$ (kg$^{0.5}.m$)</th>
<th>$\gamma_{y,y}$ (kg$^{0.5}.m$)</th>
<th>$\gamma_{z,z}$ (kg$^{0.5}.m$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.9785</td>
<td>14091</td>
<td>0.93328</td>
<td>124.95</td>
</tr>
<tr>
<td>2</td>
<td>3.2700+5.0747E-5i</td>
<td>512.81</td>
<td>3.7967</td>
<td>3384.6</td>
</tr>
<tr>
<td>3</td>
<td>4.1779+3.2040E-4i</td>
<td>32.687</td>
<td>1346</td>
<td>12.201</td>
</tr>
<tr>
<td>4</td>
<td>4.3734+0.0020710i</td>
<td>0.82534</td>
<td>2.6845</td>
<td>1.1341</td>
</tr>
<tr>
<td>5</td>
<td>4.7502+0.0037410i</td>
<td>16.283</td>
<td>2660</td>
<td>1.0304</td>
</tr>
<tr>
<td>6</td>
<td>7.9983</td>
<td>21.359</td>
<td>0.48674</td>
<td>72.986</td>
</tr>
<tr>
<td>7</td>
<td>8.5763+4.5570E-4i</td>
<td>6.7955</td>
<td>15.012</td>
<td>5.0272</td>
</tr>
</tbody>
</table>

Table 5-5. Calculation of MPF of each Mode for Rotation around the Y-axis (Rocking).

<table>
<thead>
<tr>
<th>Mode</th>
<th>$\gamma_{y,y}$ (kg$^{0.5}.m$)</th>
<th>$M_{e,i}$(kg.m$^2$)</th>
<th>MPF$_{y,y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.93328</td>
<td>0.871011558</td>
<td>8.89294E-08</td>
</tr>
<tr>
<td>2</td>
<td>3.7967</td>
<td>14.41493089</td>
<td>1.47175E-06</td>
</tr>
<tr>
<td>3</td>
<td>1346</td>
<td>1811716</td>
<td>0.184974325</td>
</tr>
<tr>
<td>4</td>
<td>2.6845</td>
<td>7.20654025</td>
<td>7.3578E-07</td>
</tr>
<tr>
<td>5</td>
<td>2660</td>
<td>7075600</td>
<td>0.722411423</td>
</tr>
<tr>
<td>6</td>
<td>0.48674</td>
<td>0.236915828</td>
<td>2.41889E-08</td>
</tr>
<tr>
<td>7</td>
<td>15.012</td>
<td>225.360144</td>
<td>2.3009E-05</td>
</tr>
<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>8887564.09</td>
<td>0.907411078</td>
</tr>
</tbody>
</table>

The calculations of the remaining MPFs are presented in Appendix D. Table 5-6 summarizes the estimated MPFs for each DOF. The highlighted cells indicate significant contributions. It can be observed that the presented modes capture most of the response fairly, with the lowest MPF of ~0.65 for the significant motions. As expected, MPF$_{y}$ and MPF$_{z,z}$, which correspond to translation along Y (longitudinal swaying) and rotation about Z (global torsion), are approximately 0. This confirms a proper assignment of boundary conditions. Furthermore, Mode#5 contributes as high as ~0.72 of MPF$_{y,y}$ (rocking), while
Mode#3 contributes ~0.18 (both modes represent 0.9 of this motion). This indicates that rocking is primarily the vibration mode of Mode#5, while also exhibiting some swaying. On the other hand, Mode#3 has predominantly translation motion (swaying) compared to Mode#5. Nevertheless, swaying-rocking is coupled to some extent in both modes.

Table 5-6. Summary of MPFs of each Mode for Translational and Rotational DOFs.

<table>
<thead>
<tr>
<th>Mode</th>
<th>MPF_x</th>
<th>MPF_y</th>
<th>MPF_z</th>
<th>MPF_{x,y}</th>
<th>MPF_{x,z}</th>
<th>MPF_{y,z}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.79886E-06</td>
<td>0.003778069</td>
<td>6.63835E-08</td>
<td>0.678519839</td>
<td>8.89294E-08</td>
<td>5.1748E-05</td>
</tr>
<tr>
<td>2</td>
<td>1.23837E-05</td>
<td>3.56323E-06</td>
<td>2.20892E-07</td>
<td>0.000898653</td>
<td>1.47175E-06</td>
<td>0.037969573</td>
</tr>
<tr>
<td>3</td>
<td>0.438255866</td>
<td>1.27803E-06</td>
<td>0.002418895</td>
<td>3.65114E-06</td>
<td>0.184974325</td>
<td>4.93414E-07</td>
</tr>
<tr>
<td>4</td>
<td>0.003515275</td>
<td>1.10738E-09</td>
<td>0.645697958</td>
<td>2.32779E-09</td>
<td>7.3578E-07</td>
<td>4.26308E-09</td>
</tr>
<tr>
<td>5</td>
<td>0.305994111</td>
<td>4.32857E-07</td>
<td>0.000675773</td>
<td>9.06041E-07</td>
<td>0.722411423</td>
<td>3.51911E-09</td>
</tr>
<tr>
<td>6</td>
<td>7.00489E-08</td>
<td>9.30383E-07</td>
<td>1.34622E-08</td>
<td>1.55898E-06</td>
<td>2.41889E-08</td>
<td>1.76563E-05</td>
</tr>
<tr>
<td>7</td>
<td>2.74277E-08</td>
<td>5.4032E-08</td>
<td>0.000318148</td>
<td>1.57805E-07</td>
<td>2.3009E-05</td>
<td>8.37671E-08</td>
</tr>
<tr>
<td>Σ-</td>
<td>0.747782532</td>
<td>0.003784328</td>
<td>0.649111074</td>
<td>0.679424768</td>
<td>0.907411078</td>
<td>0.038039562</td>
</tr>
</tbody>
</table>

5.1.2. Model Validation

After conducting the eigenfrequency study, FE analysis and simulation of T-Rex loading were conducted in the time domain, which was also used to validate the FEM model results against the experimental results. Since the boundary conditions of the footing is changing with frequency throughout the chirp, and thus with time, Eq. 5-3 was used to define frequency (ft) in the time domain.

\[ f(t) = f_{ini} - \delta f t \]  

where \( f_{ini} = 15 \) Hz, \( t \) is time, and \( \delta f \) is the frequency gradient (0.4375 Hz/s).

Fig. 5-2 shows response time histories caused by the load sweep from both the experimental and FEM model results. The overall accuracy of the DSSI-incorporating model, vs. the experimental results, is higher than that of the fixed-base model. At the early stages of loading (higher frequencies), both the fixed-base and DSSI-incorporating models produced lower amplitude motion than the experimental results. This could be attributed
to the selected values of $\alpha_M$ and $\beta_k$ for damping to represent the entire frequency range, while validating the FE model was focused on the dominant modes. However, near the dominant natural frequencies of the bridge, the DSSI model showed a better match with the experimental results, supplemented by a lower mean absolute error (MAE), especially around resonance.

![Time history of east geophone from the test, fixed-base model, and DSSI model due to 93.4 kN (21 k) transverse load.](image)

Table 5-7 presents the measured response and corresponding frequencies from the experimental results, compared with the amplitudes for DSSI-incorporating and fixed-base FE models.
Table 5-7. Transverse Response due to Transverse Loading at Frequencies of Interest from Time Trace of Frequency.

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>Response [cm/s] (% error)</th>
<th>Fixed</th>
<th>DSSI</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.16</td>
<td>2.41 (-8.1%)</td>
<td>2.53 (-3.5%)</td>
<td>2.62(-)</td>
<td></td>
</tr>
<tr>
<td>4.49</td>
<td>2.83 (32.4%)</td>
<td>2.41 (12.9%)</td>
<td>2.13 (-)</td>
<td></td>
</tr>
<tr>
<td>4.69</td>
<td>0.79 (11%)</td>
<td>0.74 (4.2%)</td>
<td>0.71 (-)</td>
<td></td>
</tr>
</tbody>
</table>

The peak response from the experiment, 2.62 cm/s (1.033 in/s), occurred at a frequency of 4.16 Hz, as seen in Table 4, indicating a resonant mode. The peak amplitude at the same frequency of the DSSI model was 2.53 cm/s (0.996 in/s), with a relative amplitude error of -3.5%. On the other hand, the response at the same frequency from the fixed-base model was 2.41 cm/s (0.949 in/s) with a relative error of -8.1%. The comparison of responses at frequencies of interest from the experimental investigation and both FE models is also presented in Table 5-7. Therefore, the fixed-base model had a higher error in the response amplitude at the predominant natural frequency, and it also exhibited a peak due to a different mode shape (or a superposition of multiple mode shapes). The decrease in resonant frequencies due to DSSI, as opposed to a fixed base model, is in agreement with experimental results from other research efforts (Chaudhary et al., 2001; Sextos et al., 2016). This is a crucial finding since, although the error in estimating the peak response might be lower, the fixed-base assumption may lead to skipping a mode of vibration. This lateral mode is attributed to soil flexibility, and its omission could lead to analysis and design errors (Chaudhary, 2017; Tongaonkar and Jangid, 2003; Wang et al., 2014). After validating the model with the response obtained from the experimental study, the model was used to estimate displacements. Fig. 5-3 shows a comparison of the response of the deck to horizontal shaking at 93.4 kN between the fixed-base and DSSI-incorporating model. It can be observed that the fixed-base model does not exhibit the multiple modes
determined from the experimental study, hence skipping some modes, although they exist from the eigenfrequency analysis. Therefore, DSSI incorporation overall leads to a better match with amplitude and dynamic behavior when compared to experimental results. For the tested bridge, the inclusion of DSSI effects led to an increase in lateral displacement relative to the fixed-base model. For the mode exhibited by both models (~4.2 Hz), this increase is approximately 14%.

After matching the model in the time domain, frequency domain studies were established to accelerate modeling and preparation for parametric sweeps. Fig. 5-4 shows results from both models compared to the experimental results for lateral deck response to lateral shaking. It can be observed that the DSSI-incorporating model better matches the amplitude at the first resonant peak than the fixed-base model. Nevertheless, both models underestimated the resonant frequency of the first peak. As for the second peak, which is of higher importance, the DSSI-incorporating model was more accurate at estimating the resonant frequency and amplitude when compared to the test results. Away from resonance, both models captured the damping correctly until ~5.5 Hz, after which the damping was higher than what was exhibited by the bridge. Nevertheless, properly estimating damping at higher frequencies is not of foremost importance in this research.
Fig. 5-5 shows another comparison of models against test results at midspan, showing vertical response to lateral shaking. The DSSI-incorporating model was better at describing the dynamic behavior of the deck.

![Graph showing velocity magnitude vs frequency for Test, DSSI, and Fixed models.]

Fig. 5-4. Results from frequency-domain models compared to experimental results for the lateral response of the deck to lateral shaking at 21 k (93.4 kN).

![Graph showing velocity magnitude vs frequency for Test, DSSI, and Fixed models.]

Fig. 5-5. Results from frequency-domain models compared to experimental results for the vertical response of the midspan to lateral shaking at 21 k (93.4 kN).

### 5.1.3. Analysis of Stresses

Although no field strain measurements on bridge components were conducted, a comparison of various stress components between the DSSI-incorporating and fixed-base
models was conducted to examine the potential implications of DSSI effects on stresses. After validating the numerical models, shear stresses within critical regions (regions with stress concentration) were extracted from the FEM simulations. As expected, the highest stress due to horizontal load was observed in the bent/bent cap connection (0.65 MPa and 1.4 MPa for DSSI and fixed-base respectively). As an illustration, the stress time history induced in the deck due to a 93.4 kN load is shown in Fig. 5-6. The DSSI-incorporating model exhibited lower maximum shear stresses in the resonant frequency range, compared to the fixed-base model. It was 64% of the fixed-base model stress at the frequency of 4.52 Hz.

![Graph](image.png)

**Fig. 5-6.** Time history of maximum shear stress in the deck near the top rebar level in both models due to 93.4 kN load.

Furthermore, the peak shear stresses were evaluated for various bridge components for the two models at the resonant frequency, as depicted in Fig. 5-7. In both models, the peak shear stresses occur at the connection between the center bent and the center bent cap. For example, the maximum shear stress in the cap/bent connection in the DSSI-incorporating models was 0.65 MPa, compared to 1.4 MPa in the fixed-base model. This indicates a 55% reduction in peak stress compared to the fixed-base model. A similar trend
was observed when comparing normal stresses. This is a result of an increased stiffness due to the end restraints in the rigid base model.

Fig. 5-7. Peak shear stresses from both models due to a 93.4 kN transverse load at the resonant frequency.

Fig. 5-8 shows a comparison of shear stress distributions due to transverse loading for the two models at their resonant frequencies for mode #3. It illustrates that the fixed-base model bridge components experienced substantially higher stress levels compared to the same in the DSSI-incorporating model. It is especially pronounced at the connections and in the girders.
Another stress component that was evaluated was von Mises stress. Fig. 5-9 presents an overall comparison of von Mises stresses evaluated at various locations on the bridge. In general, DSSI incorporation led to a decrease in stresses when compared to the fixed-base model. The stress peaks in the fixed-base model occurred at a slightly higher frequency than the DSSI model but describe the same stress component. An exception was encountered as illustrated in Fig. 5-9(a), in which the peak stress in the deck was higher in the DSSI-incorporating model compared to the fixed-base model. The highest stress increase was from 18 MPa (DSSI) to 26 MPa (fixed), which was experienced by the braces as shown in Fig. 5-9(e). Fig. 5-10 shows a comparison of the von Mises stress in the deck at the bottom and upper surfaces (near reinforcement). The peak stress levels in the DSSI (@4.12 Hz) and fixed-base (4.17 Hz) models were 1.49 MPa and 1.44 MPa respectively. Stresses concentrated at the end supports and on either side of midspan locations.
Fig. 5-9. von Mises stress comparison between the fixed-base and DSSI-incorporating model due to lateral shaking at 93.4 (kN), at the (a) deck, (b) bent, (c) girders, (d) columns, (e) braces, (f) footing-column connection, (g) column-bent connection, and (h) girder-bent cap connection.
The Dynamic Amplification Factor (DAF) was also evaluated for the lateral response of the deck to lateral shaking. The static displacement $\delta_{st}$ was obtained from running a stationary study on the fixed-base model. DAF was then calculated as shown in Eq. 5-4.

$$DAF(f) = \frac{\delta_{dyn}(f)}{\delta_{st}}$$  \hspace{1cm} Eq. 5-4

where $\delta_{dyn}(f)$ is the dynamic displacement at a given frequency $f$. Fig. 5-11 shows the DAF obtained for both the DSSI-incorporating and fixed-base models. The maximum displacement amplification was ~9.4 for the DSSI model, while it was 9.9 for the fixed-base model. The DSSI peak also occurred at a lower frequency, confirming the period increase exhibited when considering DSSI effects. In general, the DAF for the DSSI model was lower than the fixed-base model, except for frequencies $< 3$ Hz where it was slightly higher for the DSSI model.

![Comparison of von Mises stress at the top and bottom surfaces of the deck between the DSSI-incorporating and fixed-base model.](image)

Fig. 5-10. Comparison of von Mises stress at the top and bottom surfaces of the deck between the DSSI-incorporating and fixed-base model.
The results reported herein suggest that incorporating DSSI effects in the current study led to a reduction in structural demands, which is a beneficial effect that can give rise to more optimal designs. In addition, assuming a fixed base can fail to explain complex bridge motions such as rocking, especially if there was strong coupling with lateral swaying as well. This was exhibited in the current study by skipping this mode of vibration. In the following section, the effect of varying parameters pertaining to the soil and structure is examined for dimensionless ratios described in Chapter 2.

![Graph](image)

**Fig. 5-11. DAF of deck lateral displacement due to lateral shaking from both models.**

### 5.2. Parametric Study

To assess the extent to which the incorporation of DSSI would affect the response of the bridge, a parametric study was conducted in the frequency domain. The study included several sub- and superstructural elements that can alter the response. Furthermore, the parametric can facilitate the identification of several responses/outputs from bridge shaking that would serve as differentiators when comparing different parameters of hypothetical bridges to the actual bridge. The parameters considered were soil shear wave velocity ($V_s$),
structural height ($H_{tot}$), foundation half width (B), and foundation depth ($D_f$). Other parameters are crucial for correctly modeling the response of the bridge such as concrete and structural steel elastic moduli, their densities, and Rayleigh damping coefficients $\alpha$ and $\beta$. However, these parameters are a prerequisite for the correct matching of the FEM with the experimental response. Hence, these parameters were not considered. Specific combinations are shown herein rather than all possible combinations while holding other parameters constant.

Fig. 5-12 shows the effect of varying the structural height on the overall response of the bridge to a 93.4 kN (21 K) lateral load centered above the pier at the deck. The increase in structural height led to softening of the structure as expected and reduced the resonant frequency of both peaks. In addition, the 1st peak becomes stronger than the second when the height is increased beyond 5 m. This suggests that the coupling between swaying and rocking for this mode becomes stronger. Furthermore, while the 1st peak becomes stronger, the 2nd peak gets smaller with increasing height.
Fig. 5-13 shows the effect of varying the shear wave velocity of soil on the response of the bridge. The 1\textsuperscript{st} peak amplitude and frequency show negligible changes, while both the amplitude and resonant frequency of the 2\textsuperscript{nd} peak increased with increasing the shear wave velocity. The response starts to saturate beyond 250 m/s, suggesting that increasing the velocity further leads to approaching the fixed-base condition. This is the case increasing shear wave velocity increases the shear modulus, which in turn increases the static stiffness.

Fig. 5-14 shows the effect of varying the foundation depth on the lateral response of the deck. The amplitude and frequency of the 1\textsuperscript{st} peak exhibited no difference in varying the foundation depth. However, the 2\textsuperscript{nd} peak exhibited a slight increase in both amplitude and frequency. This slight difference was deemed inadequate to inform different behavior. Hence, other points on the bridge were examined.

![Graph showing the effect of varying shear wave velocity (V_s) on lateral deck response to lateral shaking.](image-url)
Fig. 5-14. Effect of varying $D_f$ (m) on lateral deck response to lateral shaking. (Shown for $H_{tot} = 5$ m, $B = 5.7$ m, $V_s = 200$ m/s). [1 in/s = 2.54 cm/s].

Fig. 5-15 shows the effect of varying the foundation depth on the lateral response of the footing. The effect of increasing the depth mainly led to high damping of the response by lowering the peak amplitudes. Through the figure, it is possible to infer a more intuitive and discernible difference in response to lateral shaking. In addition, the behavior of the footing is in agreement with the behaviors described by Wolf (1985). This is the case since damping is expected to decrease peak response and slightly increase frequency due to increased mechanical impedance (not to be confused with a lower damped frequency of SDOF systems). This result highlights the importance of having ground geophones either attached to the pier or the foundation itself if possible.
Fig. 5-15. Effect of varying \( D_f \) (m) on lateral footing response to lateral shaking. (Shown for \( H_{tot} = 5 \text{ m}, B = 5.7 \text{ m}, V_s = 200 \text{ m/s} \). [1 in/s = 2.54 cm/s].

Similar to footing depth, varying footing half-width did not lead to discernible differences at the deck level; hence, the response of the footing was considered. Fig. 5-16 shows the lateral response of the footing to lateral shaking. While there is no clear trend deducible from the results, they still show that varying the footing half-width leads to a varying response, and that helps discern the correct foundation width from an iterative process.

Fig. 5-16. Effect of varying \( B \) (m) on lateral footing response to lateral shaking. (Shown for \( H_{tot} = 5 \text{ m}, D_f = 2 \text{ m}, V_s = 200 \text{ m/s} \). [1 in/s = 2.54 cm/s].
To assess the extent to which DSSI effects alter the response, the results from the parametric study were used further to examine the effect of the following variables on the response:

- Embedment ratio
- Structure-to-Soil Stiffness Ratio (Rigidity Ratio):
- Structure-to-Soil Slenderness Ratio

The two main criteria selected to assess such effects are peak amplitude(s) modification and their respective frequencies. The change in peak frequency relative to a fixed-base case is commonly and interchangeably referred to as period lengthening/shortening or stiffness softening/hardening. This is referred to as structural hardening/softening herein. Naturally, varying parameters such as \( V_s \) or \( B \) will lead to altering multiple ratios at the same time. While damping is another key factor to examine when assessing DSSI effects, it was not considered in the current study.

Fig. 5-17(a) shows the effect of the embedment ratio (D/B) on the peak amplitude ratio of DSSI/Fixed lateral deck response, while Fig. 5-17(b) shows the same for peak frequency. Those results are shown for the 2\(^{nd}\) peak illustrated in Fig. 5-14. It can be observed that increasing the embedment ratio led to diminishing DSSI effects, which is manifested in Fig. 5-17(b) as the DSSI/Fixed peak frequency ratio increases and approaches unity. Furthermore, Fig. 5-17(a) suggests that the increase in total mechanical impedance leads to structural hardening rather than a reduction in the response due to increased damping. This is the case since both dynamic stiffness and radiation damping increased with increasing embedment depth. The extent of DSSI response alteration relative to a fixed-base scenario based on the embedment ratio can also be varied by varying the footing half-
width. This may lead to a different relationship for various depths. Hence, a combination of varying both the footing depth and half-width is of importance. However, the results presented provide adequate insights into the effect of the embedment ratio on the response.

Fig. 5-17. The extent of (D/B) ratio on altering the DSSI effect based on (a) peak amplitude and (b) peak frequency, with other parameters held constant.

Fig. 5-18 illustrates the effect of varying the Structure-to-Soil Slenderness Ratio, $\bar{h} = \frac{h}{a}$, the extent of DSSI response alteration relative to the fixed base. The results are presented
as a function of the dimensionless frequency \(a_0 = \omega B/V_s\). For the second peak which describes rocking, it is evident that increasing \(\bar{h}\) led to a reduction in DSSI alteration of response relative to fixed-base. This means that this effect diminishes for more slender structures. Rotations dictate the response, and they become more controlled by the moment arm rather than the boundary condition, i.e., fixed vs DSSI. This effect also depends on the stiffness ratio, which is discussed next. For flexible structures, rocking of the foundation is less important than for rigid structures.

As for the 1st peak, increasing \(\bar{h}\) led to increasing the DSSI alteration of response. As shown in Chapter 4, the bridge experiences stronger coupling between lateral, rocking, and vertical modes. Hence, further investigation is required to determine this effect on rotations independently. However, overall softening is observed from the 1st peak relative to a fixed base, where the resonant frequency was reduced further than the computational frequency domain considered (around detected resonant frequencies from the experimental program).

![Graph](image)

**Fig. 5-18.** Effect of varying \(\bar{h}\) on altering DSSI effects relative to a fixed base based on the peak lateral amplitude ratio of the deck due to lateral shaking. Shown for \(V_s = 200\ m/s\), \(B = 5.7\ m\), and \(D_f = 2m\).
Away from the lateral resonant frequencies, the ratio approaches 1 as $\bar{h}$ increase in the interval $a_0 > 0.9$. On the other hand, the response is reduced relative to a fixed base at the anti-resonance down to a ratio of 0.75. The valley between the two peaks corresponds to a vertical mode shape as determined in Chapter 4, which again suggests some coupling in the bridge dynamic response that counteracts the rocking behavior. Fig. 5-19 shows the effect of the $\bar{h}$ on the peak-amplitude frequency. As discussed, the effect of DSSI decreased with increasing $\bar{h}$, which also means structural softening due to DSSI effects diminish as slenderness increases when examined at the same rigidity ratio.

The rigidity ratio $\bar{s} = \frac{h_{tot} f}{V_s}$ was estimated assuming $f$ to be the frequency of the 2$^{nd}$ peak obtained from the fixed-base model (4.8 Hz). This ratio was estimated by sweeping both $V_s$ and $h_{tot}$. Fig. 5-20 shows the effect of the rigidity ratio $\bar{s}$ on the peak amplitude ratio of DSSI/Fixed lateral deck response. Increasing $\bar{s}$ led to decreasing the maximum amplitude since the rigidity of the SFS is increasing. This is in agreement with the literature since the system gets closer to the fixed-base assumption (Wolf, 1985).
Fig. 5-20. Extent of \( \bar{s} \) alteration of lateral peak amplitude caused by lateral shaking due to DSSI effects relative to a fixed base. Shown for \( B = 5.7 \text{ m}, D_f = 2\text{m}, \) and varying \( h_{\text{tot}} \) and \( V_s \).

Fig. 5-21 shows the reduction of the 2\textsuperscript{nd} peak amplitude ratio and the frequency of the peak as a function of dimensionless frequency with increasing \( \bar{s} \). This follows the expected behavior that increasing \( \bar{s} \) would eventually lead to a fixed-base condition.

Fig. 5-21. Effect of varying \( \bar{s} \) on altering DSSI effects relative to a fixed-base based on the peak lateral amplitude ratio of the deck due to lateral shaking. Shown for \( B = 5.7 \text{ m}, D_f = 2\text{m}, \) and varying \( V_s \) and \( h_{\text{tot}} \).
5.3. Effect of Superstructure Rigidity on Structural Response

A motivation for this study was the observation of bridges exhibiting highly dynamic behavior during the passing of heavy vehicles during a scheduled NDE of a bridge in Iowa (Gucunski et al., 2021). There was significant delamination while there were few signs of corrosion. The delamination was attributed to repeated high deflection leading to cracking and delamination. This finding was one of the motivations for the evaluation of actual bridges to explore the effect of the dynamic characteristics of the bridge, the rigidity of the superstructure, on stress levels in the deck. The evaluation was conducted using a large-amplitude mobile shaker. The reason for the need for a large mobile shaker was the participation of unreliable mechanisms under low-load levels as described in Chapter 2. The purpose of the numerical model development was to examine the effect of the superstructure rigidity on the stress levels in the bridge deck under dynamic loads. With the numerical model matching the measured response as presented in this chapter, a second numerical model was developed with girders of four times higher flexural stiffness than the existing ones. The natural frequencies and mode shapes for the two models were obtained and compared. As illustrated in Fig. 5-22, the increase in the girder stiffness led to an increase in the first bending mode frequency from 3.1 Hz to 5.1 Hz, with only minor changes in the mode shapes.
Vertical harmonic loading of 21500 lb (97 kN) at the two bending mode frequencies was applied at the midspans of the two models, as shown in Fig. 5-23. While the vertical displacement surfaces for the two models in Fig. 5-22 show similar shapes, the displacements for the model with the stiffer girders were about four times lower than for the existing structure. The dynamic magnification factor for the existing bridge was 2.67, while it was 1.67 for the stiffened bridge.
Longitudinal stresses at the top of the deck and Von Mises stress at the top rebar level distributions for the two models are shown in Fig. 5-24. While stress distributions are similar, the stress levels in the deck of the existing structure are two to three times higher. Similar relationships were obtained for other stress components in the deck, describing the significance of the rigidity of the supporting girders on the deflection and stresses in the deck.

Fig. 5-24. Longitudinal normal (top) and Von Mises stress (bottom) at the top of the deck of the Hobson Avenue Bridge under harmonic loading at 3.1 Hz for the existing (left) and 5.1 Hz for the hypothetical stiffer girders (right).

Fig. 5-25 shows normal stresses induced by the load in both models at the top rebar level. It is observed that having a stiffer superstructure leads to lower normal stresses at the top rebar levels. In addition, Fig. 5-25 shows that some regions are under compressive stress (blue regions), while other regions experience tensile stress (red regions). The current numerical study considers a load at a fixed position, while considering a moving load can
show regions that experience the most stress reversals. This is important in assessing the durability of concrete decks since load reversals lead to fatigue, and initiation of cracks. This in turn can potentially affect the durability of concrete decks and make them susceptible to corrosion and further deterioration. Nevertheless, the results from the current study with the fixed load position emphasize the importance of superstructure rigidity on deck durability. The models highlight the ability of highly refined FEM models to capture the dynamic response of bridges due to moving loads and estimate differences in displacements and stresses. This can provide helpful insights into monitoring bridge deck conditions and assessing expected vibrations due to operational load levels. Typically, passing vehicles would excite the structure vertically and engage the 1st mode shape which was presented in this section.

Fig. 5-25. Longitudinal normal at the top of the deck of the Hobson Avenue Bridge under harmonic loading (97 kN) at 3.1 Hz for the existing (left) and 5.1 Hz for the hypothetical stiffer girders (right).
To summarize, Chapter 5 provided results of the numerical simulations performed to match the experimental results in time and frequency domains. The models enabled capturing DSSI effects on bridge response in terms of dynamic behavior, amplitude magnification, and stress levels in various elements of the bridge. The eigenfrequency analysis shows that while it can help estimate the resonant frequencies for both the fixed and DSSI incorporating models, matching the response in frequency and time domain revealed that the fixed-base assumption can lead skipping a mode shape even if it was detected through an eigenfrequency analysis. This indicates incorrect dynamic characterization of the bridge. Equally important, the parametric study highlighted that several super- and substructural features can alter the response of the bridge and the extent to which DSSI effects alter it. This is crucial since this is the basis for identifying different SFS configurations and making it possible to discern them from each other. Furthermore, the assessed dimensionless ratios confirm that modelling DSSI effects in the current research followed expected behavior and is in agreement with the literature for rocking foundations. Lastly, the use of the models can be extended beyond rocking behavior to evaluating response to vertical excitation due to operational loads. They introduce the possibility to use large amplitude mobile shakers to monitor response and deterioration in the bridge deck by evaluating stress/strain levels. This promotes their use as a global NDE tool for bridge condition assessment.
CHAPTER 6: EXPLORATION OF UNKNOWN FOUNDATIONS AND ESTIMATION OF BEARING CAPACITY

This chapter describes the approach proposed for the exploration of unknown foundations using large-amplitude mobile shakers. The parametric sweeps of foundation half-width, half-length, and depth from numerical simulation results were compared with the experimental results from the Gate Creek Bridge in Oregon. As a result, the foundation geometry can be then estimated through iteratively matching the response from numerical models to the experimental results including eigenfrequency, time domain, and frequency domain studies. Once the geometry is known, vertical impedance functions can be used to estimate small-strain shear modulus.

6.1. Approach

A major motivation for this study was to introduce novel methods for assessing foundations' reuse potential as argued in Chapter 1. To assess the potential reuse of the substructure, the ultimate bearing capacity is the basis of judging the substructure's reusability herein. The bearing capacity can be fundamentally defined as the average contact pressure between the load-bearing soil and the bottom surface of the foundation/substructure under a given limit state. From a structural/mechanical point of view, there are primarily three limit states: i) critical state (when a failure mechanism takes place), ii) serviceability (excessive settlements), and iii) extreme events such as earthquakes (liquefaction). In this research, the critical state and serviceability conditions are examined to describe the ultimate bearing capacity herein.
Several physical properties play a crucial role in the identification of unknown foundations (Olson et al., 1998):

1. Foundation Depth – depth to the bottom soffit of footing, pile length, etc.
2. Foundation Type - shallow vs. deep
3. Foundation Geometry - embedded substructure dimensions, pile locations
4. Foundation Materials - steel, timber, concrete, and masonry
5. Foundation Integrity - corroded steel, rotted timber, cracked concrete, etc.

In addition to those, soil properties are also a crucial element in identifying unknown foundations. In this research, items #4 and #5 are excluded from the evaluation as they are deemed secondary in determining the bearing capacity. The determined dimensions are assumed to be effective dimensions that capture item #5, while item #4 was limited to concrete since it is the most common bridge foundation material. Furthermore, since this study serves as a proof of concept, the simpler case of shallow foundations is sought after rather than deep foundations, limiting item #3. In addition, a relatively rigid structure in cohesionless soil was selected for testing to avoid more complicated/indirect phenomena that can affect response such as consolidation or volume change. The Gate Creek Bridge in Oregon met those requirements and hence was selected for this study as discussed in Chapter 3.

The estimation of the bearing capacity under two different scenarios based on available information lies between two extreme cases:

- Known foundation geometry and soil properties.
- Unknown foundation geometry and soil properties.
It is proposed that bridge foundations can be explored by utilizing the T-Rex on the bridge deck, monitoring the response near the foundation, and matching it with a 3D Finite Element Model (FEM). In addition, obtaining dynamic vertical impedance functions allows for the determination of shear modulus which can be correlated to soil shear strength parameters once the foundation dimensions are known. As shown in Chapter 5, varying the substructural features leads to altering the response when DSSI is considered. This enables discerning which foundation material and geometric parameters best fit the experimental results. The main criteria that can be used for this purpose are peak amplitude(s), the frequency(s) at which they occur, and the overall fit of the time history to the experimental results. While damping can change for embedded foundations, especially for vertical vibrations based on depth, its effect was implicitly incorporated into the overall response.

Fig. 6-1 shows a flowchart for the determination of an unknown foundation’s geometry. It is an iterative process that includes multiple FEM runs to achieve the objective of minimizing errors in the estimation of peak amplitudes, resonant frequencies, and best-fitting of time histories. Once the best fit is achieved, the explored foundation is then used for the determination of bearing capacity based on a failure mode, e.g., general shear failure, or a serviceability limit such as the allowable settlement of the foundation. One of the main challenges in describing the in-situ strength and stiffness of soils is the determination of shear modulus. The hypothesis in this research proposes the use of a dynamic impedance function obtained from a frequency sweep to determine the shear modulus. As for the FEM modeling step in Fig 6-1, known information can be described as any component of the SFS system that is accessible or readily measurable. Examples of
this can be dimensions of the superstructure, end-bearing/supports, span continuity, superstructure system (i.e. reinforced concrete, structural steel, or prestressed concrete), and a general description of supporting soil. In the lack of foundation input motion (FIM), which is the case for forced vibration testing, DSSI becomes purely inertial and the substructuring method can be more appealing for its modeling simplicity to practicing engineers, in addition to a more readily parameterizable setup compared to the Direct Method. Nevertheless, the Direct Method can be used in this application to describe the behavior of the entire SFS system rather than just the effect on the response of the bridge. This can include stresses at various depths in the soil beneath the footing, foundation settlement, and contact pressure between the foundation and the soil.
6.2. Exploration of Parameters Affecting the Response of Unknown Foundations

The numerical model was established to capture the response of the complex soil/structure dynamic response problem as described in Chapter 3, and it was utilized as a vehicle for identifying parameters of an unknown foundation. This was done by sweeping various geometric and material parameters to emulate the unknown foundation. Three main parameters (2 geometric and 1 material) were included in the current study:

- Depth of the foundation (distance from the ground surface to the bottom surface of footing),
- Half-width and half-length of the foundation (geometry of footing), and
- Shear wave velocity of soil (an estimator of soil shear strength parameters).

Guidelines for the footing design/construction that are available in codes/standards were excluded from limiting the range of values of the parameters. For instance, there is a required minimum footing depth to avoid environmental effects and degradation. Additionally, there are recommended length-to-width ratios for sizing footings in design codes. Such criteria can help eliminate more variables for future studies. However, to prove the concept of the study, only physical limitations are abided by, albeit they are hypothetical. An example of this is the minimum footing dimensions considered. From a structural stability point of view, the minimum footing dimensions used were those of the pier. The only information present is what is visible from the bridge (the soil surface and the superstructure). Similarly, the minimum depth for a fully embedded footing must be its thickness. An assumption in the current study is that the bridge is supported by a shallow
foundation. Damping was also excluded since it can be determined from the response of the bridge/soil during shaking. Table 6-1 summarizes the range of values considered in the parametric sweeps in the current study. Results reported herein are from specified combinations of the parametric sweeps rather than all combinations.

Table 6-1. Summary of Parameters Swept in Numerical Models for Footing Identification

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
<th>Step</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Df</td>
<td>0.5-3</td>
<td>0.5</td>
<td>m</td>
</tr>
<tr>
<td>2L</td>
<td>0.75-5.75</td>
<td>1</td>
<td>m</td>
</tr>
<tr>
<td>2B</td>
<td>2.5-3.33</td>
<td>0.167</td>
<td>m</td>
</tr>
<tr>
<td>Vs</td>
<td>150-450</td>
<td>50</td>
<td>m/s</td>
</tr>
</tbody>
</table>

The FE analysis and simulation of T-Rex loading were conducted in the time domain, the results of which were used and compared against the experimental results. Fig. 6-2 shows the T-Rex loading force, which was an input for the model. The force magnitude was 24k. A deformed shape of the bridge response at one instant of time of bridge shaking is depicted in Fig. 6-3. It can be observed that it is possible to excite and capture a notable response at the footing level by applying the load at the deck level.

![Force input used in the numerical model.](Image)
Fig. 6-3. Deformed shape of the bridge during one time instant of shaking.

Fig. 6-4 presents the effect of footing depths indicated in Table 6-1, in comparison to the test results, as one of the geometric features of the footing.

Fig. 6-4. Effect of footing depth (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for \( V_s = 400 \text{ m/s} \), \( 2B=2.75 \text{ m} \), and \( 2L = 4.5 \text{ m} \). [1 in/s = 2.54 cm/s]
To quantitatively assess how the depth affects the response relative to the experimental results, the Root Mean Square Error (RMSE) was used as a measure. The RMSE is calculated using Eq. 6-1 as:

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N} (R_i - \hat{R}_i)^2}{n}}$$  \hspace{1cm} Eq. 6-1

where $R_i$ is the local peak velocity obtained from the numerical simulation at the time $i$, $\hat{R}_i$ is the local velocity peak obtained from experimental results at the time $i$, and $n$ is the number of peaks.

Instead of matching the entire time history, local maxima and minima were extracted and used for RMSE calculation to ensure the peaks are occurring concurrently. This is to reduce any structural hardening/stiffening due to increased damping, which may lead to phase shifts. Fig. 6-5 shows the error estimates of the fits (RMSE) and peak amplitudes (relative error) of various footing depths relative to experimental results as shown in Fig. 6-4. The error is minimum when the foundation depth is set at 2.5 m. On the other hand, the relative error of the peak with respect to experimental results is minimum when the depth is set to 3 m. From the combination of both, it is deduced that the actual depth is in the interval of 2.5-3 m (8.2-9.8 ft).

Fig. 6-5. Error estimates for the results of varying footing depth shown in Fig. 6-4 on peak amplitude and time history fit.
Fig. 6-6 displays the effect of footing width (shorter dimension) on the response, while Fig. 6-7 shows the error estimates of the fits and peak amplitudes of various footing widths relative to experimental results as shown in Fig. 6-6. The least error in peak amplitude was when the footing width was set to 2.67 m. However, the RMSE of the fit kept increasing as the footing width increased. Nevertheless, the 2.67 m width provided the best estimate if the combined error is examined.

![Graph showing effect of footing width on response](image_url)

**Fig. 6-6.** Effect of footing width (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $V_s = 200$ m/s, $D_f = 2.4$ m, and $2L = 4.5$ m.

![Graph showing error estimates](image_url)

**Fig. 6-7.** Error estimate for the results of varying footing width shown in Fig. 6-6 on peak amplitude and time history fit.
Fig. 6-8 displays the effect of footing length (longer dimension) on the response, while Fig. 6-9 shows the error estimates of the fits and peak amplitudes of various footing lengths relative to experimental results as shown in Fig. 6-8. Evidently, setting the footing length to 4.75 m led to minimizing both the fit and peak amplitude errors, when the other parameters were set as shown in the figure caption.

Fig. 6-8. Effect of footing length (in m) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for $V_s = 200$ m/s, $D_f=2.4$ m, and $2B = 2.75$ m.

Fig. 6-9. Error estimate for the results of varying footing length shown in Fig. 6-8 on peak amplitude and time history fit.
As for soil properties, the shear wave velocity was varied to emulate the variation in stiffness of the supporting soil. Fig. 6-10 shows the effect of having varied shear wave velocity. Meanwhile, Fig. 6-11 shows the error estimates of the fits and peak amplitudes of various footing lengths relative to experimental results as shown in Fig. 6-10. The selected values for Fig. 6-10 were 200, 300, and 400 m/s to avoid clutter. Such values represent the transition from soft clays to medium and dense sands.

Fig. 6-10. Effect of shear wave velocity (in m/s) on vertical footing response to a 24 K steady-state vertical load at the experimentally identified resonant frequency (26 Hz). Shown for 2L= 4.5 m, D_f=2.4 m, and 2B = 2.75 m.

The best match between the model and the experimental results in terms of fit was when the velocity was set to 400 m/s. On the other hand, the peak amplitude relative error was when the velocity was 450 m/s. This indicates that the estimated velocity lies within the interval 400 – 450 m/s.
After obtaining the parameters which led to the lowest combined errors, they were combined to compare them with the experimental results. Fig. 6-12 shows the comparison between the numerical model representing the estimated foundation parameters with the experimental results at the resonant frequency of the bridge (26 Hz). For this study, the response from the numerical simulation at the footing level and how it compares to the experimental results is of paramount importance. There is an exceptionally good match between the numerical and experimental results.

Fig. 6-12. The best-match model obtained from the parametric sweep of several footing geometries and soil properties, representing vertical footing response to a 24 k steady-state vertical force. Shown for $V_s = 425$ m/s, $2L = 4.75$ m, $2B = 2.67$ m, and $D_f = 2.75$ m.
At this point, the actual footing dimensions were obtained from the available drawing of the bridge. Table 6-2 presents a comparison between the actual and the estimated footing dimensions. The method presented herein provides an adequate estimation of the footing dimensions. The estimated length, width, and depth were off by 4%, 3%, and ~0% respectively. In terms of bearing area aspect ratio (B/L), the error was ~1% and 6%. The estimated dimensions underestimate the bearing capacity of the footing compared to the actual dimensions since the estimated footing width and aspect ratio are slightly less than the actual dimensions. Those parameters are directly involved in bearing capacity calculations through the width, B/L ratio, and Df/B ratio.

Table 6-2. Comparison Between Estimated and Actual Footing Dimensions.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>2L (m)</th>
<th>2B (m)</th>
<th>B/L</th>
<th>Df (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimated Geometry</td>
<td>4.75</td>
<td>2.67</td>
<td>0.565</td>
<td>2.75</td>
</tr>
<tr>
<td>Actual Geometry</td>
<td>4.57</td>
<td>2.74</td>
<td>0.6</td>
<td>2.74</td>
</tr>
</tbody>
</table>

To confirm the obtained shear wave velocity from the model, the experimental vertical impedance was used. The response of the bridge to dynamic shaking was used to evaluate the shear modulus of the supporting soil. This can be done in a scenario where the geometry of the footing is known either by matching the actual response to a FEM model as presented earlier or by estimating the dimensions of the footing from knowledge of the superstructure. To entertain the novelty of this method, the former procedure is adopted for this research. For the determination of the shear modulus of the supporting soil, the footing is assumed to be a massless prism on an elastic homogenous half-space. This allows for the exploitation of dynamic soil-structure interaction to determine soil properties and how they change with frequency. Since the objective is to find the low-strain shear modulus, stiffness (load divided by displacement) of the footing is obtained from the steady-state response to shaking in the low-frequency range. The steady-state displacements were
obtained by integration of the velocity records in the time domain for all tested frequencies using Simpson’s Rule. Fig. 6-13 shows a sample displacement record obtained by integration, with a steady-state response of ~0.0012 in [0.03 mm]. The steady-state force and displacement were determined for the tested frequency range, as illustrated in Fig. 6-14 (a) and (b) respectively. The total vertical impedance is defined through Eq. 6-2 as:

$$S_v = F_v(f)/U_v(f)$$

Eq. 6-2

where $F_v(f)$and $U_v(f)$ are the harmonic vertical force and displacement, respectively.

Total impedance includes a stiffness component and a damping component, as a function of frequency. Therefore, impedance can be expressed as in Eq. 6-3

$$S_v(\omega) = K(\omega) + i\omega C(\omega)$$

Eq. 6-3

where $K(\omega)$ is the dynamic stiffness and $C(\omega)$ is the sum of material and radiation damping. The complex notation is because $F_v$ is typically out of phase with $U_z$ (Gazetas, 1991; Pais and Kausel, 1988). The damping term is negligible at lower frequency ranges, which means that total impedance is solely comprised of the stiffness term. This enables the estimation of stiffness directly from the impedance function.
Fig. 6-14. Steady-state (a) force and (b) displacement at the geophone near footing due to T-Rex shaking.

Impedance as a function of frequency is shown in Fig. 6-15. It can be observed that the impedance is almost constant, about 60,000 k/in, up to a frequency of 15 Hz. This indicates that this portion of the graph is still close to static stiffness. This obtained stiffness includes the effect of footing embedment, since it is obtained experimentally, which must be considered while estimating shear modulus.

A widely-accepted expression for a fully embedded rectangular footing is shown in Eq. 6-4 (Gazetas, 1991):

\[
K_{emb} = \frac{2LG}{1-\nu}S_z \left[ 1 + \frac{D}{21B} \left( 1 + \frac{A_b}{3L^2} \right) \right] \left[ 1 + 0.19 \left( \frac{A_w}{A_b} \right)^2 \right]
\]  Eq. 6-4
where \( L \) and \( B \) are half-length and half-width of footing, \( G \) is the soil shear modulus, \( D \) is the embedment depth (from the ground surface to bottom surface of footing), \( \nu \) is Poisson’s ratio of soil, \( A_b \) is the bearing area = \( 2L \times 2B \), \( A_w \) is the sidewall contact area = \( 2(2L+2B)d \), where \( d \) is the thickness of the footing, and \( S_z \) is a vertical stiffness parameter for an equivalent circular foundation approximation = \( 0.73 + 1.54 \left( \frac{A_b}{4L^2} \right)^{0.75} \).

![Experimental impedance curve resulting from dynamic testing.](image)

The shear modulus of the soil can be approximated if appropriate values for footing geometry are plugged in Eq. 6-4. \( K_{emb} \) of \( \sim 60,000 \text{ k/in} \) and \( \nu \) of 0.33 for sand are used in this case. The geometric parameters previously obtained are used as well to determine \( G \). Using Eq. 6-4, the \( G \) estimated from the explored geometry and actual geometry is 451 MPa and 441 MPa respectively, representing an adequate approximation. Based on the obtained value of shear modulus, the soil can be classified as very dense sand. However, the values obtained for shear modulus are slightly higher than \( G \) obtained using the shear wave velocity profile shown in Fig. 3-17 using \( G = \rho V_s^2 \), which is 369.8 MPa. It is idealized that load resistance/bearing is attributed to the soil properties at the soil-footing interface. Nevertheless, there is a contribution from deeper soil elements through which failure
wedges extend, which have even higher shear modulus or strength parameters than that at the footing depth. This leads to an apparent increase in shear modulus.

Using estimated dimensions led to a fair estimation of shear modulus. This means that if the dynamic force and response are measured accurately, the dimension of the footing can be iterated to reach the best match with the shear wave velocity profile using the impedance function obtained from testing. However, running a full parametric sweep will all combinations while simultaneously the shear wave velocity and foundation dimensions is required to fully prove this concept. Nevertheless, the findings reported herein are promising since they promote the use of large-amplitude mobile shakers as a non-destructive method for exploring unknown foundations. Furthermore, once the geometry of the footing has been determined, the ultimate bearing capacity can be evaluated as illustrated in the next section.

6.3. Estimation of Ultimate Bearing Capacity

After the exploration of unknown foundation material and geometric parameters, a direct application of this information is the determination of the ultimate bearing capacity of bridge foundations. This is one of the criteria for assessing foundation reuse potential. The determination of bearing capacity can vary depending on the level of complexity of the analysis. This includes using empirical correlations between shear strength parameters such as shear wave velocity, SPT values, and shear modulus to strength parameters. In addition, a settlement-based analysis can be implemented through which FEM modeling is used to determine bearing pressure. A full nonlinear analysis involving DSSI and nonlinear/plastic soil behavior of the explored foundation using constitutive material models is another alternative for determining the bearing capacity. In this research, only
the first two approaches (general shear failure and serviceability limit) are entertained (general shear failure and serviceability limit). The goal remains the same for all approaches, which is to find a link between small strain properties typically obtained from field measurements to large strains occurring at failure, and hence shear strength parameters.

6.3.1. Determination of Bearing Capacity Using Empirical Relationships Based on General Shear Failure

For the estimation of ultimate bearing capacity, classical methods based on empirical formulations require knowledge of the geometry of the footing and soil material properties. The geometric aspects include the footing dimensions, ground slope, and depth. As for material properties, shear strength parameters (angle of internal friction and/or cohesion) are required, in addition to the unit weight of soil. Furthermore, the groundwater table position is essential in determining the overburden pressure and appropriate unit weight to be used in the bearing capacity estimate. Moreover, load inclination and loading eccentricity can also affect the ultimate bearing capacity of a footing. The only measured soil parameter that can be correlated to the shear strength in the current study was the shear wave velocity. The shear wave velocity of sand is inherently related to its shear strength parameters, although it is measured at small strain. This is due to the dependence of the angle of internal friction on the void ratio and confining pressure. There is an inverse relationship between the void ratio and shear wave velocity. Another advantage of using the shear wave velocity as an indicator of shear strength parameters in sands is that there is a negligible effect of moisture on the shear wave velocity (Dong and Lu, 2016). Hence, a changing position of the groundwater table will have minimal effects on the value of
shear wave velocity. Nevertheless, it would greatly affect the effective stresses, which must be accounted for in the calculation of ultimate bearing capacity.

A relationship correlating $V_s$ of sand to the bearing capacity $q_u$ is presented in Eq. 6-5 (Tezcan and Ozdemir, 2012).

$$q_u = 0.4\gamma V_s$$  \hspace{1cm} \text{Eq. 6-5}$$

where $\gamma$ is the unit weight of the soil, the factor 0.4 is a time factor obtained from a calibration process based on soil type, and $V_s$ is the shear wave velocity. Using Eq. 6-5 yields a value of 3,453 kPa (72 ksf). A study was performed by (Kumar et al., 2016) correlating field SPT values (N) to soil parameters. They proposed Eq. 6-6 to determine the angle of internal friction ($\varphi$).

$$\varphi = 27.12^\circ + 0.2857N$$  \hspace{1cm} \text{Eq. 6-6}$$

Using the SPT value (40) for sand reported from the test bores, the estimated angle of internal friction was found to be 38.5°. The determined values of the angle of internal friction along with the estimated shear wave velocity are reasonable for dense sand. This obtained value was then used in Vesic’s method of determining the ultimate bearing capacity (Vesić, 1973) as shown in Eq 6-7. Cohesion was set as 0 since it is a sandy site.

$$q_u = \gamma D_l N_q S_q d_q i_q b_q + 0.5\gamma B N_l S_l d_l i_l b_l$$  \hspace{1cm} \text{Eq. 6-7}$$

where $N_q$ and $N_\gamma$ are bearing capacity factors, $S_q$ and $S_\gamma$ are shape factors, $d_q$ and $d_\gamma$ are depth factors, $i_q$ and $i_\gamma$ are inclination factors, and $b_q$ and $b_\gamma$ are base factors.

As for the groundwater table, a conservative approach was used in this study and the footing was submerged, although the measurements were conducted in the dry season. This is the case because the bridge is an overpass over a channel, which may submerge the footing in the wet season. Another practical assumption undertaken was that any moment
acting on the footing is negligible compared to the axial load; load distribution is uniform on the bottom of the footing. In addition, the load was considered to be acting vertically and centric from the pier onto the footing. The ultimate bearing capacity determined using Eq. 6-7 for the explored and actual geometries is 3,277 kPa (68 ksf) and 3,432 kPa (72 ksf), respectively. The estimated bearing capacity of the unknown foundation was slightly lower than that of the actual dimensions, indicating a conservative estimate. This discussion is not intended to provide an exhaustive survey of field measurements and empirical correlations pertinent to soil strength parameters. Nevertheless, the results presented in this research show that whether using an empirical correlation of in-situ measurements (such as Eq. 6-5) or classical bearing capacity solution (such as Eq. 6-7), the bearing capacity can be estimated reasonably well through the approach proposed in this research. However, a comprehensive comparison of several suggested relationships between field measurements and correlation to soil strength parameters which are validated by soil strength tests is required to assess the bearing capacity more accurately.

6.3.2. Determination of Bearing Capacity Based on Serviceability Limits

A step further in determining bearing capacity would be a displacement-based approach. The bearing capacity in this case would be defined as the average contact pressure between the soil and the bottom of the footing for a specified allowable settlement. Soil is a nonlinear material, and its behavior is strain-dependent. Typically, this behavior is described by the modulus degradation curve, which describes how the maximum shear modulus decreases and material damping increases as shearing strain increases. Fig. 6-16 illustrates a typical degradation curve of soils.
Modeling of bridge foundations for displacement estimation should include the nonlinear behavior of soil. For modeling soil dynamics in earthquake engineering problems, a hyperbolic law is typically used to describe the secant modulus $G_s$ at any given $\gamma$. Eq. 6-8 presents the hyperbolic law used in the current study to describe the nonlinear behavior of soil.

$$G_s = G_{\text{max}} \frac{1}{1 + \left(\frac{\gamma}{\gamma_{\text{ref}}}\right)^n}$$  \hspace{2cm} \text{Eq. 6-8}$$

where $\gamma$ is the current shear strain, $\gamma_{\text{ref}}$ is the reference shear strain, and $n$ is a fitting exponent. Setting $n=1$ yields the Hardin-Drnevich model, which is one the most commonly used models to describe stiffness reduction of soil with shear strain (Hardin and Drnevich, 1972). Nevertheless, using values other than 1 for $n$ the hyperbolic law provides a better fit to the data, and those values can be experimentally determined for various soil conditions (Stokoe et al., 1999). Menq (2003) conducted an extensive study on sandy and gravelly soils using resonant column testing to determine $n$ and $\gamma_{\text{ref}}$. The reference strain $\gamma_{\text{ref}}$ can be defined as the strain at which the soil behavior shifts from linear to nonlinear, corresponding to a drop of 30% on shear modulus (or $G=0.7G_{\text{max}}$) typically.
From the results of Menq (2003), values for $\gamma_{\text{ref}} = 0.045$ and $n=0.88$ were selected based on the similarity of soil conditions. As for $G_{\text{max}}$, the lowest estimate obtained from field measurements was used (370 MPa). COMSOL Multiphysics was used to sweep the load in a stationary study to obtain a pressure-displacement curve, emulating a plate test. Fig 6-17 shows the pressure-settlement curve obtained for the tested bridge pier. According to AASHTO, the maximum allowable angular distortion (settlement/span length) for a continuous-span bridge is 0.004 (AASHTO, 2017). The span length for this bridge is 40’; hence, the allowable settlement is 48.8 mm (1.92”). The obtained bearing pressure at 48.8 mm was 3231 kPa, which is close to the bearing capacity obtained from empirical estimations.

![Fig. 6-17. Bearing pressure-settlement plot from FEM simulation of the tested bridge pier.](image)

While this outcome shows that it is possible to obtain the bearing capacity using field measurements in tandem with FEM, it serves as a proof of concept of the proposed methodology. More testing is required to verify and validate the applicability of this method in similar site conditions as well as different site conditions. Furthermore, a more representative simulation of the pressure-settlement behavior is a full dynamic study in
which the resonant frequencies and corresponding peak amplitudes will change with softening of the SFS system, in addition to dynamic amplification effects compared to a static study. On the other hand, damping will increase as the soil becomes more nonlinear. Those combined effects necessitate a fully dynamic nonlinear numerical study in the frequency domain to assess whether the response would change substantially. They can also be studied while considering DSSI effects and soil plasticity with Mohr-Coulomb or Drucker-Prager material models to describe failure.

To summarize, Chapter 6 demonstrates the applicability of large-mobile shakers combined with FEM to explore unknown foundations and estimate their bearing capacity based on field measurements and minimal known parameters. This is a crucial step in the assessment of the potential reuse of bridge foundations. Fitting time histories and comparing peak amplitudes from numerical models at the experimentally determined resonant frequencies highlighted the difference in response when super- and substructural features of the SFS system are altered. This enables the identification of the set of parameters that best fit the data and explores the foundation as a result. Once the foundation has been explored, the bearing capacity can be estimated. Two examples shown in this chapter illustrate this capability.
The primary goal of this research was to prove that large-amplitude mobile shakers in tandem with highly refined numerical simulations can be used for:

- Dynamic characterization of bridges, while informing DSSI effects.
- Exploration of unknown bridge foundations.

The technique implemented in this research promotes the use of large-amplitude mobile shakers as a global NDE tool for bridges. The quick and easy deployment of sensors at several locations on the bridge enables capturing of multiple dynamic features including the overall dynamic behavior, resonant frequencies, and damping characteristics. From this research, the specific findings are:

- The synergistic approach conducted in the current study involving the evaluation of DSSI effects and structural identification using large mobile shakers in tandem with well-defined 3D FEM models proved to be effective. This approach is applicable not just in the evaluation of bridges but also of other infrastructure and building structures assets. This was evident from the study by evaluating the quality of dynamic response, expressed through high coherence and clarity of features in the frequency response functions.

- A priori numerical modeling of a bridge facilitates better planning for field testing with an objective evaluation of the significance of DSSI effects on the bridge dynamic response. The model calibration following the field testing is essential for the development of the right conclusions about the significance of DSSI effects. This includes the evaluation of the representative shear wave velocity profile of the site.
• The vertical and transverse loading with a chirp function sweep from 15-1 Hz and 80-10 Hz enabled clear identification of natural frequencies (modes) in both bridges. Seven main modes were identified for Hobson Avenue Bridge; two lateral modes were observed at close frequencies of 4.39 Hz and 4.69 Hz, with an additional vertical mode between them at 4.44 Hz. Testing at the bridge at multiple locations can facilitate the detection of modes. In addition, chirp functions for vertical loading were utilized to identify the natural frequencies of the soil-structure system of the Gate Creek Bridge. Three modes were determined at 9.1 Hz, 14.8 Hz, and 26.2 Hz. The dominant eigenfrequency was 26.2 Hz since it resulted in the highest response under the same driving load magnitude.

• The testing conducted in this research enabled the estimation of the composite damping ratio by examining material, structural, and radiation damping. In addition, the mechanical impedance for rocking was estimated for the Hobson Avenue Bridge. There was a notable coupling between swaying and rocking, especially for the mode at 4.39 Hz.

• The inclusion of DSSI effects led to a reduction in stresses (up to 35%) compared to a fixed-base assumption for the site of study at the Hobson Avenue Bridge due to lateral loads. This was observed through the models, with higher stresses at the joints when a fixed base is assumed.

• A parametric sweep was conducted for the Hobson Avenue Bridge to assess the effect of varying super- and substructural features of the SFS system. Peak amplitude, resonant frequency, and overall time history were shown to be unique for each combination, and this allows for discerning which set of parameters fit the
experimental results. Furthermore, the results obtained from evaluating the stiffness, embedment, and slenderness ratios in this study agreed with the typical dynamic behavior reported in the literature.

- Evaluation of stresses due to vertical loading highlights the effect of superstructure rigidity on stresses in the deck. This can provide helpful insights into monitoring bridge deck conditions and assessing expected vibrations due to operational load levels. In general, having a more rigid superstructure leads to lower stress in the deck; hence, improving its durability.

- A parametric sweep was conducted for the Gate Creek Bridge to assess the effects of material and geometric parameters of the footing on the response. This was done as part of an exploration of an unknown foundation. The parameters included were footing depth, half-width, half-length, and soil shear velocity. The peak amplitude and time history fit were used to estimate relative and RMS errors respectively. These were the criteria to determine which parameters are most likely to be close to the actual conditions. Combining the least error-producing parameters produced a close estimate of the size and depth of embedment of the unknown foundation compared to the actual foundation.

- Estimating the soil shear modulus was conducted for a scenario of estimated footing dimensions based on the substructure dimensions and was compared to the actual geometry. Both cases yielded shear moduli comparable to that obtained using the shear wave velocity profile.

- Large-amplitude mobile shakers can be used for foundation exploration if the steady-state force and response of footing - hence impedance functions - are
accurately measured. This was done for the soil/bridge system evaluated in the current study, and the bearing capacity was estimated using field measurement correlations to and a settlement-based approach.

The results demonstrated in this research open opportunities for more ideas in the field of dynamic NDE characterization of bridges. Some of the ideas and challenges to be tackled in the future are:

- The ability of machine learning algorithms to produce model-free estimates of bearing capacity needs to be evaluated. This requires building a library of data considering several material and geometric features of bridge/foundation systems for both fixed and DSSI models. For that, numerous FEM parametric sweeps including such factors are required. This study can serve as a cornerstone for this idea. Furthermore, error minimization can be implemented in exploring unknown foundations when evaluating the resonant frequencies, peak amplitudes, and time-history fits from parametric sweeps. This can be done for all combinations of parameters from which the foundation can be determined.

- A full parametric study on the effects of DSSI on the vertical response of bridges like the one developed for the lateral response can be conducted. This would enable the assessment of DSSI effects on altering the bridge response under traffic loads compared to a fixed-base assumption.

- A fully dynamic nonlinear approach in numerical simulations can be used to explore unknown foundations and their bearing capacity while encapsulating general failure, serviceability limits, and extreme events such as earthquakes and liquefaction.
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APPENDIX A: Supplementary Information for Preliminary Site Investigations
Overview:

Appendix A provides the results of the MASW performed at the Hobson Avenue Bridge, New Jersey for the determination of the shear wave velocity profile shown in Fig. 3-7.
Fig. A-1. Site layout documenting the location of single station horizontal-to-vertical (H/V) spectral ratio noise measurements made in the free-field (i.e., well-separated from the structural system) and the 23-m long Multichannel Analysis of Surface Waves (MASW) linear array. Each single station measurement location is denoted relative to the Hobson Avenue Bridge over Interstate 195 (i.e., NW, NE, SW, and SE).
Fig. A-2. Shown for each inversion parameterization are the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southeast (SE) station location; (c) Vs profiles constrained solely by the experimental dispersion data; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\ln Vs}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].
Fig. A-3. Shown for each inversion parameterization are the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southeast (SE) station location; (c) Vs constrained by the experimental dispersion and the H/V data from the SE station location; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\lnVs}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].
Fig. A-4. Shown for each inversion parameterization is the 100 lowest misfits: (a) theoretical fundamental mode Rayleigh wave dispersion curves with the experimental dispersion data; (b) theoretical Rayleigh wave ellipticity with the lognormal median experimental H/V data from the southwest (SW) station location; (c) Vs constrained by the experimental dispersion and the H/V data from the SW station location; and (d) standard deviation of the natural logarithm of Vs ($\sigma_{\lnVs}$). The layering ratio ($\Xi$) representing each inversion parametrization is shown in the figure legend located in panel (d). The layering ratio is followed by the number of layers in the parameterization (inside parentheses) and the range of misfit values for the 100 lowest misfit profiles [inside brackets].
APPENDIX B: 2D FEM Model for Incorporating DSSI Effects
2D Numerical Model:

The total mechanical impedance of a soil-foundation-structure system can be represented in the $S(\omega) = K(\omega) + i\omega C(\omega)$ form, in which the real part represents the stiffness, while the imaginary part represents the damping. This impedance function can be also written in the form: $K_i = K_{is}[k(\omega) + ia_0c(\omega)]$, where $K_i$ is the total impedance for the $i^{th}$ degree of freedom, $K_{is}$ is the static stiffness of the same degree of freedom, $k(\omega)$ is the stiffness coefficient, $c(\omega)$ is the damping coefficient, $a_0 = \frac{\omega B}{V_s}$ is the dimensionless frequency, $\omega$ is the driving frequency, $B$ is the half-width (or radius) of the footing, and $V_s$ is the shear wave velocity of the soil. Gazetas (1991) has developed simplified closed-form solutions for foundations on an elastic half-space, including the effects of foundation shape, embedment, and uniformity of soil. Based on different scenarios, static stiffness is calculated by knowing the shear modulus of rigidity ($G$) and geometry of the footing, and then multiplying it by the dynamic coefficients corresponding to the dimensionless frequency of the driving excitation to obtain the dynamic stiffness of the footing. Fig. B-1 presents an example of the dynamic coefficients used to incorporate the DSSI, demonstrating the change in vertical impedance coefficients as a function of dimensionless frequency. The graphs from Gazetas (1991) were reproduced by the means of cubic splines. This was also conducted for the stiffness and damping of rocking and swaying modes.

For the 2D model shown in Fig. B-2, translational and rotational springs of the footing were modeled as those of a surface circular footing on an elastic homogeneous half-space as a preliminary approximation of the DSSI. A parametric study was conducted to assess the effect of changing $V_s$, the radius of footing ($R$), and slenderness ratio ($H/W$) where $H$ is the height, and $W$ is the width of the pier. For the current study, the height was
changed, and the pier width was held constant. Table B-1 illustrates the parameters included in the 2D FEM model of the pier plane section.

![Graphs](image)

Fig. B-1. Dynamic swaying and rocking coefficients of a surface rectangular footing.

![2D Model](image)

Fig. B-2. 2D model representing typical bridge pier frames.
Table B-1. Parameters Included in the 2D FEM Model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Expression</th>
<th>Value [units]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_s$</td>
<td>Shear Wave Velocity</td>
<td>-</td>
<td>200, 300, 400 [m/s]</td>
</tr>
<tr>
<td>R</td>
<td>Footing Radius</td>
<td>-</td>
<td>2, 3, 4 [m]</td>
</tr>
<tr>
<td>$H_c$</td>
<td>Column Height</td>
<td>-</td>
<td>3-12 @0.5 increments [m]</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Poisson’s Ratio of Soil</td>
<td>-</td>
<td>1/3</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Soil Density</td>
<td>$\rho_s (V_s)^2$</td>
<td>Varies: 7.61 x $10^7$ to 3.04 x $10^8$ [Pa]</td>
</tr>
<tr>
<td>G</td>
<td>Modulus of R rigidity</td>
<td>$\frac{8GR}{2 - \mu}$</td>
<td>Varies: 7.3 x $10^8$ to 5.84 x $10^9$ [N/m]</td>
</tr>
<tr>
<td>$K_s$</td>
<td>Static Sliding Stiffness</td>
<td>$\frac{8GR}{3(2 - \mu)}$</td>
<td>Varies: 9.73 x $10^8$ to 3.11 x $10^{10}$ [N/rad]</td>
</tr>
<tr>
<td>$K_r$</td>
<td>Static Rocking Stiffness</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. B-3 shows the at-resonance peak response at the deck level due to a unit harmonic load applied transversely at the center of the deck. It illustrates that rigidity increases as $R.V_s$ increases, while the response peak amplitude diminishes from 0.0293 m for a flexible foundation (2,200) to 0.014 m as the base becomes infinitely rigid (fixed base), which is expected. An inverse relationship is observed between the resonant frequency and $H/W$. There is also an effect on the curvature of the frequency-$H/W$ graph. Moreover, a fixed-base model would experience greater stress levels compared to the rocking foundation as illustrated by the response in Fig. B-4, by which rigidity leads to the restraint of the footing and smaller rotations. Rotation of the deck decreased from 0.0032 rad in the flexible foundation (2,200) to around 0.002 rad in the fixed-base model. The effect of changing $H/W$ on rotation is more pronounced than displacement due to the reduced overall rigidity. Therefore, determining the rocking stiffness of the foundation is crucial for predicting the overall response of the system.
Fig. B-3. Displacement [m] from 2D FEM model at variable rigidity (R,Vs); a) [2,200], b) [3,300], c) [4, 400], d) [fixed base].

Fig. B-4. Rotation [rad] from 2D FEM model at variable rigidity (R,Vs); a) [2,200], b) [3,300], c) [4, 400], d) [fixed base].
APPENDIX C: Hobson Avenue Bridge Response Evaluation for Determining Resonant Frequencies
Overview:

Appendix C provides a summary of the dynamic response of the bridge at multiple locations from varying load magnitudes, directions, and location. The main observations from summarizing the results in this manner were the determined resonant frequencies and rocking behavior of the bridge.
Fig. C-1. T-Rex centered above the pier to shake the bridge transversely and corresponding transverse response in Fig. C-2 to Fig. C-6.
Fig. C-2. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.
Fig. C-3. Transverse response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them.
Fig. C-4. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them.
Fig. C-5. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C-6. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them.
Fig. C-7. T-Rex centered above the pier to shake the bridge transversely and corresponding vertical response in Fig. C-8 to Fig. C-12.
Fig. C-8. Vertical response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.
Fig. C-9. Vertical response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them.
Fig. C-10. Vertical response to transverse shaking @ 21 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them.
Fig. C-11. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C-12. Transverse response to transverse shaking @21 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them.
Fig. C-13. T-Rex centered above the pier to shake the bridge vertically and the corresponding vertical response in Fig. C-14 to Fig. C-18.
Fig. C-14. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.
Fig. C-15. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them.
Fig. C-16. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the pier at the same side, and (d) the phase shift between them.
Fig. C-17. Vertical response to vertical shaking @12 kips with T-Rex above the pier showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the ground sensor at the same side, and (d) the phase shift between them.
Fig. C-18. Vertical response to vertical shaking @ 12 kips with T-Rex above the pier showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the ground sensor at the other side, and (d) the phase shift between them.
Fig. C-19. T-Rex centered above the mid-span to shake the bridge transversely and corresponding transverse response in Fig. C-20 to Fig. C-24.
Fig. C-20. Transverse response to transverse shaking @15 k with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.
Fig. C-21. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with same side at the mid-span, and (d) the phase shift between them.
Fig. C-22. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them.
Fig. C-23. Transverse response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C-24. Transverse response to transverse shaking at 15 kips with T-Rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them.
Fig. C-25. T-Rex centered above the mid-span to shake the bridge transversely and corresponding vertical response in Fig. C-26 to Fig. C-30.
Fig. C-26. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side of the deck, and (d) the phase shift between them.
Fig. C-27. Vertical response to transverse shaking @ 15 kips with T-Rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them.
Fig. C-28. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them.
Fig. C-29. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C-30. Vertical response to transverse shaking @15 kips with T-Rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C-31. T-Rex centered above the mid-span to shake the bridge vertically and corresponding vertical response in Fig. C-32 to Fig. C-36.
Fig. C-32. Vertical response to vertical shaking @ 10.8 kips with T-Rex above the mid-span showing: (a) time history of east deck sensor, (b) its corresponding power spectrum, (c) transfer function ratio with other side of the deck, and (d) the phase shift between them.
Fig. C-33. Vertical response to vertical shaking @ 10.8 kips with T-rex above the mid-span showing: (a) time history of west mid-span sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the mid-span, and (d) the phase shift between them.
Fig. C-34. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of west pier sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the pier, and (d) the phase shift between them.
Fig. C-35. Vertical response to vertical shaking @10.8 kips with T-rex above the mid-span showing: (a) time history of west ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the same side at the ground sensor, and (d) the phase shift between them.
Fig. C.36. Vertical response to vertical shaking @ 10.8 kips with T-rex above the mid-span showing: (a) time history of east ground sensor, (b) its corresponding power spectrum, (c) transfer function ratio with the other side at the ground sensor, and (d) the phase shift between them.
APPENDIX D: Supplementary Information from Numerical Simulations
Overview:

Appendix D provides supplementary calculations performed to determine the modal participation factors estimated in Chapter 5.
Table D-1. Calculation of MPF of each Mode for Translation along Y.

<table>
<thead>
<tr>
<th>Mode</th>
<th>$\gamma_x$ (kg${}^{0.5}$)</th>
<th>$M_{eff,i}$ (kg)</th>
<th>MPF$_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>62.363</td>
<td>3889.143769</td>
<td>0.003778069</td>
</tr>
<tr>
<td>2</td>
<td>1.9152</td>
<td>3.66799104</td>
<td>3.56323E-06</td>
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<tr>
<td>3</td>
<td>1.147</td>
<td>1.315609</td>
<td>1.27803E-06</td>
</tr>
<tr>
<td>4</td>
<td>0.66752</td>
<td>0.44558295</td>
<td>4.32857E-07</td>
</tr>
<tr>
<td>5</td>
<td>0.97864</td>
<td>0.95773625</td>
<td>9.30383E-07</td>
</tr>
<tr>
<td>6</td>
<td>0.23584</td>
<td>0.055620506</td>
<td>5.4032E-08</td>
</tr>
<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>3895.587449</td>
<td>0.003784328</td>
</tr>
</tbody>
</table>

Table D-2. Calculation of MPF of each Mode for Translation along Z.

<table>
<thead>
<tr>
<th>Mode</th>
<th>$\gamma_x$ (kg${}^{0.5}$)</th>
<th>$M_{eff,i}$ (kg)</th>
<th>MPF$_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.26141</td>
<td>0.068335188</td>
<td>6.63835E-08</td>
</tr>
<tr>
<td>2</td>
<td>0.47685</td>
<td>0.227385923</td>
<td>2.20892E-07</td>
</tr>
<tr>
<td>3</td>
<td>49.9</td>
<td>2490.01</td>
<td>0.002418895</td>
</tr>
<tr>
<td>4</td>
<td>815.28</td>
<td>664681.4784</td>
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<tr>
<td>5</td>
<td>26.375</td>
<td>695.640625</td>
<td>0.000675773</td>
</tr>
<tr>
<td>6</td>
<td>0.11772</td>
<td>0.013857998</td>
<td>1.34622E-08</td>
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<tr>
<td>7</td>
<td>18.097</td>
<td>327.501409</td>
<td>0.000318148</td>
</tr>
<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>668194.94</td>
<td>0.649111074</td>
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</table>

Table D-3. Calculation of MPF of each Mode for Rotation about the X-axis.

<table>
<thead>
<tr>
<th>Mode</th>
<th>$\gamma_x$ (kg${}^{0.5}$m)</th>
<th>$M_{eff,i}$ (kg.m$^2$)</th>
<th>MPF$_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14091</td>
<td>198556281</td>
<td>0.678519839</td>
</tr>
<tr>
<td>2</td>
<td>512.81</td>
<td>262974.0961</td>
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<tr>
<td>3</td>
<td>32.687</td>
<td>1068.439699</td>
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</tr>
<tr>
<td>4</td>
<td>0.82534</td>
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</tr>
<tr>
<td>5</td>
<td>16.283</td>
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<td>9.06041E-07</td>
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<tr>
<td>6</td>
<td>21.359</td>
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</tr>
<tr>
<td>7</td>
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<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>198821091.7</td>
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</tr>
</tbody>
</table>

Table D-4. Calculation of MPF of each Mode for Rotation about the Y-axis.

<table>
<thead>
<tr>
<th>Mode</th>
<th>$\gamma_x$ (kg${}^{0.5}$m)</th>
<th>$M_{eff,i}$ (kg.m$^2$)</th>
<th>MPF$_z$</th>
</tr>
</thead>
<tbody>
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<td>1</td>
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<tr>
<td>3</td>
<td>1346</td>
<td>1811716</td>
<td>0.184974325</td>
</tr>
<tr>
<td>4</td>
<td>2.6845</td>
<td>7.20654025</td>
<td>7.3578E-07</td>
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<tr>
<td>5</td>
<td>2660</td>
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<td>6</td>
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<td>2.41889E-08</td>
</tr>
<tr>
<td>7</td>
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<td>225.360144</td>
<td>2.3009E-05</td>
</tr>
<tr>
<td>-</td>
<td>$\Sigma$</td>
<td>8887564.09</td>
<td>0.907411078</td>
</tr>
</tbody>
</table>