

UC San Diego

UC San Diego Electronic Theses and Dissertations

Title

Effects of Liquefaction on Earthquake Ground Motions

Permalink

<https://escholarship.org/uc/item/3q3212r5>

Author

Gingery, James Robert

Publication Date

2014

Peer reviewed|Thesis/dissertation

UNIVERSITY OF CALIFORNIA, SAN DIEGO

Effects of Liquefaction on Earthquake Ground Motions

A dissertation submitted in partial satisfaction of the requirements for the
degree of Doctor of Philosophy

in

Structural Engineering

by

James Robert Gingery

Committee in charge:

Professor Ahmed Elgamal, Chair
Professor Duncan Agnew
Professor Jonathan D. Bray
Professor Patrick Fox
Professor Tara Hutchinson

2014

Copyright
James R. Gingery, 2014
All rights reserved

The Dissertation of James Robert Gingery is approved, and it is acceptable in
quality and form for publication on microfilm and electronically:

Chair

University of California, San Diego

2014

Table of Contents

Signature Page.....	iii
Table of Contents	iv
List of Figures	vii
List of Tables.....	xxiii
List of Acronyms, Abbreviations and Symbols	xxiv
Acknowledgements	xxix
Vita	xxxii
Abstract of the Dissertation	xxxv
Chapter 1 Introduction and Background.....	1
1.1 Characteristics of Soil Liquefaction Behavior	1
1.2 Review of Literature Related to Soil Liquefaction and Ground Motions.....	4
1.2.1 Youd and Carter (2005).....	5
1.2.2 Zeghal and Elgamal (1994) and Elgamal et al. (1996)	6
1.2.3 Zorapapel and Vucetic (1994)	7
1.2.4 Hartvigsen (2007).....	8
1.3 Requirements of Design Codes.....	10
1.4 Objective and Scope	11
1.5 Dissertation Outline	13
1.6 References	13
Chapter 2 Recorded Ground Motions at Liquefied Sites	16
2.1 Introduction.....	16
2.2 Ground Motion Database	17
2.2.1 Time Series Processing.....	41
2.2.2 Site and Earthquake Characteristics	42
2.3 Acknowledgment	50
2.4 References	51
Chapter 3 Temporal and Frequency Analysis of Ground Motions... 55	
3.1 Timing of Liquefaction Initiation	55
3.2 Timing of Spectral Accelerations	62

3.3	Discussion and Conclusions.....	71
3.4	References	71
	Chapter 4 Empirical Liquefaction Amplification Model	73
4.1	Evaluation of Recorded Motions at Liquefaction Sites	73
4.1.1	NGA Relationships	73
4.1.2	Directivity	74
4.1.3	Computation of Median Spectra and Residuals	80
4.1.4	Discussion	90
4.2	Liquefaction Amplification Model	98
4.3	Conclusions	110
4.4	Acknowledgment	112
4.5	References	113
	Chapter 5 Site Response Model Calibration and Validation.....	116
5.1	Introduction.....	116
5.2	Numerical Model Description.....	116
5.2.1	OpenSees Finite Element Program	116
5.2.2	UCSD Soil Constitutive Models	117
5.3	PDMY2 and PIMY Models and their Calibration.....	117
5.3.1	Drained Response.....	119
5.3.2	Undrained Response.....	142
5.3.3	Summary of Calibrated Parameters	154
5.3.4	One-Dimensional Non-Linear Site Response Modeling Protocols	155
5.4	Model Validation for Non-Liquefaction Conditions.....	158
5.4.1	Stewart et al. (2008) Validation	158
5.4.2	Validation in PRENOLIN Program	159
5.5	Model Validation Using Liquefied Site Case Histories.....	160
5.5.1	Wildlife Liquefaction Array	160
5.5.2	Kobe Port Island	176
5.6	Summary and Conclusions.....	187
5.7	References	188
	Chapter 6 Parametric Site Response Analyses	194

6.1	Introduction.....	194
6.2	Soil Profiles	195
6.3	Input Ground Motions.....	196
6.4	Analysis Procedures.....	205
6.5	Results and Conclusions	207
6.5.1	Fidelity of Models' Liquefaction Triggering Predictions.....	207
6.5.2	Example Time Histories.....	216
6.5.3	Overall Amplification Factors and r_u Dataset Filtering	222
6.5.4	Comparison of Site Response and Empirical Data.....	225
6.5.5	Examination of Parameter Dependence.....	229
6.6	Summary and Conclusions.....	240
6.7	References	241
	Chapter 7 Summary and Conclusions.....	243
7.1	Summary	243
7.2	Conclusions.....	245
7.2.1	Empirical Data Studies	245
7.2.2	Site Response Modeling.....	246
7.3	Recommendations for Future Research.....	250
7.4	References	251
	Appendix A Empirical Site and Source Information	252
	Appendix B Summary of PDMY2 and PIMY Models.....	261
	Appendix C Element Calibration Test Results	278
	Appendix D Input Ground Motions.....	364
	Appendix E Site Response Analysis Parameter Sensitivity	448

List of Figures

Figure 1-1: Undrained cyclic simple shear test of a medium dense (relative density = 63 percent) by Wu (2002).....	3
Figure 1-2: Typical hysteric behavior of drained sand and undrained clay after (Seed and Idriss 1970).....	4
Figure 1-3: Spectral Ratio predictions from Hartvigsen (2007).	10
Figure 2-1: Magnitude and distance pairs represented in the liquefaction ground motion database.	20
Figure 2-2: Hachirogata station recording of the M7.7 Nihonkai-Chibu, Japan earthquake of 1983.	21
Figure 2-3: Wildlife Liquefaction Array recording of the M6.54 Superstition Hills, California earthquake of 1987.	22
Figure 2-4: Treasure Island recording of the M6.9 M 6.93 Loma Prieta earthquake of 1989.	23
Figure 2-5: Amagasaki-G recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.	24
Figure 2-6: Amagasaki No. 4 P.P., KE recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.	25
Figure 2-7: Higashi-Kobe Bridge recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.	26
Figure 2-8: Kobe-JI-S recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.	27
Figure 2-9: Port Island ground surface recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.	28
Figure 2-10: NIG018 recording of the M6.6 Niigata-Ken Chuetsu-OkI, Japan earthquake of 2007.	29
Figure 2-11: GDLC recording of the M7.1 Darfield, New Zealand earthquake of 2010.	30
Figure 2-12: HPSC recording of the M7.1 Darfield, New Zealand earthquake of 2010.	31
Figure 2-13: SMTc recording of the M7.1 Darfield, New Zealand earthquake of 2010.	32
Figure 2-14: CBGS recording of the M6.3 Christchurch, New Zealand earthquake of 2011.	33
Figure 2-15: CCCC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.	34

Figure 2-16: CHHC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.....	35
Figure 2-17: HPSC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.....	36
Figure 2-18: PRPC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.....	37
Figure 2-19: REHS recording of the M6.3 Christchurch, New Zealand earthquake of 2011.....	38
Figure 2-20: SHLC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.....	39
Figure 3-1: From Zeghal and Elgamal (1994), WLA Superstition Hills earthquake: (a) surface time history; (b) time history at depth of 7.5 m; (c) excess pore pressure ratio at depth of YY m; and (d) average V_s over time.....	59
Figure 3-2: From Elgamal et al. (1996), Port Island recording of Kobe earthquake: surface time history in (a) EW and (b) NS directions, and (c) average V_s over time.....	61
Figure 3-3: (a) Phase lag time (t_{lag}) and (b) phase lag time error when assuming $t_{lag} = T/4$	66
Figure 3-4: Time of spectral acceleration relative to t_{liq} (a) and t_{liq} (b) for 19 liquefaction ground motions (2 components each).	69
Figure 3-5: Cumulative density functions of $t_{Sa}(T)-t_{liq}$ (a) and $t_{Sa}(T)-t_{liq}$ (b) for the 19, two-component horizontal liquefied site ground motions.	70
Figure 4-1: Examples of directivity amplification functions using Bayless & Somerville model (Spudich et al. 2013) for Mw 7 and (a) strike slip and (b) dip slip faults.	78
Figure 4-2: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.	81
Figure 4-3: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.	82
Figure 4-4: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.	83
Figure 4-5: Intra-Event residuals computed from NGA spectral accelerations. .	86
Figure 4-6: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of PGA	87
Figure 4-7: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) for various periods plotted versus V_{s30}	88

Figure 4-8: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) for various periods plotted versus M_w	89
Figure 4-9: Intra-event standard deviations for: NGA GMPEs for M7, $V_{s30}=200\text{m/s}$, $R_{rup}=5\text{km}$, vertical strike slip; liquefaction site ground motion residuals; and liquefaction ground motion residuals from proposed model.	90
Figure 4-10: Means and standard deviations of intra-event residuals computed from NGA spectral accelerations all data (gray) and only ground motions with no apparent forward directivity pulse (black).	94
Figure 4-11: Example liquefaction amplification curves for AS08 and BA08 calculated with proposed model.....	101
Figure 4-12: Example liquefaction amplification curves for CB08 and CY08 calculated with proposed model.....	102
Figure 4-13: Example liquefaction amplification curves showing M_w dependence of the proposed model.	103
Figure 4-14: Intra-Event residuals computed from NGA GMPEs adjusted with the proposed liquefaction amplification model.	106
Figure 4-15: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of PGA	107
Figure 4-16: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus V_{s30}	108
Figure 4-17: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of M_w	109
Figure 4-18: Response spectra computed using AS08 and BA08 NGA GMPEs with and without the adjustment by the proposed liquefaction amplification model. $V_{s30}=200\text{ m/s}$ for all cases.....	110
Figure 5-1: Example of the hysteretic shear stress-strain response of the PIMY model.	118
Figure 5-2: Comparison of G/G_{max} curves	124
Figure 5-3: (a) RCTS data used in development of Darendeli (2001) G/G_{max} curves and (b) shear strain ranges (Ishihara 1996).....	128
Figure 5-4: Example sand and clay profiles illustrating differences between estimated shear strengths, and shear strengths implied by a PRSH model (as calibrated by low/moderate shear strain levels).....	128

Figure 5-5: (a) Weighting values (W), and (b) G/G_{max} curves computed using the PRSH and GH models at various depths in the sand profile.....	133
Figure 5-6 Shear stress versus shear strain curves computed using the PRSH and GH models at various depths in the sand profile.....	134
Figure 5-7: (a) Weighting values (W), and (b) G/G_{max} curves computed using the PRSH and GH models at various depths in the clay profile.....	134
Figure 5-8: Shear stress versus shear strain curves computed using the PRSH and GH models at various depths in the clay profile.....	135
Figure 5-9: Strain-controlled loading pattern for drained CSS simulations.	140
Figure 5-10: Element configuration for strain-controlled cyclic simple shear simulations.....	140
Figure 5-11: Example results for $D_r = 70\%$ and $\sigma'_{v0} = 1$ atm drained cyclic simple shear simulation.....	142
Figure 5-12: Element configuration for stress-controlled cyclic simple shear simulations.....	144
Figure 5-13: Comparison of K_σ values from the calibrated model versus the relationship proposed by and Boulanger and Idriss (2004).....	146
Figure 5-14: CRR versus number of cycles computed for the calibrated model in CSS simulations at $\sigma'_{v0} = 1$ atm and various value of D_r	149
Figure 5-15: MSF values computed for calibrated model in CSS simulations compared to values recommended by Youd et al. (2001).....	150
Figure 5-16: Example of undrained cyclic simple shear test result of Monterey 0/30 Sand at $D_r = 60\%$ from Wu (2002).	152
Figure 5-17: Post-liquefaction shear strain per cycle from undrained CSS test by Wu (2002).	152
Figure 5-18: Post-liquefaction shear strain per cycle from undrained CSS simulation for $D_r = 60\%$	153
Figure 5-19: Schematic diagram of finite element mesh and boundary conditions for (a) compliant base conditions and (b) rigid base condition.	157

Figure 5-20: WLA geotechnical data used V_s and Dr values used for site response simulations (compiled from data from Youd et al. 2004 and 2007).....	163
Figure 5-21: Comparison of simulated and recorded ground motions for WLA 360 component Elmore Ranch earthquake.....	167
Figure 5-22: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 360 component Elmore Ranch earthquake.	168
Figure 5-23: Comparison of simulated and recorded ground motions for WLA 90 component Elmore Ranch earthquake.....	169
Figure 5-24: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 90 component Elmore Ranch earthquake.	170
Figure 5-25: Comparison of simulated and recorded ground motions for WLA 360 component Superstition Hills earthquake.....	171
Figure 5-26: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 360 component Superstition Hills earthquake.	172
Figure 5-27: Comparison of shear stress-strain for 360 component Superstition Hills event from (a) Zhegal and Elgamal (1994) (average over upper 7.5 m), and (b) computed for element from 2.5 to 3 m.....	173
Figure 5-28: Comparison of excess porewater pressure vs. time for Superstition Hills event (a) computed at depth of 2.75 m, and (b) measured at 2.9 m (Zeghal and Elgamal 1994).....	173
Figure 5-29: Comparison of simulated and recorded ground motions for WLA 90 component Superstition Hills earthquake.....	174
Figure 5-30: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 90 component Superstition Hills earthquake.	175
Figure 5-31: Kobe Port Island used V_s and Dr values used for site response simulations (after Nakakita and Watanabe, 1981; Iwasaki, 2009; Elgamal et al., 1996; and Cubrinovski, 1996).	178
Figure 5-32: Comparison of simulated and recorded ground motions for Port Island 00 component Kobe earthquake.	181
Figure 5-33: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 00 component Kobe earthquake.....	182

Figure 5-34: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 00 component Kobe earthquake.....	183
Figure 5-35: Comparison of simulated and recorded ground motions for Port Island 90 component Kobe earthquake.	184
Figure 5-36: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 90 component Kobe earthquake.....	185
Figure 5-37: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 90 component Kobe earthquake.....	186
Figure 5-38: Average shear stress vs. shear strain for the depth interval of 0 to 16 m calculated from the recorded PI motions by Elgamal et al. (1996).	187
Figure 6-1: Soil profiles used in the conducted site response analyses.	196
Figure 6-2: Plots showing characteristics of ground motions used as input in site response analyses.....	197
Figure 6-3: Response spectra for all the ground motions used in the site response analyses.....	203
Figure 6-4: Response spectra for each ground motion bin used in the site response analyses.....	204
Figure 6-5: Three examples (cases 432b, 341b and 379b) of initial and minimum vertical effective stress and maximum excess porewater pressure ratio profiles, and calculated factors of safety. The red-shaded zone is the potentially liquefiable layer. All have $z_{liq} = 3\text{m}$, $T_{liq} = 1\text{m}$ and groundwater is at 1 m, but differing D_r and input ground motions as noted at top of each figure.....	210
Figure 6-6: Three examples (cases 1302b, 1297b and 1297b) of initial and minimum vertical effective stress and maximum excess porewater pressure ratio profiles, and calculated factors of safety. The red-shaded zone is the potentially liquefiable layer. All have $D_r = 50\%$, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$ and groundwater is at 1 m, but input ground motions are different as noted at top of figure.....	211
Figure 6-7: Plots of excess porewater pressure ratio versus reciprocal of liquefaction factor of safety.	213
Figure 6-8: Examples of rates of r_u generation from element calibration CSS simulations.....	214
Figure 6-9: Plots of excess porewater pressure ratio versus liquefaction factor of safety.	215

Figure 6-10: Histogram of Idriss & Boulanger (2006) liquefaction factor of safety for cases with $r_u > 0.98$	216
Figure 6-11: CDF of Idriss & Boulanger (2006) liquefaction factor of safety for cases with $r_u > 0.98$	216
Figure 6-12: Site response results for case 1302 with $D_r = 50\%$, NGA No. 5483, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $\Delta u_{max} = 0.34$, $FS_{IB08} = 2.2$	219
Figure 6-13: Site response results for case 1297 with $D_r = 50\%$, NGA No. 751, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $\Delta u_{max} = 1.0$, $FS_{IB08} = 0.9$	220
Figure 6-14: Site response results for case 1311 with $D_r = 50\%$, NGA No. 1548, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $\Delta u_{max} = 1.0$, $FS_{IB08} = 0.74$, pulse-like.....	212
Figure 6-15: Response spectra and AF_{liq} for liquefied and non-liquefied cases. Black squares and error bars are mean ± 1 standard deviation.....	223
Figure 6-16: Dependency of AF_{liq} on $r_{u,max}$. Vertical dashed line is at $r_{u,max} = 0.95$	224
Figure 6-17: Response spectra and AF_{liq} for liquefied and non-liquefied cases, (only cases with $r_u \geq 0.98$). Black squares and error bars are mean ± 1 standard deviation.	226
Figure 6-18: Histograms of AF_{liq} for various periods (only $r_{u,max} \geq 0.98$).	227
Figure 6-19: Comparison of empirical and site response AF_{liq} values. Gray shaded areas and black lines are site response range and mean values. Gray circles and black squares are empirical data and mean values.....	228
Figure 6-20: Dependency of AF_{liq} on M_w (only $r_{u,max} \geq 0.98$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.	231
Figure 6-21: Dependency of AF_{liq} on PGA_{nonliq} (only $r_{u,max} \geq 0.98$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.	232
Figure 6-22: Dependency of AF_{liq} on CAV_5 (only $r_{u,max} \geq 0.98$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.	235

Figure 6-23: Dependency of AF_{liq} on D_r (only $r_{u,max} \geq 0.98$). Black squares show median values and error bars show 0.1 and 0.9 quantiles.....	236
Figure 6-24: Dependency of AF_{liq} on Z_{liq} (only $r_{u,max} \geq 0.98$). Black squares and show median values and error bars show 0.1 and 0.9 quantiles.....	237
Figure 6-25: Dependency of AF_{liq} on T_{liq} (only $r_{u,max} \geq 0.98$).....	238
Figure 6-26: Comparison of cases with $r_{u,max} \geq 0.98$ and (a) pulse-like input ground motion only, (b) non-pulse-like.....	239
Figure 6-27: Comparison of cases with $r_{u,max} \geq 0.98$ and (a) $180 \leq V_{s30} \leq 360$ m/s, (b) $V_{s30} > 400$ m/s.....	240
Figure A-1: Nihonkai-Chubu earthquake rupture plane and Hachirogata liquefaction site accelerometer station (triangle).....	253
Figure A-2: Hachirogata station soil profile after Kostodinov and Towhata (2002).	254
Figure A-3: Downhole V_s measurements from three seismic CPTs at Wildlife Liquefaction Array (Youd et al. 2007, NEES@UCSB 2012).	255
Figure A-4: Hyogoken-Nambu Kobe earthquake rupture plane and liquefaction site accelerometer stations (triangles).	256
Figure A-5: Niigata-Ken Chuetsu-Oki earthquake rupture plane and NIG018 liquefaction site accelerometer station (triangle).....	257
Figure A-6: Soil profile of the NIG018 K-Net strong motion station.....	258
Figure A-7: Darfield earthquake rupture plane and accelerometer station (triangles) used to compute inter-event residual.	259
Figure A-8: Darfield earthquake rupture plane, epicenter and liquefied site accelerometer stations (triangles).....	259
Figure A-9: Christchurch earthquake rupture plane and accelerometer station (triangles) used to compute inter-event residual.	260
Figure A-10: Christchurch earthquake rupture plane, epicenter and liquefied site accelerometer stations (triangles).....	260
Figure B-1: Conical yield surfaces in principal stress space and deviatoric plane (after Prevost 1985, Parra 1996, and Yang 2000).	263
Figure B-2: Backbone curve its piecewise-linear representation though and yield surfaces and tangent shear moduli.	266

Figure B-3: Schematic of constitutive model response showing (a) octahedral stress - effective confinement response, (b) - octahedral strain response, and (c) configuration of yield domain	269
Figure B-4: Definition of vector n in the flow rules (Eqs. B.7 and B.8).	275
Figure B-5: Examples of (a) drained and (b) undrained response of cyclic simple shear simulations.	271
Figure C-1: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=0.5$ atm, $CSR=0.086$, $\phi=23.5$, $\phi_{PT}=20$	280
Figure C-2: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	281
Figure C-3: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=1$ atm, $CSR=0.081$, $\phi=23.5$, $\phi_{PT}=20$	282
Figure C-4: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	283
Figure C-5: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=2$ atm, $CSR=0.076$, $\phi=23.5$, $\phi_{PT}=20$	284
Figure C-6: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	285
Figure C-7: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=4$ atm, $CSR=0.072$, $\phi=23.5$, $\phi_{PT}=20$	286
Figure C-8: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	287
Figure C-9: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=8$ atm, $CSR=0.069$, $\phi=23.5$, $\phi_{PT}=20$	288
Figure C-10: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	289
Figure C-11: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=16$ atm, $CSR=0.068$, $\phi=23.5$, $\phi_{PT}=20$	290
Figure C-12: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$	291
Figure C-13: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=0.5$ atm, $CSR=0.110$, $\phi=25.9$, $\phi_{PT}=20$	292
Figure C-14: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	293
Figure C-15: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=1$ atm, $CSR=0.101$, $\phi=25.9$, $\phi_{PT}=20$	294

Figure C-16: Drained CSS Simulation: PIMY, $D_r=40$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	295
Figure C-17: Undrained CSS Simulation: PDMY2, $D_r=40$, $\sigma'_{v0}=2$ atm, $CSR=0.093$, $\phi=25.9$, $\phi_{PT}=20$	296
Figure C-18: Drained CSS Simulation: PIMY, $D_r=40$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	297
Figure C-19: Undrained CSS Simulation: PDMY2, $D_r=40$, $\sigma'_{v0}=4$ atm, $CSR=0.087$, $\phi=25.9$, $\phi_{PT}=20$	298
Figure C-20: Drained CSS Simulation: PIMY, $D_r=40$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	299
Figure C-21: Undrained CSS Simulation: PDMY2, $D_r=40$, $\sigma'_{v0}=8$ atm, $CSR=0.083$, $\phi=25.9$, $\phi_{PT}=20$	300
Figure C-22: Drained CSS Simulation: PIMY, $D_r=40$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	301
Figure C-23: Undrained CSS Simulation: PDMY2, $D_r=40$, $\sigma'_{v0}=16$ atm, $CSR=0.080$, $\phi=25.9$, $\phi_{PT}=20$	302
Figure C-24: Drained CSS Simulation: PIMY, $D_r=40$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$	303
Figure C-25: Undrained CSS Simulation: PDMY2, $D_r=50$, $\sigma'_{v0}=0.5$ atm, $CSR=0.143$, $\phi=28.6$, $\phi_{PT}=20$	304
Figure C-26: Drained CSS Simulation: PIMY, $D_r=50$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	305
Figure C-27: Undrained CSS Simulation: PDMY2, $D_r=50$, $\sigma'_{v0}=1$ atm, $CSR=0.129$, $\phi=28.6$, $\phi_{PT}=20$	306
Figure C-28: Drained CSS Simulation: PIMY, $D_r=50$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	307
Figure C-29: Undrained CSS Simulation: PDMY2, $D_r=50$, $\sigma'_{v0}=2$ atm, $CSR=0.118$, $\phi=28.6$, $\phi_{PT}=20$	308
Figure C-30: Drained CSS Simulation: PIMY, $D_r=50$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	309
Figure C-31: Undrained CSS Simulation: PDMY2, $D_r=50$, $\sigma'_{v0}=4$ atm, $CSR=0.109$, $\phi=28.6$, $\phi_{PT}=20$	310
Figure C-32: Drained CSS Simulation: PIMY, $D_r=50$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	311

Figure C-33: Undrained CSS Simulation: PDMY2, Dr=50, $\sigma'_{v0}=8$ atm, CSR=0.103, $\phi=28.6$, $\phi_{PT}=20$	312
Figure C-34: Drained CSS Simulation: PIMY, Dr=50, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	313
Figure C-35: Undrained CSS Simulation: PDMY2, Dr=50, $\sigma'_{v0}=16$ atm, CSR=0.100, $\phi=28.6$, $\phi_{PT}=20$	314
Figure C-36: Drained CSS Simulation: PIMY, Dr=50, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$	315
Figure C-37: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=0.5$ atm, CSR=0.193, $\phi=31.3$, $\phi_{PT}=20$	316
Figure C-38: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	317
Figure C-39: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=1$ atm, CSR=0.170, $\phi=31.3$, $\phi_{PT}=20$	318
Figure C-40: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	319
Figure C-41: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=2$ atm, CSR=0.152, $\phi=31.3$, $\phi_{PT}=20$	320
Figure C-42: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	321
Figure C-43: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=4$ atm, CSR=0.139, $\phi=31.3$, $\phi_{PT}=20$	322
Figure C-44: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	323
Figure C-45: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=8$ atm, CSR=0.129, $\phi=31.3$, $\phi_{PT}=20$	324
Figure C-46: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	325
Figure C-47: Undrained CSS Simulation: PDMY2, Dr=60, $\sigma'_{v0}=16$ atm, CSR=0.124, $\phi=31.3$, $\phi_{PT}=20$	326
Figure C-48: Drained CSS Simulation: PIMY, Dr=60, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$	327
Figure C-49: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=0.5$ atm, CSR=0.294, $\phi=34.2$, $\phi_{PT}=20$	328

Figure C-50: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	329
Figure C-51: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=1$ atm, CSR=0.241, $\phi=34.2$, $\phi_{PT}=20$	330
Figure C-52: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	331
Figure C-53: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=2$ atm, CSR=0.210, $\phi=34.2$, $\phi_{PT}=20$	332
Figure C-54: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	333
Figure C-55: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=4$ atm, CSR=0.185, $\phi=34.2$, $\phi_{PT}=20$	334
Figure C-56: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	335
Figure C-57: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=8$ atm, CSR=0.167, $\phi=34.2$, $\phi_{PT}=20$	336
Figure C-58: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	337
Figure C-59: Undrained CSS Simulation: PDMY2, Dr=70, $\sigma'_{v0}=16$ atm, CSR=0.160, $\phi=34.2$, $\phi_{PT}=20$	338
Figure C-60: Drained CSS Simulation: PIMY, Dr=70, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$	339
Figure C-61: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=0.5$ atm, CSR=0.630, $\phi=37.4$, $\phi_{PT}=20$	340
Figure C-62: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$	341
Figure C-63: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=1$ atm, CSR=0.450, $\phi=37.4$, $\phi_{PT}=20$	342
Figure C-64: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=37.4$, PT=na	343
Figure C-65: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=2$ atm, CSR=0.370, $\phi=37.4$, $\phi_{PT}=20$	344
Figure C-66: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$	345

Figure C-67: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=4$ atm, CSR=0.310, $\phi=37.4$, $\phi_{PT}=20$	346
Figure C-68: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$	347
Figure C-69: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=8$ atm, CSR=0.265, $\phi=37.4$, $\phi_{PT}=20$	348
Figure C-70: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$	349
Figure C-71: Undrained CSS Simulation: PDMY2, Dr=80, $\sigma'_{v0}=16$ atm, CSR=0.250, $\phi=37.4$, $\phi_{PT}=20$	350
Figure C-72: Drained CSS Simulation: PIMY, Dr=80, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$	351
Figure C-73: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=0.5$ atm, CSR=1p008, $\phi=40.8$, $\phi_{PT}=20$	352
Figure C-74: Drained CSS Simulation: PIMY, Dr=90, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	353
Figure C-75: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=1$ atm, CSR=0.720, $\phi=40.8$, $\phi_{PT}=20$	354
Figure C-76: Drained CSS Simulation: PIMY, Dr=90, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	355
Figure C-77: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=2$ atm, CSR=0.592, $\phi=40.8$, $\phi_{PT}=20$	356
Figure C-78: Drained CSS Simulation: PIMY, Dr=90, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	357
Figure C-79: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=4$ atm, CSR=0.496, $\phi=40.8$, $\phi_{PT}=20$	358
Figure C-80: Drained CSS Simulation: PIMY, Dr=90, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	359
Figure C-81: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=8$ atm, CSR=0.424, $\phi=40.8$, $\phi_{PT}=20$	360
Figure C-82: Drained CSS Simulation: PIMY, Dr=90, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	361
Figure C-83: Undrained CSS Simulation: PDMY2, Dr=90, $\sigma'_{v0}=16$ atm, CSR=0.400, $\phi=40.8$, $\phi_{PT}=20$	362

Figure C-84: Drained CSS Simulation: PIMY, $D_r=90$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$	363
Figure D-1: Input motion NGA W2 Seq. No. 150, Bin No. 1.	365
Figure D-2: Input motion NGA W2 Seq. No. 285, Bin No. 1.	366
Figure D-3: Input motion NGA W2 Seq. No. 568, Bin No. 1.	367
Figure D-4: Input motion NGA W2 Seq. No. 1086, Bin No. 1.	368
Figure D-5: Input motion NGA W2 Seq. No. 2734, Bin No. 1.	369
Figure D-6: Input motion NGA W2 Seq. No. 3473, Bin No. 1.	370
Figure D-7: Input motion NGA W2 Seq. No. 4040, Bin No. 1.	371
Figure D-8: Input motion NGA W2 Seq. No. 4097, Bin No. 1.	372
Figure D-9: Input motion NGA W2 Seq. No. 28, Bin No. 2.	373
Figure D-10: Input motion NGA W2 Seq. No. 164, Bin No. 2.	374
Figure D-11: Input motion NGA W2 Seq. No. 410, Bin No. 2.	375
Figure D-12: Input motion NGA W2 Seq. No. 550, Bin No. 2.	376
Figure D-13: Input motion NGA W2 Seq. No. 594, Bin No. 2.	377
Figure D-14: Input motion NGA W2 Seq. No. 765, Bin No. 2.	378
Figure D-15: Input motion NGA W2 Seq. No. 801, Bin No. 2.	379
Figure D-16: Input motion NGA W2 Seq. No. 901, Bin No. 2.	380
Figure D-17: Input motion NGA W2 Seq. No. 957, Bin No. 2.	381
Figure D-18: Input motion NGA W2 Seq. No. 1111, Bin No. 2.	382
Figure D-19: Input motion NGA W2 Seq. No. 2622, Bin No. 2.	383
Figure D-20: Input motion NGA W2 Seq. No. 2632, Bin No. 2.	384
Figure D-21: Input motion NGA W2 Seq. No. 4133, Bin No. 2.	385
Figure D-22: Input motion NGA W2 Seq. No. 751, Bin No. 3.	386
Figure D-23: Input motion NGA W2 Seq. No. 791, Bin No. 3.	387
Figure D-24: Input motion NGA W2 Seq. No. 3507, Bin No. 3.	388
Figure D-25: Input motion NGA W2 Seq. No. 2709, Bin No. 3.	389
Figure D-26: Input motion NGA W2 Seq. No. 3300, Bin No. 3.	390
Figure D-27: Input motion NGA W2 Seq. No. 5483, Bin No. 3.	391
Figure D-28: Input motion NGA W2 Seq. No. 5474, Bin No. 3.	392
Figure D-29: Input motion NGA W2 Seq. No. 143, Bin No. 4.	393
Figure D-30: Input motion NGA W2 Seq. No. 828, Bin No. 4.	394

Figure D-31: Input motion NGA W2 Seq. No. 879, Bin No. 4.	395
Figure D-32: Input motion NGA W2 Seq. No. 1148, Bin No. 4.	396
Figure D-33: Input motion NGA W2 Seq. No. 1165, Bin No. 4.	397
Figure D-34: Input motion NGA W2 Seq. No. 1482, Bin No. 4.	398
Figure D-35: Input motion NGA W2 Seq. No. 1515, Bin No. 4.	399
Figure D-36: Input motion NGA W2 Seq. No. 1548, Bin No. 4.	400
Figure D-37: Input motion NGA W2 Seq. No. 827, Bin No. 5.	401
Figure D-38: Input motion NGA W2 Seq. No. 1202, Bin No. 5.	402
Figure D-39: Input motion NGA W2 Seq. No. 1490, Bin No. 5.	403
Figure D-40: Input motion NGA W2 Seq. No. 1520, Bin No. 5.	404
Figure D-41: Input motion NGA W2 Seq. No. 1611, Bin No. 5.	405
Figure D-42: Input motion NGA W2 Seq. No. 1612, Bin No. 5.	406
Figure D-43: Input motion NGA W2 Seq. No. 1618, Bin No. 5.	407
Figure D-44: Input motion NGA W2 Seq. No. 1787, Bin No. 5.	408
Figure D-45: Input motion NGA W2 Seq. No. 891, Bin No. 6.	409
Figure D-46: Input motion NGA W2 Seq. No. 897, Bin No. 6.	410
Figure D-47: Input motion NGA W2 Seq. No. 1166, Bin No. 6.	411
Figure D-48: Input motion NGA W2 Seq. No. 1230, Bin No. 6.	412
Figure D-49: Input motion NGA W2 Seq. No. 1256, Bin No. 6.	413
Figure D-50: Input motion NGA W2 Seq. No. 1284, Bin No. 6.	414
Figure D-51: Input motion NGA W2 Seq. No. 1350, Bin No. 6.	415
Figure D-52: Input motion NGA W2 Seq. No. 1437, Bin No. 6.	416
Figure D-53: Input motion NGA W2 Seq. No. 1463, Bin No. 6.	417
Figure D-54: Input motion NGA W2 Seq. No. 1582, Bin No. 6.	418
Figure D-55: Input motion NGA W2 Seq. No. 1613, Bin No. 6.	419
Figure D-56: Input motion NGA W2 Seq. No. 1616, Bin No. 6.	420
Figure D-57: Input motion NGA W2 Seq. No. 1626, Bin No. 6.	421
Figure D-58: Input motion NGA W2 Seq. No. 1627, Bin No. 6.	422
Figure D-59: Input motion NGA W2 Seq. No. 1786, Bin No. 6.	423
Figure D-60: Input motion NGA W2 Seq. No. 1795, Bin No. 6.	424
Figure D-61: Input motion NGA W2 Seq. No. 1833, Bin No. 6.	425
Figure D-62: Input motion NGA W2 Seq. No. 1836, Bin No. 6.	426

Figure D-63: Input motion NGA W2 Seq. No. 3750, Bin No. 6.	427
Figure D-64: Input motion NGA W2 Seq. No. 3752, Bin No. 6.	428
Figure D-65: Input motion NGA W2 Seq. No. 5842, Bin No. 6.	429
Figure D-66: Input motion NGA W2 Seq. No. 5862, Bin No. 6.	430
Figure D-67: Input motion Subduct B32, Bin No. 7.....	431
Figure D-68: Input motion Subduct B40, Bin No. 7.....	432
Figure D-69: Input motion Subduct B42, Bin No. 7.....	433
Figure D-70: Input motion Subduct B52, Bin No. 7.....	434
Figure D-71: Input motion Subduct B54, Bin No. 7.....	435
Figure D-72: Input motion Subduct B58, Bin No. 7.....	436
Figure D-73: Input motion Subduct B64, Bin No. 7.....	437
Figure D-74: Input motion Subduct B65, Bin No. 7.....	438
Figure D-75: Input motion NGA Seq. No. 169, Bin No. 9.....	439
Figure D-76: Input motion NGA Seq. No. 728, Bin No. 9.....	440
Figure D-77: Input motion NGA Seq. No. 757, Bin No. 9.....	441
Figure D-78: Input motion NGA Seq. No. 829, Bin No. 9.....	442
Figure D-79: Input motion NGA Seq. No. 850, Bin No. 9.....	443
Figure D-80: Input motion NGA Seq. No. 862, Bin No. 9.....	444
Figure D-81: Input motion NGA Seq. No. 959, Bin No. 9.....	445
Figure D-82: Input motion NGA Seq. No. 1107, Bin No. 9.....	446
Figure D-83: Input motion NGA Seq. No. 1766, Bin No. 9.....	447
Figure E-1: Response spectra computed for WLA Superstition Hills earthquake 360 component with element sizes of 0.125, 0.25 and 0.5 m.	479
Figure E-2: Response spectra computed for WLA Superstition Hills earthquake 360 component with hydraulic conductivity of 1×10^{-5} , 1×10^{-6} and of 1×10^{-7} m/s.....	480
Figure E-3: Response spectra computed for WLA Superstition Hills earthquake 360 component with and without vertical excitation	481
Figure E-4: Comparison of cyclic simple shear simulations for case of (a) $K_0 = 1$ and (b) $K_0 = 0.45$	483

List of Tables

Table 2-1: Summary of liquefied-site ground motion database for shallow crustal earthquakes	19
Table 2-2: Summary of Parameters Used in NGA GMPEs	44
Table 2-3: Summary of Directivity Parameters Used in NGA GMPEs	45
Table 4-1: Summary of Directivity Amplification Values (AF_{dir}) Used in NGA GMPEs	79
Table 4-2: Summary of Intra-Event Standard Deviations	97
Table 4-3: Proposed Model Regression Coefficients and Standard Errors for AS08 and BA08	99
Table 4-4: Proposed Model Regression Coefficients and Standard Errors for CB08 and CY08.....	100
Table 4-5: Confidence levels (%) that the model coefficients are statistically significant.....	105
Table 5-1: Summary of Calibrated Model Parameters.....	154
Table 5-2: Parameters Used in WLA Site Response Analyses.....	164
Table 5-3: Parameters Used in Port Island Site Response Analyses	179
Table 6-1: Summary of magnitude and distance bins and the number of ground motions in each bin (in brackets).	198
Table 6-2: Summary Site Response Input Ground Motions.....	200
Table 7-1: Summary of correlation between and mean AF_{liq} values and various parameters.....	249

List of Acronyms, Abbreviations and Symbols

1-D	one-dimensional
AF_{dir}	directivity amplification factor
AF_{liq}	liquefaction amplification factor
A_z	NGA source-to-site azimuth
AASHTO	America Association of State Highway Transportation Officials
ASCE	America Society of Civil Engineers
AR	Amplification ratio – recording relative to NGA prediction
AS08	Abrahamson and Silva (2008)
BA08	Boore and Atkinson (2008)
C_0, C_1	Period-dependent regression coefficients used in 2013 Bayless & Somerville directivity model
c_1, c_2, c_3, c_4	regression coefficients for liquefaction amplification model
CB08	Campbell and Bozorgina (2008)
CY08	Chiou and Youngs (2008)
DS	dip slip (fault)
DSHA	deterministic seismic hazard analysis
EL	equivalent linear
FAS	Fourier amplitude spectrum
FN	fault-normal component of ground motion
F_{RV}	NGA reverse faulting indicator
F_{MEAS}	NGA factor indicating measured V_{s30} (1) or inferred (0)
F_N	NGA normal faulting indicator
F_{HW}	NGA indicator of site on hanging wall

f_D	an amplification factor in ln units to account for fault rupture directivity, used in 2013 Bayless & Somerville directivity model
f_{geom}	geometric directivity predictor used in 2013 Bayless & Somerville model
GMPE	ground motion prediction equation
GMRotI50	a form of geometric mean response spectrum that is independent of sensor orientation (Boore et al. 2006)
G	shear stiffness
G_{tan}	tangent shear stiffness
Hz	Hertz
ICC	International Code Council
MIF	mean instantaneous frequency (optional subscripts H and V denote horizontal and vertical component of ground motion, respectively)
M_w	earthquake moment magnitude
NGA	Next Generation Attenuation Relationships
N_i	number of ground motion recordings for the i^{th} earthquake event
$P(t, f)$	Fourier power spectrum
PEER	Pacific Earthquake Engineering Research Center
PGA	peak ground acceleration
PGV	peak ground velocity
PI	Port Island, Japan recording station or plasticity index
PSHA	probabilistic seismic hazard analysis
R	spectral ratio
R_d	dynamic amplification factor for SDOF oscillator
R_{jb}	Joyner-Boore distance: closest distance from a site to the surface projection of a fault rupture plane

R_x	horizontal distance to top of a fault rupture measured perpendicular to the fault strike
R_{rup}	closest distance from a site to a fault rupture plane
r_u	excess porewater pressure ratio
Sa	pseudo-absolute acceleration response spectrum or pseudo-absolute acceleration (5% damped unless otherwise noted)
Sa_{dir}	value of Sa calculated using NGA GMPEs and adjusted to account for fault rupture directivity effects
Sa_{liq}	value of Sa adjusted to account for liquefaction
Sa_{NGA}	Sa calculated using an NGA GMPE
Sa_{nodir}	value of Sa calculated using NGA GMPEs without accounting for fault rupture directivity effects
$Sa_{recorded}$	recorded Sa
Sa_{NGA}	median value of Sa predicted using an NGA GMPE
Sd	relative displacement response spectrum or relative displacement (5% damped unless otherwise noted)
Sv	pseudo-absolute velocity response spectrum or pseudo-absolute velocity (5% damped unless otherwise noted)
SDOF	single degree of freedom
SPT	Standard Penetration Test
SS	strike slip (fault)
s	length of striking fault rupturing toward the site used in 2013 Bayless & Somerville forward directivity model
t_{lag}	phase lag between input excitation and SDOF oscillator response
t_{lag}^*	value of t_{lag} chosen for the purpose of calculating $t_{Sa}(T)$
$t_{Sa}^*(T)$	time at which the peak acceleration occurs in the response of a SDOF oscillator with a natural period T

$t_{Sa}(T)$	value of $t_{Sa}^*(T)$ corrected for phase lag between input excitation and SDOF oscillator response.
t_{liq}	time of liquefaction initiation
t_{sus}	time of suspected liquefaction initiation
T	natural period of a SDOF oscillator
T_{Az}	azimuth taper used in 2013 Bayless & Somerville forward directivity model
T_{CD}	distance taper used in 2013 Bayless & Somerville forward directivity model
T_{max}	maximum usable period of a ground motion recording
T_{Mw}	magnitude taper used in 2013 Bayless & Somerville forward directivity model
$u(t)$	displacement time history of an SDOF oscillator
$(u_{st})_0$	static displacement of an SDOF oscillator under a constant force
V_s	shear wave velocity
V_{s30}	travel time-based average shear wave velocity in upper 30 meters
W	down-dip fault rupture width
WLA	Wildlife Liquefaction Array
Z_{tor}	depth to top of a fault rupture
Z_1	depth to V_s of 1 km/s
$Z_{2.5}$	depth to V_s of 2.5 km/s
Δu	excess porewater pressure
δ	fault dip
ε_{ij}	intra-event residual (error) for the i_{th} event and j_{th} recording
ε_{tot}	total residual (error) for the i_{th} event and j_{th} recording
ϕ	phase lag in radians; internal angle of friction of soil

γ	engineering shear strain
θ	azimuth angle between the fault plane and the ray path from the epicenter to the site
η_i	inter-event residual (error) for the i_{th} event
σ	standard error computed for intra-event residuals reported by NGA developer
σ_{liq}	standard error computed for intra-event residuals in the empirical analysis of liquefaction ground motion recordings
σ'_{v0} or σ'_{vi}	initial vertical effective stress
σ'_v	vertical effective stress
σ^2	intra-event variance
τ	shear stress
τ^2	inter-event variance
τ_{oct}	octahedral shear stress
ζ	damping ratio in fraction or percent of critical

Acknowledgements

I owe the deepest debt of gratitude to my beautiful, intelligent and loving wife Staci. Her unflinching support through the process of my doctoral studies has given me the strength and courage to tackle this challenge. Her support of my PhD pursuit has come with great sacrifice on her part. On countless weekends and evenings she has cared for our children and shouldered household responsibilities in my absence. Thank you, Staci, the love of my life, for all that you have done to support my dreams.

Though they may not fully realize it, my children Jayde and Will have also sacrificed time with their father so that I might pursue this study. I hope that later, when they are old enough to understand it, the undertaking of this work will serve to inspire them to chase their life dreams. In the meantime, I'm going to repay them with gigantic ice cream sundaes.

To my advisor Professor Ahmed Elgamal I offer my deepest thanks and appreciation. His willingness to take on an unorthodox graduate student working a full time job was the water that allowed this seed to grow. He's given me support, encouragement and advice when I needed it, and the space and time to complete this work under the constraints of my work and family obligations. The technical insight he has provided me on this work has been invaluable, and his humor and friendship has enriched my personal life.

I owe a great debt to committee member Professor Jonathan Bray, whose advice and guidance during this research has been key to its quality. His thoroughness, attention to detail and perspective has not only improved the

quality of this work, it has inspired me to elevate my “engineering game” every day.

I thank the other members of my PhD committee, Professors Duncan Agnew, Patrick Fox and Tara Hutchinson for their time and effort in reviewing this work.

My employer Kleinfelder, Inc. has provided financial support for these studies and has offered me the schedule flexibility necessary for me to complete my PhD coursework, which is greatly appreciated.

Professor Fumio Yamazaki graciously provided rare time histories from liquefaction sites in Japan that were invaluable to this effort. Brian Carlton assisted by providing ground motions recordings from the NGA West 2 database and from subduction zone events, which were used as input in site response analyses.

I thank my friends and colleagues Drs. Zia Zafir, Kevin Franke, Hyungsuk Shin, Patrick Wilson and Jorge Meneses, who offered their encouragement and confidence for me to initiate these studies in the first place, and with whom I have shared many engaging discussions on personal and earthquake engineering matters. To Dr. Jinchi Lu I express my appreciation for his consultation and advice on all things Opensees.

Chapters 2 and 4, in part, contain material from an article published in *Earthquake Spectra* (Gingery et al. 2014). This material was used with permission from the Earthquake Engineering Research Institute. The dissertation author was the primary author of this paper.

The portion of Chapter 5 dealing with adjustment of shear modulus reduction curves for strength compatibility was previously published in the proceedings of IACGE 2013: Challenges and Recent Advances in Geotechnical and Seismic Research and Practices (Gingery and Elgamal 2013). This material was used with permission from American Society of Civil Engineers. The dissertation author was the primary author of this paper.

Vita

1996	B.S., <i>Cum Laude</i> Department of Civil and Environmental Engineering, San Jose State University, California
1995-1997	Intern / Staff Engineer, Donald M. Tharp and Associates, Soquel, California
1997-2000	Staff Engineer, Sampson Engineering, Inc., Watsonville, California
1999	Registered Civil Engineer C59819, State of California
2001	M.S., Department of Civil and Environmental Engineering, University of California, Berkeley
2001-2004	Senior Engineer, Haley & Aldrich, Inc., Boston Massachusetts and San Diego, California.
2003	Registered Geotechnical Engineer 2368, State of California
2004-Present	Principal Engineer, Kleinfelder, San Diego, California
2014	Ph.D., Department of Structural Engineering, University of California, San Diego, California

Journal Papers

Gingery, J.R., Franke, K.W., Meneses, J.F., Zafir, Z. (2014), "North Torrey Pines Bridge seismic retrofit: part I, seismic hazard and geotechnical design," in press, *Earthquake Spectra*, EERI.

Gingery, J.R., Elgamal, A., Bray, J.D. (2014), "Response Spectra at Liquefaction Sites during Shallow Crustal Earthquakes," in press, *Earthquake Spectra*, EERI.

Reports

Brandenberg, S.J, J. Fletcher, **J.R. Gingery**, K.W. Hudnut, T. McCrink, J.F. Meneses, D. Murbach, T.K. Rockwell, J.P. Stewart, J. Tinsley (2010). *Preliminary Report on Seismological and Geotechnical Engineering Aspects of the April 4 2010 Mw 7.2 El Mayor-Cucapah (Mexico) Earthquake*, Geotechnical Earthquake Engineering Reconnaissance Association Report No. GEER-023
http://www.geerassociation.org/Post_EQ_Reports.html

Conference Proceedings

- Dunn, R.J. & **Gingery, J.R.** (2007). "Between a rock and a hard place – the difficult task of closing the San Marcos Landfill with an evapotranspirative cover", *Proceedings Sardinia 2007, Eleventh International Waste Management and Landfill Symposium*, Cagliari, Italy; 1 - 5 October 2007.
- Gingery, J.R.**, (2003). "Embankment Failure from Liquefaction and Other Damage in the 20 April 2002 Au Sable Forks, NY Earthquake", *Proceedings of the 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering*, Cambridge, Massachusetts, June 2003,
- Gingery, J.R.** (2007). "A simplified method for estimating shear strains for ovaling and racking analysis of tunnels", *Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering*, June 25-28, 2007, Thessaloniki, Greece.
- Gingery, J.R.** and Merry, S.M. (2008). "Evaluation of a Geogrid-Reinforced Soil Mat to Mitigate Post-Liquefaction Settlements – A Case Study", *Geotechnical Engineering and Soil Dynamics IV, Sacramento California*, May 18-22.
- Gingery, J.R.**, J. Meneses and Z. Zafir, (2009). "Addressing kinematic pile bending design requirements in the International Building Code", accepted to *International Foundation Congress & Equipment Expo*, March 15-19, Orlando, Florida.
- Gingery, J.R.**, J.F. Meneses, K.W. Franke & Z. Zafir (2009). "Seismic retrofit of an historic bridge", *International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, June 15-18, Tokyo, Japan.
- Gingery, J.R.**, Rugg, S.H., Hilton, B.R. & Rockwell, T.K. (2010). "Fault Hazard Characterization for a Transportation Tunnel Project in Coronado, California," *5th Int. Conf. on Recent Advances Geotech. Earthquake Engr. and Soil Dynamics*, May 2010.
- Gingery, J.R.** and Elgamal, A. (2013). "Shear stress-strain curves based on the $G/G_{\max}$ logic: a procedure for strength compatibility," accepted to *2nd IACGE Intrnl. Conf. on Geotech. and Earthq. Engrg.*, October 25-27, Chengdu, China.
- Gingery, J.R.** and Arora, S. (2013), "Compaction grouting for the seismic retrofit of the North Torrey Pines Bridge," *Seventh Intrnl. Conf. on Case Histories in Geotech. Engr. and Symp. In Honor of Clyde Baker*, April 29- May 4, Chicago, Ill.

- Meneses, J.F., **J.R. Gingery**, K.W. Franke & Z. Zafir (2009). "Performance-based geotechnical aspects for seismic retrofit of a hospital complex", *International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, June 15-18, Tokyo, Japan.
- Nguyen, T.V., Shao, L., **Gingery, J.R.**, Robertson, P.K. (2014). "Proposed modifications to CPT-based liquefaction method for post-vibratory ground improvement," accepted to *ASCE Geo-Congress*, Atlanta, GA, February 23-26.
- Schmidt, J.M., Davis, S., **Gingery, J.**, Hoffman, J., Lingwall, B. (2012). "I-15 CORE design-build project – a case history, Provo, Utah," *Central Pennsylvania Geotechnical Conference*, October 24-26, 2012.
- Young, D., Dean, A., **Gingery, J.R.**, Gregor, T. (2010). "Development of Seismic Design Criteria for the Coronado Highway Tunnel", *North American Tunneling Conference Proceedings*, Society for Mining, Metallurgy, and Exploration, Inc.

ABSTRACT OF THE DISSERTATION

Effects of Liquefaction on Earthquake Ground Motions

By

James R. Gingery

Doctor of Philosophy in Structural Engineering

University of California, San Diego, 2014

Professor Ahmed Elgamal, Chair

Site amplification studies and building code provisions recognize that soil liquefaction can alter the characteristics of ground shaking at a site. However, guidance as to how the amplitudes of spectral accelerations are modified is lacking. To address this issue, a two-part study is undertaken.

In the first part an empirical study of ground motions recorded at liquefaction sites is undertaken. Available recorded ground motions from shallow crustal earthquakes at sites that exhibited evidence of liquefaction are compiled.

Analysis of spectral acceleration residuals of the recorded ground motions computed relative to Next Generation Attenuation (NGA) estimates reveal positive bias at longer periods, slight negative bias at intermediate periods, and slight positive bias at short periods. Trends with V_{s30} , NGA-estimated peak ground acceleration (PGA), and moment magnitude are also observed. A model is developed that removes the initially observed residual bias and reduces uncertainty. The proposed model can be used to adjust NGA-estimated acceleration response spectra to account for the effects of liquefaction on ground shaking.

In the second part of this study a series of parametric 1-D site response analyses were performed to provide a much larger synthetic dataset and to study geotechnical parameter that are typically unavailable in the empirical data. An existing constitutive model was rigorously calibrated against a widely-used semi-empirical liquefaction triggering method. The calibrated model was used to perform 2988 site response analysis pairs: one with porewater pressure generation and the other without. The resulting surface spectra are compared in a way that is analogous to the empirical study. Liquefaction amplification factors from the site response analysis results exhibit similar trends with period compared to the empirical data, but their amplitudes are systematically lower. The differences might be attributable to the inability of the 1-D site response analyses to capture 2-D and 3-D effects such as surface waves and basin effects, or possibly shortcomings the in the 1-D model's ability to faithfully represent all salient aspects of 1-D wave propagation under liquefaction

conditions. Factors that appear to influence liquefaction amplification the most include whether the input motion is pulse-like or not and the amplitude of ground shaking.

Chapter 1 Introduction and Background

1.1 Characteristics of Soil Liquefaction Behavior

Earthquake-generated soil liquefaction occurs when cyclic loading induced by ground shaking lead to contraction of the soil, and thus an increase in porewater pressure that causes significant decreases in effective stress, stiffness and strength. In liquefied soils, large shear strain excursions can cause a dilative phase to occur, whereupon excess porewater pressures decrease and the soil quickly regains its strength and stiffness. Cyclic shearing at large amplitude can lead to successive changes between contractive and dilatant conditions, known in the literature as *phase transformation behavior* (e.g., Idriss and Boulanger 2008). The behavior leads to a cycle-by-cycle accumulation of shear strains, which are referred to as *cyclic mobility* (e.g., Kramer 1996).

Figure 1-1 presents the results of an undrained cyclic simple shear test conducted on a medium dense poorly graded sand (Monterrey 0/30) at the University of California, Berkeley by Wu (2002). These results demonstrate the salient aspects of soil liquefaction, cyclic mobility and phase transformation behavior. In this figure the normalized effective vertical stress is the ratio of the instantaneous vertical effective stress (σ'_v) to the initial vertical effective stress (σ'_{vi}) and the excess porewater pressure ratio is $r_u = \Delta u / \sigma'_{vi}$, where Δu is the excess porewater pressure. When the absolute value of shear stress is increasing, the soil is contractive between the phase transformation boundary lines and dilative outside. After the peak shear stress is reached and the

shearing direction is reversed, the soil is contractive until it crosses the second phase transformation line whereupon it is dilative. As cycles of shearing occur, excess porewater pressure accumulates until $r_u \approx 1$ and $\sigma'_v/\sigma'_{vi} \approx 0$, where the state of initial liquefaction is achieved. After initial liquefaction, when the stress path crosses the phase transformation line a neutral phase occurs in which considerable shear strain accumulates without significant change in shear or confining stress. The shear strain accumulation in the neutral phase increases with successive cycles of loading.

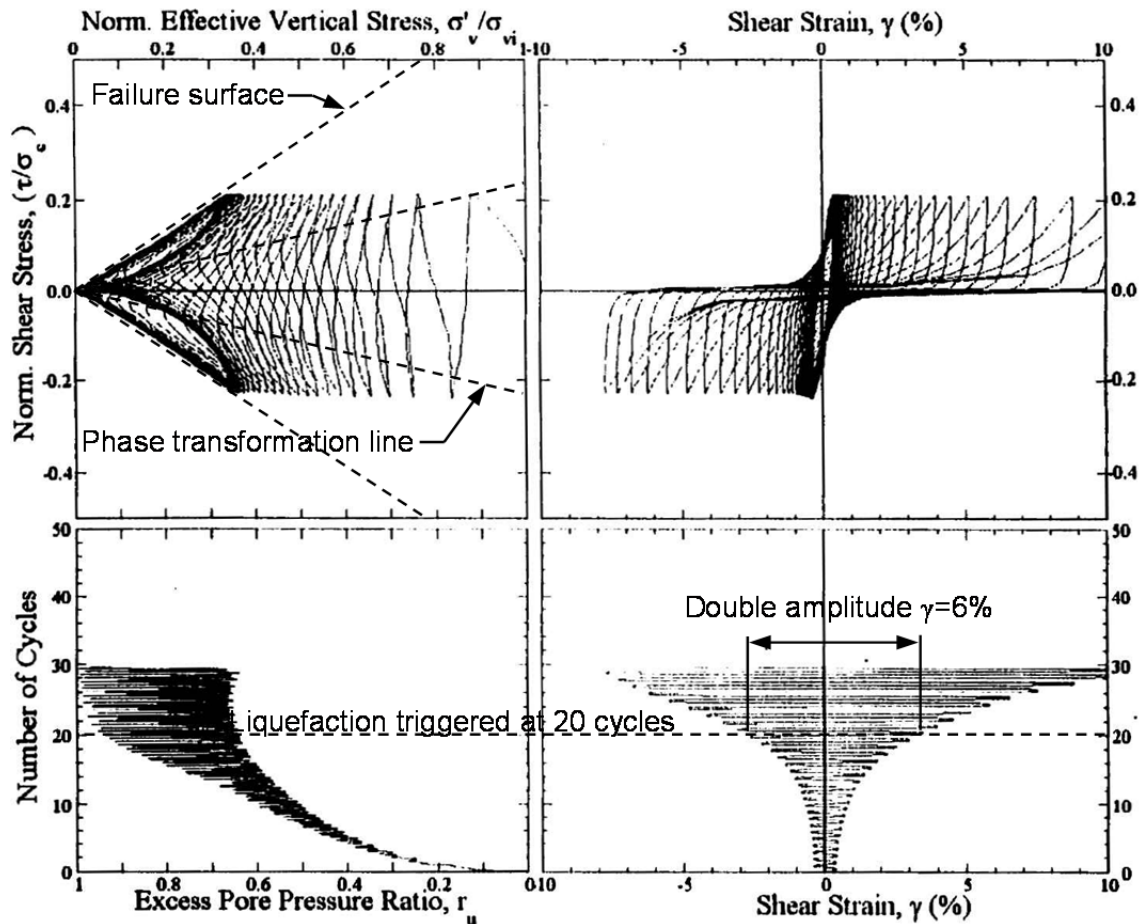


Figure 1-1: Undrained cyclic simple shear test of a medium dense (relative density = 63 percent) by Wu (2002).

The cyclic behavior demonstrated in Figure 1-1 and described above is more complicated than that of materials that do not liquefy, like drained sand or undrained clay (Figure 1-2). In liquefied soil response, tangent shear stiffness ($G_{tan} = d\tau/d\gamma$) values vary widely over time and phases occur where $G_{tan} \approx 0$. The peak-to-peak secant shear moduli of liquefied soils can be much lower than drained sand, particularly if load reversal occurs before dilation.

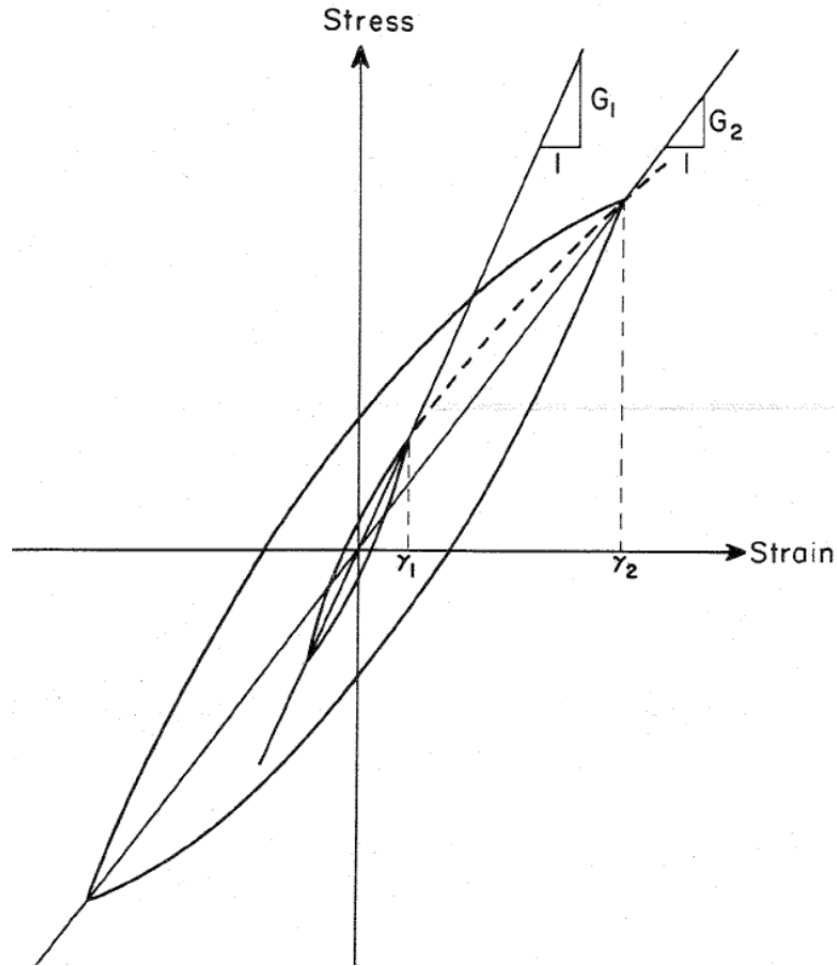


Figure 1-2: Typical hysteric behavior of drained sand and undrained clay after (Seed and Idriss 1970).

Considering the significant differences between liquefaction and non-liquefaction cyclic soil behavior, one should expect differences between earthquake ground motions at liquefaction and non-liquefaction sites.

1.2 Review of Literature Related to Soil Liquefaction and Ground Motions

The most damaging effects of liquefaction are related to damage from strength loss and deformation, and much research effort has been focused in these areas. However, relatively little research has been performed to elucidate

the effects of liquefaction on earthquake ground motions. The following presents a review of prior studies of ground motions at liquefaction sites.

1.2.1 Youd and Carter (2005)

Youd and Carter (2005) investigated ground motions at five sites where liquefaction occurred and pairs of surface and downhole or surface and rock outcrop accelerometer recordings were available. They used the downhole or outcrop ground motions as base input for equivalent linear site response analyses. The ground surface spectra from the equivalent linear response analysis were considered to represent the ground motion without the occurrence of liquefaction. The equivalent linear spectra were compared to spectra calculated from the actual ground surface accelerometer recordings. By computing spectra from pre- and post-liquefaction subsets of the time series, they were able to identify long period ground motion that was amplified as a result of liquefaction.

For the records analyzed, Youd and Carter showed that when initial liquefaction appeared to be triggered earlier in the record, deamplification of the surface spectra occurred at shorter periods. But when liquefaction appeared to be triggered later in the record, the initial site response was stiffer and short period deamplification did not occur. This study suggested that demamplification at shorter periods should not be relied upon if liquefaction is triggered later in the time history. They also compared the recorded spectra to code-based spectra from the 1998 American Association of State Highway and

Transportation Officials (AASHTO) Bridge Design Specifications and the 1997 Uniform Building Code, and concluded that these code-based design spectra are generally conservative relative to the studied ground motion case histories, provided a softer site profile such as D or E is used.

1.2.2 Zeghal and Elgamal (1994) and Elgamal et al. (1996)

Zeghal and Elgamal (1994) examined downhole array accelerometer and porewater pressure transducer data from the Wildlife Liquefaction Array site recorded during the 1987 Elmore Ranch and Superstition Hills earthquakes. Acceleration time history data from strong motion seismometers positioned below and above a liquefiable layer were used to calculate and plot gross average shear stress versus shear strain for the interval between the seismometers. Porewater pressure transducer time history data was used to calculate changes in vertical effective stress, which when combined with the shear stress data provided effective stress path history plots.

The data clearly showed that the overall shear stiffness decreased with the increase in porewater pressure. It is expected that this decrease in shear stiffness would translate to a reduced site period, which in turn could amplify ground motions at larger periods. Also observed in these ground motions were post-liquefaction phases of strain hardening at larger strain that coincided with transient reductions in the excess porewater pressure and sharp peaks in the acceleration time history record. Zeghal and Elgamal concluded that these

effects were associated with the dilational stage of phase transformation behavior.

Elgamal et al. (1996) performed a similar study of ground motions recorded in the Port Island downhole array in liquefiable soils during the 1995 Hyogoken-Nambu Kobe earthquake in Japan. Porewater pressure recordings were not available for this case, surface manifestations of liquefaction had been observed in the field. Stress-strain data in the interval between the surface and a depth of 16 m showed a rapid decay of stiffness and strength along with large shear strains. Post-liquefaction average shear wave propagation velocities calculated in the upper 16 meters typically ranged from 20 to 50 m/s, indicating a significantly softened soil response.

The Zeghal and Elgamal (1994) and Elgamal et al. (1996) studies did not examine ground motion in terms of acceleration response spectra.

1.2.3 Zorapapel and Vucetic (1994)

Zorapapel and Vucetic (1994) evaluated excess porewater pressure recordings and Fourier spectra for ground motions recorded in strong earthquakes at the Wildlife Liquefaction Array site and the Lotung array in Taiwan. The Fourier spectra were calculated for several short time intervals. They found that shifting of the predominant period from higher to lower values occurred as excess porewater pressure was generated. They concluded that the excess porewater pressure buildup altered both the frequency content and magnitude of the surface motions.

1.2.4 Hartvigsen (2007)

Hartvigsen (2007) performed set of non-linear site response analyses on nine hypothetical sand soil profiles with a potentially liquefiable layer of varying thickness and relative density. The potentially liquefiable layer began at 2 m below ground surface and was 4, 9, or 14 m thick, with $(N_1)_{60}$ standard penetration test (SPT) blow counts of 8, 16 and 24 blows per 30 cm. The potentially liquefiable layer was underlain by very dense sand that extended to a depth of 20 m. Each of the nine soil profiles was subjected to a total 105 base input ground motions covering a wide range of magnitudes and distances.

A one-dimensional (1-D) non-linear site response program called PNSL, which was under development by EduPro Civil Systems, Inc., was used for the site response analyses. Limited details on this software were provided by Hartvigsen (2007): the program used either the finite element or finite difference method to solve the 1-D wave equation problem; a hyperbolic backbone curve is used for nonlinear stress-strain behavior; unloading-reloading varies from Masing (1926) type behavior at low strain levels to Cundall-Pyke-type behavior (Pyke 1979) at high strain levels; and shear strength is governed by the Mohr-Coulomb model. For saturated soils, the software calculated incremental volumetric strain that translated to porewater pressure changes. Both contractive and dilative phases of behavior are incorporated.

For each combination of soil profile and ground motion, the site response was performed with the porewater pressure generation feature of the program

disabled and enabled. For a series of acceleration response spectral ordinates, the “Spectral Ratio” (R) was computer as:

$$R = \frac{Sa(T,L)_{withpwp}}{Sa(T,L)_{nopwp}} \quad (1-1)$$

where $Sa(T,L)_{withpwp}$ and $Sa(T,L)_{nopwp}$ are the spectral accelerations calculated with and without porewater pressures generation, respectively. These values were calculated for various periods, T , and for a “Loading Parameter”, L , which is essentially the inverse of the triggering factor of safety calculated using Youd et al. (2001). Regression analyses were performed to develop a predictive relationship for R as a function of L and T . In the resulting relationship at shorter periods, R tends to decrease as L increases. At longer periods, R tends to increase with L and T . The division between shorter and longer periods varies from about 0.13 seconds at $L = 0.5$ to about 1.7 seconds at $L = 2$. A plot of the mean Spectral Ratio predictions is presented in Figure 1-3. The Spectral Ratio predictive relationship was suggested for use as a modifier to response spectra calculated in site response analyses without consideration of porewater pressure generation, such as the total stress equivalent linear method.

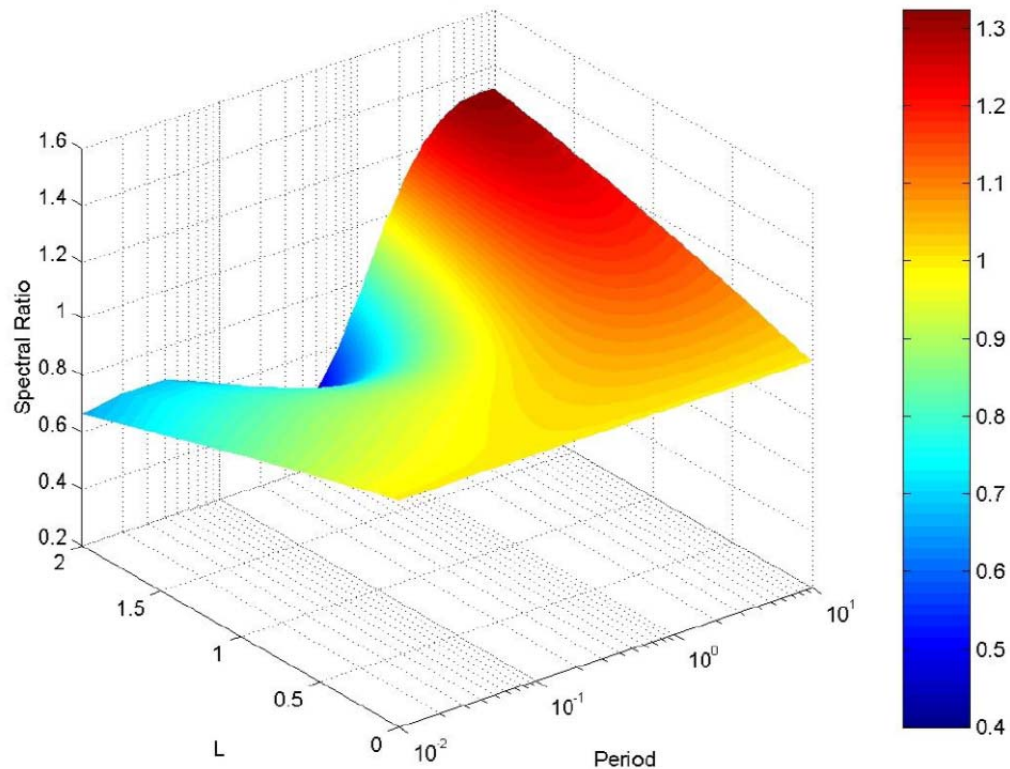


Figure 1-3: Spectral Ratio predictions from Hartvigsen (2007).

1.3 Requirements of Design Codes

In most modern building codes, design ground motions are based on deterministic and/or probabilistic seismic hazard analyses (DSHA, PSHA). Inherent in the DSHA and PSHA methodology are ground motion prediction equations (GMPEs), which provide estimates of spectral accelerations conditioned upon earthquake magnitude and focal mechanism, site-source distance, and site characteristics. While most modern GMPEs account for shallow soil site conditions through the V_{S30} (the average shear wave velocity in the upper 30 meters) parameter, there currently are no GMPEs capable of

accounting for the effect of liquefaction on ground motions. Since GMPEs do not account for liquefaction effects, many contemporary design codes (for example ASCE 2010, International Building Code [ICC 2012], Caltrans 2013) designate sites with liquefaction-prone soils as “Site Class F”, and require that non-linear effective stress site response analyses be performed to develop design earthquake ground motions. But effective stress site response analyses are time consuming and expensive to perform. Moreover, protocols for nonlinear code usage not very well established (especially for liquefaction cases), and this has restricted their use in practice (Kramer and Paulsen 2004).

AASHTO (2011) acknowledged these difficulties and indicated that “limited case-history data and analysis results indicate that liquefaction reduces spectral response rather than increases it, except at long periods in some cases. (e.g., $T = 1$ sec)”. Based on this assessment, AASHTO (2007) and later editions eliminated liquefiable soil as a condition to qualify for Site Class F. ASCE 7 (2010) allows Site Class D or E spectra to be used for liquefiable sites where the fundamental period of the structure is less than 0.5 seconds; which implies that liquefaction does not amplify short period ground motions. However, considering the limited amount of previous research, the basis of this assumption has not been very well established.

1.4 Objective and Scope

The previous studies described above have provided some insight into ground motions at liquefaction sites. However, more work is needed develop a

more complete understanding of liquefaction effects on ground motions. The main objectives of this research are to quantify the effects of liquefaction on earthquake ground motions and identify parameters that are important to liquefaction site response. These objectives are pursued using two complimentary approaches: compilation and analysis of empirical data; and parametric numerical modeling of one-dimensional site response. In pursuit of the main objective, several subordinate objectives are also addressed:

- Assemble a dataset of ground motions recorded at sites that liquefied during shallow crustal earthquakes.
- Develop a method for evaluating the timing of spectral accelerations relative to the onset of liquefaction.
- Develop a new method for estimating response spectra at liquefaction sites using an adjustment to ground motion prediction equation estimates that do not account for liquefaction.
- Make advancements in computational modeling of liquefaction site response through rigorous constitutive model calibration.
- Evaluate whether the calibration of a constitutive model for liquefaction triggering still holds when a broader site response system is considered.
- Develop a new adjustment to shear modulus degradation curves that assures compatibility with shear strength.

1.5 Dissertation Outline

The empirical part of this work is described in Chapters 2 through 4. Chapter 2 presents a database of strong ground motions recorded at sites where evidence of liquefaction was observed. In Chapter 3, analysis of the empirical database is presented which examines the temporal and frequency characteristics of the ground motions. Chapter 4 presents a model regressed from the empirical data that can be used to modify acceleration response spectra from ground motion prediction equations to account for liquefaction conditions. The material in Chapters 2 and 4 has been published by Gingery et al. (2014). A rigorous calibration of a nonlinear constitutive model for liquefaction and non-liquefaction conditions is presented in Chapter 5. Chapter 5 includes a new method for adjusting shear modulus reduction curves to produce accurate shear strength, which was previously published by Gingery & Elgamal (2013). Chapter 6 describes use of the calibrated model to perform finite element-based parametric nonlinear site response analyses for liquefaction and non-liquefaction conditions. Finally, Chapter 7 presents the conclusions of this study and makes recommendations for future work.

1.6 References

- American Association of State Highway and Transportation Officials (AASHTO) (2007). *AASHTO LRFD Bridge Design Specifications*, AASHTO, Washington DC.
- American Association of State Highway and Transportation Officials (AASHTO) (2011). *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, AASHTO, Washington DC.

- American Society of Civil Engineers (ASCE) (2010). *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-10, ASCE Reston, Virginia.
- California Department of Transportation (Caltrans) (2013). *Caltrans Seismic Design Criteria*, Version 1.7.
- Gingery, J.R. and Elgamal, A. (2013). "Shear stress-strain curves based on the $G/G_{\max}$ logic: a procedure for strength compatibility," *2nd IACGE Intrnl. Conf. on Geotech. and Earthq. Enrg.*, October 25-27, Chengdu, China.
- Gingery, J.R., Elgamal, A. and Bray, J.D. (2014), "Response Spectra at Liquefaction Sites during Shallow Crustal Earthquakes," in press, *Earthquake Spectra*, EERI.
- Hartvigsen, A.J. (2007). Influence of pore pressures in liquefiable soils on elastic response spectra. Master's thesis, University of Washington.
- International Code Council (ICC) (2012). *International Building Code*.
- Kramer, S. L., and S. B. Paulsen (2004). "Practical Use of Geotechnical Site Response Models," *NSF/PEER Int. Workshop on Uncertainties in Nonlinear Soil Properties and their Impact on Modeling Dynamic Soil Response*, University of California at Berkeley, 18–19 March.
- Masing, G. (1926). "Eigenspannungen und verfestigung beim messing (Self stretching and hardening for brass)," *Proc. of the 2nd Intrnl. Congress for Applied Mech.*, Zurich, Switzerland, pp. 332–335 (in German).
- Pyke, R.M. (1979). "Nonlinear soil mode ls for irregular cyclic loadings," *J. Geotech. Eng.*, ASCE, 105(GT6), pp 715-726.
- Seed, H.B. and Idriss, I.M. (1970). "Soil Modulus and Damping Factors for Site Response Analysis," *Earthquake Engineering Research Center Report No. EERC 70-10*, University of California, Berkeley.
- Wu, J. (2002). "Liquefaction triggering and post-liquefaction deformation of Monterey 0/30 sand under unidirectional cyclic simple shear loading," PhD dissertation, University of California, Berkeley.
- Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., (2001). "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils," *J. Geotech. and Geoenvr. Eng.*, ASCE 127(10), pp 817–33.

- Youd, T.L. and Carter, B.L. (2005). "Influence of soil softening and liquefaction on spectral acceleration," *J. Geotech. and Geoenvr. Engr.*, ASCE, 131(7).
- Zeghal, M. and Elgamal, A. (1994). "Analysis of site liquefaction using earthquake records," *J. Geotech. and Geoenvr. Engr.*, ASCE, 120(6), pp. 996-1017.
- Zorapapel, G.T. and Vucetic, V. (1994). "The effects of seismic pore water pressure on ground surface motion," *Earthquake Spectra*, Earthquake Engineering Research Institute, Vol. 10, No. 2.

Chapter 2 Recorded Ground Motions at Liquefied Sites

2.1 Introduction

Ground response recorded at sites where liquefaction occurred provide an opportunity for direct observation of liquefaction effects on ground motions. Strong ground motion recording sites with documented liquefaction manifestation represent a very small fraction of the global ground motion records. For instance, the database used for development of the Pacific Earthquake Engineering Research Center's (PEER) Next Generation Attenuation (NGA) relationships (Chiou et al. 2008) included a total of 3,551 recordings, of which only four are known or suspected to have been recorded at sites with liquefaction: Wildlife Liquefaction Array in the 1987 Superstition Hills earthquake; Treasure Island in the 1989 Loma Prieta earthquake; and Amagasaki-G and Port Island in the 1995 Hyogoken-Nambu Kobe earthquake. Nonetheless, the continued accumulation in recent years of earthquake response recordings from liquefied sites provides an unprecedented opportunity for study of liquefaction effects on ground motions.

With the focus placed on shallow crustal earthquakes, this chapter presents a database of strong ground motion recordings from liquefied sites, which was compiled as part of this study. Recording site and earthquake source characteristics are described, and horizontal geometric mean acceleration response spectra are calculated for each recording. These data are used later in Chapter 4 for comparisons with response spectra computed with ground motion prediction equations.

2.2 Ground Motion Database

A database of 19 strong ground motions recorded at stations where liquefaction occurred was compiled for this study. Empirical observations of surface manifestations of liquefaction such as sand boils, sand-filled ground fissures, large permanent settlements or displacements, floating tanks/pipes, foundation failures, etc. have been reported in the literature for each of these stations. Table 2-1 presents a summary of the liquefied-site ground motion recordings used in this study. The recordings were taken from a total of seven shallow crustal earthquakes with magnitudes ranging from 6.3 to 7.7 and site-to-fault rupture plane distances (R_{rup}) ranging from 1.7 to 78.9 km. The distribution of magnitude and distance pairs is presented on Figure 2-1. Plots of acceleration, velocity and displacement time histories, and acceleration response spectra are presented in Figures 2-2 through 2-20. These figures also include time history plots of mean instantaneous frequency (MIF); this parameter is explained in Chapter 3. Additional details of the earthquakes and recording stations for these ground motions are discussed in Section 2.2.2: *Site and Earthquake Characteristics*.

As described in Chapter 4, the ground motions in this database were compared with ground motion estimations provided by 2008 NGA GMPEs. This study was limited to shallow crustal earthquakes to maintain consistence with the NGA relationships that are currently available, which will be used for comparisons. The ground motions selected were also limited to “free-field” sites,

as defined by Geomatrix instrument structure types *I*, *A* or *B* (Chiou et al. 2008).

The compiled dataset represents all recordings currently available that meet the specified free-field and shallow crustal earthquake criteria.

Table 2-1: Summary of liquefied-site ground motion database for shallow crustal earthquakes

Earthquake	M _w	Fault Style Note (a)	Date	Station	GMX C1 ^(b)	Source of Liquefaction Confirmation	Pulse-Like Forward Directivity ^(e)
Nihonkai-Chubu	7.7	R	26-May-1983	Hachirogata	Note (c)	Yanagisawa et al. (1984) ^(f)	No
Superstition Hills	6.54	SS	24-Nov-1987	Wildlife	I	Holzer et al. (1989) ^(f)	No
Loma Prieta	6.93	RO	18-Oct-1989	Treasure Island	B	Darragh & Shakal (1991) ^(f)	No
Kobe	6.9	SS	17-Jan-1995	Amagasaki-G	Note (c)	Sato et al. (1998) ^(f)	No
Kobe	6.9	SS	17-Jan-1995	Amag. No. 3 PP,	Note (c)	ECRHED (1998) ^(f)	No
Kobe	6.9	SS	17-Jan-1995	Higashi-Kobe	Note (c)	Hagiwara et al. (1997) ^(f)	Yes
Kobe	6.9	SS	17-Jan-1995	Kobe-JI-S	Note (c)	Hamada et al. (1995) ^(f)	Yes
Kobe	6.9	SS	17-Jan-1995	Port Island GL	I	Shibata et al. (1996) ^(f)	Yes
Niigata-Ken Chuetsu-	6.6	R	16-Jul-2007	NIG018 (K-Net)	Note (d)	Kayen et al. (2009) ^(g)	No
Darfield, NZ	7.1	SS	3-Sep-2010	GDLC	A	Bradley (2012)	Yes
Darfield, NZ	7.1	SS	3-Sep-2010	HPSC	A	GEER (2011), Bradley (2012)	No
Christchurch, NZ	6.3	RO	21-Feb-2011	CBGS	A	Cubrinovski & Taylor (2011)	No
Christchurch, NZ	6.3	RO	21-Feb-2011	CCCC	B	Cubrinovski & Taylor (2011)	Yes ^(h)
Christchurch, NZ	6.3	RO	21-Feb-2011	CHHC	B	Cubrinovski & Taylor (2011)	Yes ^(h)
Christchurch, NZ	6.3	RO	21-Feb-2011	HPSC	A	Bradley (2012)	Yes ^(h)
Christchurch, NZ	6.3	RO	21-Feb-2011	NNBS	A	Bradley (2012)	Yes ^(h)
Christchurch, NZ	6.3	RO	21-Feb-2011	PRPC	A	Bradley (2012)	Yes
Christchurch, NZ	6.3	RO	21-Feb-2011	REHS	A	Cubrinovski & Taylor (2011)	Yes
Christchurch, NZ	6.3	RO	21-Feb-2011	SHLC	A	Bradley (2012)	Yes ^(h)

Notes:

(a) R=reverse; SS=strike slip; RO=reverse oblique

(b) Geomatrix instrument structure type (Chiou et al. 2008): I = Free-field instrument at or near ground surface. A = One-story structure of lightweight construction, instrument is located at the lowest level and within several feet of the ground surface; B = Two- to four-story structure of lightweight construction, or tall one-story warehouse-type building, instrument is located at the lowest level and within several feet of the ground surface.

(c) Described as "free field" by Kostadinov & Yamazaki (2001), which is assumed to mean GMX C1 type I, A or B.

(d) Station is part of Japan's K-Net seismograph network which does not report structure type. However a GMX C1 classification of I, A or B was deduced based on review of Google Earth aerial and street-view imagery, which shows only parking lots and 1 to 2 story buildings in the vicinity of the station coordinates.

(e) Indicates whether the ground motion is "pulse-like" based on Baker (2007) wavelet analysis classification.

(f) Confirmation of liquefaction by these references is reported in Kostadinov and Yamazaki (2001).

(g) Electronic supplement Google Earth kmz file.

(h) Shahi and Baker (2012) indicate apparent pulse may be influenced by basin effects, particularly because the strongest pulse was observed in the fault parallel direction.

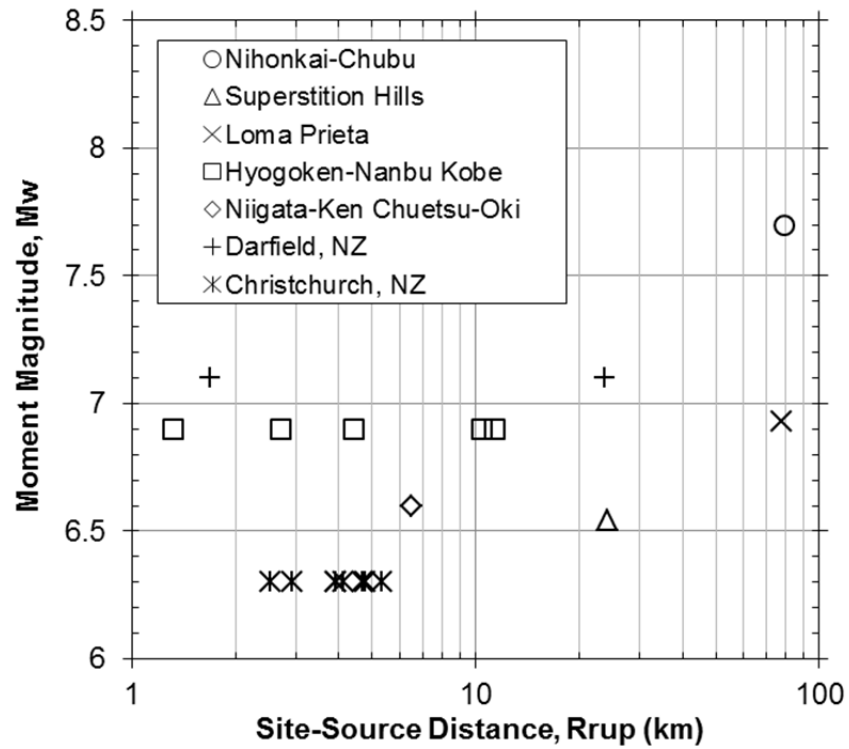


Figure 2-1: Magnitude and distance pairs represented in the liquefaction ground motion database.

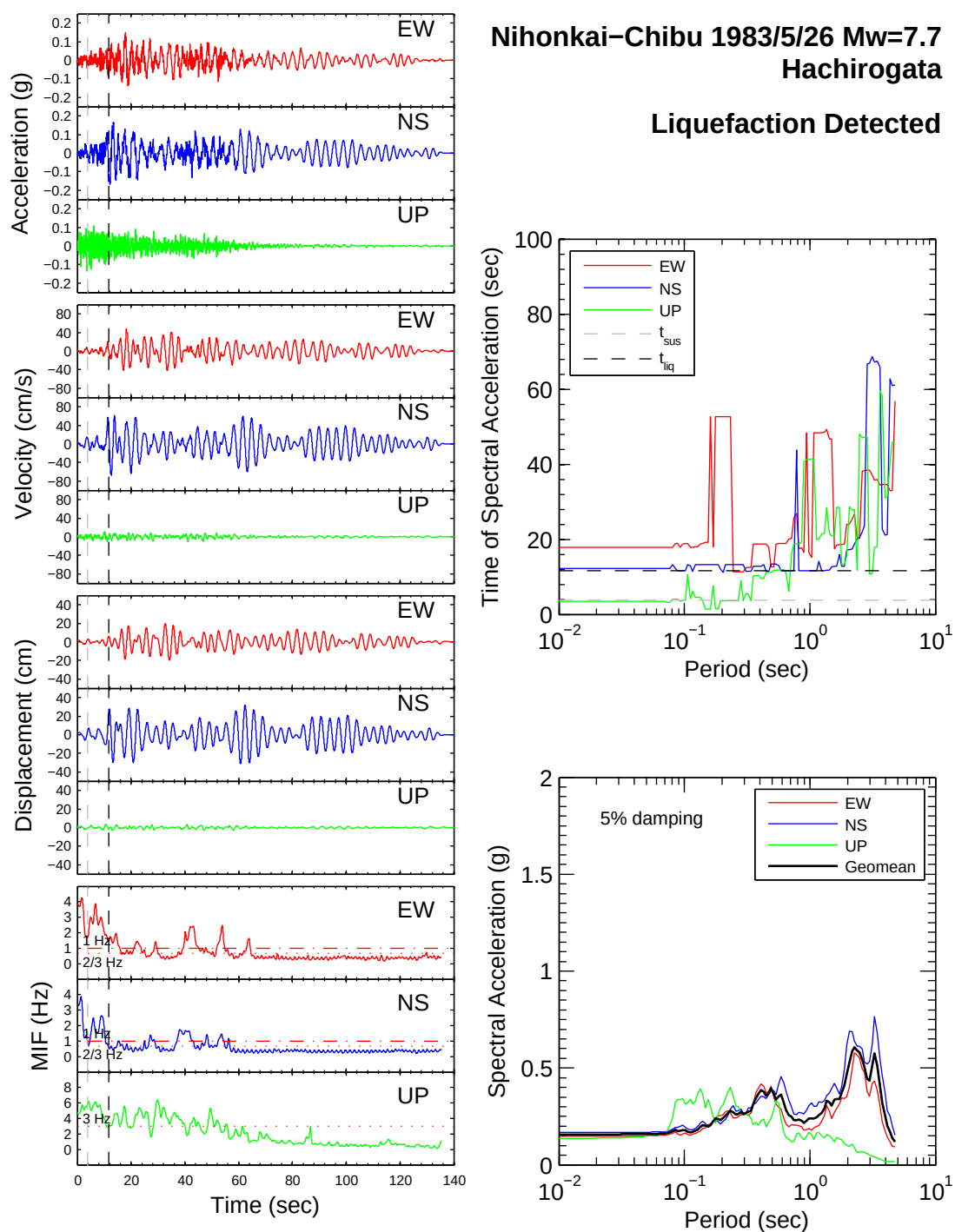


Figure 2-2: Hachirogata station recording of the M7.7 Nihonkai-Chibu, Japan earthquake of 1983.

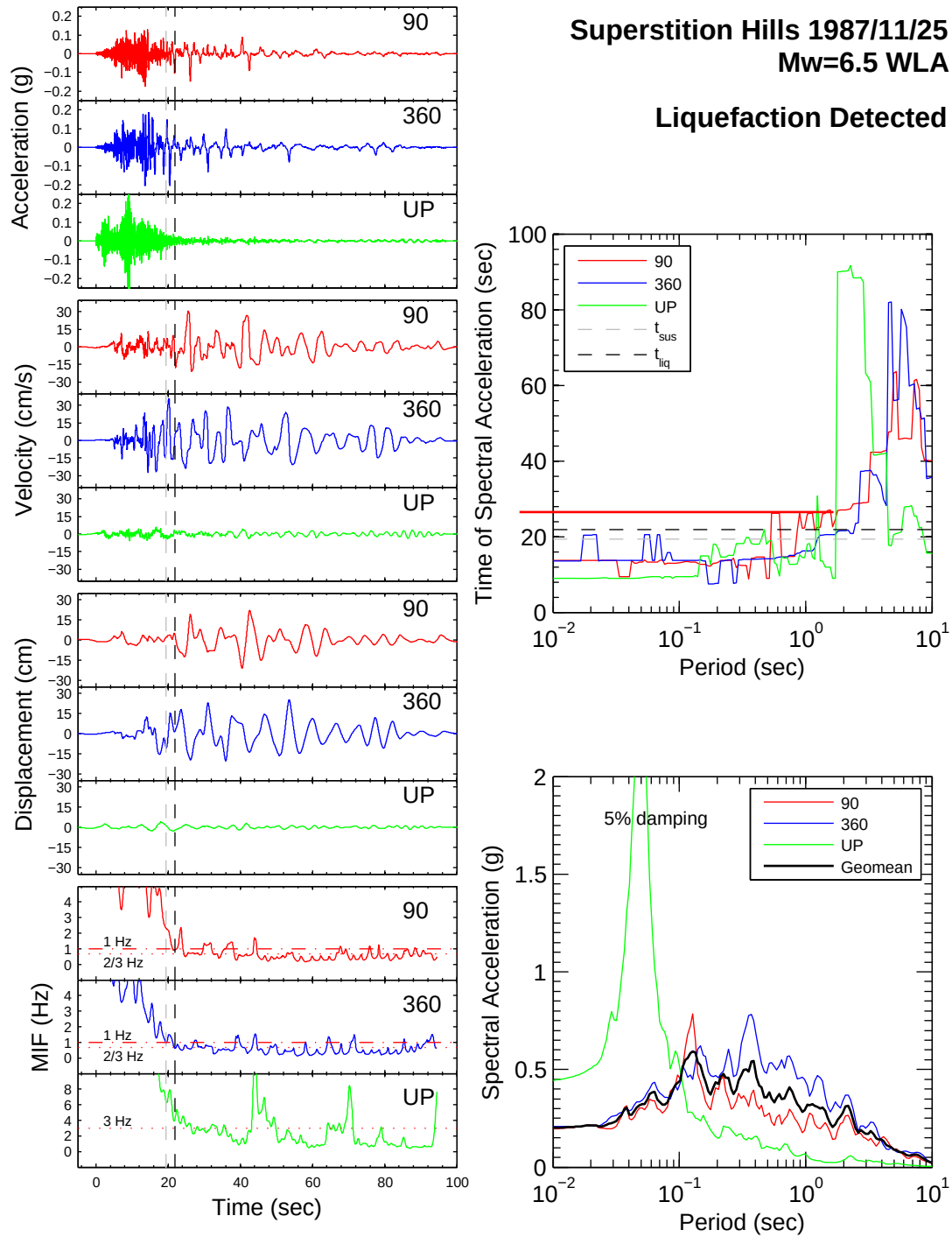


Figure 2-3: Wildlife Liquefaction Array recording of the M6.5 Superstition Hills, California earthquake of 1987.

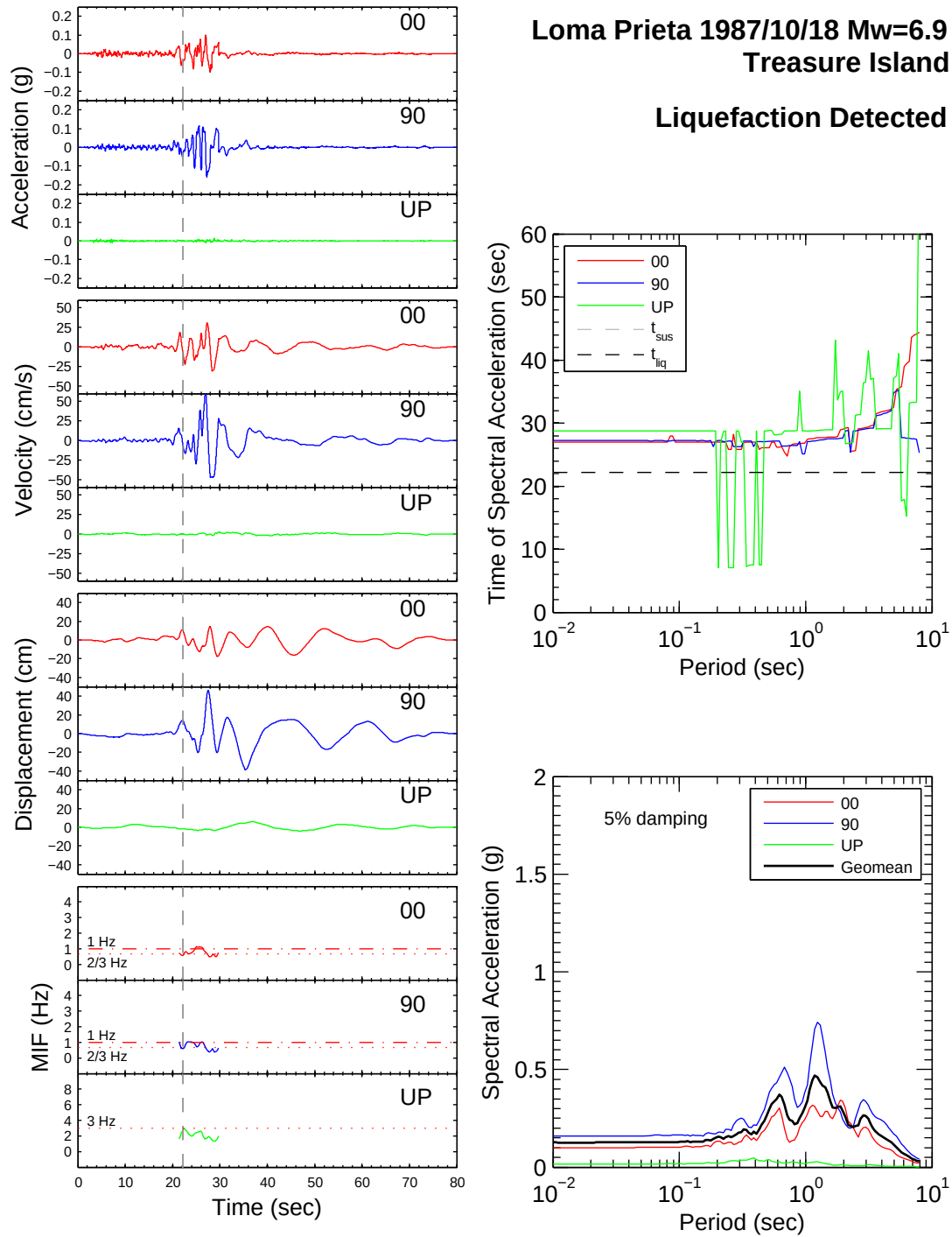


Figure 2-4: Treasure Island recording of the M6.9 M 6.93 Loma Prieta earthquake of 1989.

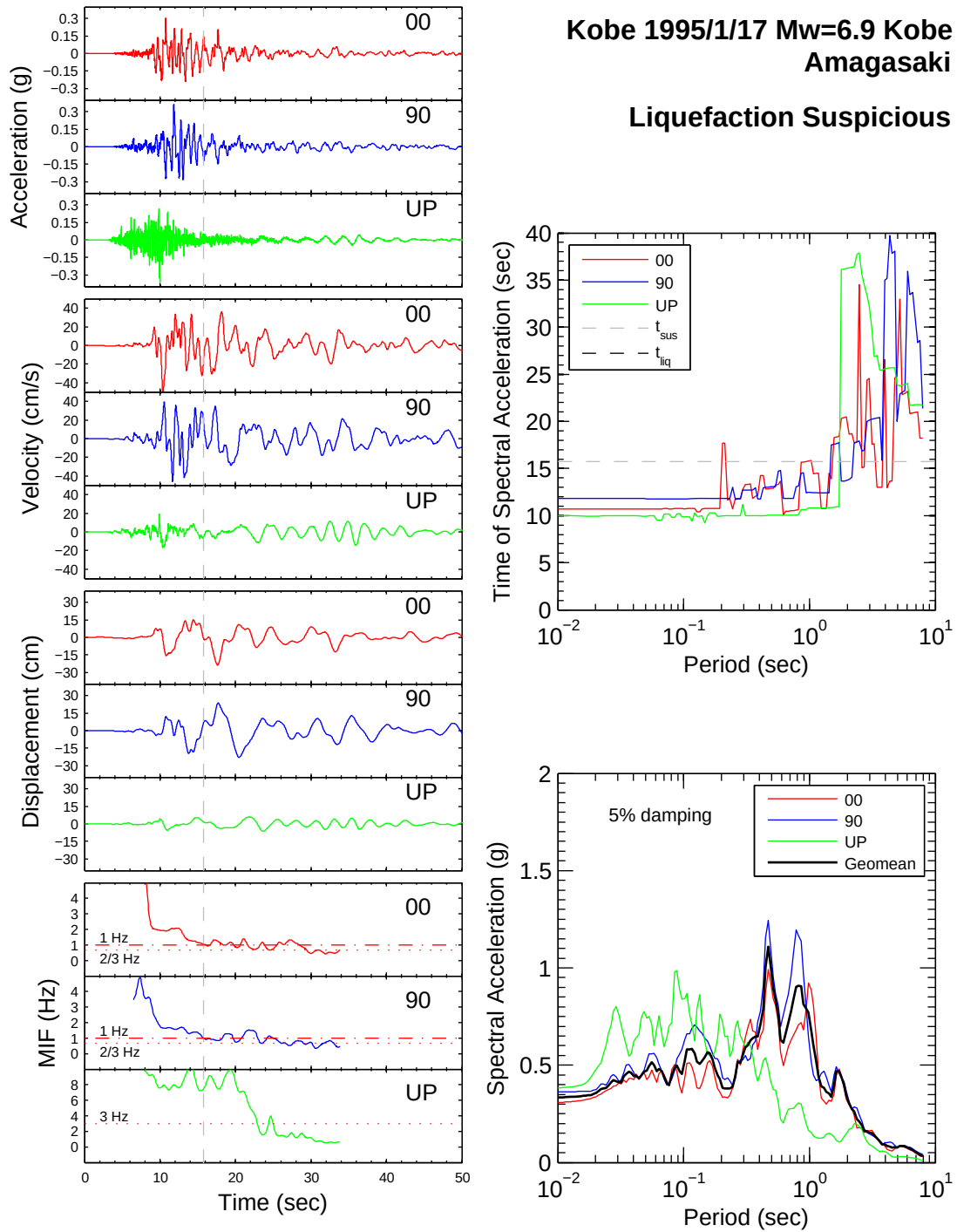


Figure 2-5: Amagasaki-G recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

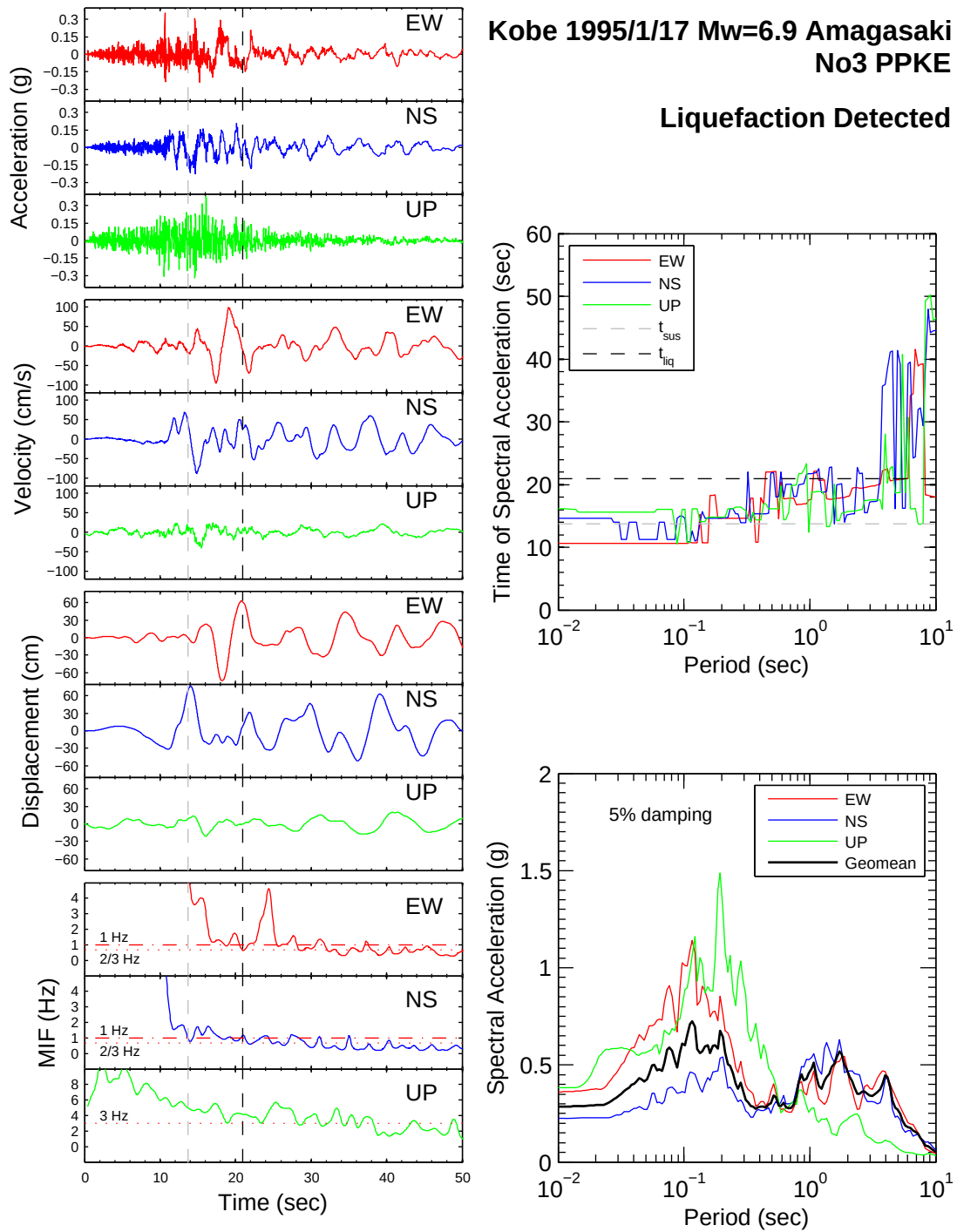


Figure 2-6: Amagasaki No. 4 P.P., KE recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

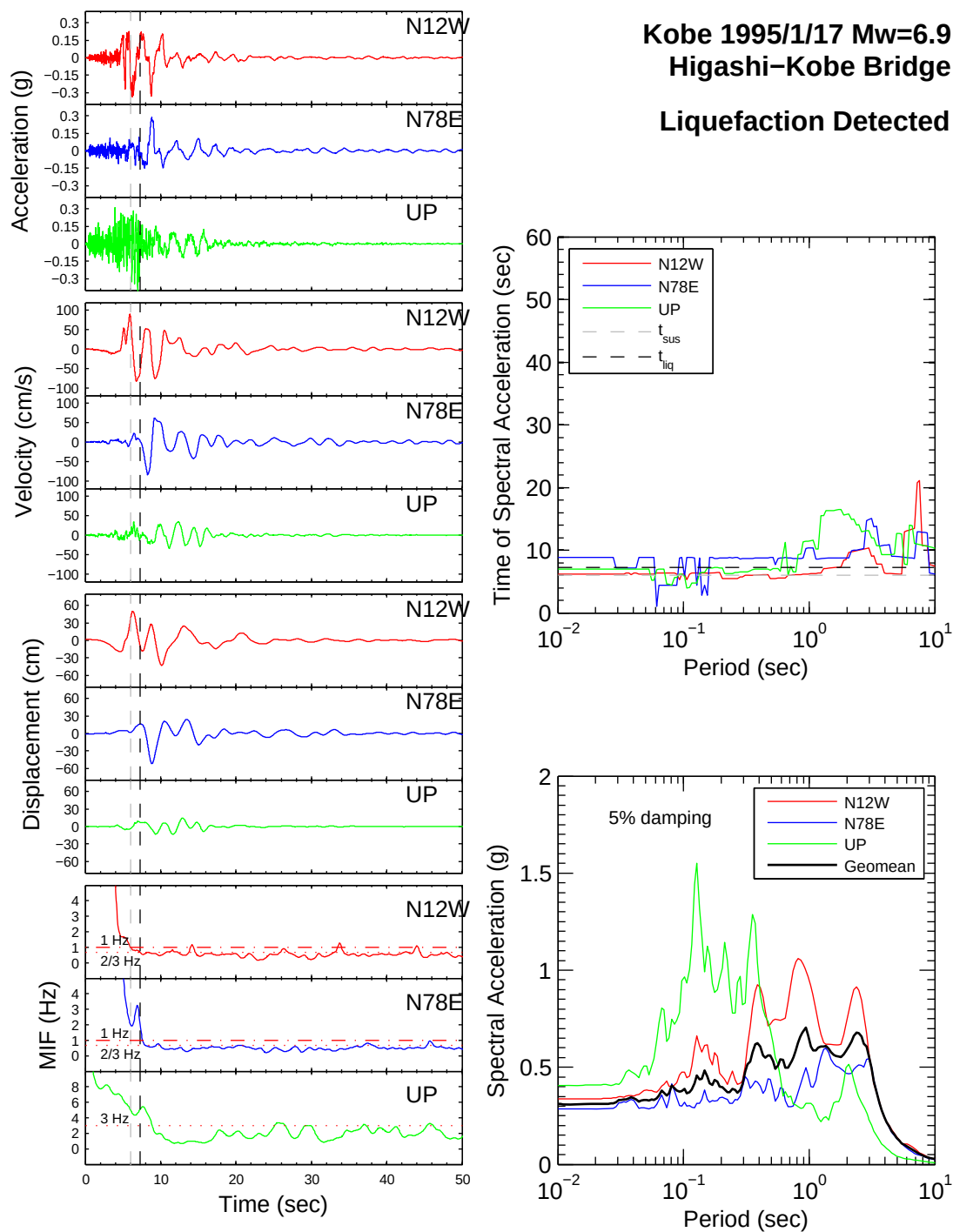


Figure 2-7: Higashi-Kobe Bridge recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

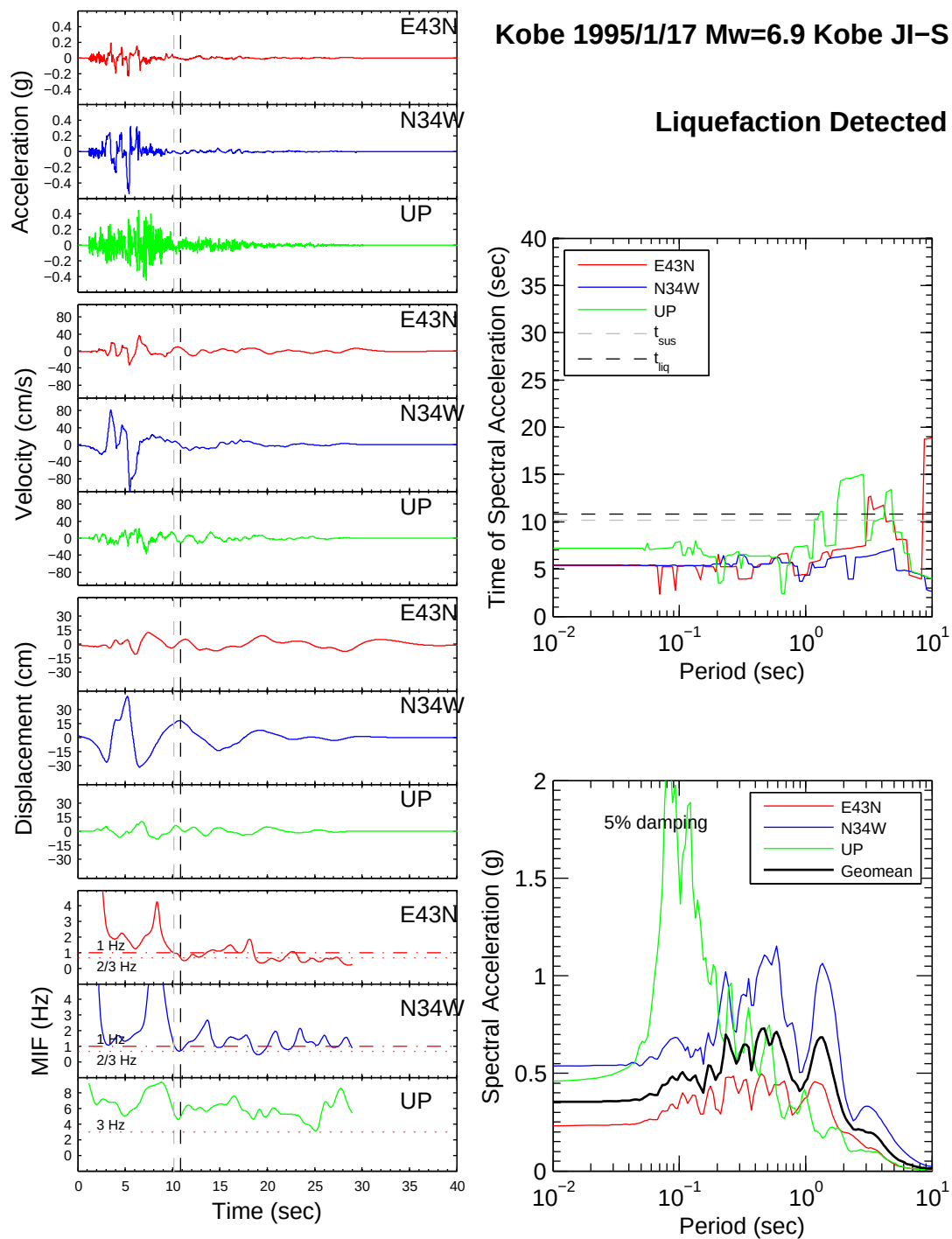


Figure 2-8: Kobe-JI-S recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

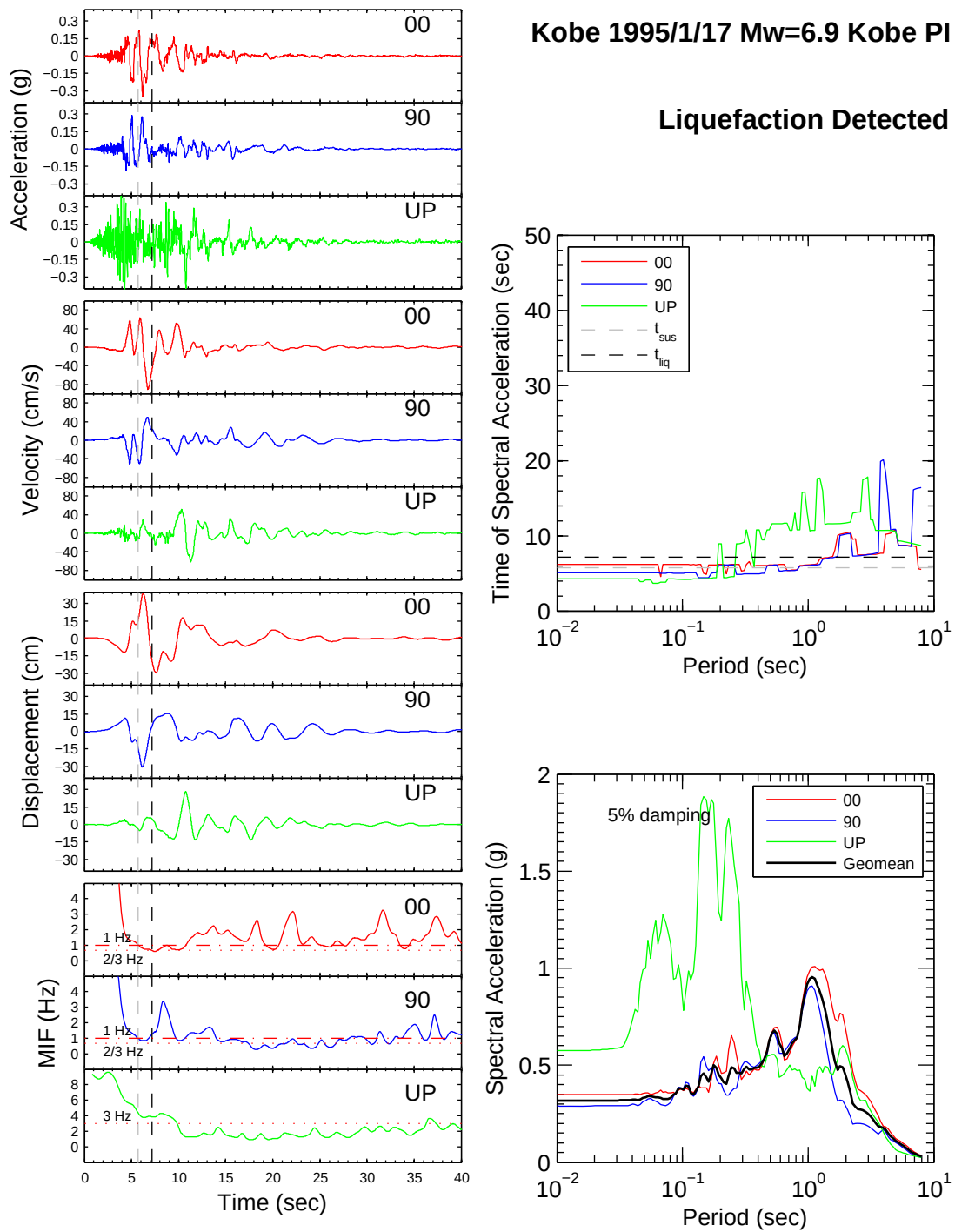


Figure 2-9: Port Island ground surface recording of the M6.9 Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

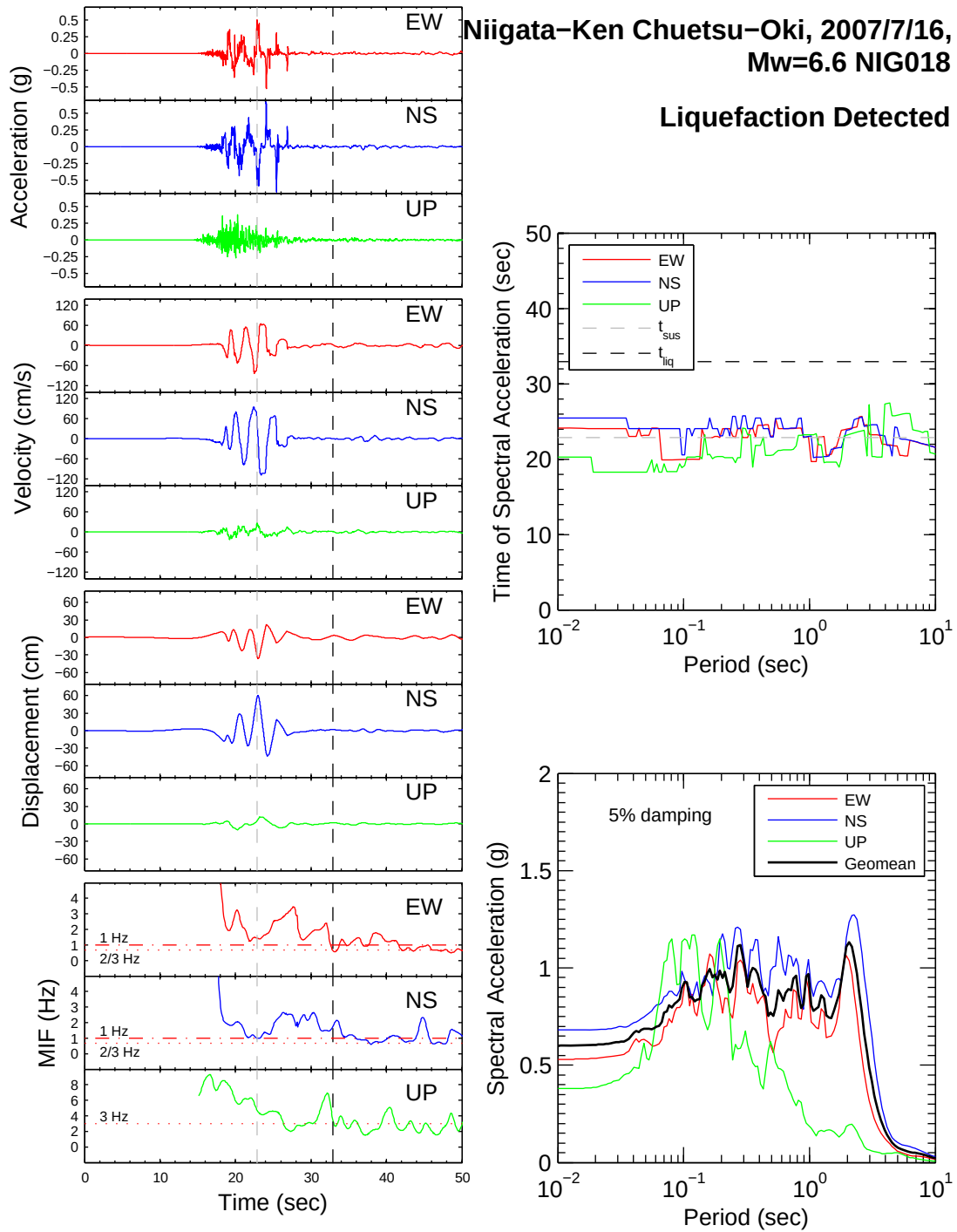


Figure 2-10: NIG018 recording of the M6.6 Niigata-Ken Chuetsu-Oki, Japan earthquake of 2007.

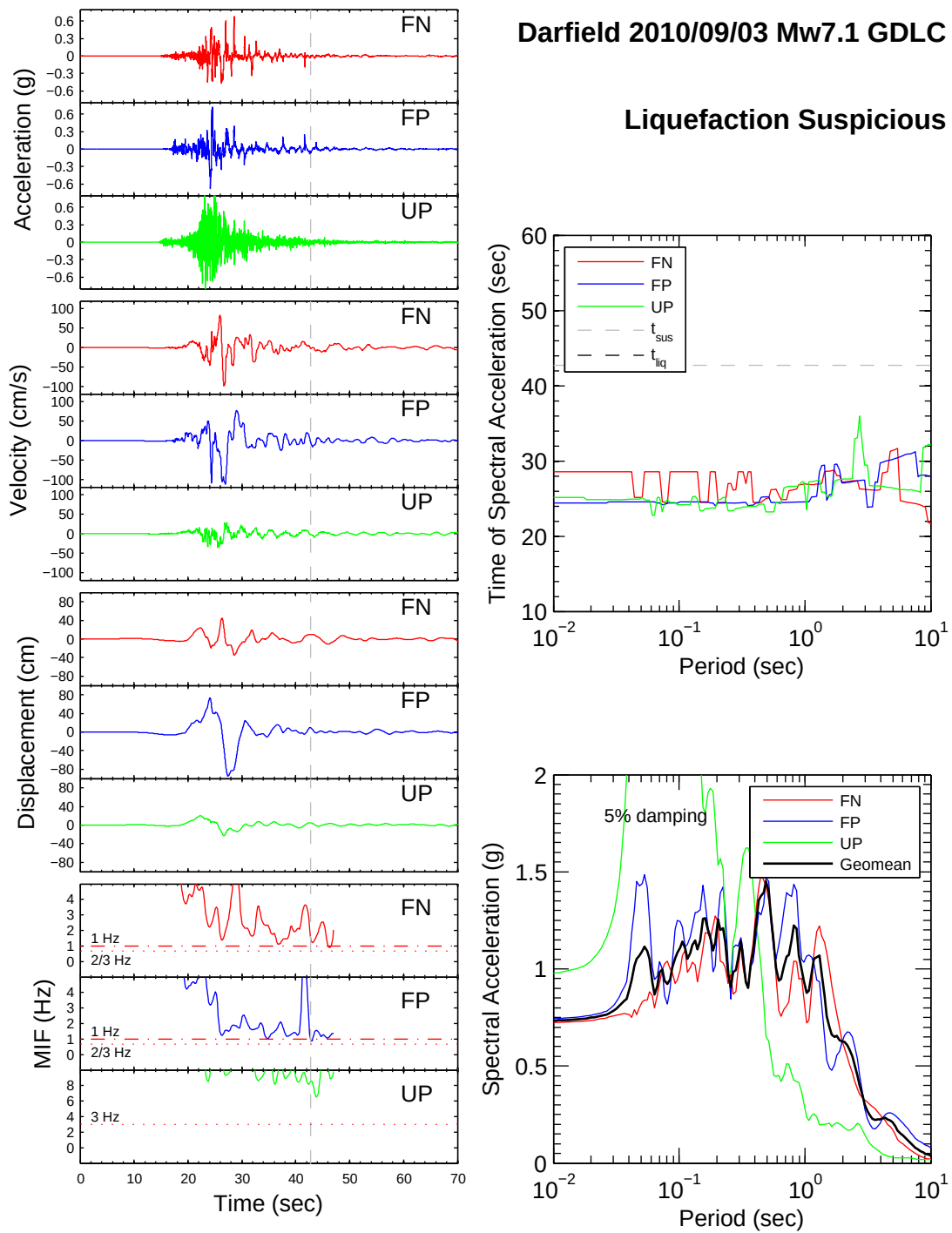


Figure 2-11: GDLC recording of the M7.1 Darfield, New Zealand earthquake of 2010.

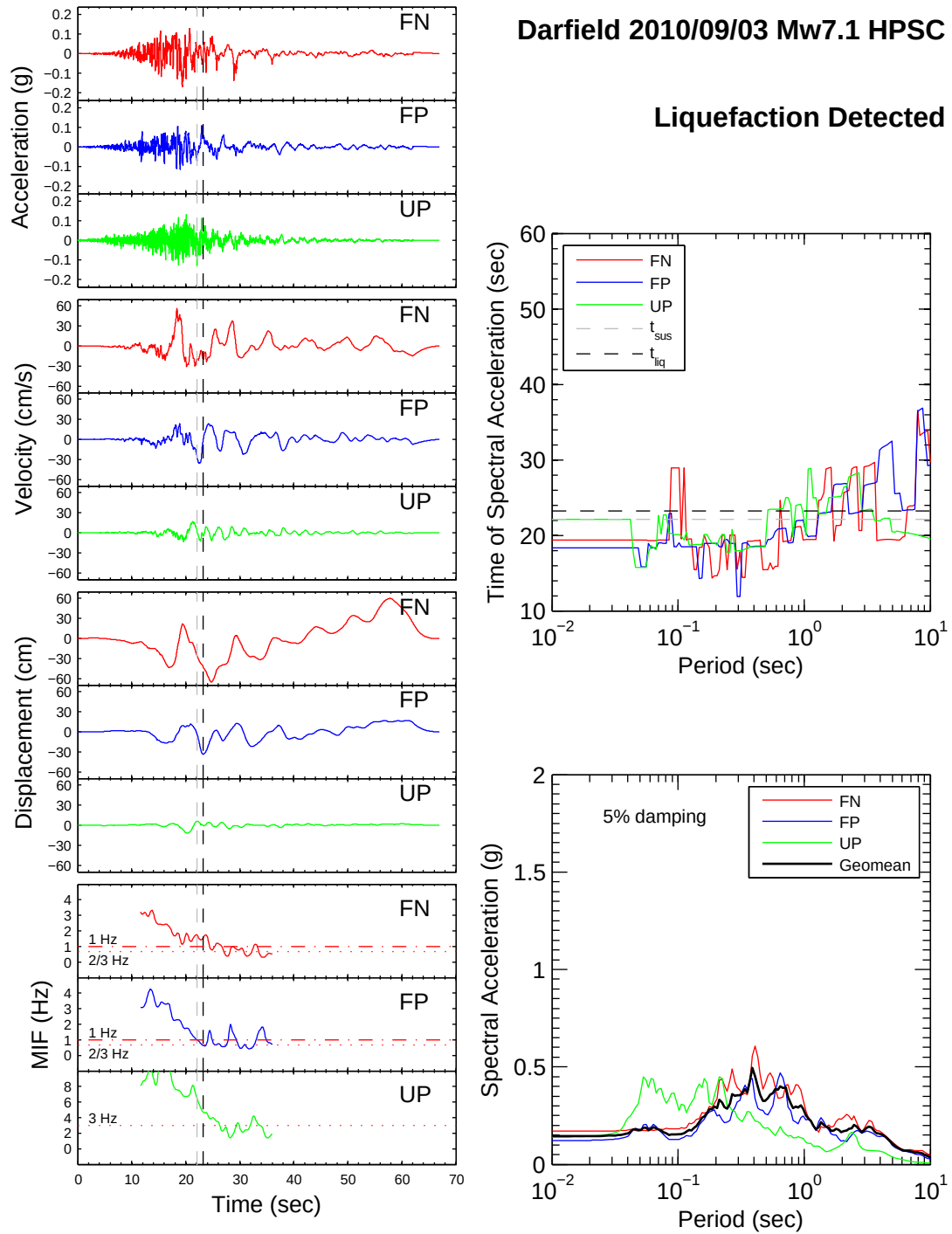


Figure 2-12: HPSC recording of the M7.1 Darfield, New Zealand earthquake of 2010.

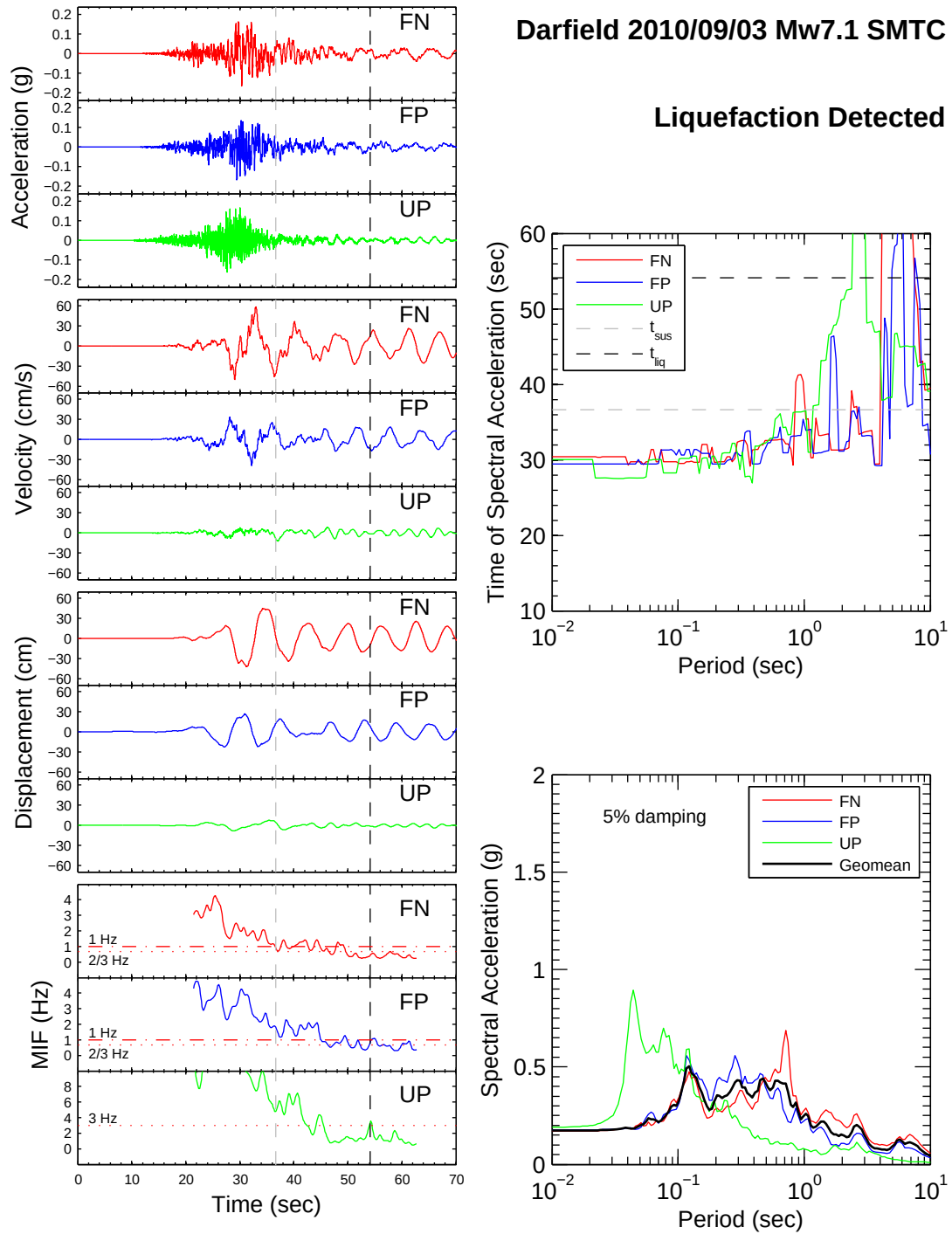


Figure 2-13: SMTc recording of the M7.1 Darfield, New Zealand earthquake of 2010.

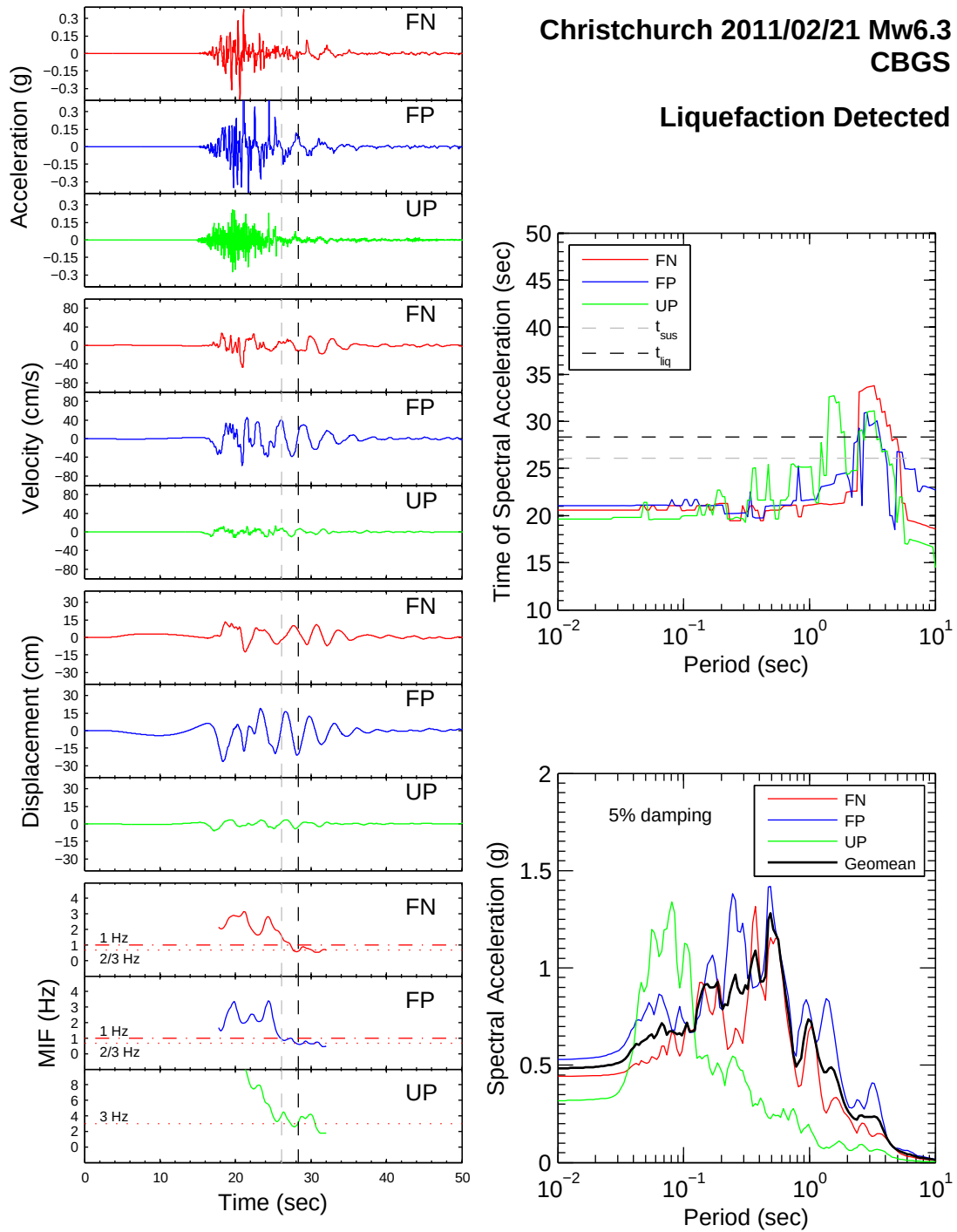


Figure 2-14: CBGS recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

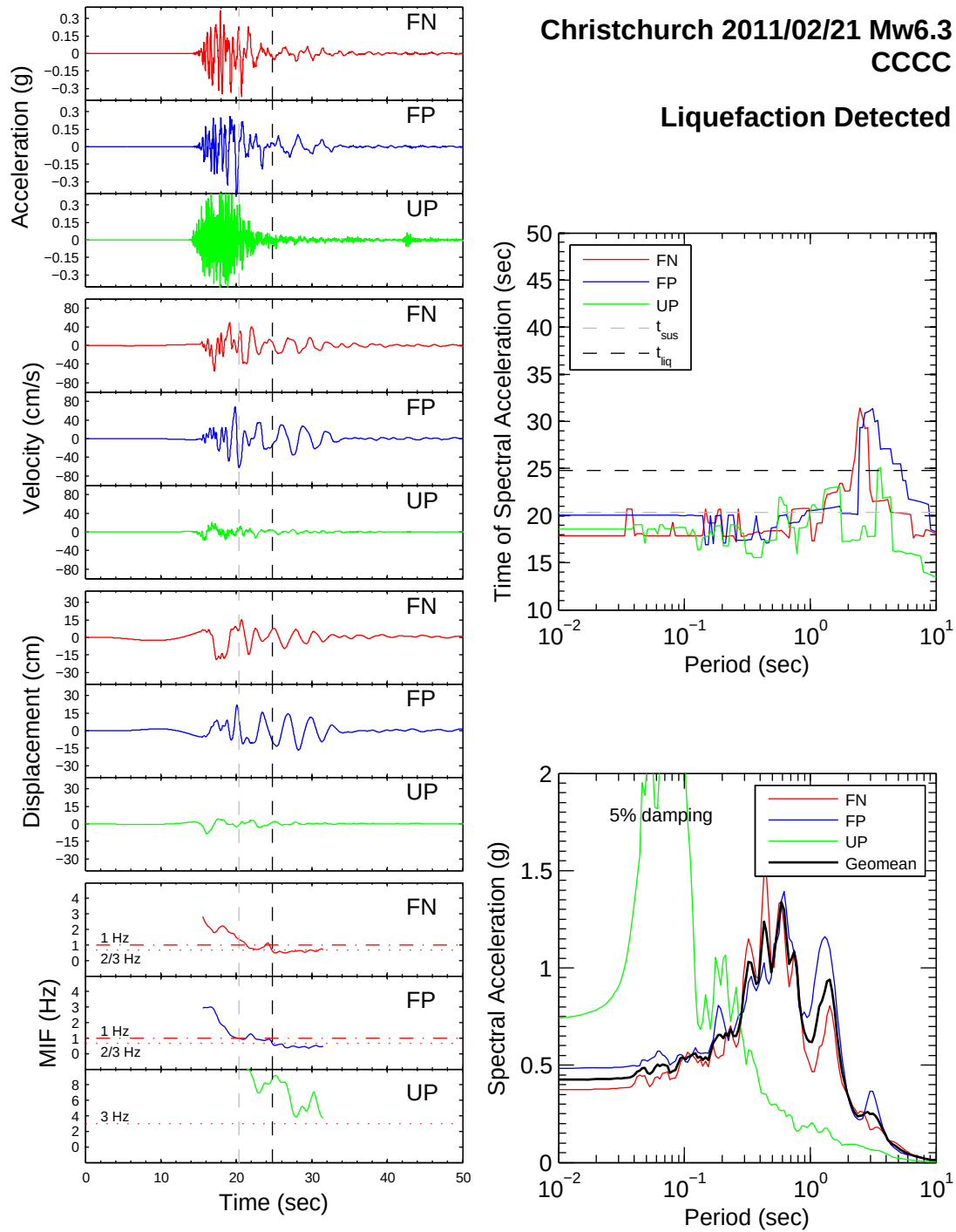


Figure 2-15: CCCC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

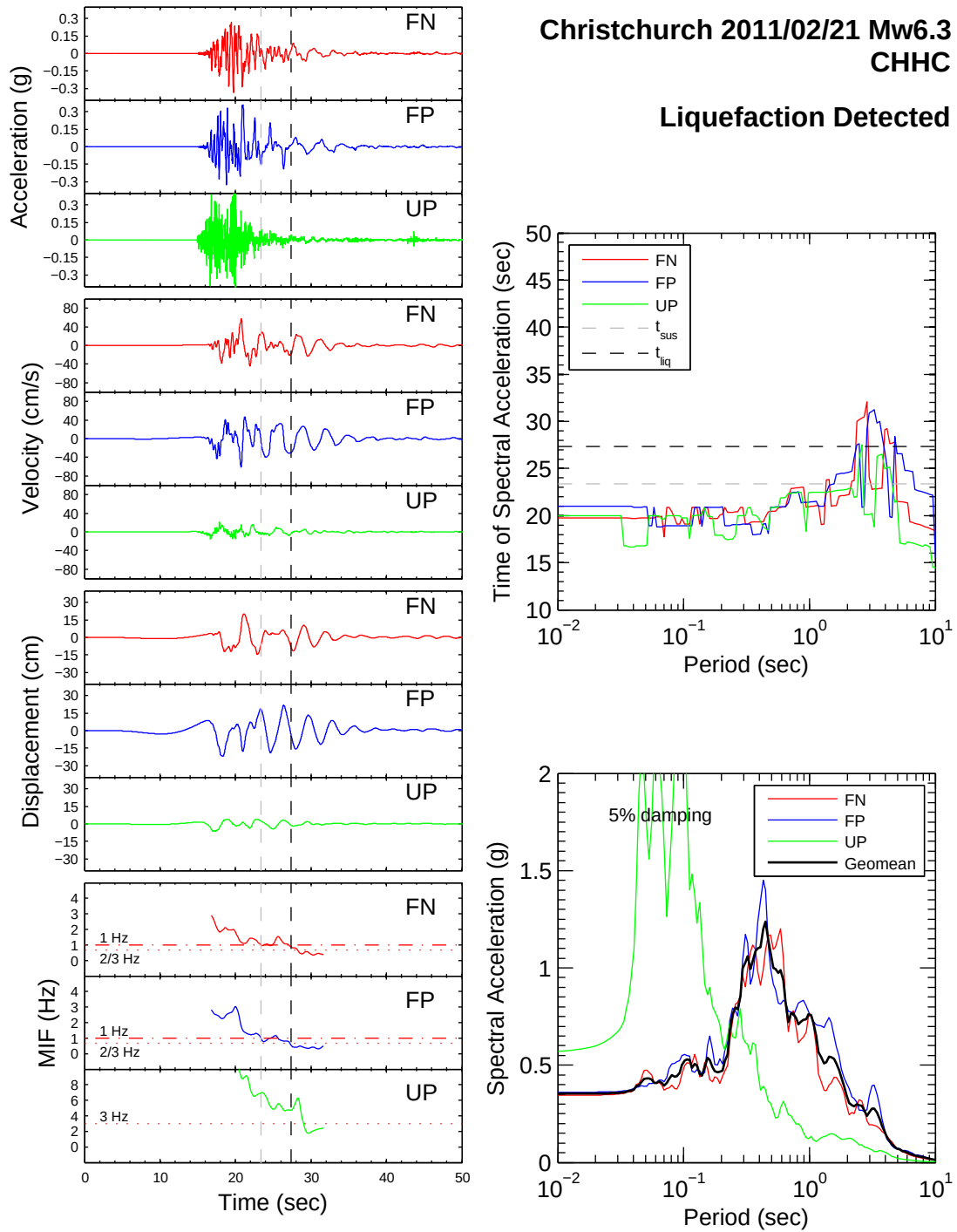


Figure 2-16: CHHC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

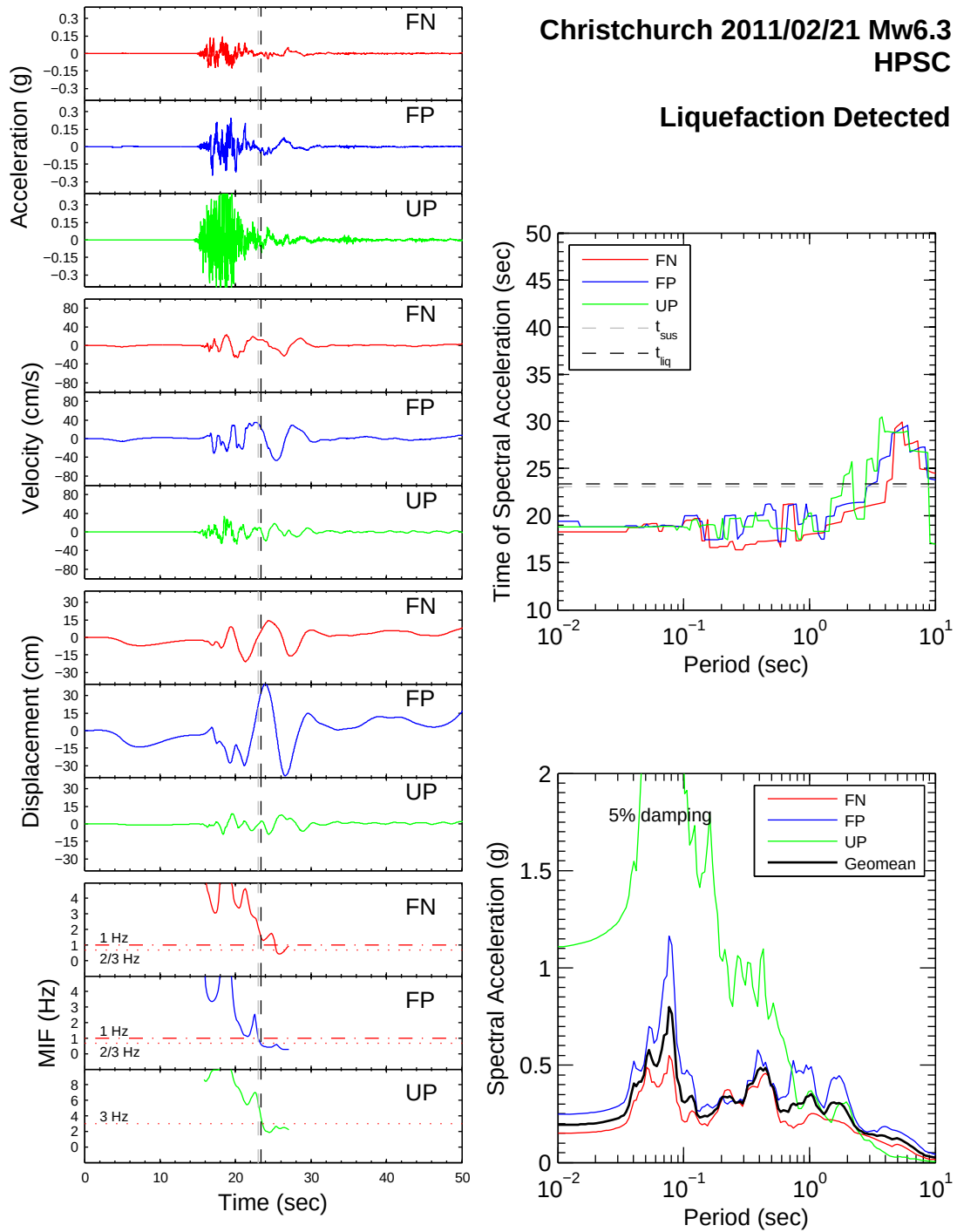


Figure 2-17: HPSC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

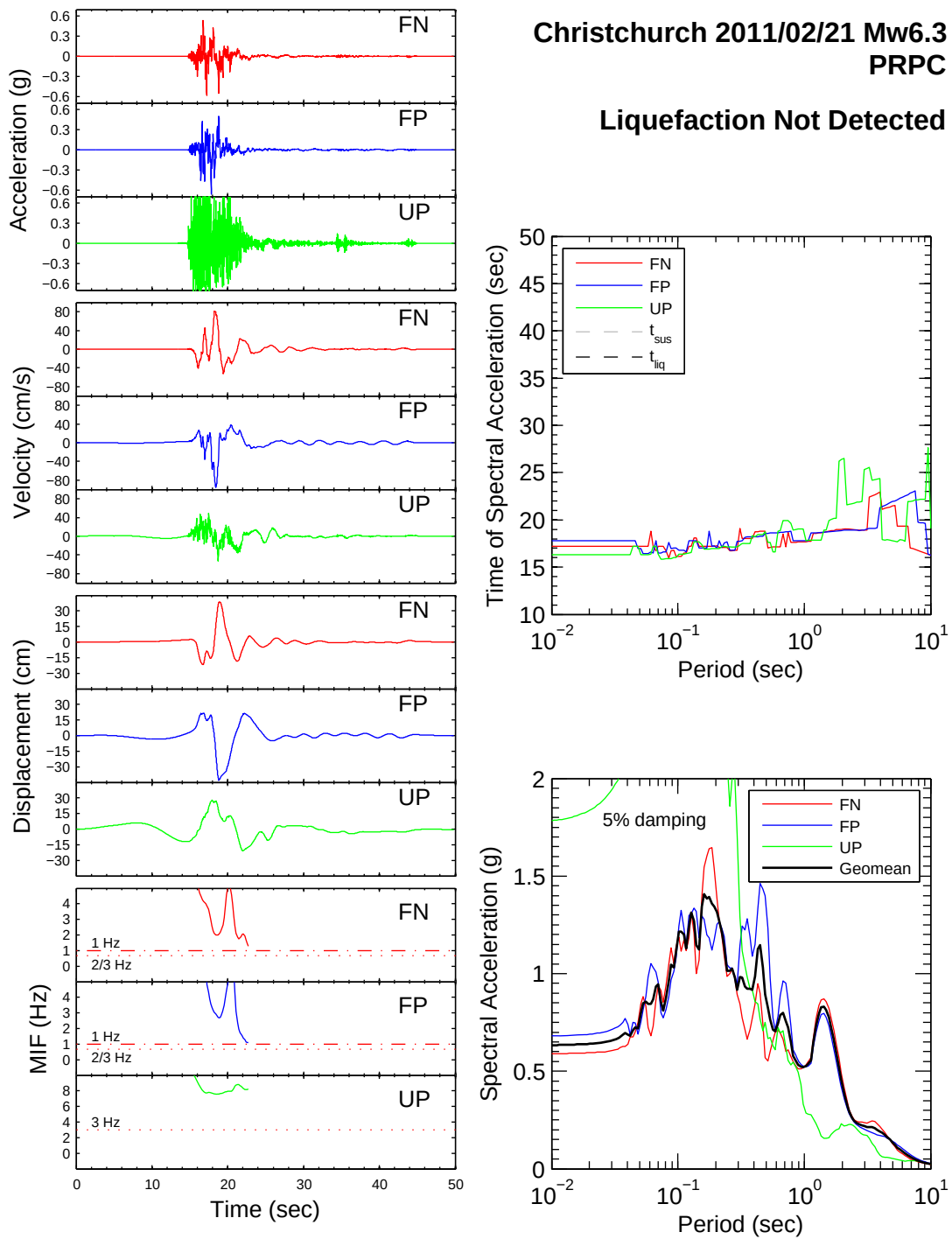


Figure 2-18: PRPC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

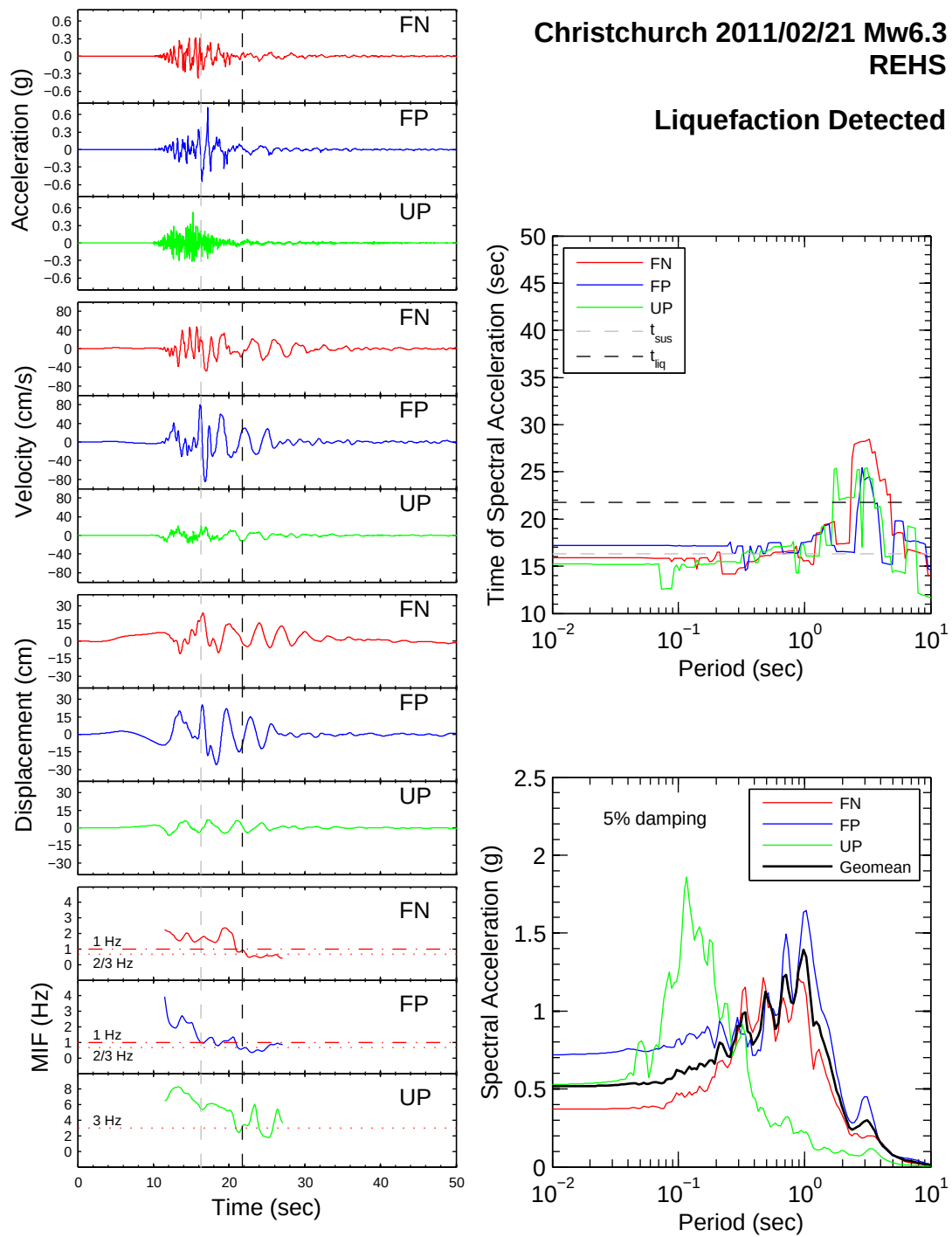


Figure 2-19: REHS recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

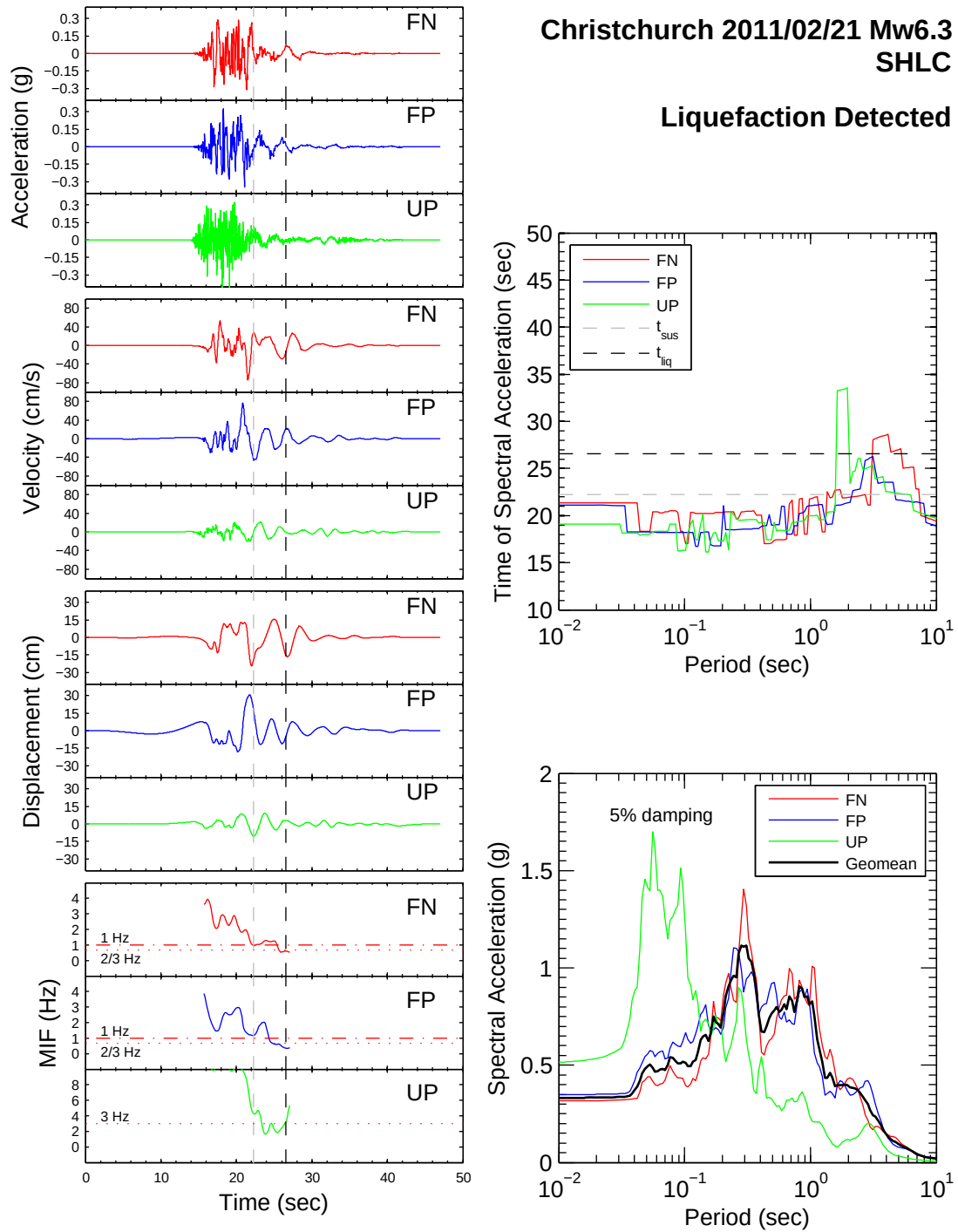


Figure 2-20: SHLC recording of the M6.3 Christchurch, New Zealand earthquake of 2011.

Ground motion recordings at liquefied sites are also available for subduction zone earthquakes. The Bureau of Ports and Harbours (1968) reported liquefaction at the Aomori Harbor S-235 recording station during the M8.35 Tokachi-Oki, Japan earthquake of 1968. Iai et al. (1995) reported that liquefaction occurred at the Kushiro-G recording station during the 1993 M7.6 Kushiro-Oki earthquake in Japan. The SDS and SDW station sites were reported to have liquefied during the 2001 Nisqually earthquake in the State of Washington (Frankel et al. 2002). Cox et al. (2013) identified 22 strong ground motion recording sites where liquefaction was confirmed following the M9.0 Tohoku, Japan earthquake of 2011. While these subduction zone event ground motions represent a valuable dataset, they were not included in this study, because currently available GMPEs for subduction zone earthquakes have relatively high levels of epistemic uncertainty (Atkinson 2012) which would lead to unreliable residual analyses. Efforts are ongoing to develop improved subduction zone GMPEs using an expanded dataset which includes recent significant events such as the M8.8 Maule, Chile earthquake of 2012 and the M9.0 Tohoku, Japan earthquake of 2011 (Bozorgnia 2012). The exclusion of subduction events was based solely on concerns regarding the uncertainty of the subduction zone GMPEs, rather than on a suspicion that faulting mechanism would play a significant role in liquefaction effects on response spectra. These recordings at liquefied sites should be investigated in a similar manner when more robust NGA-type GMPEs are developed for subduction zone earthquakes.

It is noted that surface manifestations of liquefaction are not always present, particularly when liquefaction occurs at significant depth below the ground surface. For example, Ishihara (1985) developed a chart to estimate the minimum thickness of a non-liquefiable crust necessary for liquefaction-related surface damage not to occur. Considering that liquefaction could occur at ground motion recording stations without observable surface manifestations, it is not possible to identify and include all of the potentially available liquefied-site ground motion recordings.

2.2.1 Time Series Processing

The Superstition Hills, Amagasaki-G, and Port Island recordings were obtained from PEER and had already been processed in accordance with the NGA West procedures outlined in Chiou et al. (2008). The remainder of the ground motions was obtained from a variety of sources and has been subjected to various levels of processing. It was assumed that the non-PEER records had already been corrected for instrument response considerations. To promote uniformity in processing characteristics and consistency with NGA relationships, the non-PEER ground motions were processed in general accordance with the NGA database procedures (Chiou et al. 2008). The processing sequence generally involved: 1) subtract the mean of the acceleration time series; 2) perform filtering; 3) perform baseline correction; and 4) (optional) remove zero pads added during filtering.

An important aspect of the time series processing involved high-pass filtering and evaluation of the largest usable period, because liquefaction generally results in softening that can amplify long period ground motions. All filtering was done using a fourth order high-pass acausal Butterworth filter, with zero pads added and largest usable period evaluated in accordance with the recommendations of Boore and Bommer (2005). Low frequency “noise” was differentiated from signal by examining the decay of the Fourier amplitude spectrum (FAS) at low frequencies. The high-pass corner frequency was selected where the FAS decay diverged from a straight line proportional to the frequency squared (Boore and Bommer 2005). The largest usable period was taken as the period at which the filter response was $-1/2$ dB less than (or 94.4 percent of) the maximum response (Chiou et al. 2008). Where high-pass filtering did not remove permanent drift, a baseline correction was applied by fitting a second or third order polynomial to the acceleration time history and subtracting it from the signal. The processing did not significantly alter the record’s peak ground acceleration (PGA) or other key ground motion parameters. Primarily, it eliminated displacement drift.

2.2.2 Site and Earthquake Characteristics

Site and earthquake parameters are necessary to make ground motion estimates using the NGA GMPEs. These parameters include fault mechanism, earthquake moment magnitude (M_w), fault width (W), various site-source distance measurements (R_{rup} , R_{jb} , R_x), depth to top of rupture (Z_{TOR}), the average

shear wave velocity in the upper 30 meters (V_{s30}), and depth to material with shear wave velocities of 1 km/s and 2.5 km/s (Z_1 and $Z_{2.5}$) (Abrahamson et al. 2008). A summary of the parameters used in Chapter 4 to calculate median acceleration response spectra using GMPEs is provided in Tables 2-2 and 2-3. Data sources and calculation methods used to develop these parameters are discussed in the subsequent sections.

2.2.2.1 Nihonkai-Chubu Earthquake

The M_w 7.7 Nihonkai-Chubu earthquake occurred on 26 May 1983 in the Japan Sea approximately 80 km west of the coast of the Aomori and Akita Prefectures in northern Honshu, Japan. Based on the fault plane interpretation of Sato (1985), a two-segment reverse rupture dipping to the east at 20 degrees was used in this study to estimate site-source distance parameters (Figure A-1 in Appendix A). Sato (1985) reported a complex 2 or 3 phase rupture pattern for this event with seismic moments ranging from 7.5×10^{27} to 8.0×10^{27} dyne-cm, which corresponds with a moment magnitude of about 7.9. The USGS (2013) report seismic moments of 1.2×10^{27} and 4.6×10^{27} dyne-cm, which correspond to moment magnitudes of 7.4 and 7.8, respectively. Based on these data, a moment magnitude of 7.7 was used in this study for the purpose of estimating ground motions using NGA GMPEs.

Table 2-2: Summary of Parameters Used in NGA GMPEs

Earthquake	Station	M_w	R_{rup} (km)	R_{jb} (km)	R_x (km)	U	F_{RV}	F_N	F_{HW}	Z_{TOR} (km)	δ ($^\circ$)	V_{s30} (m/s)	F_{MEAS}	Z_1 (m)	$Z_{2.5}$ (km)	W (km)	$T_{max}^{(b)}$
Nihonkai-Chubu	Hachirogata	7.7	78.9	78.0	113	0	1	0	1	0	20	180	0	850/337 ^(a)	3.6	35	4.8
Superstition Hills	Wildlife	6.54	24.1	11.8	11.8	0	1	0	0	0	90	181	1	200	3	12	8
Loma Prieta	Treasure Island	6.93	77.4	77.3	77.3	0	0	0	0	3.8	70	155	1	850/337 ^(a)	3.6	18	8
Kobe	Amagasaki-G	6.9	10.4	10.4	10.4	0	0	0	0	0.2	85	256	0	600	1.2	20	8
Kobe	Amagasaki No. 3 P.P., KE	6.9	11.3	11.3	11.8	0	0	0	0	0.2	85	250	0	600	1.2	20	10
Kobe	Higashi-Kobe Bridge	6.9	4.4	4.4	4.4	0	0	0	0	0.2	85	250	0	600	1.2	20	10
Kobe	Kobe-JI-S	6.9	1.3	1.3	1.3	0	0	0	0	0.2	85	194	0	600	1.2	20	10
Kobe	Port Island GL	6.9	2.7	2.7	2.7	0	0	0	0	0.2	85	194	1	600	1.2	20	8
Niigata-Ken Chuetsu-Oki	NIG018 (K-Net)	6.6	6.5	0.0	5.5	0	1	0	0	5	36	182	1	850/337 ^(a)	3.5	25.3	10
Darfield, NZ	GDLC	7.1	1.7	0.0	0	0	0	0	0	0	73.5	305	0	1000	2	26	10
Darfield, NZ	HPSC	7.1	23.8	22.5	22.5	0	0	0	0	0	73.5	200	0	1000	2	26	10
Christchurch, NZ	CBGS	6.3	4.7	4.6	-4.6	0	1	0	0	1	67	240	0	1000	2	7	10
Christchurch, NZ	CCCC	6.3	2.9	2.8	-2.8	0	1	0	0	1	67	252	0	1000	2	7	10
Christchurch, NZ	CHHC	6.3	3.9	3.8	-3.8	0	1	0	0	1	67	150	0	1000	2	7	10
Christchurch, NZ	HPSC	6.3	4.1	4.0	-4.0	0	1	0	0	1	67	200	0	1000	2	7	10
Christchurch, NZ	NNBS	6.3	3.9	3.8	-3.8	0	1	0	0	1	67	24	0	1000	2	7	10
Christchurch, NZ	PRPC	6.3	2.5	2.4	-2.4	0	1	0	0	1	67	223	0	1000	2	7	10
Christchurch, NZ	REHS	6.3	4.8	4.7	-4.7	0	1	0	0	1	67	259	0	1000	2	7	10
Christchurch, NZ	SHLC	6.3	5.3	5.3	-5.3	0	1	0	0	1	67	205	0	1000	2	7	10

Notes:

(a) Calculated from Vs30 using values recommended by AS08 (first value) and CY08 (second value).

(b) T_{max} is the highest usable period as determined according to Boore and Bommer (2005).

Table 2-3: Summary of Directivity Parameters Used in NGA GMPs

Earthquake	Station	Bayless & Somerville Parameters					AF _{dir}					
		s for SS or L for SS		θ (deg)	Az (deg)	Rake (deg)	T≤0.75s	T=1s	T=2s	T=3s	T=5s	T=10s
		[d] for DS [W] (km)	(km)									
Nihonkai-Chubu	Hachirogata	-	-	-	-	~90	1.00	1.00	1.00	1.00	1.00	1.00
Superstition Hills	Wildlife	17.2	20	54.2	-	180	1.00	1.00	1.00	1.00	1.00	1.00
Loma Prieta	Treasure Island	-	-	-	-	140	1.00	1.00	1.00	1.00	1.00	1.00
Hyogoken-Nanbu Kobe	Amagasaki-G	34.3	60	19.4	-	180	1.00	1.12	1.09	1.08	1.07	1.06
Hyogoken-Nanbu Kobe	Amagasaki No. 3 P.P., KE	31.6	60	23.2	-	180	1.00	1.10	1.07	1.06	1.04	1.04
Hyogoken-Nanbu Kobe	Higashi-Kobe Bridge	25.9	60	13.6	-	180	1.00	1.12	1.09	1.07	1.06	1.06
Hyogoken-Nanbu Kobe	Kobe-JI-S	18.6	60	7.9	-	180	1.00	1.10	1.06	1.05	1.04	1.03
Hyogoken-Nanbu Kobe	Port Island GL	16.3	60	14.2	-	180	1.00	1.08	1.04	1.03	1.01	1.00
Niigata-Ken Chuetsu-Oki	NIG018 (K-Net)	[10.1]	[25.3]	-	90	90	1.00	1.00	1.08	1.19	1.23	1.26
Darfield, NZ	GDLC	1	43.9	57.14	-	180	1.00	1.00	1.00	1.00	1.00	1.00
Darfield, NZ	HPSC	30	43.9	6.34	-	180	1.00	1.13	1.11	1.09	1.08	1.08
Christchurch, NZ	CBGS	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.08	1.06	1.06
Christchurch, NZ	CCCC	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.08	1.06	1.06
Christchurch, NZ	CHHC	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.08	1.05	1.05
Christchurch, NZ	HPSC	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.07	1.05	1.04
Christchurch, NZ	NNBS	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.08	1.05	1.05
Christchurch, NZ	PRPC	[4.4]	[7.0]	-	90	135	1.00	1.00	1.04	1.09	1.07	1.07
Christchurch, NZ	REHS	[4.4]	[7.0]	-	90	135	1.00	1.00	1.03	1.07	1.04	1.03
Christchurch, NZ	SHLC	[4.4]	[7.0]	-	90	135	1.00	1.00	1.03	1.06	1.03	1.02

The Hachirogata strong ground motion recording station is in an area that has been mapped with late Pleistocene to Holocene dune deposits overlaying late Pleistocene to Holocene marine and non-marine sedimentary deposits (GSJ 2012). Liquefaction was reported to have occurred at this station during the Nihonkai-Chubu earthquake (Yanagisawa 1984, Kostadinov and Yamazaki 2001). Kostadinov and Towhata (2002) indicate this site is underlain by approximately 10 meters of loose to medium dense sand, then about 15 meters of soft silty clay, then 6 meters of stiffer sandy silt and silty clay (Figure A-2 in Appendix A). Shear wave velocity data was not available, so a value of $V_{s30}=180$ m/s was estimated based on the soil profile data. A digital record of the acceleration time history recorded at this site during this earthquake was provided by Professor Fumio Yamazaki (personal communication, 2012).

2.2.2.2 Superstition Hills Earthquake

With the exception of V_{s30} , all of the site and source parameters for the November 24, 1987 M_w 6.54 Superstition Hills earthquake were based on the NGA database (Power et al. 2008, PEER 2012). A V_{s30} of 181 m/s was calculated for the WLA site based on the average V_s values measured in three seismic CPTs (Figure A-3 in Appendix A) performed at the site (Youd et al. 2007, NEES@UCSB 2012). The stratigraphic sequence at the site from the ground surface downward generally consist of 2.5 m of lean clay to silt (CL-ML), 4.3 m of loose to medium dense silty sand to sandy silt, and then lean clay. Liquefaction at the WLA site during the Superstition Hills earthquake has been

well-documented in the sandy silt to silty sand layer based on analysis of porewater pressure and acceleration/velocity/displacement records (Holtzer, et al. 1989, Zeghal and Elgamal 1994).

2.2.2.3 Hyogoken-Nambu Kobe Earthquake

Earthquake fault source parameters for the 17 January 1995 *Mw* 6.9 Hyogoken-Nambu Kobe (for simplicity, sometimes denoted herein as “Kobe” earthquake) in the Hyogo Prefecture of Japan were based on Power et al. (2008) and PEER (2012). Of the 5 strong ground motion recordings from this earthquake used in this study, 2 were included in the NGA database: Amagasaki-G and Port Island (ground surface). The remaining three ground motions – Amagasaki No. 3 P.P., KE, Higashi-Kobe Bridge, and Kobe-JI-S were obtained courtesy of Professor Fumio Yamazaki (personal communication, 2012).

Values of V_{s30} used in the NGA predictions of response spectra were taken from various sources. The Amagasaki-G site V_{s30} of 256 m/s and all of its other site and site-source parameters were based on the NGA flatfile (PEER 2012). For the Higashi-Kobe Bridge station, the V_{s30} value was based on the V_s profile reported by Ganey et al. (1998). The Amagasaki No. 3 P.P., KE station V_{s30} was inferred to be the same as that of the Higashi-Kobe Bridge station due to its proximity and similar geologic setting. The V_{s30} value used for Port Island was calculated from the V_s profile reported in Iwasaki (2009) and the V_{s30} value for Kobe-JI-S was assumed to be the same as Port Island due to their proximity and similar geology. Site-source geometry for Port Island and Amagasaki-G were

based on the values reported the NGA flatfile (PEER 2012). For the other three sites the site-source parameters were measured based on the fault geometry shown on Figure A-4.

Sources for liquefaction at the Kobe earthquake stations are presented in Table 2-1. With the exception of the Amagasaki-G station where the ground conditions were uncertain, all of the Kobe stations were situated in reclaimed land areas, and therefore were underlain by young hydraulically-placed sandy fills.

2.2.2.4 Niigata-Ken Chuetsu-Oki Earthquake

The 16 July 2007 Niigata-Ken Chuetsu-Oki earthquake consisted of an *Mw* 6.6 occurring on a reverse fault on the west coast of Japan in the southern Niigata Prefecture. Aftershock locations and source inversion showed conjugate faults, one dipping southeast at 36 degrees and the other northwest at 54 degrees, with most of the slip originating on the southeast plane (Miyake et al. 2010). The southeast plane (Figure A-5) was assumed for the purpose of calculating site-source distance parameters for use with NGA GMPEs.

The soil profile at the K-Net NIG018 station consists of approximately 7 meters of loose to medium dense sand over 5.5 meters of medium dense to dense sand overlying clay that is at least 7.5 meters deep. A V_{s30} value of 182 m/s was calculated based on the K-Net V_s profile (Figure A-6 in Appendix A). Post-earthquake reconnaissance team observations at the RK27 site, which is in the same block as the NIG018 recording station, indicated liquefaction evidence

that included ground cracking and an uplifted underground storage tank (Kayen et al. 2008, electronic kmz supplement).

2.2.2.5 Darfield Earthquake

The *Mw* 7.1 Darfield earthquake occurred in the Canterbury Plains region east of Christchurch, New Zealand on 4 September 2010. This *Mw* 7.1 event caused strike-slip on the previously unrecognized Greendale fault. Based on surface fault rupture mapping and ground motion inversion analysis, Hayes (2010) interpreted a single fault plane with a strike azimuth of 85.1 degrees and a dip of 82.2 degrees to the south. Using InSAR and geodetic measurement along with inversion modeling, Holden et al. (2011) interpreted rupture along a primary strike-slip fault plane with an azimuth of 87.14 degrees and a dip to the north of 73.5 degrees, and two minor northeast striking reverse faults. The fault plane used in this study for measuring site-source distances was interpreted from the Hayes (2010) and Holden et al. (2011) models and has a strike azimuth of 87.1 degrees and dips to the north at 73.5 degrees (Figures A-7 and A-8 in Appendix A). Based on the recommendations of Chiou et al. (2008), the westerly and easterly limits of the fault were truncated at the point where less than 50 cm of slip occurred. Since the point of <50 cm of slip differed for the Hayes (2010) and Holden et al. (2011) models, the midpoints between the two were used to define the longitudinal termini of the fault rupture plane.

Values of V_{s30} for the three ground motion recording stations from the Darfield (and Christchurch) events were estimated based on those reported by

Segou and Kalkan (2011), which were taken from topographically correlated values by Allen and Wald (2007). Evidence of liquefaction at the three stations was based on GEER (2011) and personal communication (Bradley 2012) (see Table 2-1).

2.2.2.6 Christchurch Earthquake

The M_w 6.3 reverse-oblique Christchurch, New Zealand earthquake occurred on 21 February 2011. Beavan et al. (2011) conducted inversion modeling to develop a fault rupture model with a dip of 67 degrees to the southeast, a width of 7 km and a length of 16 km (Figures A-9 and A-10 in Appendix A). Site-source distances were calculated based on this fault model. As with the Darfield earthquake, V_{s30} values at recording stations were taken from Segou and Kalkan (2011) and Allen and Wald (2007). As detailed in Table 2-1, evidence of liquefaction occurrence at 7 recording stations was based on liquefaction zone mapping by Cubrinovski and Taylor (2011) and personnel communication (Bradley 2012).

2.3 Acknowledgment

Chapter 2, in part, contain material from an article published in *Earthquake Spectra* (Gingery et al. 2014). This material was used with permission from the Earthquake Engineering Research Institute. The dissertation author was the primary author of this paper.

2.4 References

- Abrahamson, N.A., Atkinson, G., Boore, D., Bozorgina, Y., Campbell, K., Chiou, B., Idriss, I., Silva, W. and Youngs, R. (2008). "Comparison of the NGA ground motion relationships," *Earthquake Spectra*, 24(1), pp 45-66.
- Allen, T. I., and Wald, D. J. (2007). Topographic Slope as a Proxy for Seismic Site Conditions (V_{s30}) and Amplification around the Globe, U.S. Geological Survey, Open File Report 2007-1357, 69 pp.
- Atkinson, G. (2012). "Overview of subduction GMPE issues for Cascadia," presentation made at the *Pacific Northwest Workshop for the Update of the National Seismic Hazard Maps*, United States Geologic Survey.
- Baker, J.W. (2007). "Quantitative classification of near-fault ground motions using wavelet analysis," *Bulletin of the Seismological Society of America*, 97(5), pp. 1486-1501.
- Beavan, J., Fielding, E., Motagh, M., Samsonov, S., Donnelly, N. (2011). "Fault location and slip distribution of the 22 February 2011 Mw 6.2 Christchurch, New Zealand, earthquake from geodetic data," *Seismological Research Letters*, 82(6), pp 789-799.
- Boore, D.M. and Bommer, J.J. (2005). "Processing of strong-motion accelerograms: needs, options and consequences," *Soil Dynamics and Earthquake Engineering*, Vol. 25, pp 93-115.
- Bozorgnia, Y. (2012). "Plan for Next Generation Attenuation for Subduction Earthquakes, NGA-Sub," presentation made at the *Pacific Northwest Workshop for the Update of the National Seismic Hazard Maps*, United States Geologic Survey.
- Bradley, B. (2012). Personal communication to A. Elgamal, relayed by M. Cubrinovski.
- Bureau of Ports and Harbours, Ministry of Transport and Hokkaido Development Bureau (1968). "Damage to harbor structures by the 1968 Tokachi-Oki Earthquake," Hokkaido Development Agency (in Japanese).
- Chiou, B., Darragh, R., Gregor, N., Silva, W. (2008). "NGA project strong motion database," *Earthquake Spectra*, EERI, 24(1).
- Cox, B.R., Boulanger, R.W., Tokimatsu, K., Wood, C.M., Abe, A., Ashford, S., Donahue, J., Ishihara, K., Kayen, R., Katsumata, K., Kishida, T., Kokusho, T., Mason, H.B., Moss, R., Stewart, J.P., Tohyama, K. and Zekkos, D. (2013). "Liquefaction at Strong Motion Stations and in Urayasu City during

- the 2011 Tohoku-Oki Earthquake," *Earthquake Spectra*, 29(S1), pp S55-S80.
- Cubrinovski, M. and Taylor M. (2011). "Liquefaction map of Christchurch based on drive-through reconnaissance after the 22 February 2011 earthquake." University of Canterbury.
- Cubrinovski, M and Hughes, M. (2011). "Liquefaction map of Christchurch based on drive-through reconnaissance after the 13 June 2011 earthquake." University of Canterbury.
- Frankel, A.D., Carver, D.L., Williams, R.A. (2002). "Nonlinear and linear site response and basin effects in Seattle for the M 6.8 Nisqually, Washington Earthquake," *Bull. Of the Seis. Soc. of Amer.*, 92(6), PP 2090-2109.
- Ganev, T., Yamazaki, F., Ishizaki, H. and Kitazawa, M. (1998). "Response analysis of the Higashi-Kobe Bridge and surrounding soil in the 1995 Hyogoken-Nanbu earthquake," *Eq. Engr. and Soil Dyn.*, 27, pp 557-576.
- Geological Survey of Japan (GSJ) (2012). "Seamless Digital Geological Map of Japan (1:200,000)," http://riodb02.ibase.aist.go.jp/db084/index_en.html.
- Geo-Engineering Extreme Event Reconnaissance (GEER) (2011). "Geotechnical reconnaissance of the 2010 Darfield (New Zealand) earthquake, Version 1," M. Cubrinovski and R. Green editors.
- Hagiwara, R., Tamura, K., Honda, R., Usami, J. (1997). "Earthquake and earthquake ground motion, Report on the disaster caused by the 1995 Hyogoken Nanbu earthquake," *Journal of Research PWRI*, 33: pp 5-14.
- Hamada, M., Isoyama, R., Wakamatsu, K. (1995). "The 1995 Hyogoken-Nanbu (Kobe) Earthquake | Liquefaction, Ground Displacement and Soil Condition in Hanshin Area," Association for Development of Earthquake Prediction, Japan.
- Hayes, G. (2010). "Finite fault model, updated results of the September 3, 2010, Mw 7.0 Darfield, South Island New Zealand earthquake," http://earthquake.usgs.gov/earthquakes/eqinthenews/2010/us2010atbj/finite_fault.php.
- Holzer, T.L., Youd, T.L., Hanks, T.C. (1989). "Dynamics of liquefaction during the 1987 Superstition Hills, California, earthquake," *Science*, 244; 56-59.
- Holden, C., Beavan, J., Fry, B., Reyners, M., Ristau, J., Van Dissen, R. and Villamor, P. (2011). "Preliminary source model of the Mw 7.1 Darfield Earthquake from geological, geodetic and seismic data," *Prof. of the 9th*

Pac. Conf. on Eq. Engr., Building an Eq.-Resilient Society, Auckland, New Zealand.

- Iai, S., Morita, T., Kameoka, T., Matsunaga, Y., Abiko, K. (1995). "Response of a dense sand deposit during 1993 Kushiro-Oki earthquake," *Soils and Foundations*, 35(1), pp 115-131.
- Ishihara, K. (1985). "Stability of natural deposits during earthquakes," *Proc., 11th Int. Conf. on Soil Mech. and Found. Engr.*, University of Missouri, St. Louis, Vol. 3, pp. 1129-1143.
- Iwasaki, Y. (2009). "Vertical array records during 1995 Hyogoken-Nambu earthquake by Kobe city, KEPCO and other organizations," *Eq. Geotech. Case Hist. for Performance-Based Design*, Kokusho (ed).
- Kayen, R., Brandenberg, S.J., Collins, B.D., Dickens, S., Ashford, S., Kawamata, Y., Tanaka, Y., Koumoto, H., Abrahamson, N., Cluff, L. and Tokimatsu, K. (2009). "Geoengineering and seismological aspects of the Niigata-Ken Chuetsu-Oki earthquake of 16 July 2007," *Earthquake Spectra*, EERI, 25(4), pp. 777-802.
- Kostadinov, M.V. and Yamazaki, F. (2001). "Detection of soil liquefaction from strong motion records," *Earthquake Engr. and Struc. Dyn*, Elsevier, Vo. 30, pp 173-193.
- Kostadinov, M.V. and Towhata, I. (2002). "Assessment of liquefaction-inducing peak ground velocity and frequency of horizontal ground shaking at onset of phenomenon," *Soil Dyn. and Eq. Engr*, Vol. 22, pp 309-322.
- Pacific Earthquake Engineering Research Center (PEER) (2012). NGA Flatfile Used for Development of NGA Models, http://peer.berkeley.edu/ngawest/nga_flatfiles.html.
- Power, M., Chiou, B., Abrahamson, N., Bozorgina, Y., Shantz, T. and Roblee, C. (2008). "An overview of the NGA project," *Earthquake Spectra*, EERI, 24(1), pp. 3-21.
- Sato, T. (1985). "Rupture characteristics of the 1983 Nihokai-Chubu (Japan Sea) earthquake as inferred from strong motion accelerograms," *J. Phys. Earth*, 33, pp 527-557.
- Sato, Y., Ichii, K., Hoshino, Y., Sato, Y., Miyata, M., Morita, T., Iai, S. (1998). "Strong-motion earthquake records on the 1995 Hyogoken Nambu earthquake in port areas," *Technical Note of the Port and Harbour Research Institute, Ministry of Transport, Japan*, Vol. 907.

- Segou, M. and Kalkan, E. (2011). "Ground motion attenuation during M 7.1 Darfield and M 6.2 Christchurch, New Zealand earthquakes and performance of global predictive models," *Seis. Res. Let.*, 85(6), pp 866-874.
- Shahi, S. and Baker, J.W. (2012). Pulse Classifications from NGA West2 database, data summary at http://www.stanford.edu/~bakerjw/pulse_classification_v2/Pulse-like-records.html.
- Shibata, T, Oka, F., Ozawa, Y. (1996) "Characteristics of ground deformation due to liquefaction," *Soils and Foundations*. - *Special Issue*, pp 65-79.
- Somerville, P., Smith, N., Graves, R., Abrahamson, N.A. (1997). "Modification of empirical strong ground motion attenuation relationships to include the amplitude and duration effects of rupture directivity," *Seismological Research Letters*, 68(1), pp. 199-222.
- United States Geological Survey (USGS) (2013). Significant earthquakes of the world webpage http://earthquake.usgs.gov/earthquakes/eqarchives/significant/sig_1983.php
- Yanagisawa, E., Ishihara, K., Tobita, Y., Nakamura, S. (1984). "On the vibration characteristics of front dike of Hachirogata reclamation in relation to the Nihonkai-Chibu earthquake," *Tsuchi-to Kiso*, 32(9), pp 41-44 (in Japanese).
- Youd, T.L., Steidl, J.H. and Steller, R.A. (2007). "Instrumentation of the Wildlife Liquefaction Array," 4th *Int. Conf. on Eq. Geotech. Engr.*, Thessaloniki, Greece.
- Zeghal, M. and Elgamal, A. (1994). "Analysis of site liquefaction using earthquake records," *J. Geotech. and Geoenvr. Engr.*, ASCE, 120(6), pp. 996-1017.

Chapter 3 Temporal and Frequency Analysis of Ground Motions

In this chapter, ground motions in the compiled liquefaction database are analyzed to evaluate their time-varying frequency content, the apparent timing of liquefaction triggering, and the temporal occurrence of spectral accelerations at various periods. The comparison of the temporal occurrence of spectral accelerations relative to the initiation of liquefaction provides insight into period ranges potentially affected by liquefaction. For instance, if long period spectral accelerations consistently occur after liquefaction is initiated, then this provides evidence that liquefaction controls the long period site response. The data in this chapter is intended to compliment and corroborate the spectral acceleration amplification results presented Chapters 4 and 6.

3.1 Timing of Liquefaction Initiation

Several methods have been developed to detect liquefaction in ground motion recordings (Suzuki et al. 1998, Miyajima et al. 1998, Ozaki 1999, Kostadinov and Yamazaki 2001). Typically, these methods employ some measure of time-varying frequency content along with criteria for a frequency threshold below which liquefaction is judged to have occurred. The development of these methods has been focused on real-time applications for use in early warning systems to identify areas of potential liquefaction damage for emergency response purposes.

The method of Kostadinov and Yamazaki (2001) involves calculation of the mean instantaneous frequency (*MIF*), which is the measure of the mean frequency computed over a short time window (2.56 seconds):

$$MIF(t) = \frac{\int f P(t, f) df}{\int P(t, f) df} \quad (3-1)$$

where $P(t, f)$ is the Fourier power spectrum calculated for the time window at time t , and f is frequency. The *MIF* is calculated for the two horizontal components (MIF_H) and the vertical component (MIF_V) of acceleration at each time increment between the first and last exceedance of horizontal acceleration of 40 cm/s^2 . Kostadinov and Yamazaki's (2001) criteria for judging the occurrence of liquefaction are:

1. If $PGV \geq 10 \text{ cm/s}$ and $MIF_H \leq 2/3 \text{ Hz}$ and $MIF_V \geq 3 \text{ Hz}$ for at least 0.1 seconds, 'liquefaction' is detected.
2. If the first criterion is not met, $PGV \geq 10 \text{ cm/s}$, and both $MIF_H \leq 1 \text{ Hz}$ and $MIF_V \geq 3 \text{ Hz}$ for at least 0.1 seconds, 'liquefaction suspicious' is detected.
3. If the first two criteria are not met, 'no liquefaction' is detected.

Kostadinov and Yamazaki (2001) tested their method on a collection of 17 ground motions from sites where liquefaction was confirmed (11 sites, 9 of which were free field and 2 were within structures) or suspected (6 sites, 5 of which were free field and 1 were within structures), and 66 recordings from non-liquefied sites. All of these ground motions had peak ground velocities of at least

15 cm/s and peak ground accelerations of at least 150 cm/s^2 . Using their methodology, 91 percent of the 66 non-liquefied site ground motions were correctly identified as such. The remaining 8.3 and 1.7 percent were identified as *liquefaction suspicious* and *liquefaction*, respectively. Of the 9 liquefied free field sites, the method identified 8 of them as *liquefaction* and 1 as *liquefaction suspicious*. Of the 5 free field liquefaction suspicious sites, 2 were identified as *liquefaction* and 3 were identified as *liquefaction suspicious*. Kostadinov and Yamazaki (2001) showed that predictions using their method were more reliable than those of Suzuki et al. (1998), Myajima et al. (1998), and Ozaki (1999).

In this study, the Kostadinov and Yamazaki (2001) procedure was used to estimate the time at which liquefaction was initiated. This is accomplished by assuming that the instance of liquefaction initiation is the first time at which either the *liquefaction* (t_{liq}) or *liquefaction suspicious* (t_{sus}) criteria are met. The validity of this assumption was evaluated by examining the Wildlife Liquefaction Array (WLA) recording of the Superstition Hills, California earthquake of 1987 and the Port Island downhole array recording from the Hyogoken-Nanbu Kobe, Japan earthquake of 1995.

Zeghal and Elgamal (1994) analyzed the Wildlife Liquefaction Array (WLA) recordings, which included acceleration-time histories near the top and bottom of a liquefiable soil layer, and pore-water pressure transducer time histories. They used acceleration- and displacement-time histories data to estimate the average shear stress versus shear strain over the interval of the liquefiable layer. They identified four stages in the time history (Figure 3-1). In Stage 1 from 0 to 13.7

seconds, amplitudes were lower and little to no stiffness degradation occurred. Stage 2 from 13.7 to 20.6 seconds was an interval where pore-water pressure increased rapidly and average shear wave velocity decreased rapidly. Stage 3, which extended from 20.6 to 40.0 seconds, was identified as having a longer period and occasional spikes in the acceleration record. The excess pore-water pressure ratio at the end of Stage 2 is recorded as approximately 0.5. In this current study, values of

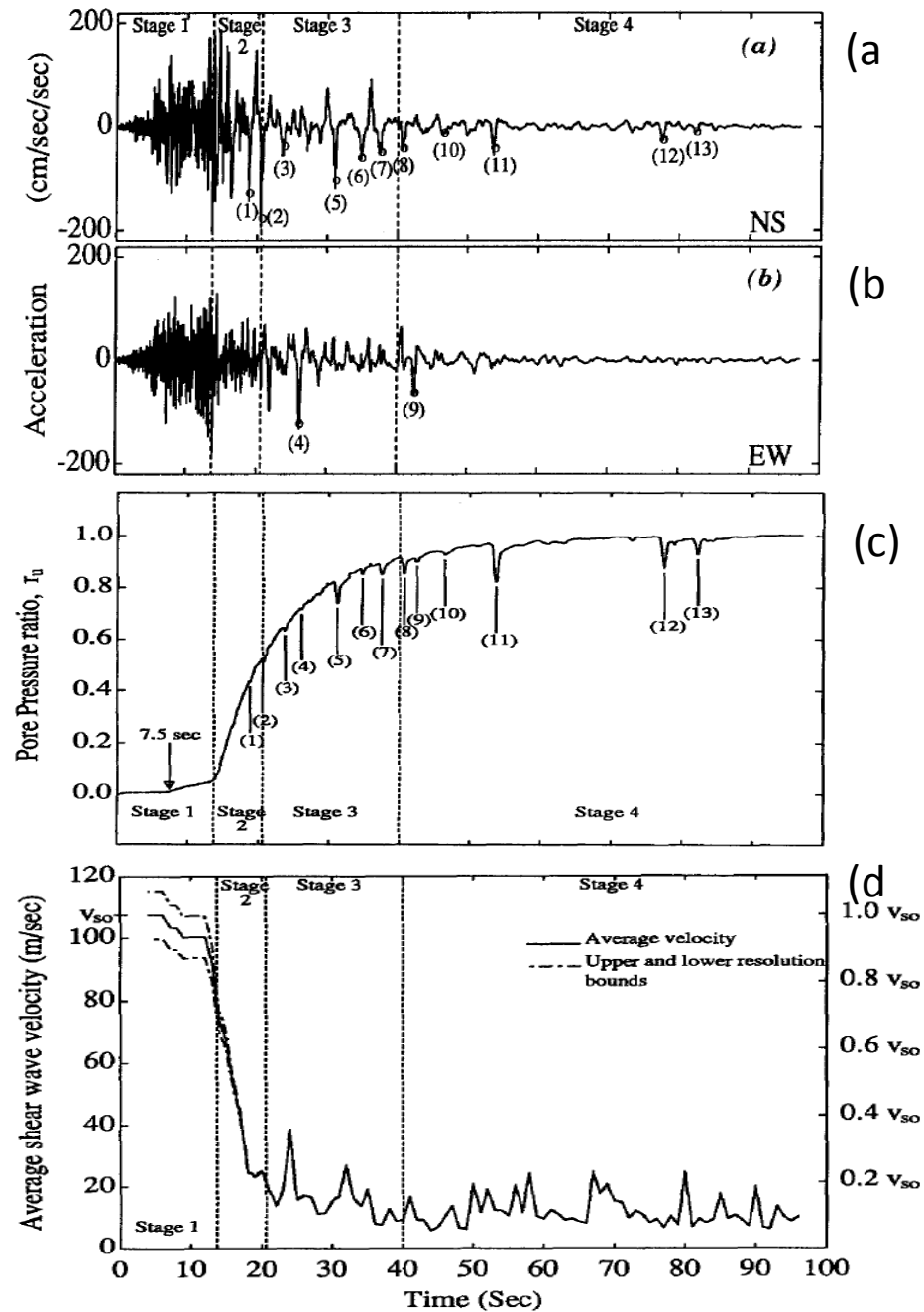


Figure 3-1: From Zeghal and Elgamal (1994), WLA Superstition Hills earthquake: (a) surface time history; (b) time history at depth of 7.5 m; (c) excess pore pressure ratio at depth of 2.9 m; and (d) average V_s over time.

$t_{sus} = 19.4$ seconds and $t_{liq} = 21.9$ seconds were calculated for the WLA surface recording (Figure 2-3). These times correspond to an excess pore-water

pressure ratio measurement of 0.5 to 0.6, approximately to the end of Zeghal and Elgamal's Stage 2 (based on the employed excess pore-pressure record). Zeghal and Elgamal (1994) showed that the first shear stress-strain loop exhibiting cyclic mobility-type behavior occurs in the interval of 18.65 to 20.38 seconds, which corresponds closely with t_{sus} and t_{liq} . The first occurrences of cyclic mobility dilation-related spikes in the acceleration time history also coincide with the t_{sus} to t_{liq} interval. Based on these observations, the t_{sus} to t_{liq} appears to provide a reasonable estimate of the onset of liquefaction for the WLA recordings of the Superstition Hills earthquake.

Elgamal et al. (1996) studied the down-hole accelerometer array data from Port Island (PI) in the 1995 Hyogoken-Nanbu earthquake in Kobe, Japan. They identified three stages in the acceleration-time history and stress-strain response (Figure 3-2). In Stage 1 the amplitude was relatively small and little to no stiffness degradation occurred. Stronger shaking occurred in Stage 2 and stiffness decreased dramatically in the liquefiable soils. In Stage 3 no further stiffness degradation occurred. While pore-water pressure recordings were not available, Elgamal et al. (1996) performed numerical modeling of the response that showed pore-water pressure increased dramatically during Stage 2. From these results, it can be inferred that liquefaction was triggered at about the end of Stage 2, which occurred at a time of 7 seconds. This is in very good agreement with the calculated values of $t_{liq} = 7.2$ seconds and $t_{sus} = 5.7$ seconds (Figure 2-9).

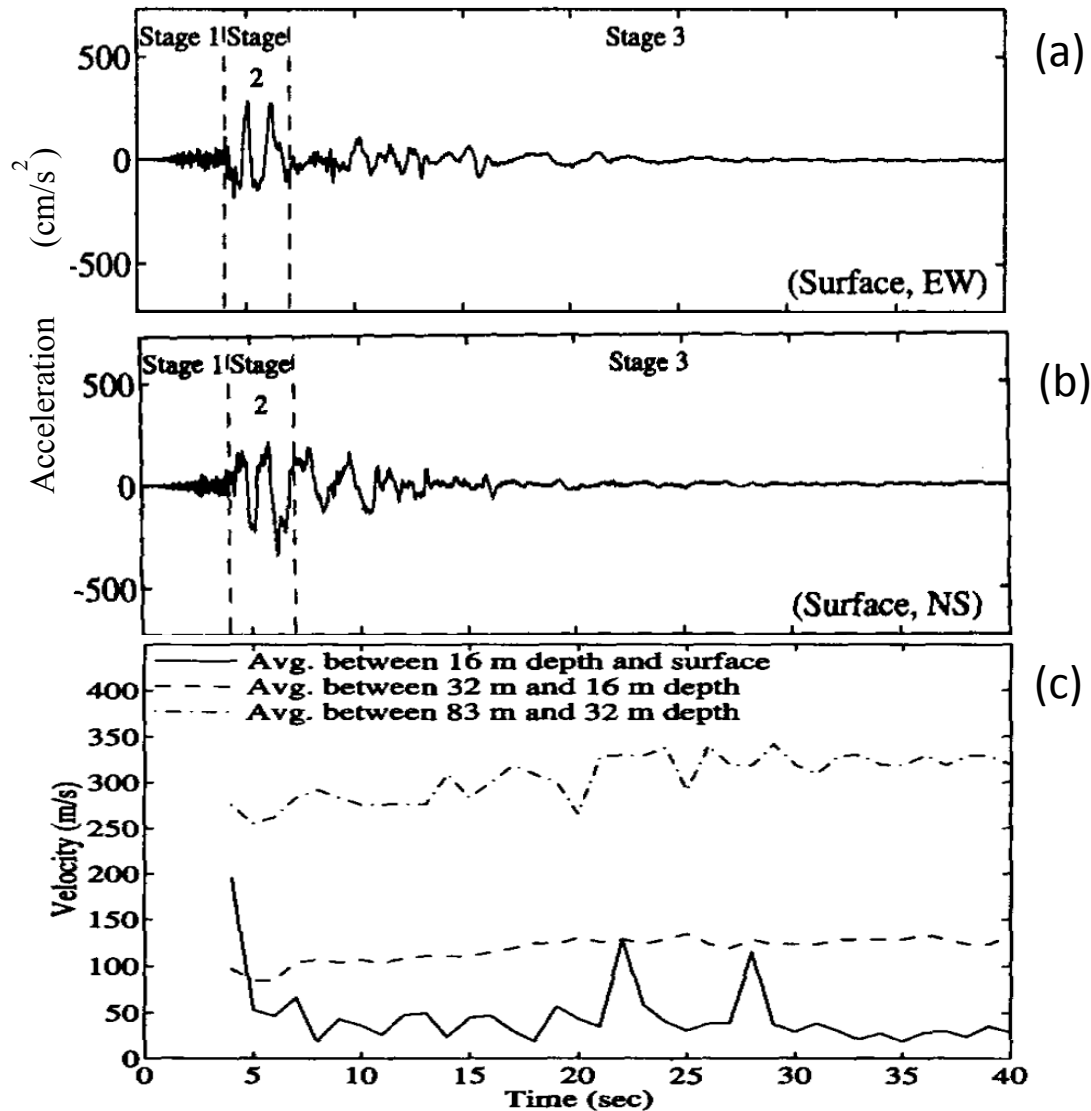


Figure 3-2: From Elgamal et al. (1996), Port Island recording of Kobe earthquake: surface time history in (a) EW and (b) NS directions, and (c) average V_s over time.

Unfortunately, additional sites with downhole and pore-water pressure recordings during liquefaction events beyond WLA and PI are not currently available to further validate the use of t_{sus} and/or t_{liq} as predictors of liquefaction onset. But based on the good agreement with the WLA and PI data, t_{sus} and t_{liq} appear to be reasonable and useful indices for liquefaction onset. In the

subsequent section, t_{sus} and t_{liq} are used to compare the relative timing of spectral accelerations and liquefaction onset.

3.2 Timing of Spectral Accelerations

This discussion starts with some clarifications regarding the types of response spectra used in this study. The *pseudo-absolute acceleration response spectrum* (S_a) of a damped single degree of freedom (SDOF) oscillator and the *pseudo-relative velocity response spectrum* (or more simply, the *pseudo-velocity response spectrum*, S_v), are related to the relative displacement (S_d) and period (T) by (Chopra 2001): $(2\pi/T)S_d = S_v = (2\pi/T)^{-1}S_a$. The peak *true relative velocity spectrum* can be calculated by differentiating the relative displacement time histories for a series of single-degree-of-freedom (SDOF) oscillators with varying periods, and taking the peak values for each period. The *true absolute acceleration* of the oscillator can be calculated by differentiating the true relative velocity, adding it to the base acceleration and taking the peak values for a range of SDOF periods.

Chopra (2001) showed that S_a and the true absolute acceleration response spectra are nearly identical for low values of damping and periods less than about 10 seconds. The use of S_a as a computationally convenient approximation of the true absolute acceleration response spectrum has been widely adopted in practice. In particular, most GMPEs use S_a including the NGA GMPEs that are used later in this study (Power et al. 2008). For these reasons,

this study uses S_a , and any reference made to “acceleration response spectrum” or “response spectrum” refers to S_a unless otherwise noted.

In this study, the time at which the peak oscillator acceleration occurs for a given period is referred to as the *time of spectral acceleration*, $t_{Sa}(T)$. While S_a is used to represent spectra to maintain consistency with standard practice, the true absolute acceleration time history is used to calculate the $t_{Sa}(T)$ because the time at which S_d occurs may not coincide with the time of peak absolute acceleration (Chopra 2001).

By comparing $t_{Sa}(T)$ with t_{liq} and t_{sus} , inferences can be made regarding spectral acceleration period ranges affected by liquefaction. Before these comparisons are made, however, some clarifications are useful. There is a phase lag between the time of the input impulse causing a spectral acceleration and the occurrence of the spectral acceleration. When examining the timing of spectral acceleration relative to t_{liq} or t_{sus} , the time of the input excitation causing the spectral acceleration is of more interest than the time at which the spectral acceleration occurs in the SDOF oscillator. It is therefore preferable to correct $t_{Sa}(T)$ so that the oscillator response phase lag is subtracted, and thus $t_{Sa}(T)$ is representative of the input ground motion causing the spectral acceleration.

$$t_{Sa}(T) = t_{Sa}(T)^* - t_{lag}^* \quad (3-2)$$

where $t_{Sa}(T)^*$ is the uncorrected time of S_a and t_{lag}^* is the value of t_{lag} chosen for correction purposes.

The selection of the value of t_{lag}^* can be guided by considering the solution of the time-varying displacement of a SDOF oscillator in response to a harmonic base acceleration is (Chopra 2001):

$$u(t) = (u_{st})_0 R_d \sin\left(\frac{2\pi}{T} - \phi\right) \quad (3-3)$$

where the phase lag is

$$\phi = \tan^{-1} \left[\frac{2\zeta (T/T_{GM})}{1 - (T/T_{GM})^2} \right] \quad (3-4)$$

and $(u_{st})_0$ is the displacement if the maximum value of the harmonic excitation were applied statically; R_d is the dynamic amplification which is a function of T/T_{GM} and ζ ; T_{GM} is the period of the harmonic base motion (ground motion); and ζ is the damping ratio. The phase lag is can be expressed in terms of the oscillator period as

$$\frac{\phi}{2\pi} = \frac{t_{lag}}{T} \quad (3-5)$$

where t_{lag} is the phase lag expressed in seconds. Substituting Eqn. (3-5) into Eqn. (3-4) and solving for t_{lag} yields

$$t_{lag} = \frac{T}{2\pi} \tan^{-1} \left[\frac{2\zeta (T/T_{GM})}{1 - (T/T_{GM})^2} \right] \quad (3-6)$$

Earthquake time histories are not harmonic and the ground motion causing S_a at a particular period may be an aggregation of multiple frequencies.

Since T_{GM} and the ratio T/T_{GM} are not known, Eqn. (3-6) cannot be used to calculate an exact value of t_{lag}^* . Therefore, it is useful to examine results from Eqn. (3-6) over a range of T/T_{GM} as in Figure 3-3 (a). For the case of $T/T_{GM} = 1$, $t_{lag} = T/4$ and this value is a median between the extreme values of t_{lag} at $T/T_{GM} = 0$ and $T/T_{GM} = \infty$. The error resulting from using $t_{lag}^* = T/4$ is computed as $t_{lag,error} = t_{lag}^* - t_{lag}$ and is presented graphically in Figure 3-3 (b). It can be seen that use of $t_{lag}^* = T/4$ provides a compromise in terms of potential error resulting for cases where $T/T_{GM} \neq 1$. In many cases, it is expected that local peaks in response spectra would likely occur at or near $T/T_{GM} = 1$, where $t_{lag,error} = 0$. At periods less than 1 second the absolute value of error is less than within ± 0.25 seconds, which is negligible when considering the uncertainty in t_{sus} and t_{liq} predictions. At periods of 4 and 10 seconds the absolute values of the maximum error is 1 and 2.5 seconds, respectively. These results suggest that potential for correction error associated with $t_{lag}^* = T/4$ is insignificant for periods of about 2 seconds and less, and that some caution should be exercised when interpreting corrections at periods greater than 2 or 4 seconds.

Figures 2-2 through 2-20 present acceleration-, velocity-, displacement-, and MIF-time histories, acceleration response spectra, and time of liquefaction initiation data for the 19 ground motion recordings in the database. The time series plots in these figures show t_{liq} and t_{sus} as dashed and dotted vertical lines, respectively. The upper right chart presents $t_{sa}(T)$ for the two horizontal and one

vertical component and the t_{liq} and t_{sus} , which can be used to visualize the relative timing of spectral accelerations and liquefaction initiation. Where the $t_{Sa}(T)$ lines are above the horizontal t_{liq} or t_{sus} lines, the spectral acceleration occurred after the onset of liquefaction, and visa versa.

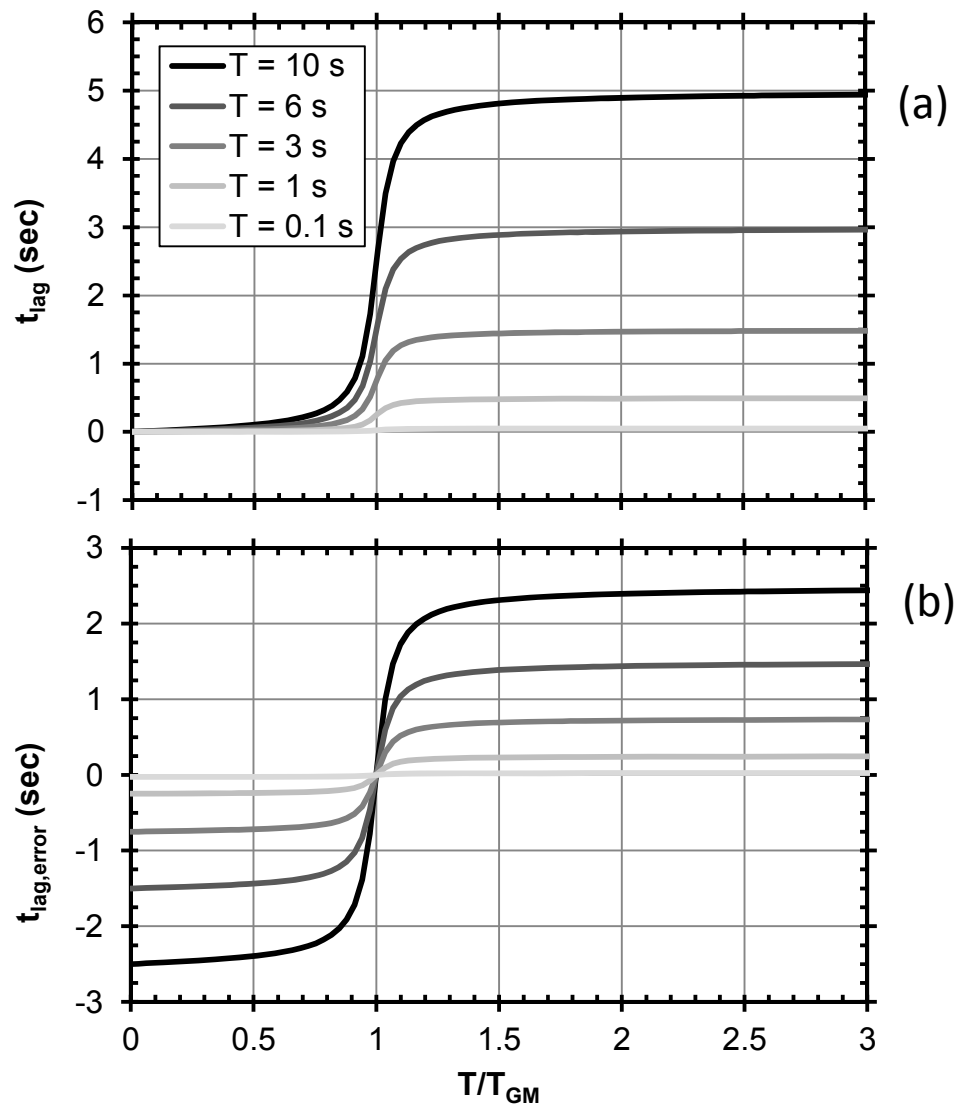


Figure 3-3: (a) Phase lag time (t_{lag}) and (b) phase lag time error when assuming $t_{lag} = T/4$.

As explained earlier, in building codes such as the 2012 IBC and ASCE 7-10 there is an exception to designation of Site Class F and the corresponding requirement to perform site-specific ground response analysis if the fundamental period of the building under consideration is 0.5 seconds or less. This follows the reasoning suggested by Youd and Carter (2005) that the softening involved in the response of liquefied sites does not lead to amplification of shorter periods spectral accelerations. If this is true, one would expect to find $t_{Sa}(T > 0.5s)$ to occur with greater frequency temporally anterior to liquefaction triggering rather than posterior. Conversely, if $t_{Sa}(T > 0.5s)$ is amplified by liquefaction, one would expect to find $t_{Sa}(T > 0.5s)$ values occurring more frequently posterior to liquefaction triggering. To examine this, the timing of spectral acceleration relative to apparent liquefaction triggering is calculated as:

$$t_{Sa}(T) - t_{liq} \quad (3-7)$$

and

$$t_{Sa}(T) - t_{sus} \quad (3-8)$$

Figure 3-4 presents $t_{Sa}(T) - t_{liq}$ and $t_{Sa}(T) - t_{sus}$ versus period for the each of the two horizontal components of the 19 liquefaction time histories and the mean values. Figure 3-4 (a) shows that the mean values of $t_{Sa}(T) - t_{sus}$ occur at about -2 to -2.5 seconds for periods less than about 0.5 seconds. Values of $t_{Sa}(T) - t_{sus}$ increase above a period of about 0.5 seconds to maximum of +9.4

seconds at a period of 5 seconds. Figure 3-4 (b) shows the mean values of $t_{Sa}(T) - t_{liq}$ range from about -4.7 to -5.5 seconds below a period of about 0.5 seconds. Higher than a period of 0.5 seconds values of $t_{Sa}(T) - t_{sus}$ gradually increase to a peak value of +6 seconds at a period of 4.8 seconds. The mean values of $t_{Sa}(T) - t_{sus}$ and of $t_{Sa}(T) - t_{liq}$ crossover the zero value at periods of 1.2 and 2.4 seconds, respectively.

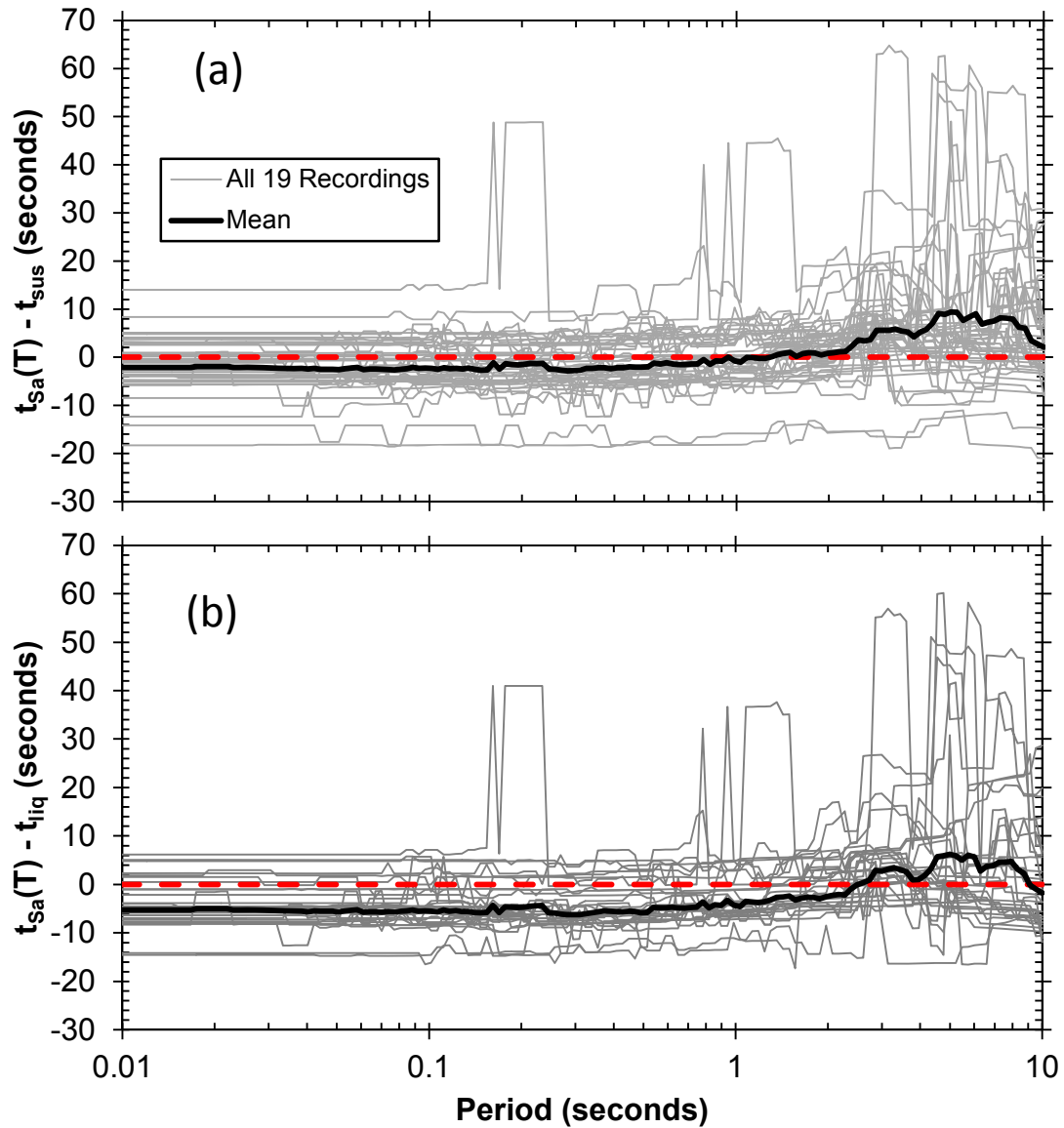


Figure 3-4: Time of spectral acceleration relative to t_{liq} (a) and t_{liq} (b) for 19 liquefaction ground motions (2 components each).

In Figure 3-5 the spectral acceleration and liquefaction timing data are presented as cumulative density functions (CDF) for five periods. These figures show that for periods of 0.5 seconds and less, about 70% of $t_{sa}(T)$ values occur before t_{sus} , and 85% of $t_{sa}(T)$ occur before t_{liq} . The data show that the CDF generally shifts to the right for increasing values of T greater than about 0.5

seconds, which indicates an increasing occurrence of spectral accelerations that occur after liquefaction onset at longer periods. For $T = 4$ seconds, about 78% of $t_{Sa}(T)$ values occur after t_{sus} , and 53% of $t_{Sa}(T)$ occur after t_{liq} .

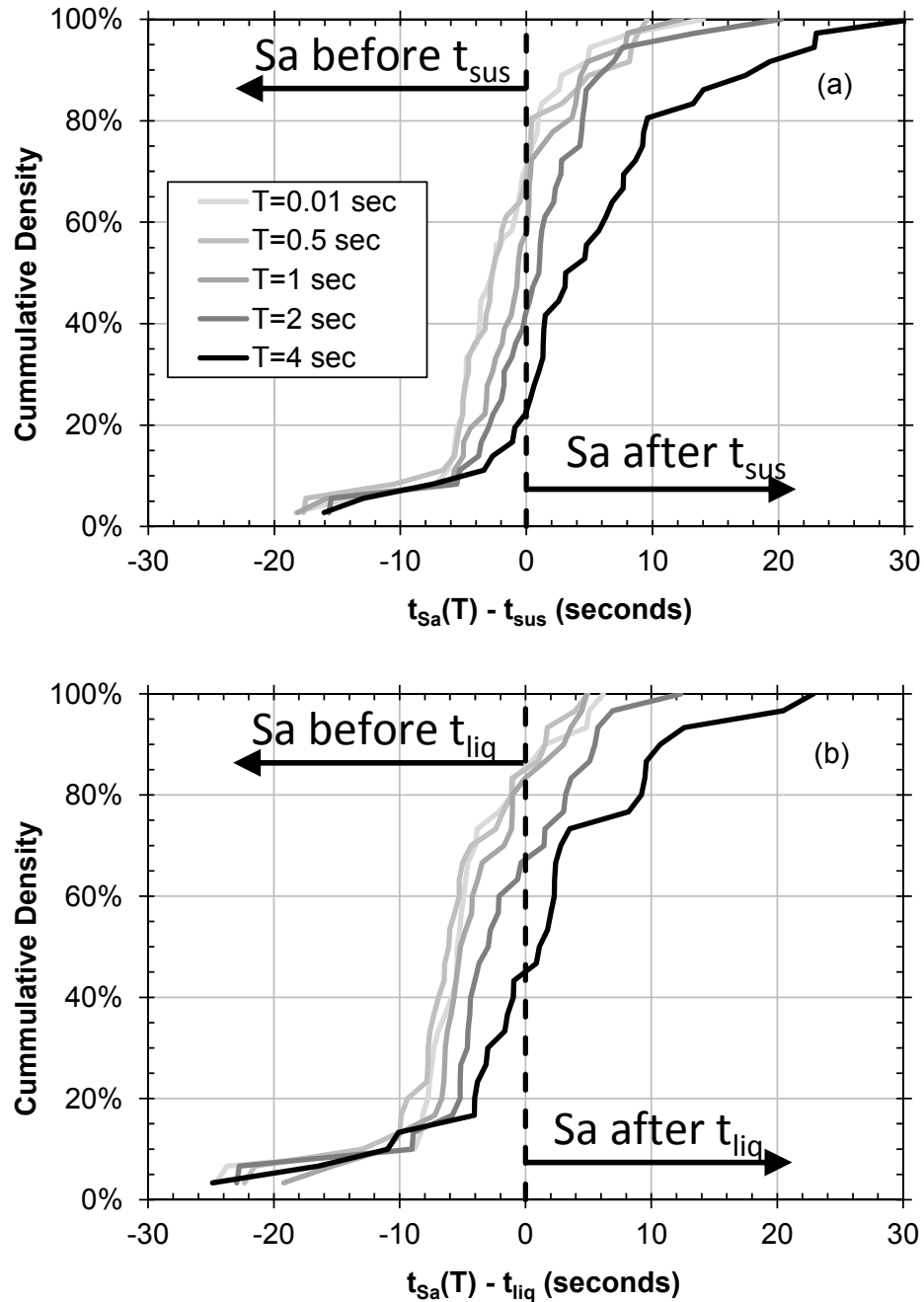


Figure 3-5: Cumulative density functions of $t_{Sa}(T)-t_{liq}$ (a) and $t_{Sa}(T)-t_{liq}$ (b) for the 19, two-component horizontal liquefied site ground motions.

3.3 Discussion and Conclusions

This section presents a method for comparing the timing of spectral accelerations to the apparent timing of liquefaction onset. This comparison sheds light on the period ranges potentially affected by liquefaction. Figures 3-4 and 3-5 provide some evidence that spectral accelerations at longer periods occur more frequently after liquefaction onset compared to shorter periods. This is consistent with conventional wisdom that liquefaction tends to soften a site and amplify longer period ranges. The data in Figure 3-5 show that spectral accelerations at short periods ($T = 0.01$, i.e., the *PGA*) occur more frequently, but not entirely, prior to liquefaction onset. That short period spectral accelerations also occur after liquefaction onset may be an indication that liquefaction-induced dilation acceleration spikes dominate the *PGA* in some cases. This calls into question the validity of the assumption in the building codes that S_a at periods less than 0.5 seconds are not amplified. The effect of liquefaction on short period spectral accelerations and *PGA* is further examined in Chapter 4.

3.4 References

- Chopra, A.K. (2001), *Dynamics of Structures, Theory and Application to Earthquake Engineering*, 2nd Edition, Pearson Education.
- Elgamal, A., Zeghal, M. and Parra, E. (1996). "Liquefaction of reclaimed island in Kobe, Japan," *J. of Geotech. Engr.*, ASCE, 122(1), pp 39-49.
- Kostadinov, M.V. and Yamazaki, F. (2001). "Detection of soil liquefaction from strong motion records," *Earthquake Engr. and Struc. Dyn.*, Elsevier, Vo. 30, pp 173-193.
- Kostadinov, M.V. and Towhata, I. (2002). "Assessment of liquefaction-inducing peak ground velocity and frequency of horizontal ground shaking at onset of phenomenon," *Soil Dyn. and Eq. Engr*, Vol. 22, pp 309-322.

- Miyajima, M., Kitaura, M., Nozu, S. (1998). "Detective method of liquefaction using strong ground motion records," *Proceedings of the 3rd China-Japan-US Trilateral Symposium on Lifeline Earthquake Engineering*, Kunming, China, August 1998; pp 133-140.
- Ozaki, R. (1999). "Study on real-time earthquake mitigation | liquefaction monitoring and earthquake countermeasures," PhD Thesis, Kobe University (in Japanese).
- Power, M., Chiou, B., Abrahamson, N., Bozorgina, Y., Shantz, T. and Roblee, C. (2008). "An overview of the NGA project," *Earthquake Spectra*, EERI, 24(1), pp. 3-21.
- Suzuki, T., Shimizu, Y., Nakayama, W. (1998). "Characteristics of strong motion records at the liquefied sites and judgment for liquefaction," *11th European Conference on Earthquake Engineering*, Paris, France, 6-11 September, Balkema: Rotterdam.
- Youd, T.L. and Carter, B.L. (2005). "Influence of soil softening and liquefaction on spectral acceleration," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 131(7).
- Zeghal, M. and Elgamal, A. (1994). "Analysis of site liquefaction using earthquake records," *J. Geotech. and Geoenvr. Engr.*, ASCE, 120(6), pp. 996-1017.

Chapter 4 Empirical Liquefaction Amplification Model

4.1 Evaluation of Recorded Motions at Liquefaction Sites

4.1.1 NGA Relationships

GMPEs are used in research and practice to make estimates of acceleration response spectra. The NGA West GMPEs (Power et al. 2008) have found wide use in engineering practice in Western North America, and have also been used in other active tectonic regions that are subject to shallow crustal earthquakes. They have been adopted in the 2008 United States National Seismic Hazard Maps (Petersen et al. 2008), which are the basis for the design ground motions in the 2009 IBC and 2010 CBC. Caltrans (2012) uses the NGA GMPEs as the basis for their design ground.

As discussed in Chapter 2, only 4 of the 3,551 ground motion recordings in the 2008 NGA database are known or suspected to have been recorded at sites with liquefaction: Wildlife Liquefaction Array in the 1987 Superstition Hills earthquake; Treasure Island in the 1989 Loma Prieta earthquake; and Amagasaki-G and Port Island in the 1995 Hyogoken-Nambu Kobe earthquake. As such, it is clear that the 2008 NGA relationships are not suitable or intended for prediction of ground motions at liquefaction sites. However, NGA ground motions predictions can be compared to actual ground motion recordings for liquefaction sites, and differences in trend can be examined and quantified. In this study, the four NGA GMPEs developed for soil conditions are used: Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and

Bozorgnia (2008) and Chiou and Youngs (2008), which are referred to herein as AS08, BA08, CB08 and CY08, respectively. Each of these GMPEs provides an estimate of median 5% damped rotation-independent geometric mean ($Sa_{GMRot150}$) pseudo spectral accelerations for a range of response periods from 0.01 to 10 seconds. The median estimates are made based on the fault style, earthquake magnitude, site-source distance, site conditions and other minor parameters. Abrahamson et al. (2008) provide a summary and comparison of the various parameters used by the NGA West GMPEs. Additional detail can be found in AS08, BA08, CB08, and CY08.

4.1.2 Directivity

Fault rupture propagation occurs at a velocity close to that of shear waves. Where a significant portion of the fault rupture occurs toward a site and where the site is situated close to the fault, seismic energy tends to arrive over a short duration. This *forward directivity* effect is typically expressed in the velocity time history as a large amplitude pulse occurring early in the time history and as amplification of spectral accelerations at periods greater than about 0.6 seconds (e.g., Somerville et al. 1997; Bray and Rodriguez-Marek 2004; Baker 2007; Spudich and Chiou 2008; Spudich et al. 2013; Hayden et al. 2014). The radiation pattern is such that the forward directivity velocity pulse generally occurs in the fault normal (FN) direction, but Baker (2007) and Hayden et al. (2014) showed that velocity pulses can be significant in orientations other than the geographical FN.

Forward directivity can affect the same period range as liquefaction-related amplification, and thus it was treated carefully in this study. In their seminal paper, Somerville et al. (1997) analyzed 65 ground motions with forward directivity and regressed relationships to modify GMPEs to account for directivity effects. Two types of modifications are possible: one adjustment accounts for the fact that the average of the two horizontal components of acceleration response spectra at periods greater than 0.6 seconds is larger for forward directivity ground motions; and another adjustment accounts for the fact that FN and FP components are typically higher and lower, respectively, than the average spectrum. Bayless and Somerville (Spudich et al. 2013) updated and enhanced the 1997 model using an expanded ground motion dataset. The updated model can be used to compute average horizontal component, fault normal and fault parallel directivity effects. Three alternate directivity models were also presented in Spudich et al. (2013). For the current study, the model of Bayless and Somerville was adopted because of its simple form and because it has been shown to be effective in reducing long period intra-event standard deviation for the 2008 NGA GMPEs (Spudich et al. 2013). The equations of the 2013 Bayless and Somerville model are:

$$\ln(Sa_{dir}) = \ln(Sa_{nodir}) + f_D \quad (4-1)$$

where Sa_{nodir} is a GMPE prediction of spectra acceleration without directivity, Sa_{dir} is an estimation of spectral acceleration with directivity, and f_D represents the directivity effect and has the form:

$$f_D = (C_0 + C_1 f_{geom}) T_{CD} T_{Mw} T_{Az} \quad (4-2)$$

where C_0 and C_1 are period-dependent regression coefficients. For strike slip faults the geometric directivity predictor is:

$$f_{geom} = \ln(s)(0.5 \cos(2\theta) + 0.5) \quad (4-3)$$

where s is the length of striking fault rupturing toward the site, θ is the azimuth angle between the fault plane and the ray path from the epicenter to the site, the distance taper is:

$$\begin{aligned} T_{CD} &= 1 && \text{for } R_{rup} / L < 0.5 \\ T_{CD} &= 1 - (R_{rup} / L - 0.5) / 0.5 && \text{for } 0.5 < R_{rup} / L < 1.0 \\ T_{CD} &= 0 && \text{for } R_{rup} / L > 1.0 \end{aligned} \quad (4-4)$$

the magnitude taper is:

$$\begin{aligned} T_{Mw} &= 1 && \text{for } Mw > 6.5 \\ T_{Mw} &= 1 - (6.5 - Mw) / 1.5 && \text{for } 5.0 < Mw < 6.5 \\ T_{Mw} &= 0 && \text{for } Mw < 5.0 \end{aligned} \quad (4-5)$$

and the azimuth taper is:

$$T_{Az} = \sin^2(|Az|) \quad (4-6)$$

For dip slip faults the geometric directivity predictor is:

$$f_{geom} = \ln(d) \cos(R_x / W) \quad (4-7)$$

where d is the length of the dipping fault rupturing toward the site, W is the fault width, and R_x is the horizontal distance in km from the top edge of the rupture.

The distance taper is:

$$\begin{aligned} T_{CD} &= 1 && \text{for } R_{rup} / W < 1.5 \\ T_{CD} &= 1 - (R_{rup} / W - 1.5) / 0.5 && \text{for } 1.5 < R_{rup} / W < 2.0 \\ T_{CD} &= 0 && \text{for } R_{rup} / W > 2.0 \end{aligned} \quad (4-8)$$

Where R_{rup} is the closest distance from the site to the rupture plane, the magnitude taper is:

$$\begin{aligned} T_{Mw} &= 1 && \text{for } Mw > 6.5 \\ T_{Mw} &= 1 - (6.5 - Mw) / 1.5 && \text{for } 5.0 < Mw < 6.5 \\ T_{Mw} &= 0 && \text{for } Mw < 5.0 \end{aligned} \quad (4-9)$$

and the azimuth taper is:

$$T_{Az} = \sin^2(|Az|) \quad (4-10)$$

The quantity $\exp(f_D)$ is a directivity amplification factor (AF_{dir}) for Sa_{nodir} . Examples of amplification factors produced using the Eqns. (4-1 through (4-10) are presented in Figure 4-1 (i.e., $Sa_{dir} = AF_{dir} Sa_{nodir}$). The 2013 Bayless and Somerville average component directivity modifications were applied in the median NGA ground motion estimations made in this study. The parameters

used in the directivity calculations are presented in Table 2-3 and the resulting AF_{dir} values are presented in Figure 4-1

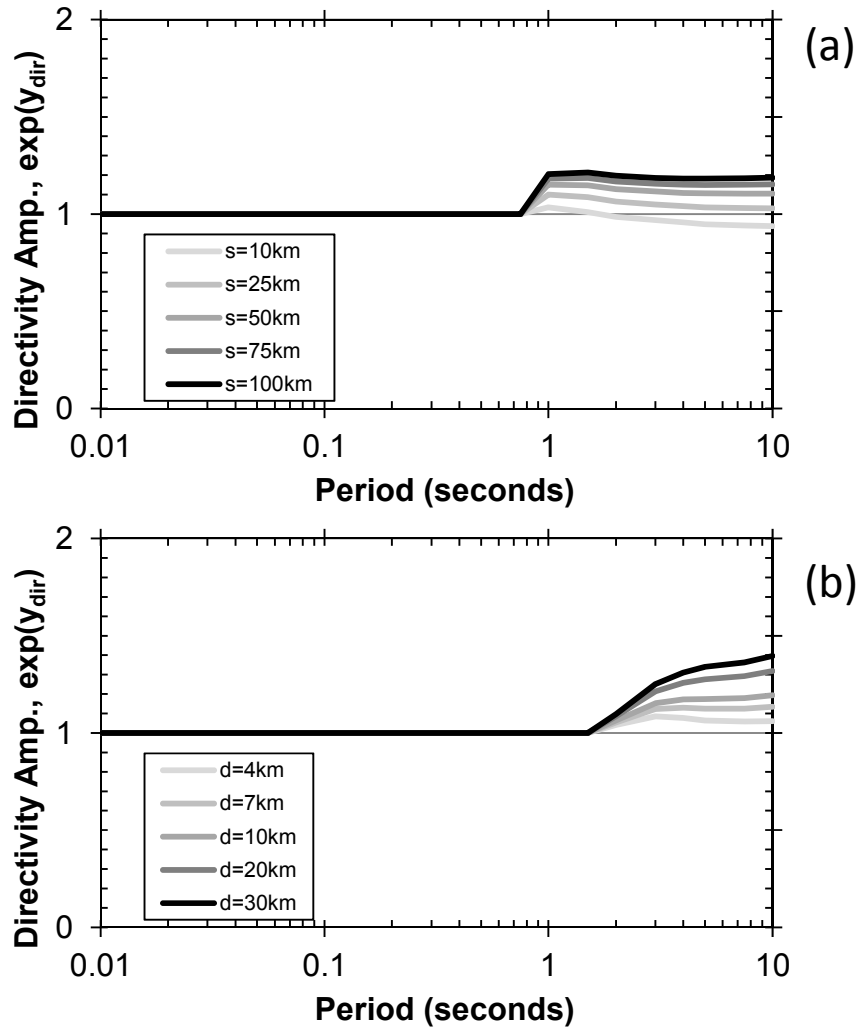


Figure 4-1: Examples of directivity amplification functions using Bayless & Somerville model (Spudich et al. 2013) for Mw 7 and (a) strike slip and (b) dip slip faults.

There was no directivity amplification ($AF_{dir}=1.0$ for all periods) for 4 of the 19 ground motions. One recording had a maximum AF_{dir} value of 1.26, 8 had maximum AF_{dir} values of 1.02 to 1.09, and 6 had maximum AF_{dir} values of 1.10 to 1.13. For strike-slip events (i.e., the Kobe and Darfield earthquakes), the peak

Table 4-1: Summary of Directivity Amplification Values (AF_{dir}) Used in NGA GMPEs

Earthquake	Station	T≤0.5	T=0.75	T=1.0	T=1.5	T=2.0	T=3.0	T=4.0	T=5.0	T=7.5	T=10
Nihonkai-Chubu	Hachirogata	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Superstition Hills	Wildlife	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Loma Prieta	Treasure Island	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Hyogoken-Nanbu Kobe	Amagasaki-G	1.00	1.00	1.12	1.11	1.09	1.08	1.07	1.07	1.06	1.06
Hyogoken-Nanbu Kobe	Amagasaki No. 3 P.P., KE	1.00	1.00	1.10	1.09	1.07	1.06	1.05	1.04	1.04	1.04
Hyogoken-Nanbu Kobe	Higashi-Kobe Bridge	1.00	1.00	1.12	1.11	1.09	1.07	1.06	1.06	1.06	1.06
Hyogoken-Nanbu Kobe	Kobe-JI-S	1.00	1.00	1.10	1.09	1.06	1.05	1.04	1.04	1.03	1.03
Hyogoken-Nanbu Kobe	Port Island GL	1.00	1.00	1.08	1.06	1.04	1.03	1.02	1.01	1.00	1.00
Niigata-Ken Chuetsu-Oki	NIG018 (K-Net)	1.00	1.00	1.00	1.00	1.08	1.19	1.22	1.23	1.24	1.26
Darfield, NZ	GDLC	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Darfield, NZ	HPSC	1.00	1.00	1.13	1.12	1.11	1.09	1.09	1.08	1.08	1.08
Darfield, NZ	SMTC	1.00	1.00	1.13	1.12	1.10	1.09	1.08	1.08	1.08	1.08
Christchurch, NZ	CBGS	1.00	1.00	1.00	1.00	1.04	1.08	1.08	1.06	1.06	1.06
Christchurch, NZ	CCCC	1.00	1.00	1.00	1.00	1.04	1.08	1.08	1.06	1.06	1.06
Christchurch, NZ	CHHC	1.00	1.00	1.00	1.00	1.04	1.08	1.07	1.05	1.04	1.05
Christchurch, NZ	HPSC	1.00	1.00	1.00	1.00	1.04	1.07	1.06	1.05	1.04	1.04
Christchurch, NZ	PRPC	1.00	1.00	1.00	1.00	1.04	1.09	1.08	1.07	1.06	1.07
Christchurch, NZ	REHS	1.00	1.00	1.00	1.00	1.03	1.07	1.05	1.04	1.03	1.03
Christchurch, NZ	SHLC	1.00	1.00	1.00	1.00	1.03	1.06	1.04	1.03	1.02	1.02

value of AF_{dir} occurs at a period of 1 second. For the remaining earthquakes, which are reverse or reverse-oblique events, the peak AF_{dir} values occurs at a period of 3 seconds, except for the NIG018 recording of the Niigata-Ken Chuetsu-Oki earthquake which has its peak AF_{dir} at T=10 seconds.

Evidence of the presence of pulse-like forward directivity in the ground motion recordings was evaluated based on the Baker's (2007) wavelet-based classification method. The far right column in Table 2-1 indicates whether or not the Baker (2007) method classifies the ground motion as pulse-like. Of the 19 ground motions in the database, 11 were classified to be pulse-like.

4.1.3 Computation of Median Spectra and Residuals

Median GMRot150 acceleration response spectra were calculated for each of the 19 liquefaction sites using AS08, BA08, CB08, and CY08. The NGA parameters used to make these calculations are summarized in Table 2-2. For source-site combinations not included within the PEER 2008 NGA database, source-site distances were calculated using a version of the *dist_3df* Fortran code by Boore (2008) based on finite fault model geometry available in the literature. Figures 4-2 through 4-4 present comparisons of calculated median NGA response spectra versus recorded ground motions for each of the 19 sites.

The total residual error (ε_{tot}) of the recorded ground motion ($Sa_{recorded}$) relative to the median NGA prediction (Sa_{NGA}) was calculated as:

$$\varepsilon_{tot} = \ln(Sa_{recorded}) - \ln(Sa_{NGA}) \quad (4-11)$$

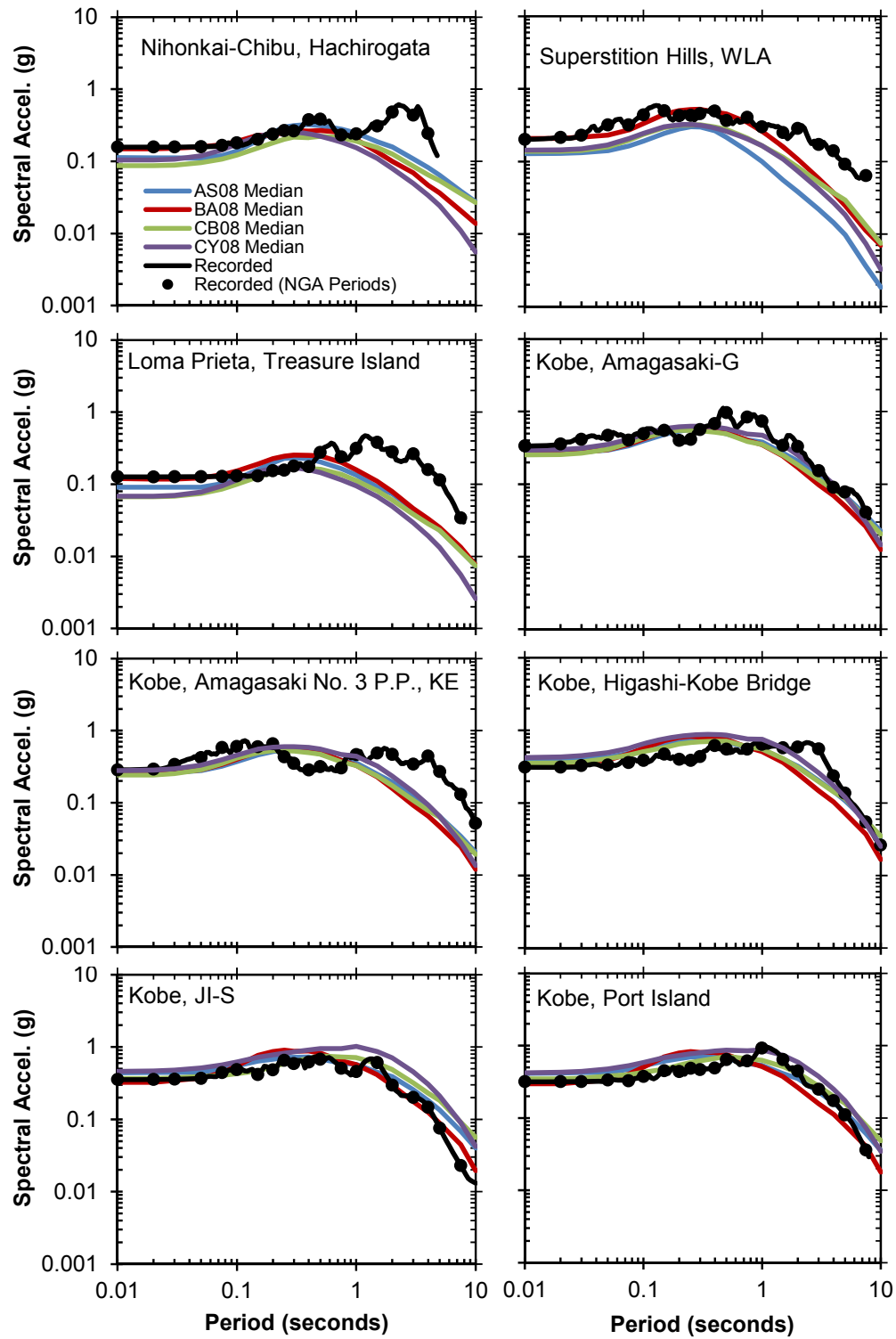


Figure 4-2: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.

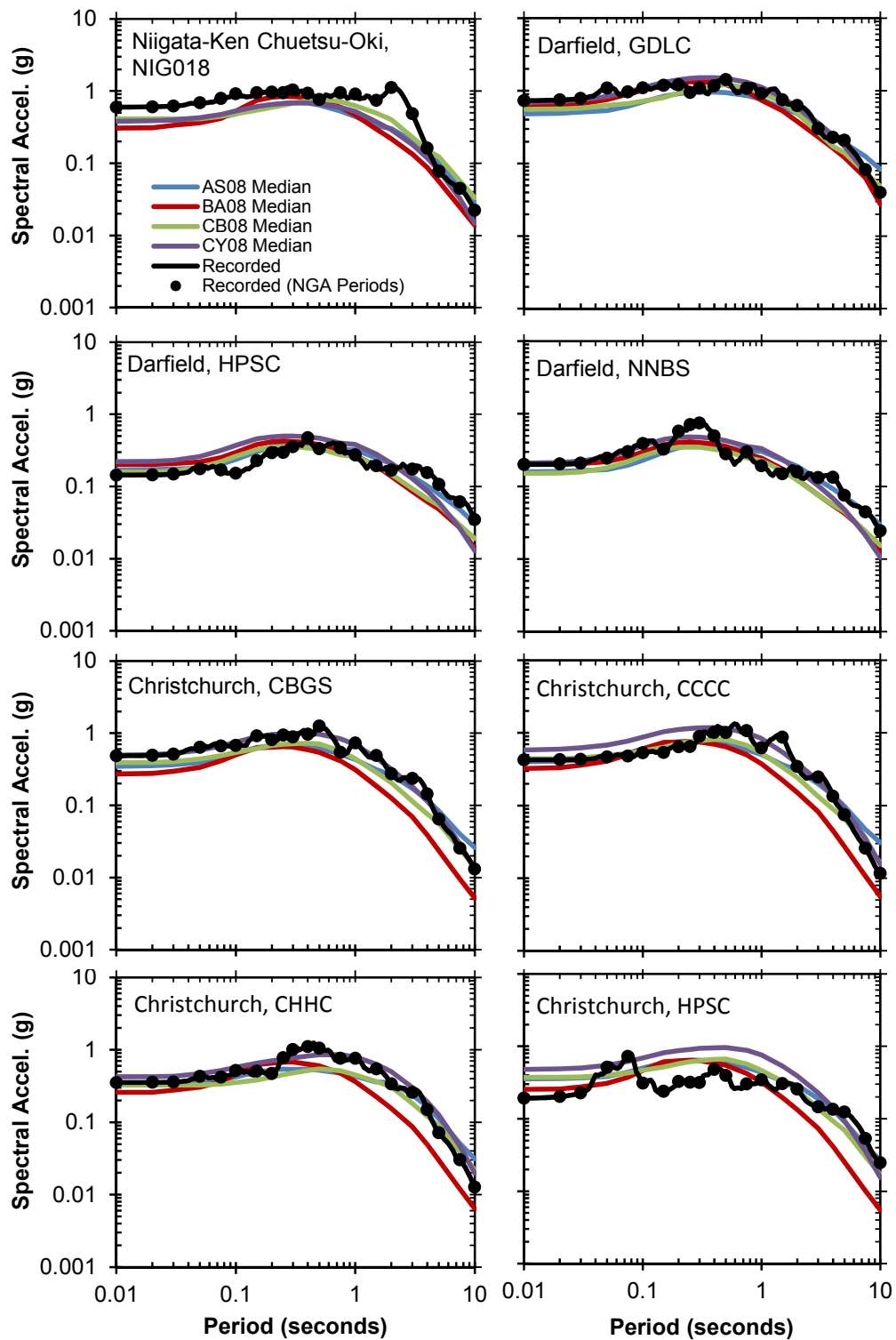


Figure 4-3: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.

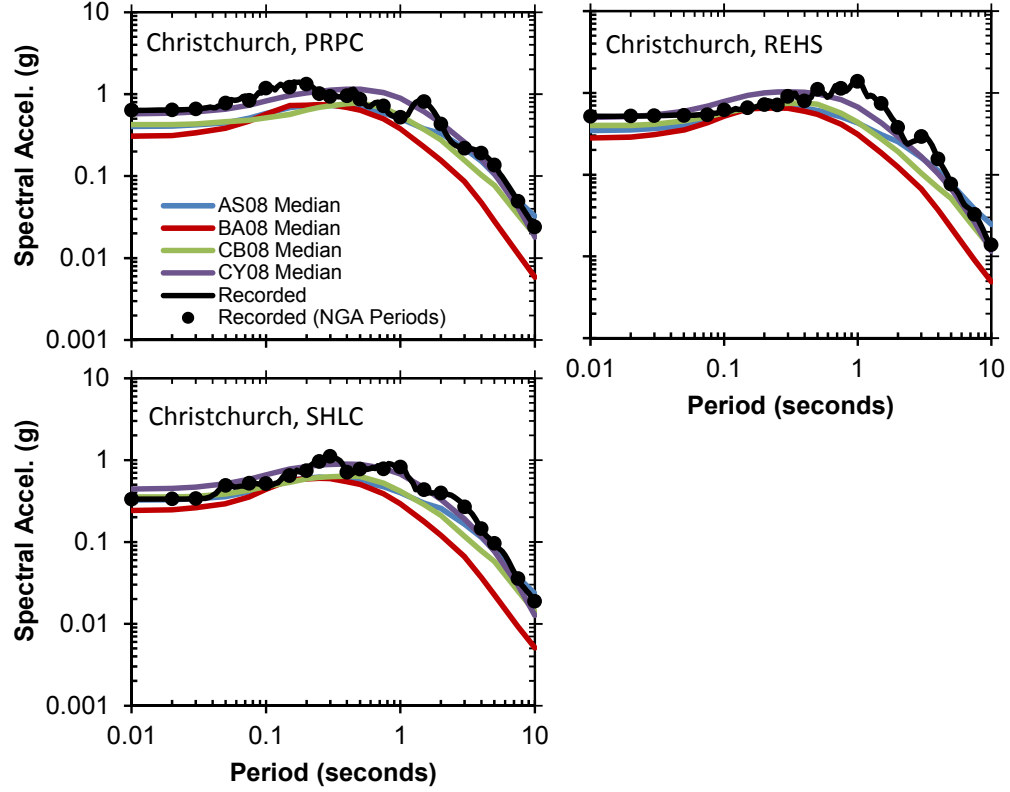


Figure 4-4: Comparison of acceleration response spectra (geometric mean) for recorded motions and NGA median predictions.

Abrahamson and Youngs (1992) described a method to partition the total error into two parts: an inter-event term (η_i) that represents the random effect for the i^{th} (earthquake) event; and the intra-event term (ε_{ij}) that represents the variation from station to station (j^{th} station) within an event. Each of these terms is typically assumed to be an independent and normally distributed variate. The total error is the sum of the inter-event and intra-event terms:

$$\varepsilon_{tot} = \eta_i + \varepsilon_{ij} \quad (4-12)$$

This separation of error terms is desirable for ground motion and site response studies because the inter-event term can be strongly influenced by

earthquake effects unrelated to site response, such as stress drop, and site-source path and attenuation effects. Abrahamson and Youngs (1992) used an iterative maximum likelihood procedure to simultaneously estimate calibration coefficients and the intra- and inter-event term variances.

Once the model parameters are established (as they already have been for NGA GMPEs), then the inter-event residual error for the i^{th} event can be calculated by:

$$\eta_i = \frac{\tau^2 \sum_{j=1}^{N_i} \epsilon_{tot}}{N_i \tau^2 + \sigma^2} \approx \frac{1}{N_i} \sum_{j=1}^{N_i} \epsilon_{tot} \quad (4-13)$$

where N_i is the number of recordings for the i^{th} event, and τ^2 and σ^2 are the inter-event and intra-event variances, respectively. The approximation on the right side of Eqn. (4-13) is true when N_i is large. Thus, intra-event residuals were calculated using Eqn. (4-11 through (4-13 (with Eqn. (4-11 solved for ϵ_{ij}). Note that Eqn. (4-11 through (4-13 are used to calculate residuals independently for each period of interest, though a period subscript has been dropped for convenience.

For events contained within the PEER 2008 NGA ground motion database (i.e., Superstition Hills, Loma Prieta, and Kobe) values of ϵ_{tot} and η_i were calculated based on all the stations within the database. For the Darfield and Christchurch events, which were not in the 2008 NGA database, ground motions from sites within 200 km of the epicenters were compiled and inter-event residuals were calculated for each of the four NGA GMPEs used.

Multiple ground motion recordings were not readily available for the Nihonkai-Chubu and Niigata-Ken Chuetsu-Oki earthquakes, so the inter-event residual were not calculated. Therefore, to facilitate the incorporation of these data into the analysis, ε_{ij} was taken as ε_{tot} for the Hachirogata and NIG018 station recordings (i.e, $\eta_i = 0$). Since only one ground motion was used from each of these two events, any bias introduced by not accounting for the inter-event residuals is not expected to have a significant effect on the overall results and their interpretation. Residuals are related to the intra-event ratio of amplification of the recorded ground motion relative to the NGA prediction (AR) by:

$$AR = \frac{Sa_{recorded}}{Sa_{NGA}} = \exp(\varepsilon_{ij}) \quad (4-14)$$

Figure 4-5 presents the intra-event residuals versus period for each of the ground motion recordings relative to AS08, BA08, CB08, and CY08 NGA estimations. Also shown on these plots are the mean and mean $\pm$ one standard deviation values. The residuals are plotted against PGA , V_{s30} and M_w for various periods in Figures 4-6, 4-7 and 4-8, respectively. Also included in these plots are linear least square regression of the mean trend and the 95 percent confidence intervals for the mean trend. Standard deviations calculated for the liquefaction ground motion intra-event residuals are compared with intra-event standard deviations reported by NGA developers in Figure 4-9.

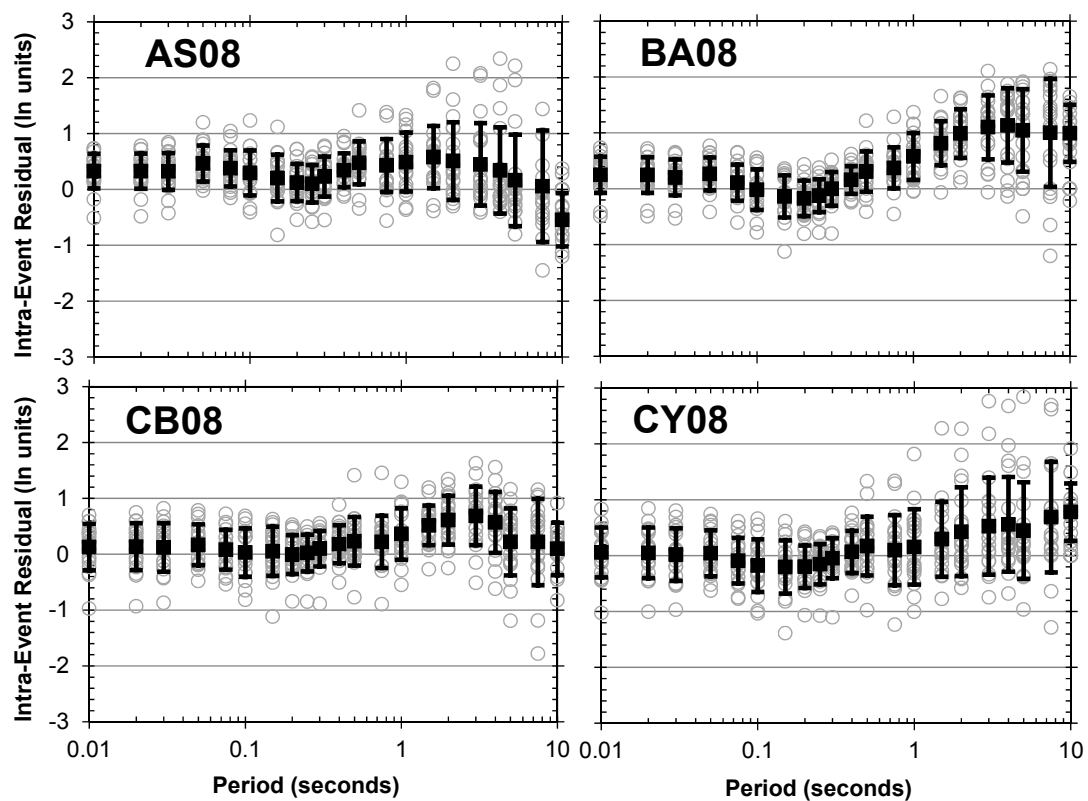


Figure 4-5: Intra-Event residuals computed from NGA spectral accelerations.

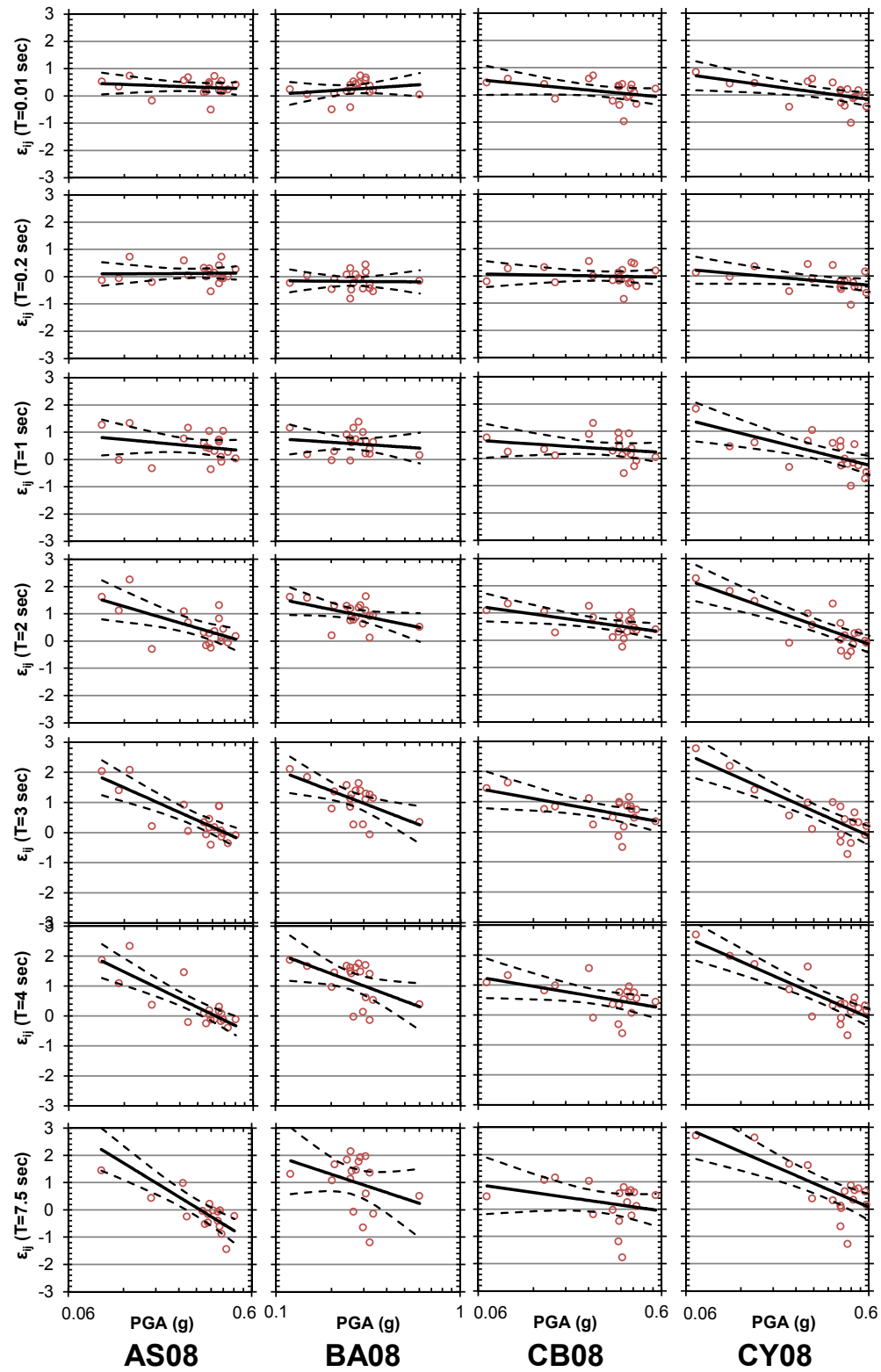


Figure 4-6: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of PGA.

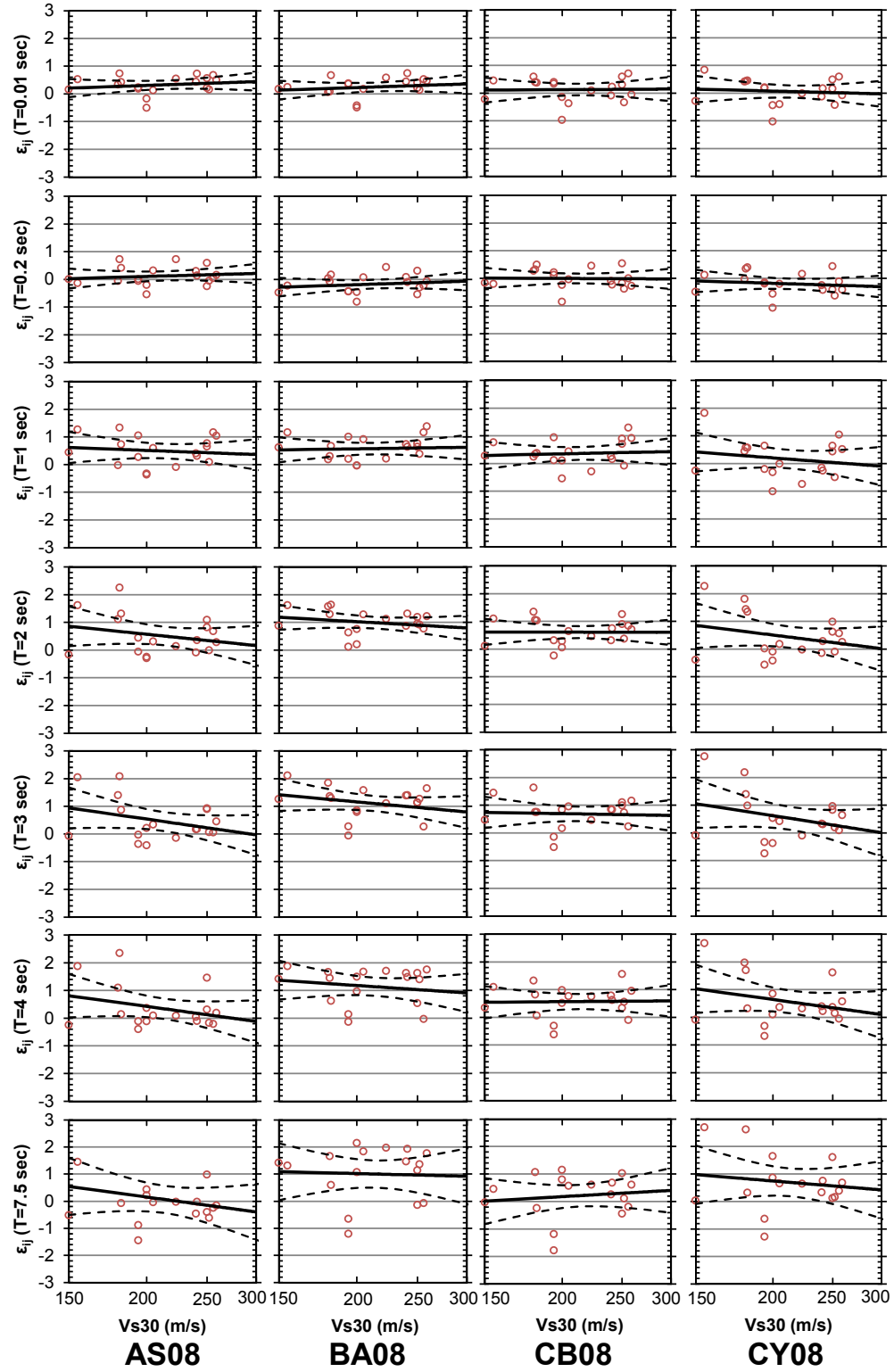


Figure 4-7: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) for various periods plotted versus V_{s30} .

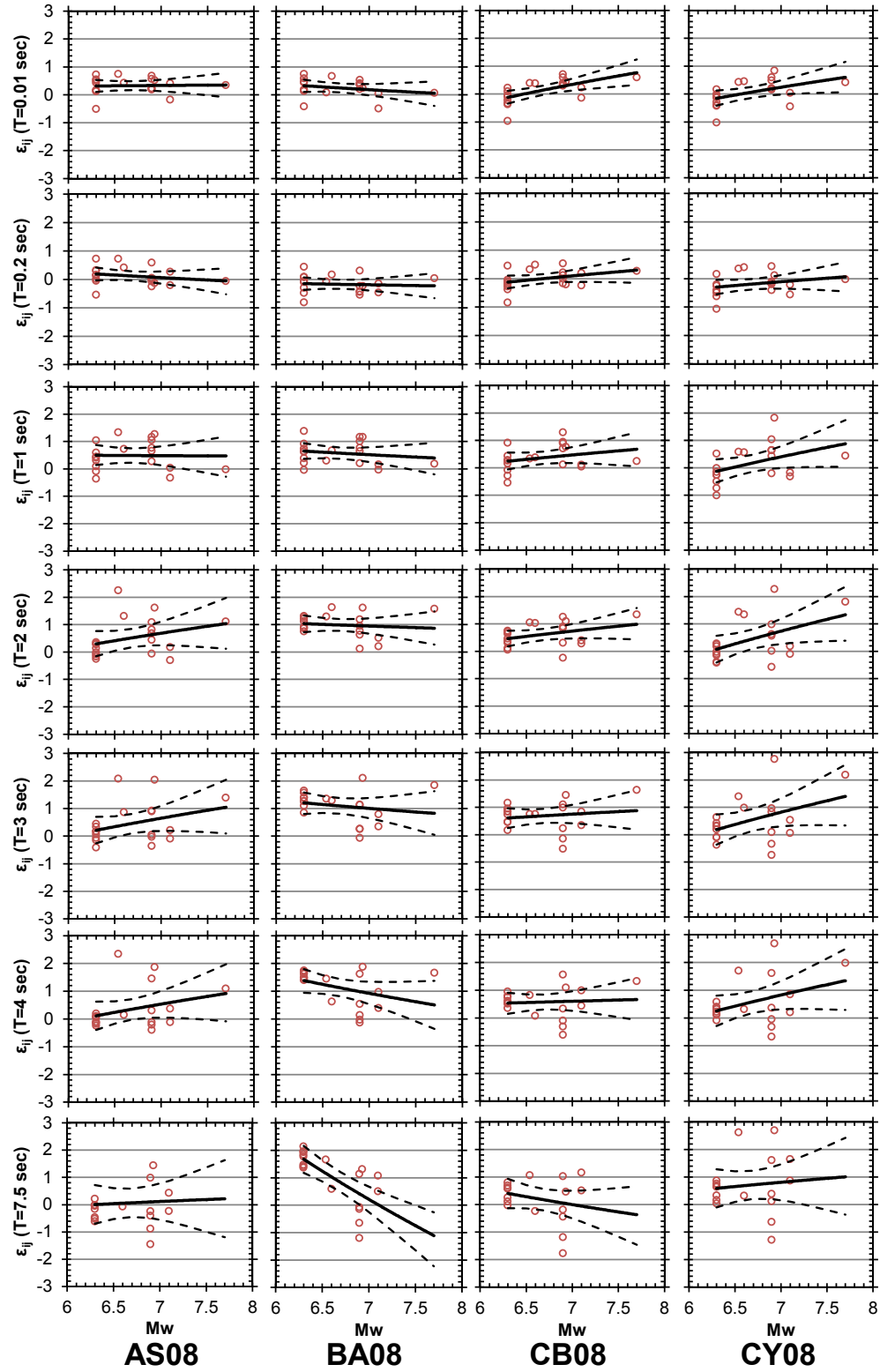


Figure 4-8: Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) for various periods plotted versus M_w .

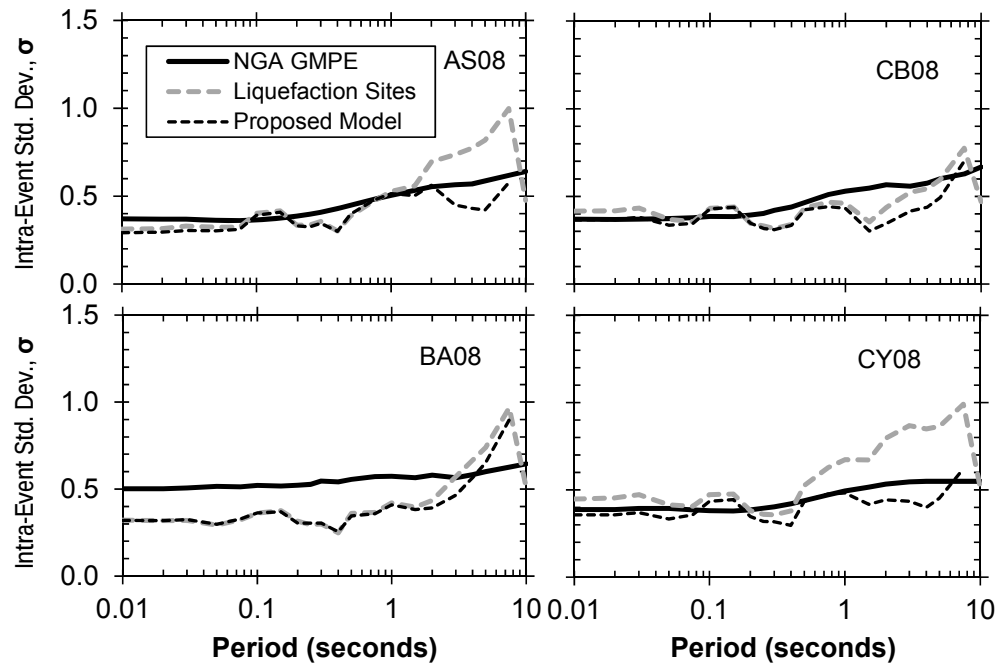


Figure 4-9: Intra-event standard deviations for: NGA GMPEs for M7, $V_{s30}=200\text{m/s}$, $R_{rup}=5\text{km}$, vertical strike slip; liquefaction site ground motion residuals; and liquefaction ground motion residuals from proposed model.

4.1.4 Discussion

4.1.4.1 Residuals

Examination of the intra-event residuals plotted versus period (Figure 4-4) reveals some distinct bias and trends, the most pronounced of which is the positive bias exhibited for all four NGAs at periods of about 0.4 seconds and greater. This observation is generally consistent with Youd and Carter's (2005) and Hartvigsen's (2007) findings that long period ground motions tend to be amplified relative to site response predictions that do not consider porewater pressure generation and liquefaction.

Maximum values of mean intra-event residuals for the AS08, BA08, CB08, and CY08 are 0.58, 1.14, 0.68, and 0.78, respectively, which correspond to *AR* values of 1.79, 3.13, 1.97 and 2.18. The mean maxima for all but CY08 occur between periods of 1.5 and 4 seconds, while the maximum mean value for CY08 occurs at a period of 10 seconds. At short periods ($T \leq 0.05$ seconds) the mean residual values ranged from 0.02 to 0.47 (mean *AR* values of 1.02 to 1.60) for BA08, CB08, and CY08. At intermediate periods between about 0.05 and 0.4 seconds, results from all four of the GMPEs showed a trough in the mean residual values. For BA08, CB08, and CY08, the trough extended to minimum mean values of -0.01 to -0.20 (mean *AR* of 0.99 to 0.82). The minimum mean value in the mid-period trough for AS08 was slightly positive at 0.1 (mean *AR* = 1.11).

Examination of the residuals plotted against NGA-predicted PGA in Figure 4-6 reveals some bias and trend at certain periods. At short period ($T = 0.01$ seconds) there is a small positive bias and insignificant trend. At $T = 0.2$ seconds, which is typically near the bottom of the deamplification trough described earlier, the trend is neutral and the bias is slightly negative for 3 of 4 GMPEs. Positive bias and a negative trend are apparent at $T = 3$ seconds (the same is true for periods between 1 and 7.5 seconds). The negative trend at long periods suggests some dependency of liquefaction amplification on the amplitude of the shaking. Long period liquefaction amplification is generally higher at lower ground shaking levels and lower at higher ground shaking levels. The trends described above are generally consistent across the four NGA GMPEs, which

suggests that these effects are characteristic of the liquefaction ground motion dataset rather than features of particular GMPEs.

Residuals plotted versus V_{s30} generally show negative trends in the long period range and are neutral at short to intermediate periods (Figure 4-7). There is a weak positive bias at the PGA, neutral to slightly negative bias at $T = 0.2$ seconds, and positive bias at $T = 3$ seconds (same for $T \geq 1.0$ seconds). Lower shear wave velocity is associated with higher liquefaction susceptibility (e.g., Andrus and Stokoe 2000), so it is reasonable that higher amplification occurs for lower V_{s30} . The parameter V_{s30} is not appropriate for predicting liquefaction triggering, because it is calculated by averaging V_s over a 30 meter depth, an interval which would typically encompass both liquefiable and non-liquefiable layers. Use of cone penetration test data or standard penetration test data to calculate factors of safety against liquefaction triggering may have proven to be better predictors, but these data are not available for any of the sites except for Port Island and WLA..

Residuals plotted versus M_w (Figure 4-8) generally show positive bias and neutral to positive trends at $T = 3$ seconds. At $T = 0.01$ and 0.2 seconds, there is a weak positive bias and neutral to slightly positive trend.

4.1.4.2 Standard Deviations

The intra-event standard deviation values calculated for the liquefaction ground motions for the four GMPEs ranged from 0.25 to 0.48 natural log (ln) units at period of 0.4 seconds and less (Figure 4-9). At periods greater than 0.4

seconds, standard deviation values generally increased to values up to 1.0. With the exception of BA08, the standard deviation values calculated for periods less than about 0.4 seconds are within about ± 0.1 of the values reported by the NGA GMPE developers for their relationships. In this period range the standard deviation values calculated for the liquefaction ground motion dataset are 0.14 to 0.29 lower than those reported by BA08. At periods greater than about 0.4 seconds, there are more significant differences between liquefaction dataset standard deviations and the values reported by the GMPE developers. In particular, liquefaction dataset values are up to 0.38 and 0.44 in units higher than the reported GMPE values for AS08 and CY08, respectively.

4.1.4.3 Forward Directivity Considerations

The potential effect of forward directivity is an important factor in this analysis. As described earlier, an attempt was made to account for amplification at long periods that could be attributable to forward directivity. However, the 2013 Bayes and Somerville directivity modification is a median correction, and use of this correction may not necessarily account precisely for directivity amplification for a specific ground motion.

A subset of data from sites without evidence of forward directivity pulse-like ground motions was examined to evaluate whether residual directivity effects could introduce significant bias into the results. The subset consists of the 8 recordings which were classified by the Baker (2007) wavelet method as not pulse-like as noted in Table 2-1. As can be observed in Table 4-1, according to

the Bayes and Somerville criteria, three of these recordings had directivity amplification values of 1.0 for all periods and the other five had directivity amplification values greater than unity at longer periods. The calculated directivity amplification values were maintained in the four ground motions for the subset analysis. This is conservative for the purpose of showing that the same liquefaction residual/amplification trends are present in this no-directivity subset, because including the directivity amplification would tend to shift residuals downward. Figure 4-10 compares the means and standard deviations of the directivity-only dataset to those of the complete dataset.

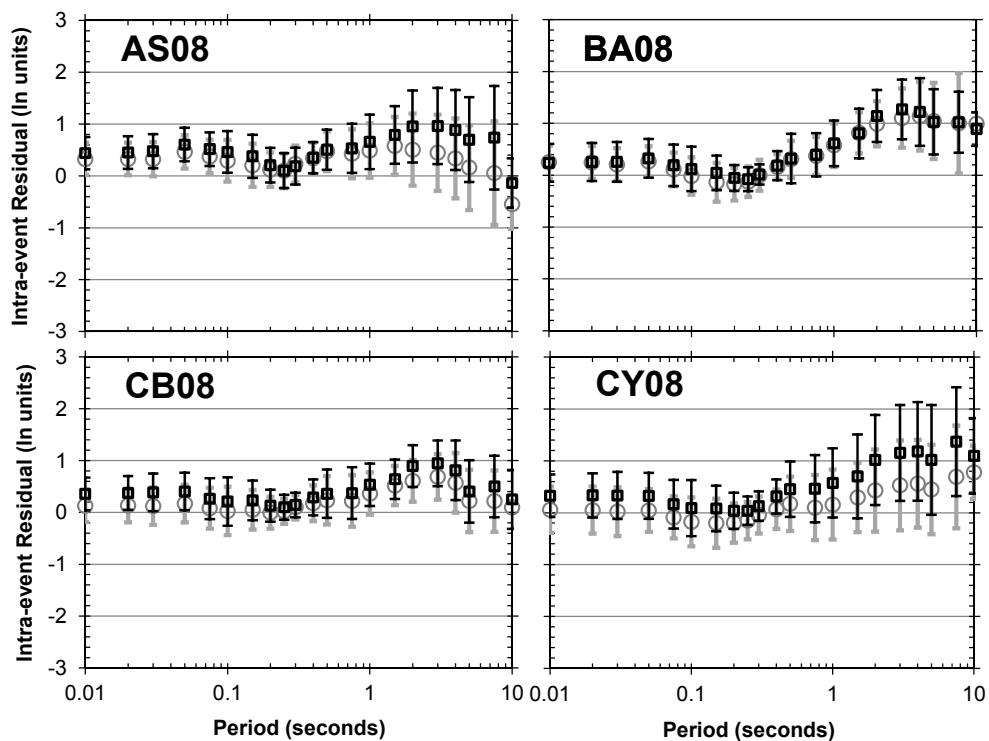


Figure 4-10: Means and standard deviations of intra-event residuals computed from NGA spectral accelerations all data (gray) and only ground motions with no apparent forward directivity pulse (black).

Examining Figure 4-10 it is apparent that similar bias and trends are present in the residuals calculated for ground motions without apparent directivity pulses as for the entire liquefaction dataset. Based on these observations, directivity effects do not appear to impart undue positive bias in the residuals. In fact, the mean residuals are generally larger for the no-directivity subset, even with the application of directivity amplification in 5 of the 8 ground motions. These comparisons suggest that the directivity correction of Bayless and Somerville (Spudich et al. 2013) is adequate for this dataset.

4.1.4.4 Basin and Surface Wave Considerations

Amplification from basin effects, including surface waves, can affect spectral accelerations in the same long period range as liquefaction. The NGA database includes ground motions with surface waves, and thus the NGA GMPEs capture in part amplification due to surface waves. The Z_1 and $Z_{2.5}$ factors used in the AS08, CB08 and CY08 GMPEs attempt to account partially for deep soil basin effects. However, these parameters cannot fully account for two- and three-dimensional basin effects, especially edge effects. More detailed and quantitative accounting of basin effects generally requires two- or three-dimensional numerical modeling, which was beyond the scope of this study.

Horizontal and vertical displacement-time histories were examined for evidence of Rayleigh waves using mean Fourier frequencies calculated over a moving 2-second window. The time at which spectral accelerations occur for each period were compared to those windows containing significant Rayleigh

waves, and residuals were not systematically greater for time histories that show evidence of surface waves.

Based on examination of displacement-time histories, Holzer and Youd (2007) concluded that Love waves occurred late in the WLA record, and similar evidence of surface waves is present in some of the other time histories in the liquefaction ground motion database. The period of a site softened by liquefaction may correspond with the long period of surface waves and lead to additional amplification. Such amplification would not be captured by the NGA relationships, which generally exclude sites with liquefaction. Therefore, the higher long-period spectral acceleration values observed in the liquefaction recordings is likely due to liquefaction effects on body and surface waves. The long period residuals observed for BA08 are larger relative to the other GMPEs, which may be attributable to the absence of a deep basin term in BA08. For AS08, CB08, and CY08, it is possible that basin effects not fully accounted for by the Z_1 and $Z_{2.5}$ NGA parameters are manifested in some of the liquefaction ground motions. However, we consider the inclusion of these data to be conservative, because basins tend to amplify long period spectral accelerations.

Table 4-2: Summary of Intra-Event Standard Deviations

Period, T (seconds)	AS08			BA08			CB08			CY08		
	$\sigma_{liq}^{(a)}$	$\sigma^{(b)}$	$\Delta\sigma^{(c)}$	$\sigma_{liq}^{(a)}$	$\sigma^{(b)}$	$\Delta\sigma^{(c)}$	$\sigma_{liq}^{(a)}$	$\sigma^{(b)}$	$\Delta\sigma^{(c)}$	$\sigma_{liq}^{(a)}$	$\sigma^{(b)}$	$\Delta\sigma^{(c)}$
0.01	0.312	0.371	-0.059	0.322	0.502	-0.180	0.416	0.370	0.046	0.447	0.387	0.060
0.02	0.316	0.368	-0.052	0.319	0.502	-0.183	0.418	0.369	0.049	0.453	0.387	0.066
0.03	0.330	0.368	-0.038	0.320	0.507	-0.187	0.433	0.370	0.063	0.472	0.393	0.079
0.05	0.323	0.362	-0.039	0.293	0.516	-0.223	0.368	0.373	-0.005	0.413	0.393	0.020
0.075	0.324	0.361	-0.037	0.324	0.513	-0.189	0.356	0.379	-0.023	0.406	0.385	0.021
0.1	0.402	0.365	0.037	0.362	0.520	-0.158	0.432	0.387	0.045	0.472	0.380	0.092
0.15	0.416	0.375	0.041	0.376	0.518	-0.142	0.440	0.385	0.055	0.475	0.380	0.095
0.2	0.334	0.386	-0.052	0.316	0.523	-0.207	0.346	0.393	-0.047	0.380	0.386	-0.006
0.25	0.336	0.397	-0.061	0.294	0.527	-0.233	0.331	0.403	-0.072	0.360	0.394	-0.034
0.3	0.357	0.409	-0.052	0.299	0.546	-0.247	0.317	0.421	-0.104	0.357	0.403	-0.046
0.4	0.301	0.429	-0.128	0.248	0.541	-0.293	0.340	0.439	-0.099	0.379	0.420	-0.041
0.5	0.386	0.448	-0.062	0.360	0.555	-0.195	0.435	0.465	-0.030	0.524	0.437	0.087
0.75	0.473	0.483	-0.010	0.367	0.571	-0.204	0.466	0.511	-0.045	0.627	0.471	0.156
1	0.526	0.505	0.021	0.420	0.573	-0.153	0.460	0.530	-0.070	0.673	0.494	0.179
1.5	0.558	0.534	0.024	0.396	0.566	-0.170	0.355	0.548	-0.193	0.670	0.515	0.155
2	0.696	0.553	0.143	0.435	0.580	-0.145	0.436	0.567	-0.131	0.794	0.533	0.261
3	0.736	0.565	0.171	0.572	0.566	0.006	0.523	0.558	-0.035	0.868	0.546	0.322
4	0.772	0.570	0.202	0.665	0.583	0.082	0.544	0.576	-0.032	0.848	0.549	0.299
5	0.819	0.587	0.232	0.738	0.601	0.137	0.597	0.601	-0.004	0.863	0.549	0.314
7.5	0.999	0.618	0.381	0.967	0.626	0.341	0.775	0.628	0.147	0.990	0.549	0.441
10	0.476	0.640	-0.164	0.509	0.645	-0.136	0.470	0.667	-0.197	0.514	0.549	-0.035

Table 4-2 Notes:

(a) Standard error computed for the intra-event residuals of the 19 liquefaction ground motions.

(b) Standard error of the intra-event residual reported by the NGA developer.

(c) Standard error reduction (negative) or increase (positive), $\Delta\sigma = \sigma_{liq} - \sigma$

4.2 Liquefaction Amplification Model

Based on the systematic bias and trends observed in the plot of residuals against PGA , V_{s30} , and M_w , a regression analysis was performed with these parameters to develop a straightforward amplification model that could be used to modify NGA estimates to account for liquefiable soil conditions. The regression was performed using the multi-linear (in log space) least squares method, where the amplification function takes the following form:

$$\ln(AF_{liq}) = c_3 \ln(PGA) + c_2 \ln(V_{s30}) + c_1 M_w + c_0 + \varepsilon_{ij} \quad (4-15)$$

where PGA is in units of g , V_{s30} is in units of m/s, and ε_{ij} is the intra-event model residual. A model of this form can be used to adjust a value of Sa calculated from an NGA GMPE to account for liquefaction effects in a manner similar to adjustments made for directivity, using the following formula:

$$\ln(Sa_{liq}) = \ln(Sa_{NGA}) + \ln(AF_{liq}) \quad (4-16)$$

The coefficients were first calculated for the four NGAs using the least squares method, then manually smoothed to minimize erratic undulations in $AF_{liq}(T)$ versus T plots. The resulting coefficients and their standard errors are presented in Tables 4-3 and 4-4. Example plots of $AF_{liq}(T)$ versus T for the four NGA GMPEs are presented in Figures 4-11 and 4-12 (illustrating PGA and V_{s30} dependency) and Figure 4-13 (illustrating M_w dependency). Model coefficients were not calculated for $T = 10$ seconds because of the scarcity of data at that period.

Table 4-3: Proposed Model Regression Coefficients and Standard Errors for AS08 and BA08

Period, T (sec)	AS08					BA08				
	C ₃	C ₂	C ₁	C ₀		C ₃	C ₂	C ₁	C ₀	
0.01	-0.294±0.20	0.715±0.47	-0.123±0.21	-3.047±2.67		0.062±0.33	0.225±0.57	-0.182±0.21	0.337±3.46	
0.02	-0.324±0.20	0.741±0.47	-0.122±0.21	-3.234±2.68		0.012±0.33	0.260±0.57	-0.158±0.20	-0.089±3.45	
0.03	-0.358±0.21	0.748±0.48	-0.165±0.22	-2.981±2.75		-0.031±0.34	0.263±0.58	-0.177±0.21	-0.030±3.51	
0.05	-0.362±0.20	0.670±0.46	-0.206±0.21	-2.279±2.66		-0.081±0.30	0.216±0.51	-0.200±0.18	0.264±3.11	
0.075	-0.347±0.21	0.565±0.49	-0.236±0.22	-1.508±2.78		-0.057±0.34	0.114±0.58	-0.190±0.21	0.701±3.50	
0.1	-0.304±0.27	0.534±0.63	-0.208±0.29	-1.557±3.59		-0.094±0.39	0.242±0.67	-0.107±0.24	-0.722±4.06	
0.15	-0.249±0.28	0.519±0.65	-0.211±0.30	-1.473±3.74		-0.154±0.40	0.434±0.68	-0.054±0.24	-2.278±4.12	
0.2	-0.125±0.22	0.258±0.52	-0.257±0.24	0.319±2.96		-0.204±0.33	0.412±0.56	-0.102±0.20	-1.943±3.40	
0.25	-0.041±0.21	0.022±0.49	-0.363±0.22	2.404±2.79		-0.190±0.30	0.255±0.52	-0.193±0.19	-0.433±3.14	
0.3	0.011±0.21	-0.165±0.49	-0.359±0.22	3.521±2.81		-0.146±0.31	0.078±0.52	-0.219±0.19	0.860±3.18	
0.4	-0.026±0.20	0.034±0.46	-0.279±0.21	2.000±2.66		-0.201±0.26	0.352±0.44	-0.159±0.16	-0.941±2.68	
0.5	-0.106±0.27	0.146±0.62	-0.172±0.29	0.646±3.57		-0.234±0.37	0.513±0.64	-0.107±0.23	-2.070±3.87	
0.75	-0.236±0.33	0.227±0.76	-0.177±0.35	0.141±4.33		-0.367±0.39	0.726±0.66	-0.144±0.24	-3.001±4.00	
1	-0.416±0.35	0.144±0.82	-0.216±0.37	0.647±4.67		-0.482±0.42	0.619±0.73	-0.268±0.26	-1.586±4.40	
1.5	-0.628±0.35	0.117±0.80	-0.150±0.37	0.122±4.60		-0.673±0.37	0.528±0.63	-0.318±0.23	-0.811±3.81	
2	-0.925±0.39	0.176±0.90	-0.100±0.41	-0.918±5.14		-0.972±0.40	0.525±0.69	-0.397±0.25	-0.488±4.19	
3	-1.233±0.31	0.382±0.72	-0.113±0.33	-2.394±4.14		-1.336±0.44	0.790±0.75	-0.559±0.27	-1.202±4.57	
4	-1.514±0.29	0.622±0.68	-0.147±0.31	-3.891±3.90		-1.583±0.47	1.193±0.81	-0.959±0.29	-1.021±4.91	
5	-1.798±0.30	0.904±0.68	-0.328±0.37	-4.654±4.01		-1.629±0.51	1.531±0.84	-1.491±0.37	0.598±5.37	
7.5	-2.142±0.39	1.152±0.89	-0.657±0.48	-4.331±5.22		-1.600±0.67	1.766±1.10	-2.111±0.49	3.421±7.03	

Table 4-4: Proposed Model Regression Coefficients and Standard Errors for CB08 and CY08

Period, T (sec)	CB08					CY08				
	C ₃	C ₂	C ₁	C ₀		C ₃	C ₂	C ₁	C ₀	
0.01	-0.135±0.23	0.374±0.57	0.545±0.27	-5.681±3.04		-0.439±0.23	0.615±0.63	0.186±0.29	-4.950±3.30	
0.02	-0.158±0.23	0.407±0.58	0.530±0.27	-5.790±3.06		-0.472±0.23	0.658±0.63	0.172±0.29	-5.138±3.30	
0.03	-0.162±0.24	0.360±0.60	0.462±0.28	-5.076±3.19		-0.486±0.23	0.657±0.64	0.108±0.30	-4.722±3.39	
0.05	-0.105±0.22	0.172±0.56	0.405±0.26	-3.636±2.99		-0.457±0.21	0.553±0.59	0.038±0.27	-3.717±3.11	
0.075	-0.006±0.23	-0.070±0.58	0.380±0.27	-2.079±3.07		-0.403±0.23	0.414±0.64	-0.017±0.29	-2.610±3.35	
0.1	0.098±0.28	-0.185±0.72	0.438±0.34	-1.752±3.81		-0.355±0.28	0.373±0.78	-0.004±0.36	-2.505±4.10	
0.15	0.146±0.28	-0.214±0.72	0.460±0.34	-1.716±3.84		-0.323±0.29	0.348±0.79	0.016±0.36	-2.507±4.15	
0.2	0.214±0.22	-0.432±0.56	0.444±0.27	-0.354±3.00		-0.250±0.22	0.049±0.62	0.033±0.29	-0.937±3.26	
0.25	0.241±0.20	-0.600±0.52	0.364±0.24	1.126±2.74		-0.210±0.20	-0.222±0.56	0.003±0.26	0.814±2.97	
0.3	0.253±0.21	-0.678±0.53	0.399±0.25	1.385±2.81		-0.200±0.21	-0.398±0.57	0.069±0.26	1.415±2.98	
0.4	0.165±0.20	-0.294±0.52	0.455±0.24	-1.080±2.75		-0.301±0.19	-0.052±0.51	0.157±0.24	-1.031±2.70	
0.5	0.045±0.26	0.105±0.67	0.487±0.32	-3.549±3.58		-0.463±0.27	0.337±0.75	0.242±0.34	-3.804±3.92	
0.75	-0.115±0.29	0.503±0.75	0.344±0.35	-4.862±3.98		-0.681±0.30	0.761±0.84	0.157±0.39	-5.717±4.40	
1	-0.295±0.30	0.745±0.76	0.104±0.36	-4.696±4.04		-0.894±0.31	0.962±0.86	-0.006±0.40	-5.887±4.50	
1.5	-0.470±0.21	0.909±0.52	-0.050±0.25	-4.634±2.78		-1.095±0.27	1.100±0.73	-0.122±0.34	-5.955±3.84	
2	-0.682±0.23	1.126±0.59	-0.243±0.28	-4.668±3.13		-1.314±0.28	1.242±0.78	-0.243±0.36	-6.024±4.08	
3	-0.922±0.27	1.360±0.69	-0.481±0.32	-4.555±3.65		-1.518±0.27	1.484±0.74	-0.373±0.34	-6.576±3.91	
4	-1.030±0.27	1.753±0.68	-0.658±0.32	-5.726±3.63		-1.673±0.23	1.817±0.63	-0.544±0.29	-7.363±3.29	
5	-1.071±0.30	2.095±0.76	-0.944±0.40	-5.962±4.11		-1.816±0.26	2.230±0.73	-0.792±0.39	-8.004±3.90	
7.5	-1.149±0.40	2.371±1.01	-1.303±0.54	-5.285±5.50		-2.000±0.35	2.662±0.97	-1.140±0.51	-8.061±5.16	

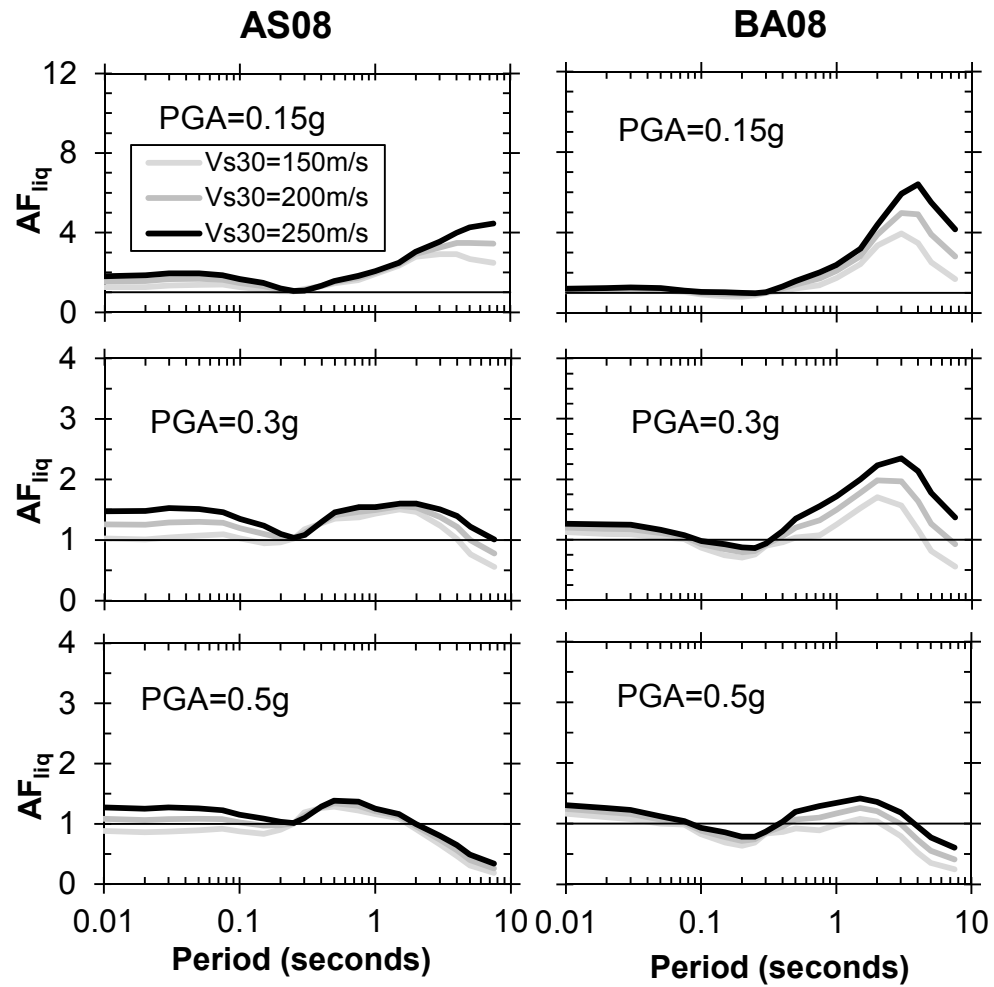


Figure 4-11: Example liquefaction amplification curves for AS08 and BA08 calculated with proposed model.

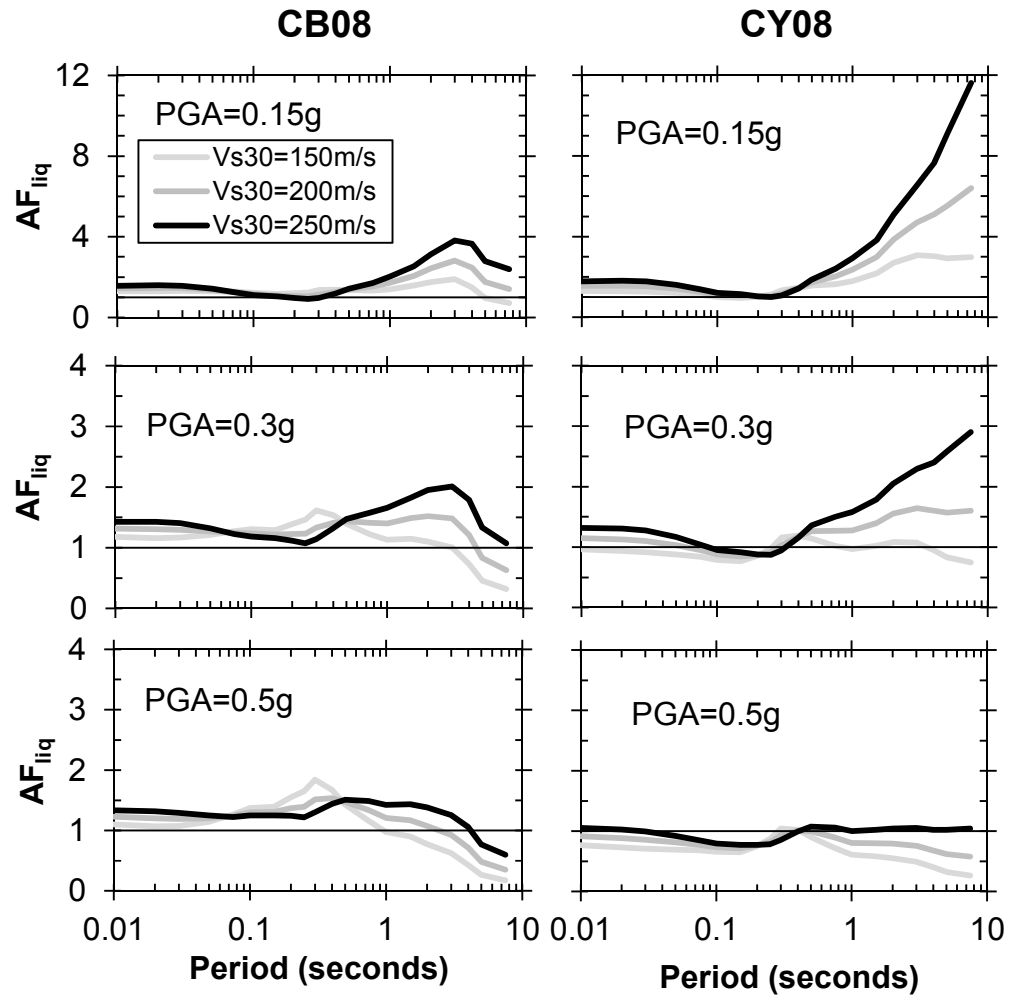


Figure 4-12: Example liquefaction amplification curves for CB08 and CY08 calculated with proposed model.

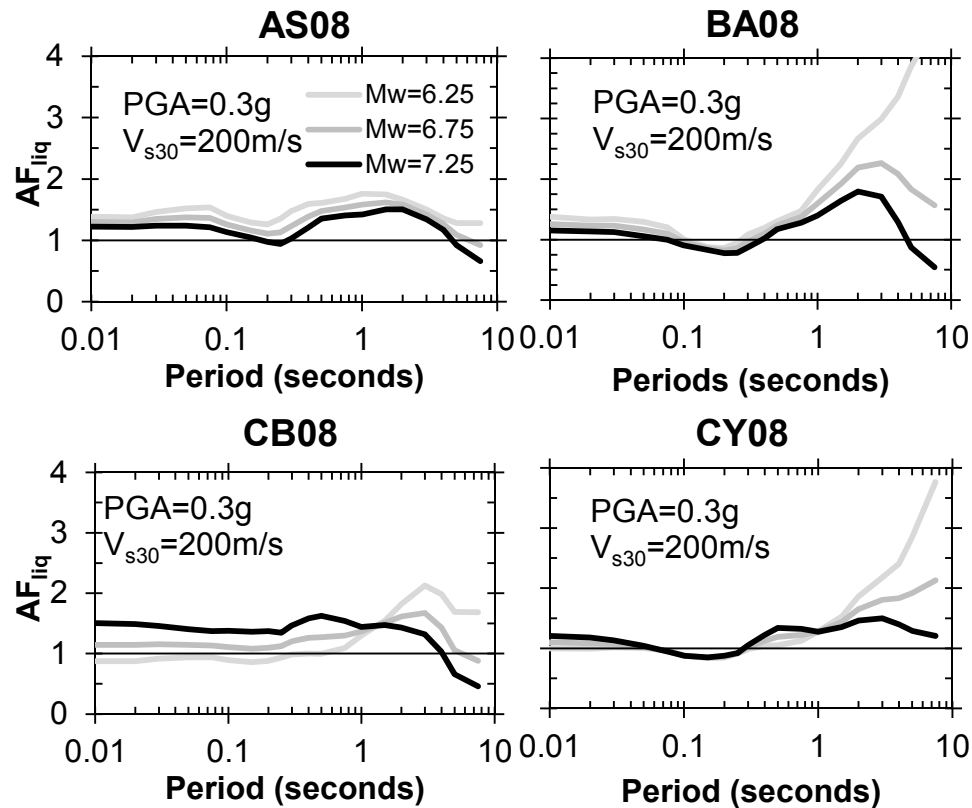


Figure 4-13: Example liquefaction amplification curves showing M_w dependence of the proposed model.

Figures 4-11 and 4-12 show a trend decreasing amplitude of AF_{liq} at long periods with decreasing V_{s30} . It may seem initially counterintuitive that less amplification occurs at softer sites. However, NGA GMPE site factors apply more long period amplification for lower values of V_{s30} . So the contrast in stiffness for a liquefied site condition is greater when the non-liquefied V_{s30} value is higher. Hence, the model's higher long period AF_{liq} values at higher V_{s30} account for the more pronounced stiffness differential compared to lower V_{s30} sites.

The statistical significance of the model was assessed by calculating two-tailed t -statistics for the model coefficient to evaluate the probability (p) that the null hypothesis (i.e., that the model coefficient is zero) cannot be rejected. This

calculation incorporates sample size, which is important given the limited amount of available data. Values of $1-p$, which are essentially equivalent to the confidence level that the model coefficient is statistically significant (i.e., not zero), are reported in Table 4-5. Each of the model parameter coefficients shows at least an 80 percent confidence level of statistical significance at several periods, and many show confidence levels exceeding 90 percent. Thus, the parameters selected for the model are justified, because the statistical significance of the parameters is demonstrated over several periods.

The NGA predictions of spectral accelerations for each of the 19 liquefaction ground motions were adjusted using Eqns. 4-15 and 4-16, and residuals were calculated. Residuals for all of the ground motions plotted versus period are presented in Figure 4-14. Residuals are plotted versus PGA , V_{s30} and M_w in Figures 4-15 through 4-17, respectively. Examination of these residual plots reveals that the bias and trends have been basically eliminated, and dispersion at longer periods is reduced.

The intra-event standard deviations for the amplification models are plotted in Figure 4-9. At longer periods where liquefaction amplification is more significant, the model standard deviations are significantly reduced compared to the standard deviations calculated for the liquefaction sites using the residuals from un-adjusted NGA predictions. The model standard deviations are generally less than or equal to the standard deviations for the original NGA models.

Table 4-5: Confidence levels (%) that the model coefficients are statistically significant

Period, T (sec)	AS08				BA08				CB08				CY08			
	C ₃	C ₂	C ₁	C ₀	C ₃	C ₂	C ₁	C ₀	C ₃	C ₂	C ₁	C ₀	C ₃	C ₂	C ₁	C ₀
0.010	84	85	43	73	14	30	61	8	44	48	94	92	93	66	47	85
0.020	87	86	44	74	8	29	59	7	50	50	93	92	94	67	43	85
0.030	90	87	39	78	14	43	42	20	55	53	92	92	95	72	40	87
0.050	93	87	75	60	17	34	78	13	48	32	78	75	96	68	2	75
0.075	86	68	73	28	27	10	74	26	5	22	79	34	88	38	11	44
0.10	74	53	51	26	2	4	28	8	30	30	79	26	76	30	1	39
0.15	64	66	42	47	34	62	4	62	43	12	85	48	74	45	7	57
0.20	53	60	71	20	62	74	33	68	52	34	87	33	78	31	6	45
0.25	21	47	85	84	35	5	77	34	86	91	88	67	46	69	14	60
0.30	5	11	95	81	33	21	81	19	75	74	71	49	69	43	17	46
0.40	4	19	74	66	47	21	68	11	64	64	95	15	82	40	63	6
0.50	16	34	27	9	58	76	17	68	16	25	90	78	87	44	62	76
0.75	55	28	36	4	45	64	33	48	30	55	69	81	96	67	38	84
1.0	73	12	51	18	80	69	69	41	65	63	21	71	99	74	7	79
1.5	89	3	43	19	90	53	92	5	97	91	47	84	100	83	40	81
2.0	96	16	6	27	94	38	68	5	99	91	25	89	100	87	33	88
3.0	100	32	26	37	100	67	92	24	100	93	84	77	100	93	73	86
4.0	100	64	57	59	100	86	99	29	100	98	94	86	100	99	93	95
5.0	100	80	16	86	99	92	100	4	100	99	95	90	100	99	89	96
7.5	100	78	81	58	97	87	100	37	99	98	98	70	100	98	96	86

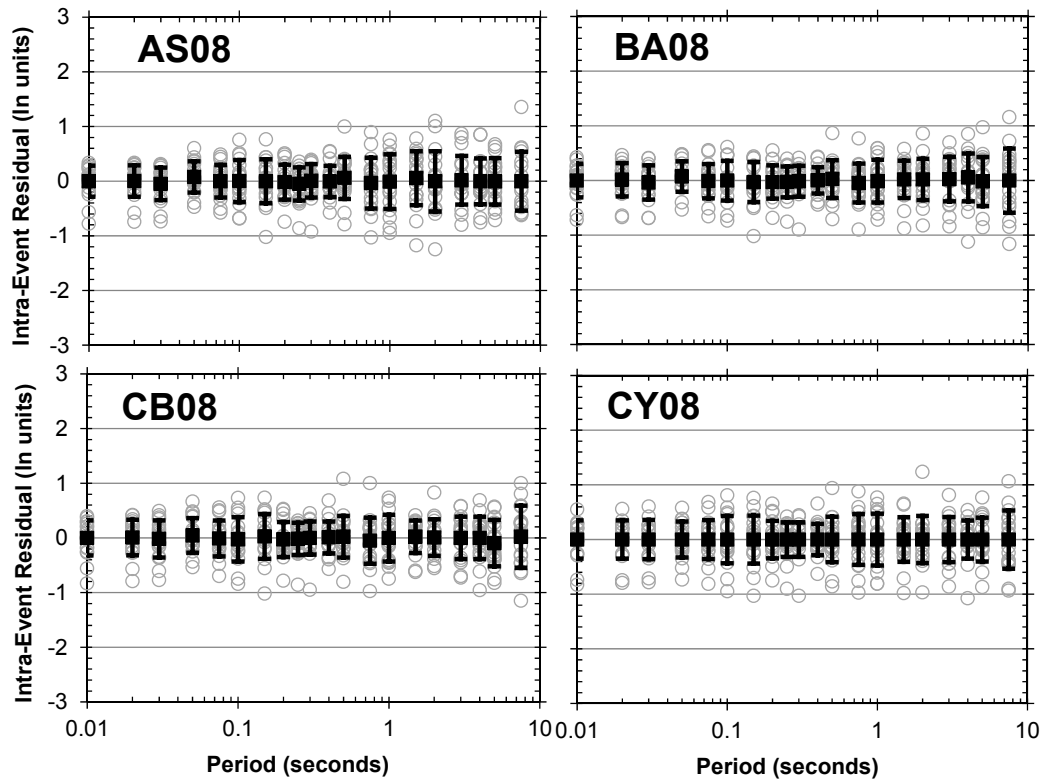


Figure 4-14: Intra-Event residuals computed from NGA GMPEs adjusted with the proposed liquefaction amplification model.

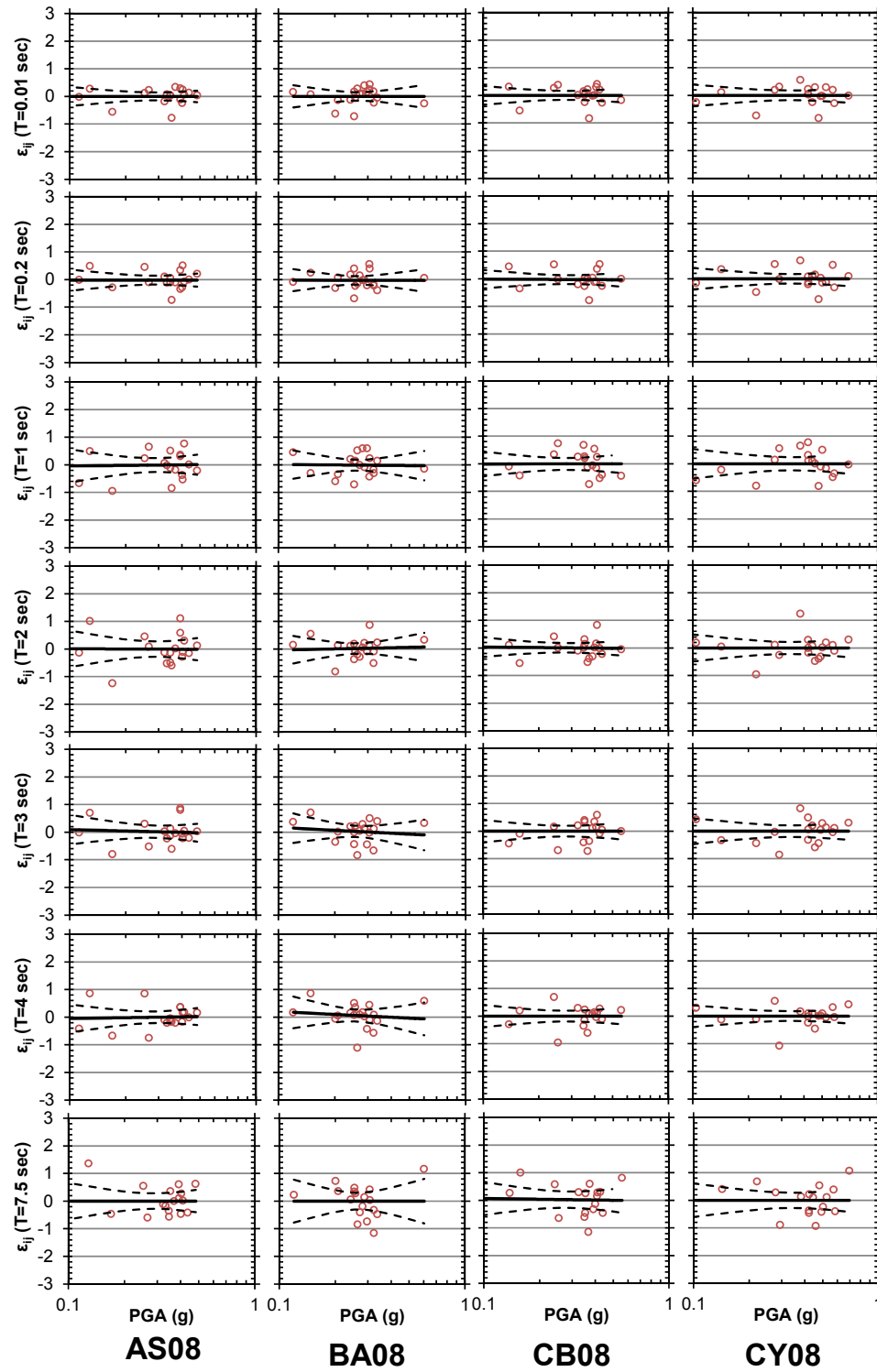


Figure 4-15: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of PGA.

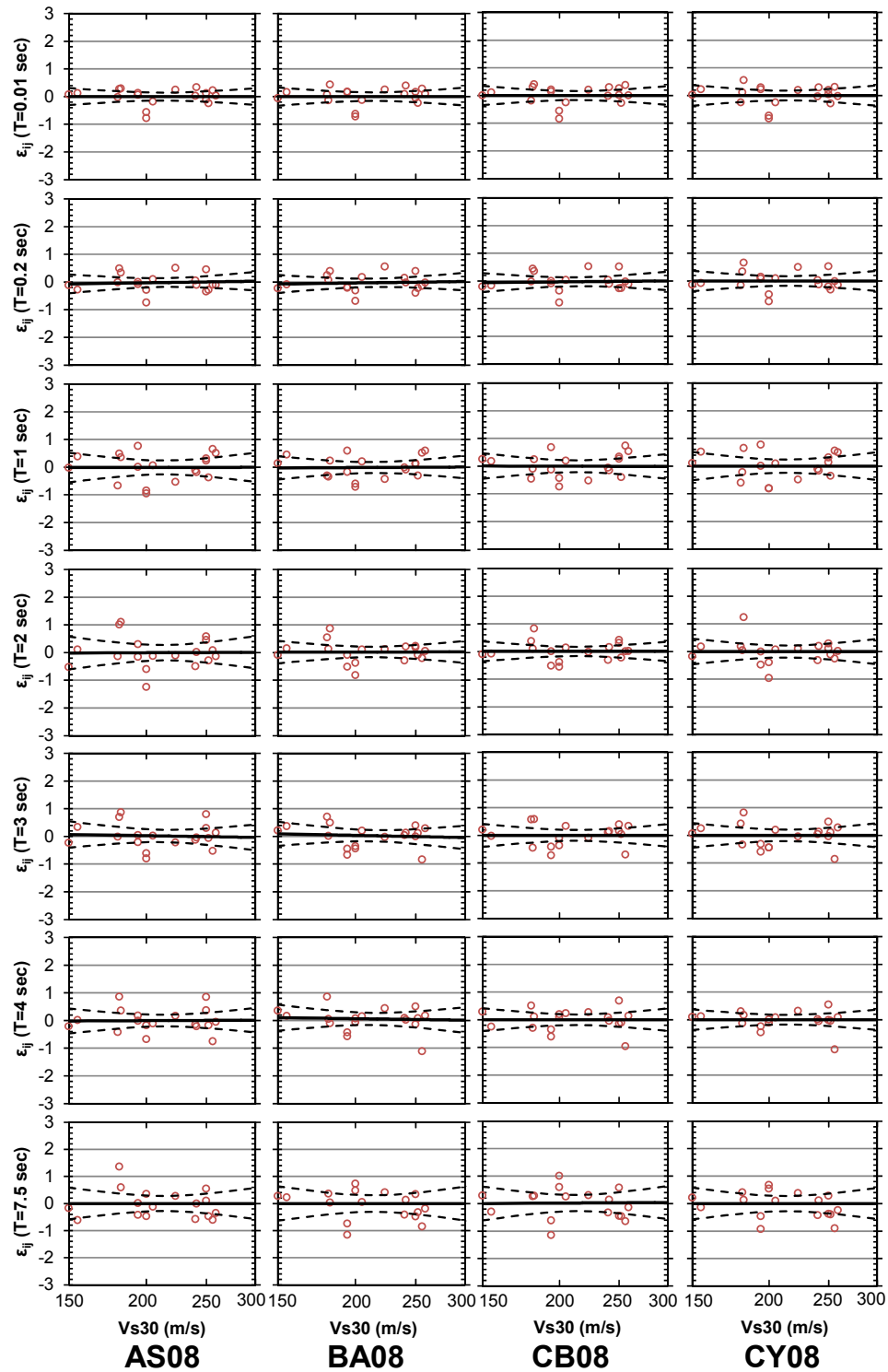


Figure 4-16: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus V_{s30} .

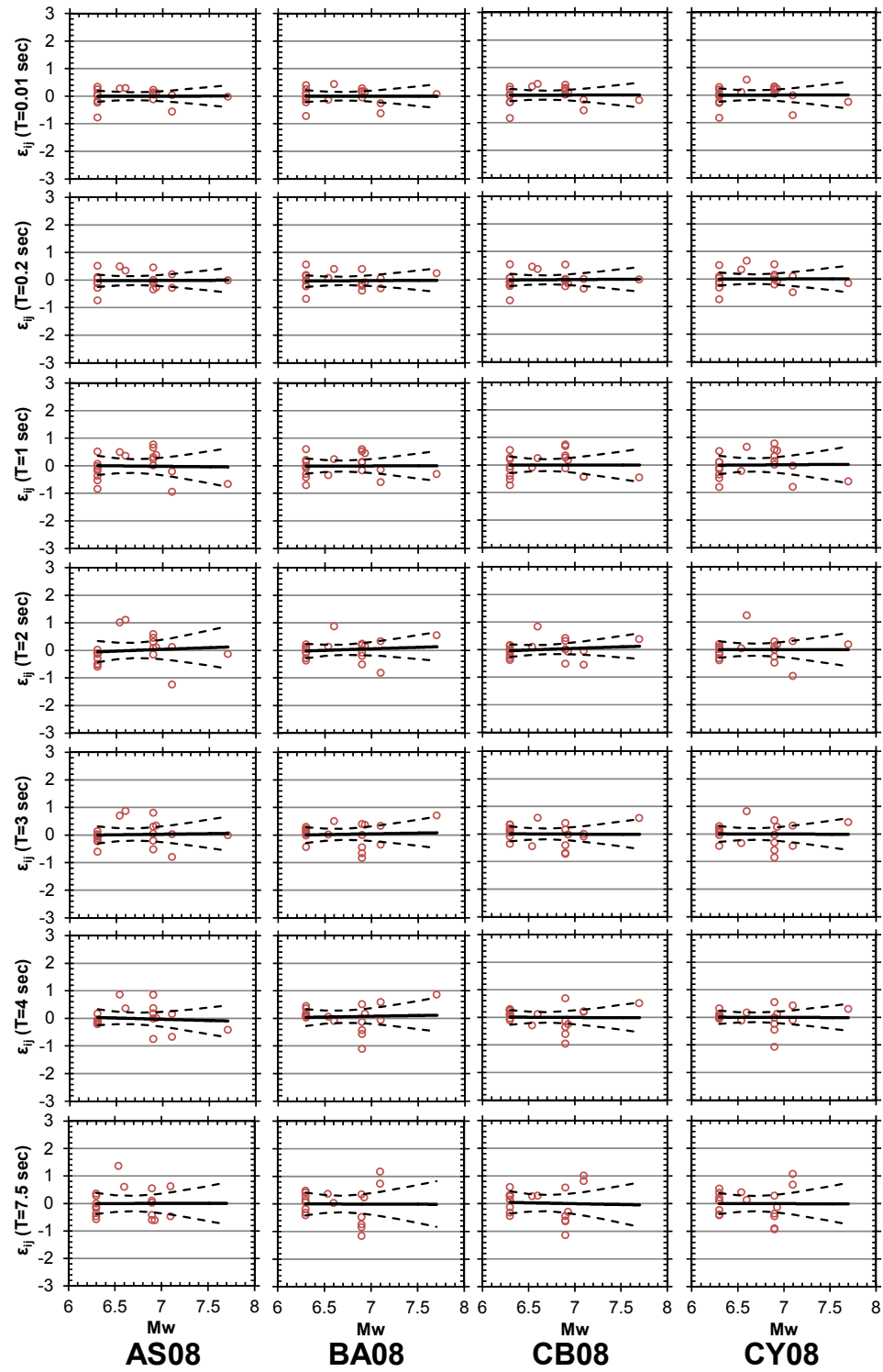


Figure 4-17: Proposed Model Intra-Event residuals, linear regression (solid line) and 95% confidence interval (dashed) plotted versus NGA prediction of M_w .

Figure 4-18 presents examples of response spectra computed using two of the NGA GMPEs with and without the adjustment by the proposed model. Three strike slip scenarios were used that produce PGA values spanning the range of the model.

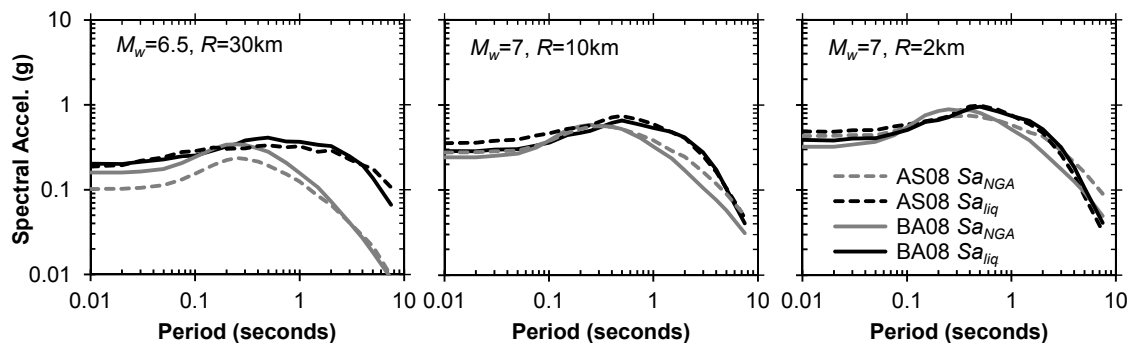


Figure 4-18: Response spectra computed using AS08 and BA08 NGA GMPEs with and without the adjustment by the proposed liquefaction amplification model. $V_{s30}=200$ m/s for all cases..

4.3 Conclusions

The GMPE residual analysis results from this study are consistent with earlier findings (i.e., Youd and Carter 2005; Hartvigsen 2007) that the occurrence of liquefaction tends to amplify longer-period spectral accelerations relative to a non-liquefiable site that is otherwise equivalent. The positive mean long-period residuals begin at a period of about 0.4 seconds, increasing with period until it either plateaus or descends at a period of about 2 or 3 seconds. Maximum mean long period amplification factors range from about 1.8 to 3.1 for the four NGA GMPEs in this study.

Mean residuals with a slightly positive bias were observed at short periods (i.e., $T \leq 0.05$ seconds), suggesting that short period spectral acceleration and

PGA may also be amplified by liquefaction. This observed amplification may be the result of acceleration spikes that occur in association with the dilational part of liquefaction phase transformation behavior.

A trough-like trend is present in the GMPE mean residual data at intermediate periods between about 0.05 to 0.4 seconds. For most of the GMPEs, the trough dips to small negative residual values (deamplification). The combination of the mid-period deamplification trough and long period amplification is characteristic of a period shift that occurs as a result of soft-site amplification (i.e., Silva et al. 1999). The amplification factors at the minima of the troughs for the four GMPEs ranged from 0.82 to 1.11.

Intra-event residual standard deviation values for the liquefaction ground motion dataset are similar to those of the GMPEs for short to moderate periods. However, in the longer period ranges where liquefaction amplification is pronounced, higher levels of dispersion occur, and the standard deviation values are generally greater than those of the GMPEs. This is not surprising, because NGA GMPEs exclude parameters that could affect liquefaction site response and thus reduce residual dispersion, such as liquefaction layer thickness and relative density.

Multi-linear least squares regression was performed to develop simple preliminary amplification functions for NGA GMPEs. These functions can be used as an “add-on” to the 2008 NGA GMPEs to modify response spectra for liquefaction effects, in a manner similar to directivity modifications (e.g., Somerville et al. 1997). The modification is made using Eqns. 4-15 and 4-16, and

the coefficients in Tables 4-3 and 4-4. A spreadsheet implementing the proposed model can be downloaded at this link: <https://www.dropbox.com/sh/8pyzt74dkzj63sx/6llp4KvrE8>. The proposed model is easily incorporated into the framework of DSHA or PSHA. Based on the similarity of the intra-event standard deviations calculated for this model with those of the original NGA GMPEs, it is not necessary to modify the NGA intra-event standard deviation when using the proposed model. The model should be updated with the NGA West 2 GMPEs, when they become available. In the meantime, the current model is considered reasonable for use with NGA West 2 models for cases where spectra computed by the NGA 2008 and NGA West 2 models are in good agreement.

The proposed model is based on limited available data so caution should be exercised in its use. Moreover, the model is not recommended for use outside of the bounds of the parameters in the liquefaction database: shallow crustal earthquakes in active tectonic regions; NGA-estimated PGA of 0.1 g to 0.6 g ; V_{s30} within 150 m/s to 260 m/s; and M_w between 6.3 and 7.7. It is recommended to perform site-specific nonlinear effective stress response analysis at F Sites. However, the proposed model is useful in that it provides a preliminary estimate of the expected modification of the ground motions at a site due to liquefaction.

4.4 Acknowledgment

Chapter 4, in part, contains material from an article published in *Earthquake Spectra* (Gingery et al. 2014). This material was used with permission from the

Earthquake Engineering Research Institute. The dissertation author was the primary author of this paper.

4.5 References

- Abrahamson, N.A. and Silva, W.J. (2008). "Summary of the Abrahamson and Silva NGA Ground-Motion Relations," *Earthquake Spectra*, 24 (1), pp. 67-97.
- Abrahamson, N.A., Atkinson, G., Boore, D., Bozorgina, Y., Campbell, K., Chiou, B., Idriss, I., Silva, W. and Youngs, R. (2008). "Comparison of the NGA ground motion relationships," *Earthquake Spectra*, 24(1), pp 45-66.
- Abrahamson, N.A. and Youngs, R.R. (1992). "A stable algorithm for regression analyses using the random effects model," *Bulletin of the Seismological Society of America*, 82(1), pp 505-510.
- American Association of State Highway and Transportation Officials (AASHTO) (2011). *AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition*.
- American Society of Civil Engineers (ASCE) (2010). *Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)*.
- Baker, J.W. (2007). "Quantitative classification of near-fault ground motions using wavelet analysis," *Bulletin of the Seismological Society of America*, 97(5), pp. 1486-1501.
- Boore, D.M. and Atkinson, G.M. (2008). "Ground-Motion Prediction equations for the average horizontal component of PGA, PGV, and 5%-damped PSA at spectral periods between 0.01 s and 10.0 s," *Earthquake Spectra*, 24 (1), pp. 99-138.
- Campbell, K.W. and Bozorgnia, Y. (2008), NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 0s, *Earthquake Spectra*, 24,(1), pp. 139-171.
- Caltrans (2012). Methodology for Developing Design Response Spectrum for use in Seismic Design Recommendations.

- Chiou, B. S, and Youngs, R. R., (2008), "An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra," *Earthquake Spectra*, 24 (1), pp. 173-215.
- Hayden, C., Bray, J.D., and Abrahamson, N.A. (2014). "Selection of Near-Fault Pulse Motions," *J. of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 140(7).
- Hartvigsen, A.J. (2007). Influence of Pore Pressures in Liquefiable Soils on Elastic Response Spectra. Master's thesis, University of Washington.
- Kostadinov, M.V. and Towhata, I. (2002). "Assessment of liquefaction-inducing peak ground velocity and frequency of horizontal ground shaking at onset of phenomenon," *Soil Dyn. and Eq. Engr*, Vol. 22, pp 309-322.
- Pease, J.W. and O'Rourke, T.D. (1997). "Seismic response of liquefaction sites," *J. Geotech. and Geoenv. Engr.*, ASCE 123(1), pp. 37-45.
- Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Y., Boyd, O.S., Perkins, D.M., Luco, N., Field, E.H., Wills, C.J., and Rukstales, K.S. (2008), Documentation for the 2008 Update of the United States National Seismic Hazard Maps, U.S. Geological Survey Open-File Report 2008-1128, 61 p.
- Power, M., Chiou, B., Abrahamson, N., Bozorgina, Y., Shantz, T. and Roblee, C. (2008). "An overview of the NGA project," *Earthquake Spectra*, EERI, 24(1), pp. 3-21.
- Silva, W., Li, S., Darragh, B., Gregor, N. (1999). Surface geology based strong motion amplification factors for the San Francisco Bay and Los Angeles areas, Report to PG&E/CEC/Caltrans by Pacific Engr. and Analysis, 118 pp.
- Somerville, P., Smith, N., Graves, R., Abrahamson, N.A. (1997). "Modification of empirical strong ground motion attenuation relationships to include the amplitude and duration effects of rupture directivity," *Seismological Research Letters*, 68(1), pp. 199-222.
- Spudich, P. and Chiou, B. (2008). "Directivity in NGA earthquake ground motions: analysis using isochrone theory," *Earthquake Spectra*, 24(1), pp 279-298.
- Spudich, P., Bayless, J.R., Baker, J.W., Chiou, B.S.J., Rowshandel, B., Shahi, S.K., Somerville, P. (2013). "Final report of the NGA-West2 Directivity

Working Group,” Pacific Earthquake Engineering Center Report PEER 2013/09.

- Youd, T.L. and Carter, B.L. (2005). “Influence of soil softening and liquefaction on spectral acceleration,” *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 131(7).
- Zeghal, M. and Elgamal, A. (1994). “Analysis of site liquefaction using earthquake records,” *J. Geotech. and Geoenvr. Engr.*, ASCE, 120(6), pp. 996-1017.

Chapter 5 Site Response Model Calibration and Validation

5.1 Introduction

This chapter describes characteristics of the site response model used to perform the parametric analyses described in Chapter 6. Element-level calibrations which were performed for both drained and undrained conditions are described. As part of this calibration, a new method is presented for assuring that modulus reduction curves are compatible with soil shear strength. The calibrated model is then verified against exact solutions for the linear case, and validated against down-hole array recording case histories.

5.2 Numerical Model Description

5.2.1 OpenSees Finite Element Program

OpenSees (Open System for Earthquake Engineering Simulation) is an open-source software platform for performing finite element and other engineering analyses (Mazzoni, et al. 2010). Its development has been coordinated by the Pacific Earthquake Engineering Center (PEER) with support from the National Science Foundation (NSF). OpenSees can be used to analyze linear and non-linear soil, structure, and soil-structure systems under static and dynamic conditions. As an open-source program, numerous developers have contributed and continue to contribute to the capabilities of OpenSees. OpenSees uses the fully programmable scripting language *tcl* for input files, which provides a great deal of power and flexibility. Many of the original geotechnical capabilities, in terms of elements and constitutive models, were

developed by researchers at UCSD (Yang et al. 2008). All of the finite element simulations performed for this research used the OpenSees software, and UCSD elements and constitutive models.

5.2.2 UCSD Soil Constitutive Models

5.3 PDMY2 and PIMY Models and their Calibration

The PressureIndependentMultiYield (PIMY) and PressureDependentMultiYield02 (PDMY2) models are based on the multi-surface plasticity framework of Prevost (1985) and have been developed and implemented in OpenSees by researchers at UCSD (Elgamal et al. 2002, 2003; Yang et al. 2003; Yang et al. 2008). A brief description of these models is provided below and a more detailed version is provided in Appendix B.

The PIMY model exhibits plasticity only in the deviatoric stress-strain response, and the volumetric stress-strain response is elastic and independent of the deviatoric response. The yield surfaces in the PIMY model are defined either as Drucker-Prager type for frictional soils or von Mises type for cohesive soils. The PIMY model is suitable for use with soils whose behavior is insensitive to changes in mean effective stress, such as the undrained response of clays. An example of the cyclic shear stress-strain response in a 2-D quadrilateral finite element using the PIMY model is presented in Figure 5-1. In this figure, τ_{xh} is the shear stress on the xy (vertical and horizontal) plane, σ'_{v0} is the initial vertical effective stress and γ_{xy} is the engineering shear strain.

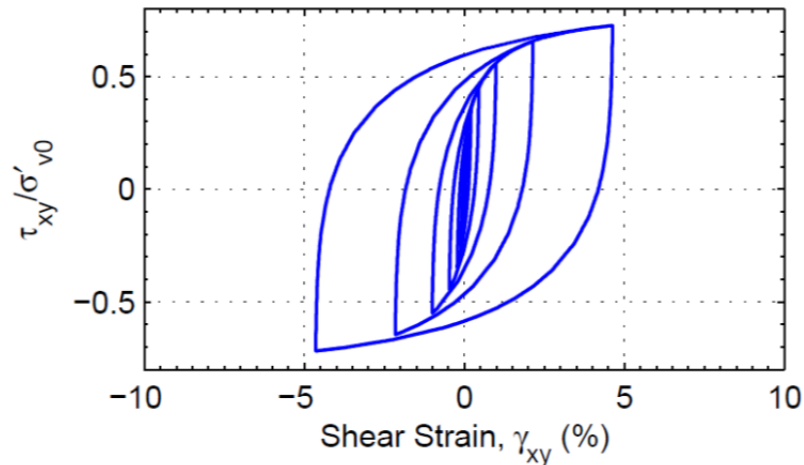


Figure 5-1: Example of the hysteretic shear stress-strain response of the PIMY model.

The PDMY2 model is differentiated from the PIMY model in its use of a non-associative flow rule that allows for volumetrically contractive or dilative response due to shear loading. When employed with fluid-solid finite elements, the contractive/dilative response causes porewater pressure changes (excluding fully drained response scenarios). The model employs a strain-space mechanism that controls the cyclic accumulation of shear strain under liquefaction cyclic mobility conditions. These features permit modeling of salient characteristics of undrained or partially-drained cyclic response of liquefiable soils, such as shear-induced contraction and dilation, excess porewater pressure development, and cyclic mobility.

The PDMY2 model was calibrated herein in a way that was optimized for the conducted 1-dimensional site response analyses. The calibration was performed at the element level with focus on obtaining reasonable fidelity to widely-accepted non-linear response relationships in both drained and undrained

cases. Previous model calibration (Yang and Elgamal 2008, Yang et al. 2008) work had focused on response under undrained conditions. Since this current study evaluates differences between site response with liquefaction (undrained) and without liquefaction (drained), the drained condition is equally important and warranted further re-calibration of the model.

5.3.1 Drained Response

The first step in the model calibration was to obtain reasonable response in drained cyclic simple shear (CSS) element tests. The drained CSS calibration focused on: 1) obtaining a reasonable match to shear modulus reduction and material damping curves that are widely accepted and frequently used in research and practice; and 2) representing the soil shear strength accurately.

5.3.1.1 Internally-Generated G/G_{max} curves in the PDMY2 and PIMY Models

The hyperbolic model for shear stress versus shear strain backbone curves has found wide use in fitting laboratory test data (Konder and Zelasko 1963; Hardin and Drnevich 1972; Darendeli 2001; Menq 2003; Roblee and Chiou 2004) and in nonlinear site response analysis programs (e.g., Matasovic 1993; Hashash and Park 2001; Yang et al. 2003). The hyperbolic model proposed by Hardin and Drnevich (*HD*) for shear modulus reduction (G/G_{max}) as a function of γ has the form (Hardin and Drnevich 1972):

$$\left(\frac{G}{G_{max}} \right)_{HD} = \frac{1}{1 + \frac{\gamma}{\gamma_{ref}}} \quad (5-1)$$

where the reference strain, $\gamma_{ref} = \tau_{ff} / G_{max}$, τ_{ff} is the shear stress at failure (peak or maximum shear stress) and G_{max} is the small strain maximum shear modulus. With $\tau = \gamma \times G$ it is apparent that Eqn. (5-1) results in τ approaching τ_{ff} asymptotically at large values of γ (at $\gamma = \text{infinity}$). This form of the hyperbolic model has the advantage that it can be easily calibrated to satisfactorily match shear strength, but it is often inconsistent with laboratory test data at small and moderate shear strains, as shown later below.

In the PDMY2 and PIMY model, shear stress-shear strain response is either governed by internally-generated shear modulus reduction ($G/G_{max} - \gamma$) curves, or by user defined $G/G_{max} - \gamma$ curves. The internally-generated $G/G_{max} - \gamma$ curves are based on the Hardin and Drnevich (1972) model, but with an additional term that allows for confinement or pressure-dependent scaling:

$$\tau_{oct} = \frac{G_{max,oct} \gamma_{oct}}{1 + \frac{\gamma_{oct}}{\gamma_{ref}} \left(\frac{p'_r}{p'} \right)^d} \quad (5-2)$$

where $G_{max,oct} = \tau_{oct} / \gamma_{oct}$ is the octahedral shear modulus, p'_r is the reference pressure, p' is the mean effective stress and the exponent d is typically taken as 0.5. The octahedral shear stress (τ_{oct}) and strain (γ_{oct}) are defined as:

$$\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{xx} - \sigma_{zz})^2 + (\sigma_{yy} - \sigma_{zz})^2 + 6\sigma_{xy}^2 + 6\sigma_{yz}^2 + 6\sigma_{xz}^2} \quad (5-3)$$

$$\gamma_{oct} = \frac{2}{3} \sqrt{(\varepsilon_{xx} - \varepsilon_{yy})^2 + (\varepsilon_{xx} - \varepsilon_{yy})^2 + (\varepsilon_{xx} - \varepsilon_{yy})^2 + 6\varepsilon_{xy}^2 + 6\varepsilon_{yz}^2 + 6\varepsilon_{xz}^2} \quad (5-4)$$

For the simple shear conditions used in the site response analyses performed for this study, the shear strain on the xy plane, $\gamma_{xy} = \varepsilon_{xy} \times 2$ and all other strain components of γ_{oct} are zero, such that $\gamma_{oct} = (\sqrt{6}/3)\gamma_{xy}$. If a condition of $K_0 = 1$ is assumed, then $\tau_{oct} = (\sqrt{6}/3)\sigma_{xy} = (\sqrt{6}/3)\tau_{xy}$ and $G_{oct} = \tau_{oct}/\gamma_{oct} = \tau_{xy}/\gamma_{xy} = G$. The assignment of $K_0 = 1$ is convenient since it means that the conventional definitions of shear modulus and maximum shear modulus can be used without modification ($G_{oct} = G$ and $G_{oct,max} = G_{max} = \rho(V_S)^2$) and that traditional shear modulus reduction curves can be used directly without modification for octahedral conditions. If $K < 1$ then $G_{max,oct} = f(K)G_{max}$. In cyclic simple shear simulations with $K_0 < 1$, K increases as the number of cycles of loading increases and K reaches 1 prior to liquefaction is triggering. Thus, $G_{max,oct}$ will vary over the course of the loading, and a user-defined $G/G_{max} - \gamma$ curve can only produce the intended shear stress-strain backbone curve for initial small-strain cycles (when $K \approx K_0$) or for later conditions when K approaches unity.

Many liquefiable soils are geologically young (Youd and Perkins 1978) and normally consolidated, and thus are expected to have values of $K_0 \approx 0.45$. However, the fidelity of the horizontal stresses in the 1-D site response analyses

employed in this study are assumed of secondary minor importance. Considering its attractive advantages and assumed minor/minimal disadvantages, the $K_0=1$ condition assumption was used in the calibration and site response work performed in this study. Heretofore, G , γ and $G_{\max}$ are used interchangeably for G_{oct} , γ_{oct} and $G_{\max,oct}$, respectively.

Considering that $G = \tau / \gamma = \tau_{oct} / \gamma_{oct}$ and rearranging terms in Eqn. (5-2) yields:

$$\frac{G}{G_{\max}} = \frac{1}{1 + \frac{\gamma}{\gamma_{ref}} \left(\frac{p'_r}{p'} \right)^d} \quad (5-5)$$

In the PDMY2 and PIMY models, $G_{\max} = G_r (p'/p'_r)^d$ where G_r is reference shear modulus defined at a reference confinement p'_r , and d is a material parameter (= 0.5 typically for sand, Kramer 1996). Similarly, the bulk modulus has the form $B = B_r (p'/p'_r)^d$.

The reference shear strain can be selected such that the shear stress is equal to the shear strength (τ_f , expressed in terms of p'_r and the drained internal friction angle of the model, ϕ) at the reference pressure and failure shear strain ($\gamma_{\max}$) (Yang et al. 2008):

$$\tau_f = \frac{2\sqrt{2}p'_r \sin \phi}{3 - \sin \phi} = \frac{G_r \gamma_{\max}}{1 + \gamma_{\max} / \gamma_{ref}} \quad (5-6)$$

or

$$\gamma_{ref} = \frac{2\sqrt{2} \sin \phi \cdot p'_r \gamma_{max}}{G_r \gamma_{max} - 2\sqrt{2} \sin \phi \cdot p'_r} \quad (5-7)$$

Figure 5-2 presents examples of the PDMY2 model internally generated $G/G_{max} - \gamma$ curves along with curves by EPRI (1993) and Darendeli (2001). The curves are shown for mean initial confining pressures (p'_0 , in atmospheres). The Darendeli curves were calculated using Plasticity Index $PI=0$ and Over Consolidation Ratio $OCR = 1.0$. It is evident that the EPRI and Darendeli curves are relatively consistent, while the internally generated PDMY2 curves are significantly stiffer.

Thus, the formulation of the hyperbolic-relation internally-generated PDMY2 and PIMY curves is such that they capture shear strength at large strain, but it is not possible to fully control the shape of the $G/G_{max} - \gamma$ curve at low to moderate strains. For the site response analyses performed in this study, the ability to control the curves in the low to moderate shear strain range was considered important. Therefore, it was decided to use the user-defined shear modulus reduction curve option in the PDMY2 and PIMY models.

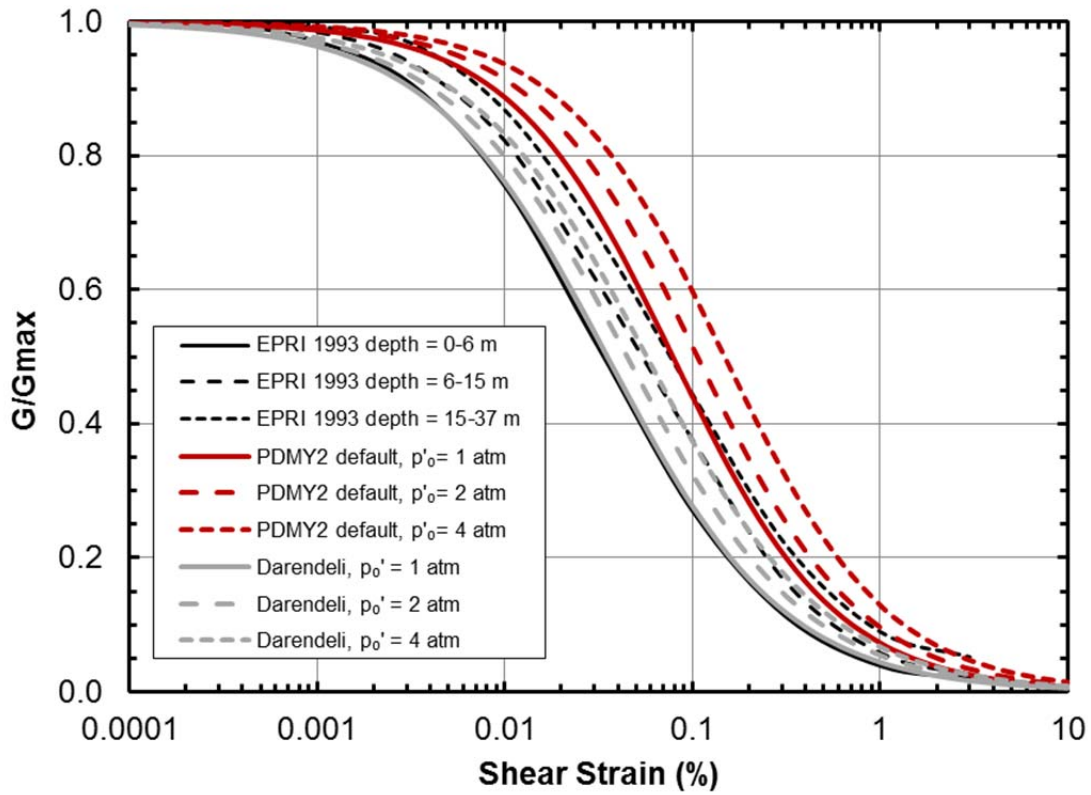


Figure 5-2: Comparison of G/G_{max} curves

5.3.1.2 Strength-Compatible User-Defined G/G_{max} Curves

5.3.1.2.1 Background and currently available relationships

Darendeli (2001), Menq (2003) and Roblee and Chiou (2004) used an alternate form of the hyperbolic model which provides added flexibility that allows a better fit to laboratory test data at small to moderate strains:

$$\left(\frac{G}{G_{max}} \right)_{PRSH} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r} \right)^\alpha} \quad (5-8)$$

where γ_r is a “pseudo-reference strain” and α is an additional fitting parameter. Models of the form of Eqn. (5-8) are referred to herein as “pseudo-reference strain hyperbolic” (PRSH) models. The form of Eqn. (5-8) is such that a value of $(G/G_{\max})_{PRSH} = 1/2$ occurs at $\gamma = \gamma_r$. Darendeli (2001) developed a family of $G/G_{\max} - \gamma$ curves based on regression of data from numerous resonant column torsional shear (RCTS) tests conducted at the University of Texas, Austin. The parameter α was set at 0.919 and values of γ_r were reported as a function of soil plasticity index (PI), mean confining stress and overconsolidation ratio (OCR). Menq (2003) used the same form of Eqn. (5-8) to fit RCTS test data for sands and gravels with values of γ_r and α calculated as a function of uniformity coefficient (C_u) and mean effective stress. Roblee and Chiou (2004) used Darendeli’s dataset, but developed values of γ_r and α for three soil-type groups and six depth bins.

The laboratory test data used to curve-fit these PRSH model coefficients becomes sparse to non-existent at moderate to high shear strain levels that are associated with soil shear strengths. For instance, Figure 5-3 (a) from Darendeli (2001) shows that very little RCTS data was available above a shear strain of 0.3 percent, while Figure 5-3 (b) from Ishihara (1996) shows that the failure strain range begins at strains on the order of 2 to 3 percent. Therefore, it is evident that shear strains achieved in most of the RCTS tests were not sufficient to mobilize the full shear strength of the soil. Thus, PRSH models developed from such

laboratory testing data sets require further information in the large shear strain regime in order to provide accurate representation of shear strength and the corresponding nonlinear soil response.

For the purpose of the study herein, shear strength on the xy -plane ($\tau_{xy,f}$) is calculated from the shear modulus degradation curve as:

$$\tau_{xy,f} = G_{\max} \cdot (G / G_{\max})_{\gamma_f} \cdot \gamma_f \quad (5-9)$$

where $(G / G_{\max})_{\gamma_f}$ is the shear modulus degradation value at a user-defined simple shear failure strain γ_f . In conducting SHAKE-type (Schnabel et al. 1972) equivalent linear analyses: i) commonly available $G/G_{\max} - \gamma$ curves are used, which include no special consideration of shear strength, and ii) $G_{\max}$ is calculated from small strain shear wave velocity information which might not strongly correlate with shear strength. Thus, any accuracy in $\tau_{xy,f}$ would only result from a fortuitous combination of $G_{\max}$ and the specified variation of $G/G_{\max}$ along the curve (at γ_f , beyond the range of the 0.3% or so data from experimentation).

The degree to which shear strength can be inaccurately estimated by a PRSH model is schematically illustrated in Figure 5-4. The figure presents $G_{\max}$ and shear strength for a loose unsaturated sand profile and a fully saturated clay profile. The sand has a relative density of 40%, a friction angle of 30.6 degrees,

and a density of 1.77 Mg/m^3 . The clay has a PI of 30, is normally consolidated, and has a saturated density of 1.77 Mg/m^3 . The shear strength used in the sand profile is based on the Mohr-Coulomb definition for frictional materials, $\tau_{xy,f} = \sigma'_{v0} \tan(\phi_{DSS})$ (Holtz and Kovacs 1981), where σ'_{v0} , is the initial vertical effective stress and ϕ_{DSS} is the friction angle in direct simple shear. Shear strength in the clay profile was calculated using the SHANSEP method, where $\tau_{xy,f} = S_u = S \times OCR^{0.8}$ (Ladd and Foot 1974), with $S = 0.22 \times 80\% = 0.18$ computed based on the recommendations of Idriss and Boulanger (2008) for cyclic direct simple shear in clay (i.e., $S_{u,DSS}$, the shear strength used in the profile) and $S = 0.31$ based on recommendations (Ladd and DeGroot 2003) for triaxial compression (i.e., $S_{u,TXC}$, the shear strength used to compute $G_{\max}$ as described later).

The sand profile used $G_{\max} = 1000K(p')^{0.5}$ with $K = 40$ according to Seed and Idriss (1971) while the clay profile used $G_{\max}/S_{u,TXC} = 700$ for $PI = 30$ clay based on Weiler (1988). In Figure 5-4, the shear strength implied by the Darendeli (2001) PRSH model ($\tau_{xy,f,PRSH}$) is calculated using Eqns. (5-8) and (5-9) with $\gamma = \gamma_f = 6\%$ (results are similar for other values of γ_f).

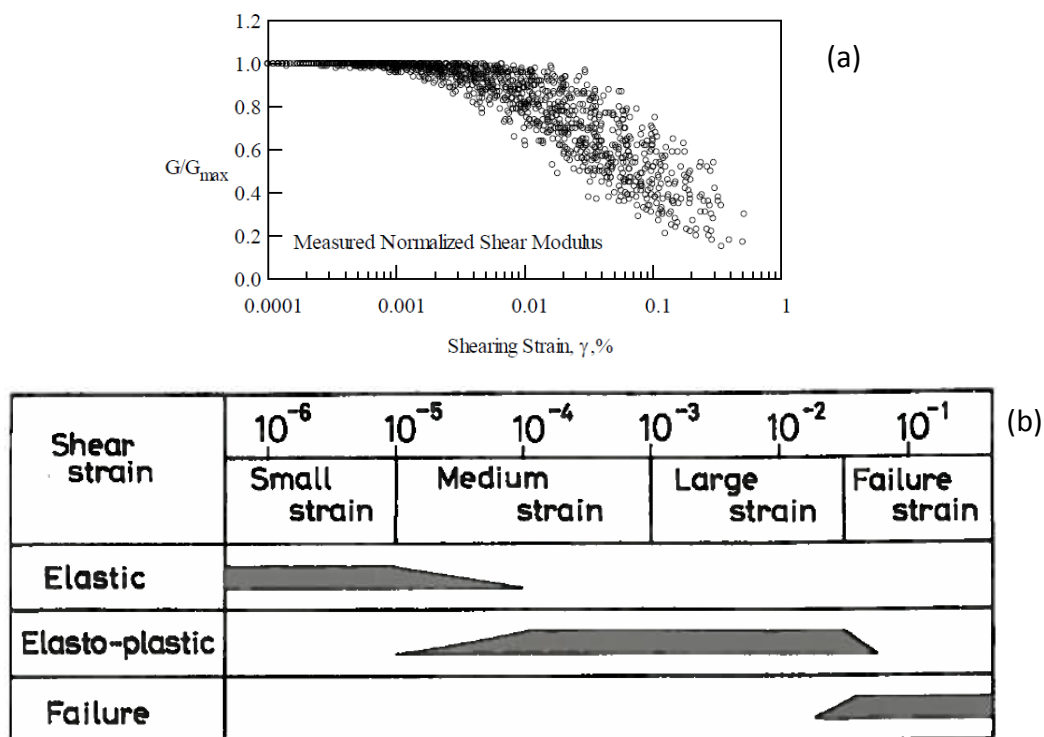


Figure 5-3: (a) RCTS data used in development of Darendeli (2001) G/G_{max} curves and (b) shear strain ranges (Ishihara 1996).

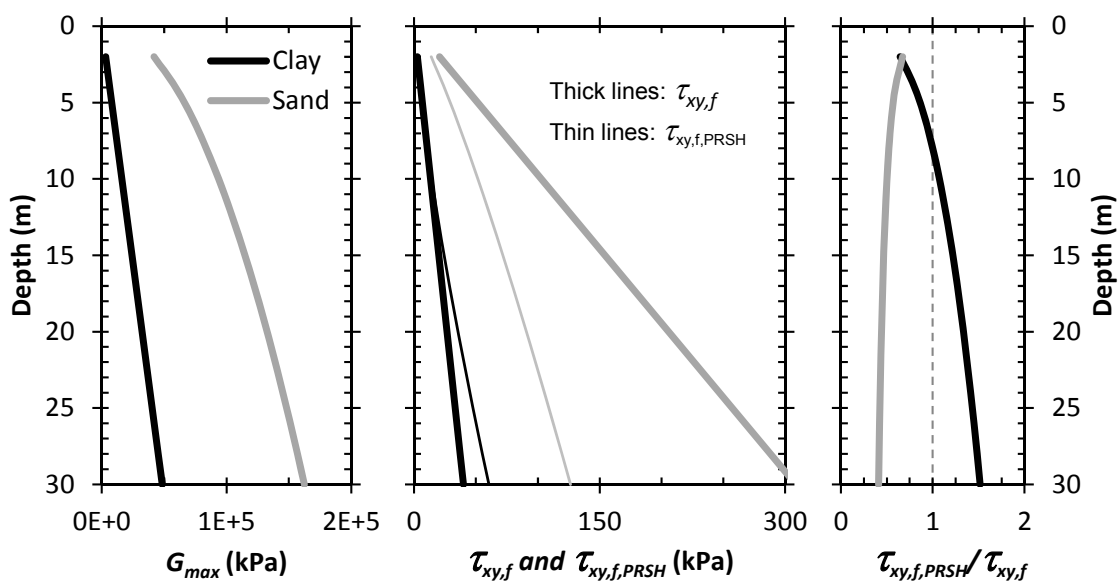


Figure 5-4: Example sand and clay profiles illustrating differences between estimated shear strengths, and shear strengths implied by a PRSH model (as calibrated by low/moderate shear strain levels).

It is evident from Figure 5-4 that the shear strength implied by the Darendeli (2001) PRSH model is 41% to 67% of the actual shear strength for the sand profile. For the clay profile, the Darendeli (2001) implied shear strengths are 64% to 152% of the actual shear strengths. Such shear strength discrepancies could result in significant errors for equivalent linear or non-linear site response analyses where shear strains occur that approach or exceed the shear strength. Similar results can be obtained from any PRSH model (on account of the limitations of curve fitting as dictated by the underlying hyperbolic-model logic).

To address this important issue, Stewart et al. (2008) recommended a $G/G_{\max} - \gamma$ curve adjustment procedure in which Eqn. (5-8) is used below a shear strain of 0.1 to 0.3%, Eqn. (5-1) is used at strain greater than the failure strain, and an interpolation scheme is used to transition from Eqn. (5-8) to (5-1) at intermediate shear strains. However, in some instances, the Stewart et al. (2008) interpolation scheme unintentionally introduces a superfluous “kink” in the shear stress-strain backbone curve. If so, manual smoothing of the kink is required, which presents difficulties for implementation within an automated computer code. Yee et al. (2013) extended this approach by formulating a version of Eqn. (5-1) that assured a smooth transition between the two backbone curves.

5.3.1.2.2 New $G/G_{\max}$ Relationship

A new interpolation scheme is presented herein (Gingery and Elgamal 2013) that produces $G/G_{\max} - \gamma$ curves that simultaneously match: 1) the PRSH models where they are well constrained by data at small to moderate strains; and 2) the shear strength at large strains. The proposed model provides a smooth, “kinkless” hyperbolic-like curve to transition between moderate and failure-level shear strains. The Darendeli (2001) model is used in this study, but the proposed modification scheme can be applied to any PRSH model of the form shown in Eqn. (5-8) (such as Roblee and Chiou 2004 or Menq 2007). The proposed “GH model” scales Eqn. (5-8) using a raised cosine function to force the curve to intercept a $G/G_{\max}$ value at γ_2 (i.e., the failure strain) that produces the correct shear strength given $G_{\max}$, c' , ϕ' and σ'_{v0} . The GH model uses Eqn. (5-8) for $\gamma < \gamma_1$. Between γ_1 and γ_2 Eqn. (5-8) is multiplied by the raised cosine weighting function (W), and for $\gamma > \gamma_2$ values of $G/G_{\max}$ are provided that result in $\tau = \tau_{ff}$, viz.:

$$\left(\frac{G}{G_{\max}} \right)_{GH} = W \cdot \left(\frac{G}{G_{\max}} \right)_{PRSH} \quad (5-10)$$

where

$$W = \begin{cases} = 1 & \text{for } \gamma \leq \gamma_1 \\ = 1 + \left[\frac{(G/G_{\max})_{\tau_{xy,f}, \gamma=\gamma_2}}{(G/G_{\max})_{PRSH, \gamma=\gamma_2}} - 1 \right] \cdot \left\{ \frac{1}{2} - \frac{1}{2} \cos \left[\pi \frac{\ln(\gamma) - \ln(\gamma_1)}{\ln(\gamma_2) - \ln(\gamma_1)} \right] \right\}^n & \text{for } \gamma_1 < \gamma \leq \gamma_2 \\ = \frac{\tau_f}{\gamma G_{\max} (G/G_{\max})_{PRSH, \gamma}} & \text{for } \gamma > \gamma_2 \end{cases} \quad (5-11)$$

On this basis, the $G/G_{\max}$ value that provides the correct shear strength (τ_{ff}) at γ_2 is:

$$\left(\frac{G}{G_{\max}} \right)_{\tau_{xy,f}, \gamma=\gamma_2} = \frac{\tau_{xy,f}}{\gamma_2 \cdot G_{\max}} \quad (5-12)$$

and $(G/G_{\max})_{PRSH, \gamma=\gamma_2}$ is the shear modulus reduction value predicted by the PRSH model at γ_2 . The exponent n can be used to manipulate the shape of the raised cosine weighting function, and in the current implementation n is taken as 1. The parameter γ_1 must be selected judiciously so that the adjustment does not produce $\tau_{xy}/\tau_{xy,f} - \gamma$ curves with strain-softening behavior. A value of $\gamma_1 = 0.05\%$ has been found to avoid strain softening over a fairly wide range of conditions. The shape of the $\tau_{xy}/\tau_{xy,f} - \gamma$ curves are less sensitive to the γ_2 parameter, which should be chosen at or near the shear strain where the shear strength is reached.

The proposed modification scheme is illustrated by applying it at depths of 4, 8, 16 and 30 m within the sand and clay profiles from Figure 5-4. The PRSH model G/G_{max} curves were computed using Darendeli (2001) and the soil parameters described earlier. Weighting values were calculated using Eqn. (5-11) with $\gamma_1 = 0.05\%$ and $\gamma_2 = 6\%$, and G/G_{max} values adjusted according to the proposed model were calculated using Eqn. (5-10). The results are presented for the sand profile in Figure 5-5 and Figure 5-6, and in Figure 5-7 and Figure 5-8 for the clay profile. Figure 5-5 also shows corresponding hysteretic damping curves calculated by cyclic simple shear simulations in OpenSees using Masing hysteresis rules.

The adjusted $(G/G_{max})_{GH} - \gamma$ curves gradually diverge from the $(G/G_{max})_{PSRH} - \gamma$ curves above γ_1 and converge to $(G/G_{max})_{\tau_{xy,f}, \gamma=\gamma_2}$ at γ_2 . The $\tau_{xy}/\tau_{xy,f} - \gamma$ curves maintain a smooth hyperbolic-like shape without strain softening and converge to the shear strength at γ_2 .

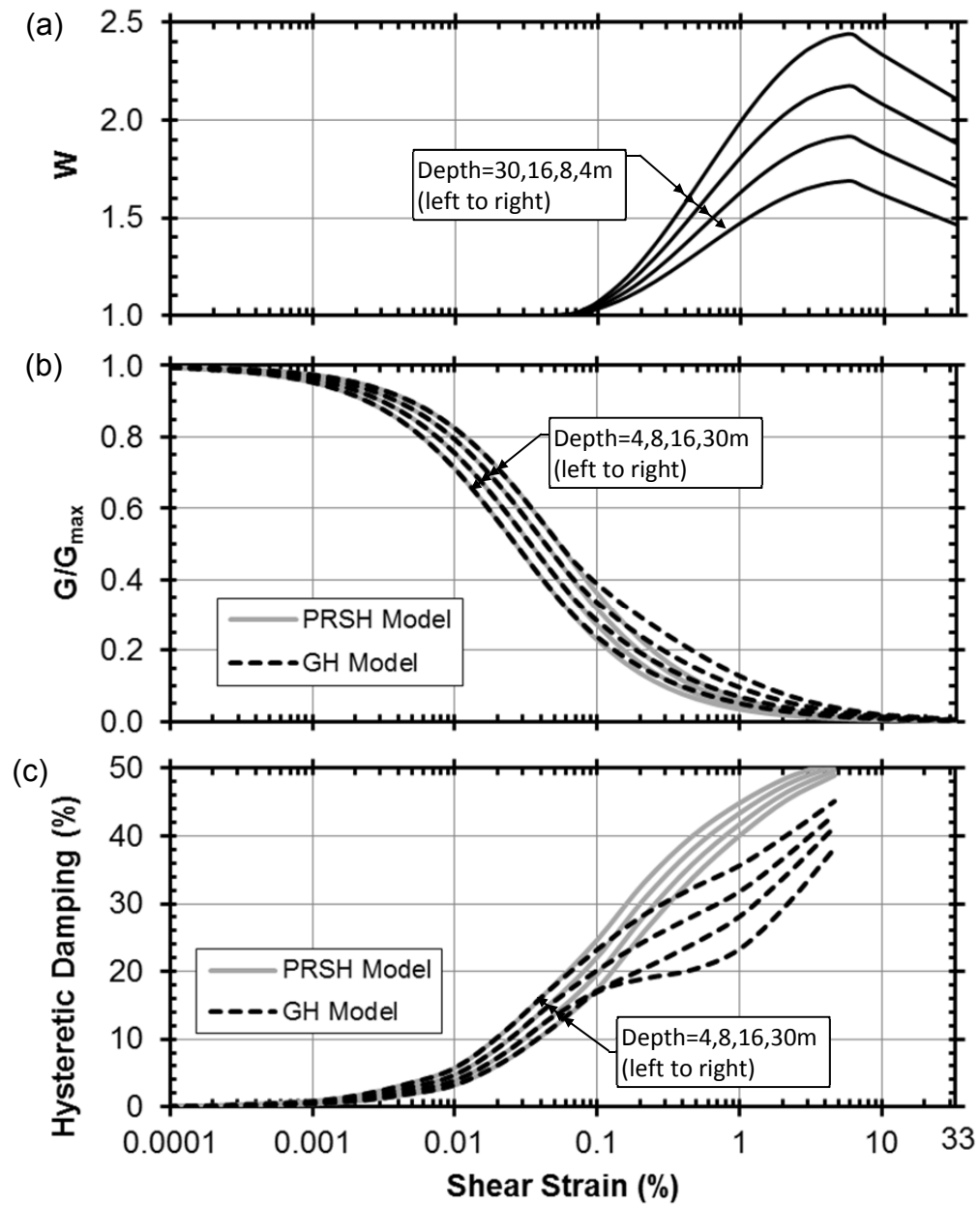


Figure 5-5: (a) Weighting values (W), (b) G/G_{max} curves and (c) hysteretic damping curves computed using the PRSH and GH models at various depths in the sand profile.

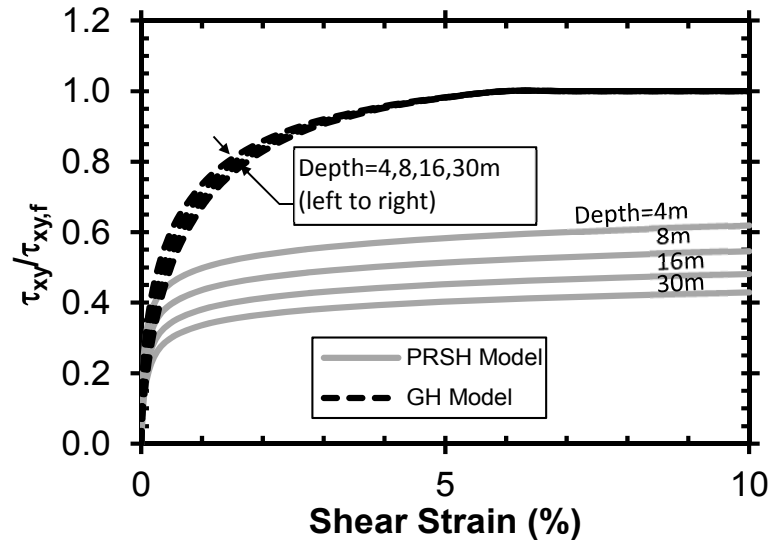


Figure 5-6 Shear stress versus shear strain curves computed using the PRSH and GH models at various depths in the sand profile.

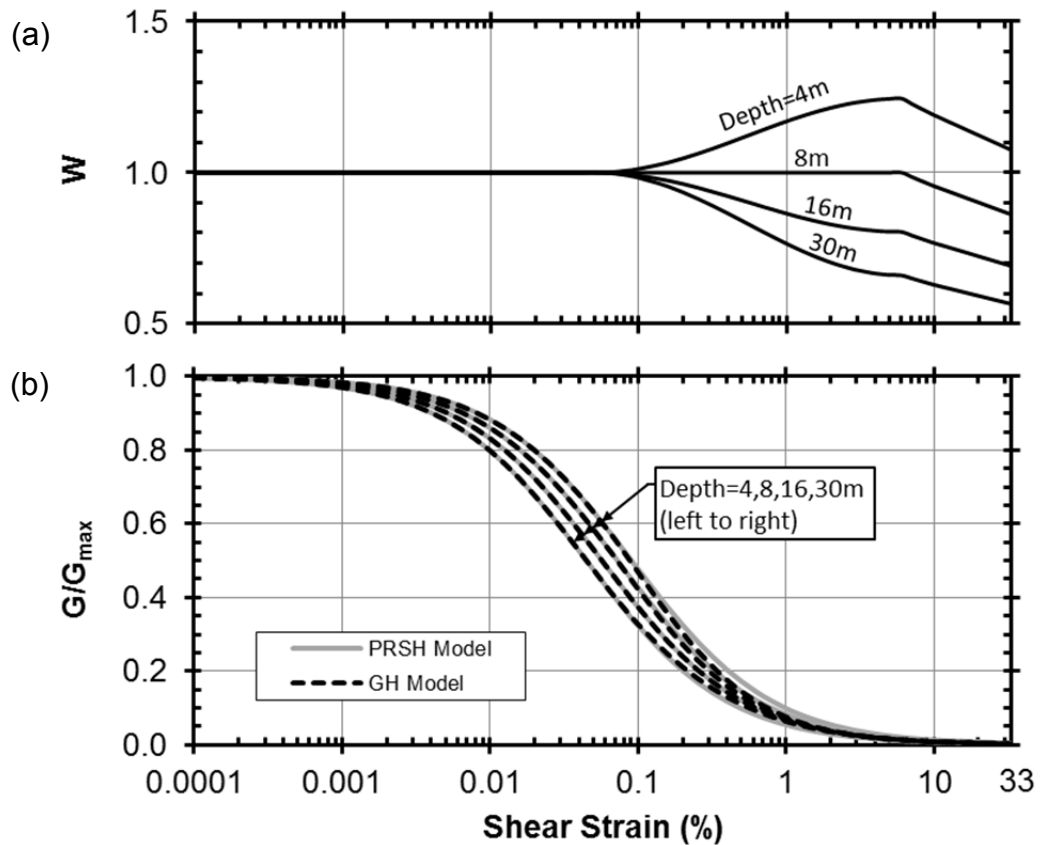


Figure 5-7: (a) Weighting values (W), and (b) G/G_{max} curves computed using the PRSH and GH models at various depths in the clay profile.

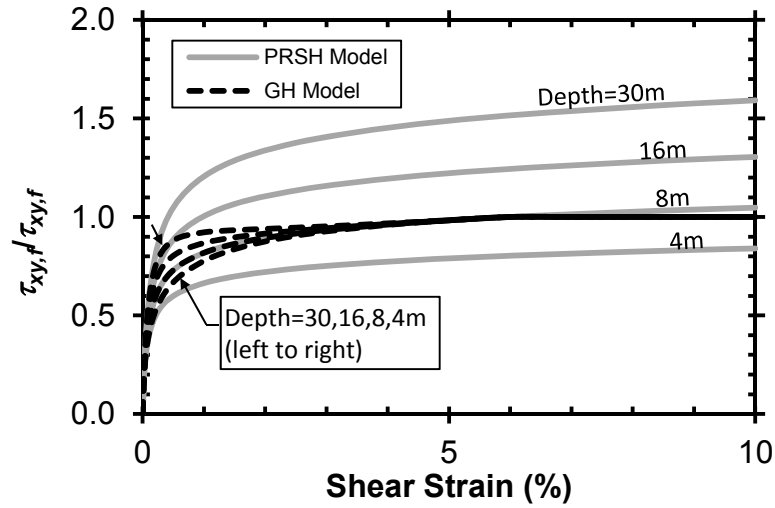


Figure 5-8: Shear stress versus shear strain curves computed using the PRSH and GH models at various depths in the clay profile.

The new method to adjust PRSH model $(G/G_{\max})-\gamma$ curves presented herein produces $\tau_{xy}/\tau_{xy,f}-\gamma$ curves that match the material shear strength exactly at large strain levels while following the PRSH models at small to moderate shear strain levels where they are well constrained by laboratory data. Provided that parameters γ_1 and γ_2 are judiciously selected, the adjusted $\tau_{xy}/\tau_{xy,f}-\gamma$ curves exhibit a smooth, hyperbolic-like shape without strain softening. The developed adjustment scheme can be easily implemented in a spreadsheet or similar tool for routine use in engineering applications.

5.3.1.3 Model Parameters and Element Calibration

The Darendeli (2001) $(G/G_{\max})-\gamma$ curves adjusted per the GH model were used for the soils in the site response modeling described in Section 6. The

values of γ_1 and γ_2 used for the calibrations in this study were 0.05% and 6%, respectively. Calibrations were performed for a generic poorly graded sand soil with $(N_1)_{60}$ values of 4.1, 7.4, 11.5, 16.6, 22.5, 29.5 and 37.3. Using the Idriss and Boulanger (2008) correlation, $D_r = \sqrt{(N_1)_{60}/46}$, the $(N_1)_{60}$ values correspond to relative densities of 30, 40, 50, 60, 70, 80 and 90 percent. The friction angle in direct simple shear (ϕ_{DSS}) was established based on a correlation with $(N_1)_{60}$ by Sabatini et al. (2002) that was developed based on the work of Hatanaka and Uchida (1996):

$$\phi_{DSS} = \sqrt{15.4(N_1)_{60}} + 20^\circ \quad (5-13)$$

Eqn. (5-13) was originally developed based on the results of triaxial compression tests. Kulhawy and Mayne (1990) proposed a relationship between the friction angle in triaxial compression (ϕ_{TXC}) and direct simple shear: $\phi_{DSS} = \tan^{-1}[\tan(1.12\phi_{TXC})\cos\phi_{CV}]$, where ϕ_{CV} is the triaxial compression constant volume friction angle. With typical ϕ_{CV} values of 31 to 33 degrees (Bolton 1986), $\phi_{DSS} = \phi_{TXC} \pm 1$ degree. Therefore, Eqn. (5-13) was considered valid for estimation of ϕ_{DSS} .

With the PDMY2 model's user-defined modulus reduction option, the final pair of $(G/G_{\max}) - \gamma$ values defines the shear strength used in the model:

$$\sin \phi = \frac{3\sqrt{3} \tau_{xy,f} / p'_r}{6 + \sqrt{3} \tau_{xy,f} / p'_r} \quad (5-14)$$

where, $\tau_{xy,f} = \sigma'_{v0} \tan \phi_{DSS}$, and $\sigma'_v = p'_r$. In this study, values of p'_r were set equal to σ'_{v0} . The relationship between direct simple shear friction angle (ϕ_{DSS}) and the friction angle used to define the failure surface in the model (ϕ) is presented in Eqn. (5-15). The derivations for Eqns. (5-14) and (5-15) are presented Appendix B.

$$\phi_{DSS} = \tan^{-1} \left[\frac{2\sqrt{3} \sin \phi}{3 - \sin \phi} \right] \quad (5-15)$$

Following Yang et al. (2008), the phase transformation angle (ϕ_{PT}) was set to a constant value. While Yang et al. (2008) used a value of 26 degrees, in the current study a value of $\phi_{PT} = 20$ degrees was found to produce good results.

Small strain shear moduli were calculated according to $G_{\max} = \rho(V_s)^2$. Since p'_r (see Eqs. 5.6 and 5.7) was set equal to σ'_{v0} , conveniently $G_r = G_{\max}$. The shear wave velocity V_s was calculated from the SPT blow count with a 60 percent hammer efficiency (N_{60}) and σ'_{v0} (in kPa) according to Brandenberg et al. (2010):

$$\ln(V_s) = 4.045 + 0.096 \ln(N_{60}) + 0.236 \ln(\sigma'_{v0}) \quad (5-16)$$

The bulk modulus was calculated based on elasticity theory, $B_r = 2G_r(1+\nu)/[3(1-2\nu)]$, where ν is Poisson's ratio. For the initial static consolidation phase in element tests and site response analyses, ν was set equal to 0.49 to produce essentially isotropic stress ($K_0 = \nu/(1-\nu) \cong 1$) conditions. In the element tests performed for this study, laterally adjacent nodes constrained to equal translational displacements ($\varepsilon_{xx} = 0$, where for these 2-D elements, x is the in-plane horizontal direction, y is the vertical direction) in order to force a simple shear deformation mode. With the horizontally constrained single elements, contraction and dilation can lead to spurious and unrealistic changes in horizontal effective stress under drained conditions. To minimize the tendency of horizontal volumetric strains, ν was reset to zero for the dynamic phase. It should be noted that porewater pressure generation rates are partially dependent upon ν , so the use of other values of ν require adjustment of the model parameters to maintain the model calibration under undrained conditions.

A series of single-element drained cyclic simple shear test simulations was performed using Opensees to verify the implementation of the specified soil shear modulus reduction and strength in the model. The tests were also used to evaluate the hysteretic damping generated by the model response. The simulations were performed using the four-node quadrilateral u-p elements (Yang et al. 2008). The bottom nodes were fixed in the x and y (horizontal and vertical) directions and the top nodes were constrained by the equal displacement

command such that a simple shear displacement mode can be enforced. The horizontal, vertical and out-of-plane dimensions of the elements were all set to 1 m. The porewater pressure degree of freedom was fixed at the bottom nodes (resulting in an impervious boundary) and free at the top nodes (allows excess porewater pressure generation). For the drained tests, the hydraulic conductivity was set to an arbitrarily high value of $1e15$ m/s to assure complete drainage during the simulation. To initialize stresses, the element behavior was first set to elastic, and the vertical forces at the top nodes representing the vertical effective stress were applied. With the initial porewater pressure equal to zero, the horizontal stresses were allowed to develop based on the vertical loading according to elasticity theory, i.e., $K_0 = \nu/(1-\nu) = 1$ approximately.

In the dynamic phase, the Poisson's ratio was set to zero and the simulations were run as strain-controlled. As shown in Figure 5-9, the applied strain function $\gamma_{xy}(t)$ was specified such that two sinusoidal cycles occurred at each strain increment, with three evenly spaced strain increments per log cycle and minimum and maximum strain levels of 10^{-5} and 4.7 percent, respectively. Strain control was achieved by applying a force ($F = k[y_{el}\gamma_{xy}(t)]$) to one of the top nodes, where y_{el} is the y-dimension of the element and k is an arbitrarily large spring stiffness (several orders of magnitude larger than the shear stiffness of the element which assures that the F is large enough to impose the desired displacement). The element configuration, boundary conditions and loading are presented in Figure 5-10.

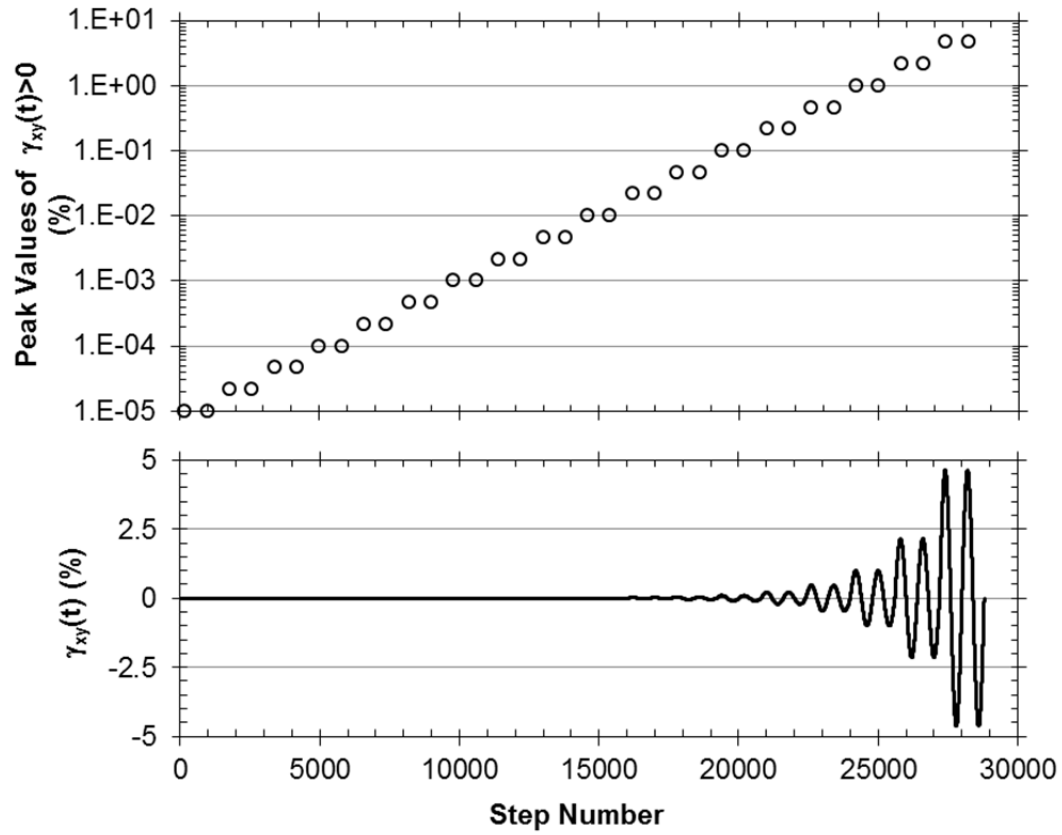


Figure 5-9: Strain-controlled loading pattern for drained CSS simulations.

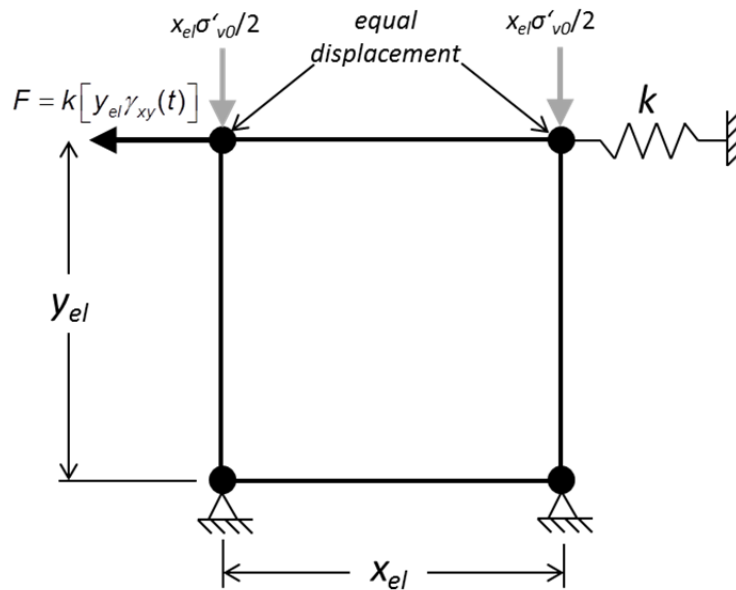


Figure 5-10: Element configuration for strain-controlled cyclic simple shear simulations.

The simulations were run using the PIMY model. Identical shear stress-strain, modulus reduction and damping results can be achieved using the PDMY2 model with the $\phi_{PT} = \phi$ to prevent dilation.

Results for Relative Densities $D_r = 30\%, 40\%, 50\%, 60\%, 70\%, 80\%$, and 90% , at $\sigma'_{v0} = 0.5, 1, 2, 4, 8$, and 16 atm are included in Appendix C. Example results for the case of $D_r = 70\%$ and a vertical effective stress of 1 atm are presented in Figure 5-11. The shear modulus reduction curves from the simulations successfully match the target curve.

A purely deviatoric kinematic hardening rule based on Mroz (1967) and Prevost (1985) is used in the PIMY model (Elgamal et al. 2003) dictates the hysteretic response. As an implementation of the Masing (1926) rule, the hysteretic response produces levels of damping at larger strain levels that are greater than those from the Darendeli (2001) model (Ishihara 1996). For example, Figure 5-11 shows model predictions of higher hysteretic damping compared to Darendeli (2000) values for a sand with $D_r = 70\%$. The disparity in damping predictions is more pronounced at lower confining stresses, and less pronounced at higher confining stresses. Stewart et al. (2008) described an alternative method, where a better fit to the damping curve can be obtained by relaxing the modulus reduction curve matching, thus achieving a compromise between matching modulus reduction and damping. In this current study, close

matching to the $G/G_{\max} - \gamma$ curve was deemed a priority, and the compromise matching procedure suggested by Stewart et al. (2008) was not used.

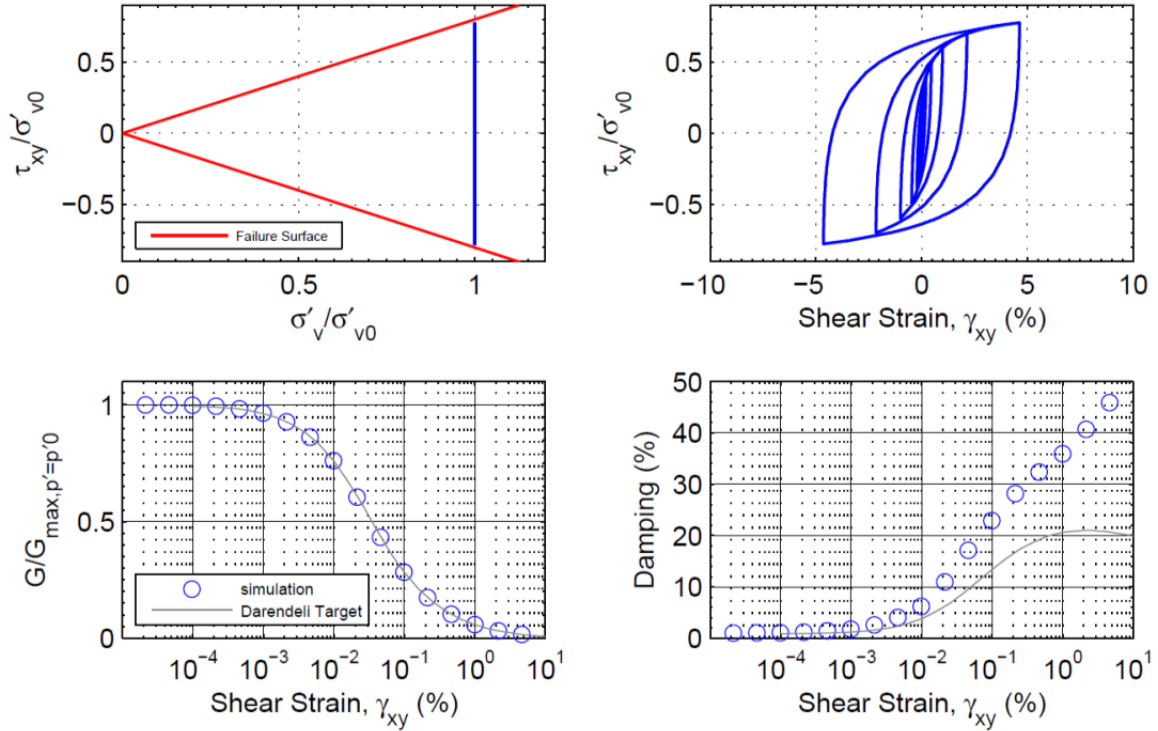


Figure 5-11: Example results for $D_r = 70\%$ and $\sigma'_{v0} = 1$ atm drained cyclic simple shear simulation

5.3.2 Undrained Response

The contraction and dilation parameters of the PDMY2 model were calibrated so that single element undrained cyclic simple shear simulations produced reasonable response relative to the semi-empirical liquefaction triggering criteria of Idriss and Boulanger (2006). The formulation and significance of the contraction (c_1 , c_2 and c_3) and dilation (d_1 , d_2 and d_3) parameters used in the PDMY2 model are described in detail in Appendix B. The calibration was performed through an iterative, manual process using single

element, stress-controlled CSS test simulations. The element layout, boundary conditions and loading are depicted in Figure 5-12. To exclude any inertial effects in the simulations, the loading frequency (f) was set to 0.05 Hz. The stresses were initialized under drained elastic conditions, then sinusoidal shear loading ($F = x_{el} z_{el} CSR \sigma'_{v0} \sin(2\pi ft)$, where z_{el} is the element thickness in the out of plane direction) was applied under undrained conditions.

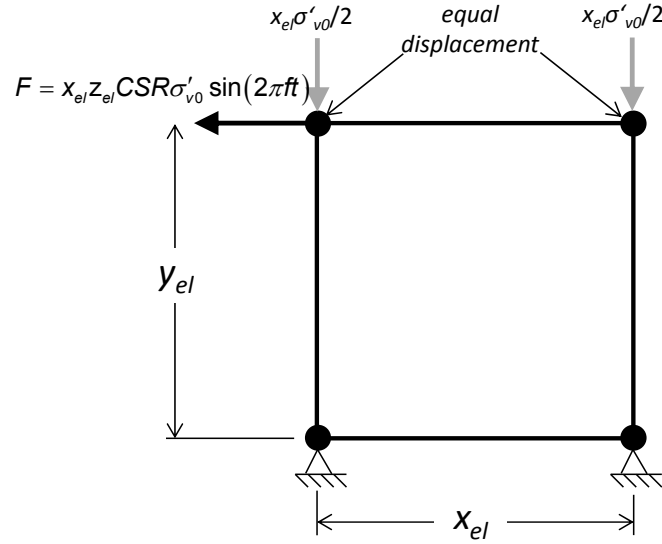


Figure 5-12: Element configuration for stress-controlled cyclic simple shear simulations.

In the CSS tests, initial liquefaction was defined as the first occurrence of double amplitude strain of 6 percent (Ishihara 1993, Wu 2002). It was observed in the simulation results that the double amplitude strain of 6 percent generally corresponded to an excess porewater pressure ratio ($r_u = \Delta u / \sigma'_{v0}$) of 95 to 98 percent. The parameters were calibrated such that initial liquefaction was achieved at 15 ± 0.3 cycles of stress controlled CSS loading at $\sigma'_{v0} = 1$ atm and at a cyclic stress ratio ($CSR = \tau_{xy,cyc} / \sigma'_{v0}$) set to the cyclic resistance ratio (CRR) defined by the Idriss and Boulanger (2008) liquefaction triggering curve:

$$CRR_{M=7.5, \sigma'_{v0}=1} = \exp \left[\left(\frac{(N_1)_{60cs}}{14.1} \right) + \left(\frac{(N_1)_{60cs}}{126} \right)^2 - \left(\frac{(N_1)_{60cs}}{23.6} \right)^3 + \left(\frac{(N_1)_{60cs}}{25.4} \right)^4 - 2.8 \right] \quad (5-17)$$

For converting between $(N_1)_{60cs}$ and D_r , the relationship from Idriss and Boulanger (2008) was used:

$$D_r = \sqrt{\frac{(N_1)_{60}}{46}} \quad (5-18)$$

where, for clean sands $(N_1)_{60} = (N_1)_{60cs}$.

Increased confining stresses are recognized to increase contractive tendencies and thus affect liquefaction resistance (e.g., Seed 1983, Seed and Harder 1990). The Idriss and Boulanger liquefaction triggering criteria account for this effect through the use of an overburden stress correction factor (Boulanger and Idriss 2004):

$$K_\sigma = \frac{CRR_{\sigma'_{v0}}}{CRR_{\sigma'_{v0}=1atm}} \quad (5-19)$$

where $CRR_{\sigma'_{v0}}$ is the cyclic resistance ratio for an initial vertical effective stress σ'_{v0} and $CRR_{\sigma'_{v0}=1atm}$ is the cyclic resistance ratio at an initial vertical effective stress of 1 atmosphere. Idriss & Boulanger (2008) present a relationship for K_σ based on D_r and σ'_{v0} :

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{v0}}{p_a}\right) \leq 1.1 \quad (5-20)$$

where

$$C_\sigma = \frac{1}{18.9 - 17.3D_r} \leq 0.3 \quad (5-21)$$

Figure 5-13 plots Eqns. 5-20 and 5-21 along with data from CSS simulations calculated by Eqn. 5-19 against σ'_{v0} normalized by 1 atm of pressure (P_a). The c_3 and d_3 parameters, which adjust contraction and dilation rates for overburden stress levels, were calibrated so that a reasonable fit to Eqns. 5-20 and 5-21 was achieved as shown in Figure 5-13. Based on the current formulation of the PDMY2 model, it was not possible to match the K_σ from Eqn. 5-20 and 5-21 exactly at all confining stresses. With these limitations in mind, the calibration focused on achieving a good match at vertical effective stresses between 0.5 and 8 atm, which represents the range over which many effective stress site response analyses would be performed in practice.

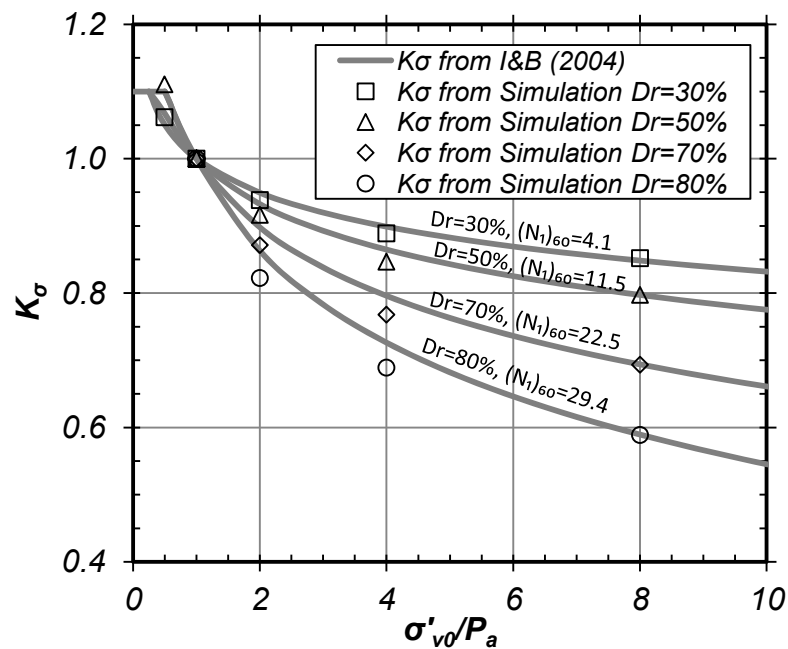


Figure 5-13: Comparison of K_σ values from the calibrated model versus the relationship proposed by and Boulanger and Idriss (2004).

Larger magnitude earthquakes have long been recognized to produce more cycles of loading compared to smaller magnitude earthquakes (Ambraseys 1988, Arrango 1996, Andrus and Stokoe 1997, Youd et al. 2001, Idriss and Boulanger 2006). Undrained cyclic laboratory tests have shown that porewater pressure build-up and liquefaction triggering are dependent on the number of loading cycles. Laboratory tests are typically performed at various *CSRs*, and the number of cycles to liquefaction (*N*) are plotted versus *CRR*. In semi-empirical liquefaction triggering analyses, the magnitude scaling factor ($MSF = CRR_M / CRR_{M=7.5}$) has been used to account for the effect of magnitude and number of cycles. Idriss (1999) proposed a relationship between the number of equivalent uniform stress cycles versus earthquake magnitude which can be represented by the following equation:

$$N = 0.0476 \exp(0.769M_w) \quad (5-22)$$

A series of single element CSS simulations were performed to evaluate *CRR* versus the number of cycles to liquefaction and *MSFs* that result from the calibrated model. The simulations were performed for $D_r = 30\%, 50\%, 70\%$ and 80% and $\sigma'_{v0} = 0.5, 1, 2$ and 4 atm.

Figure 5-14 presents CSS simulation results for the number of cycles necessary to reach liquefaction versus *CRR* for $\sigma'_{v0} = 1$ atm and each of the four relative densities. Also included in the plot are least square best fit regression lines of the form $CRR = aN^{-b}$. The values of *b* range from 0.46 to 1.13, and are

higher than the typical value suggested by Idriss and Boulanger of 0.34 for laboratory test results. This difference translates to *MSF* vs. *M_w* curves that, under some conditions, are somewhat steeper than published values, as described in the following paragraph.

By solving Eqn. (5-22) for *M_w*, the magnitude corresponding to the number of cycles in a CSS simulation can be estimated so that the simulation results can be compared with *MSF* relationships used in liquefaction triggering methods. Figure 5-15 presents the results of the CSS simulations along with *MSF* versus *M_w* relationships recommended by Youd et al. (2001) and Idriss and Boulanger (2008). The CSS results are in good agreement with the Youd et al. (2001) *MSF* relationships for *Dr* of 30, 50 and 70 percent. At *Dr* = 80%, the slope of the CSS results is steeper than that of the Youd et al. (2001) relationship. The slope of the Idriss and Boulanger (2008) *MSF* versus *M_w* relationship is less steep at than that of the Youd et al. (2001) curve and the CSS simulation data. Overall, these results show that the model is in reasonable agreement with proposed commonly used *MSF* relationships for *Dr* of 30 and 50 percent. At *Dr* of 70 and 80 percent, the model shows steeper *MSF* trends, which indicates increased sensitivity of liquefaction triggering to the number of cycles and magnitude.

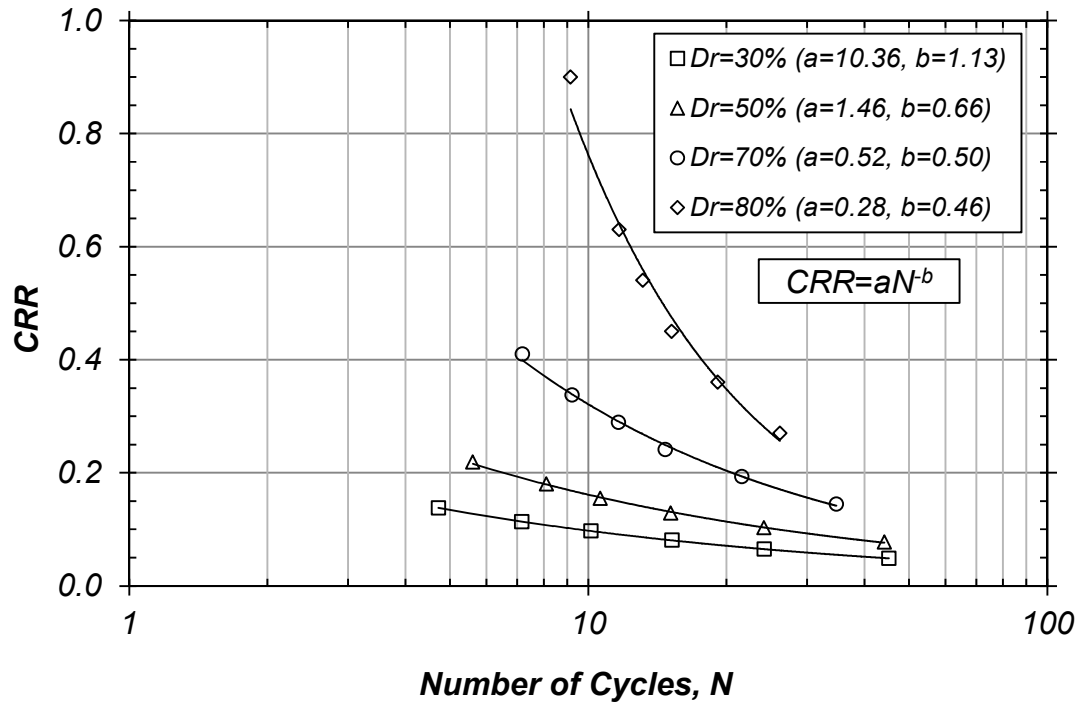


Figure 5-14: CRR versus number of cycles computed for the calibrated model in CSS simulations at $\sigma'_{v0} = 1$ atm and various value of Dr .

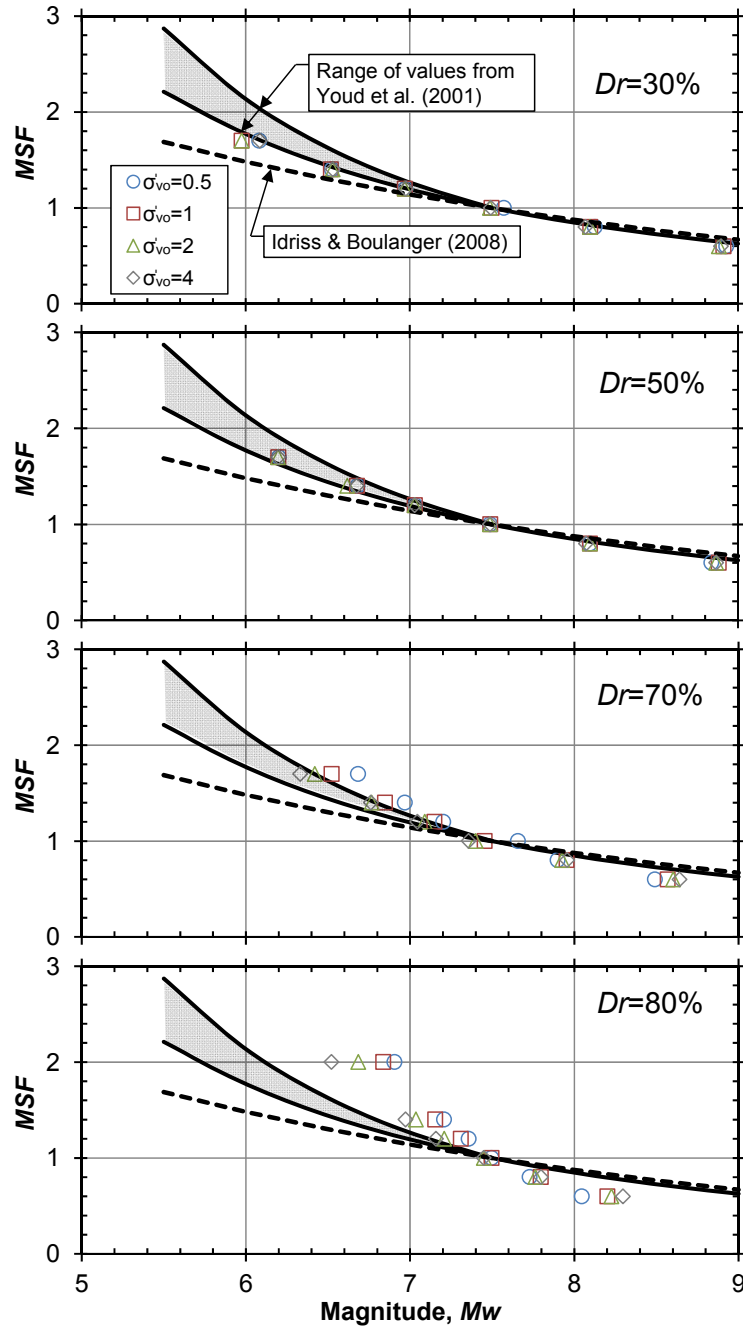


Figure 5-15: MSF values computed for calibrated model in CSS simulations compared to values recommended by Youd et al. (2001).

An additional PDMY2 model parameter, y_1 , controls the amount of strain accumulation that occurs during neutral phases (phases 4-5 and 7-8 described in

Appendix B) when the stress state is on the phase transformation line, between contraction and dilation phases. This parameter is important for post-liquefaction wave propagation studies because it affects the amount of post-liquefaction plastic shear deformation, and thus the post-liquefaction soil stiffness.

Empirical liquefaction triggering relationships do not provide sufficient information to fully constrain post-liquefaction shear deformation and the y_1 parameter. Instead, results from CSS test results performed at U.C. Berkeley by Wu (2002) were used to calibrate the y_1 parameter. In Wu's undrained stress-controlled CSS tests without shear stress bias, the rate of cyclic shear strain accumulation per cycle (γ/cyc) is approximately constant after the onset of liquefaction. This can be observed in the constant slope shown by the red line in Figure 5-16. Figure 5-17 presents γ/cyc measured versus relative density and a least-square log-linear fit to the data, which produces the following relationship:

$$\gamma/cyc = 27.418 \exp(-0.063D_r) \quad (5-23)$$

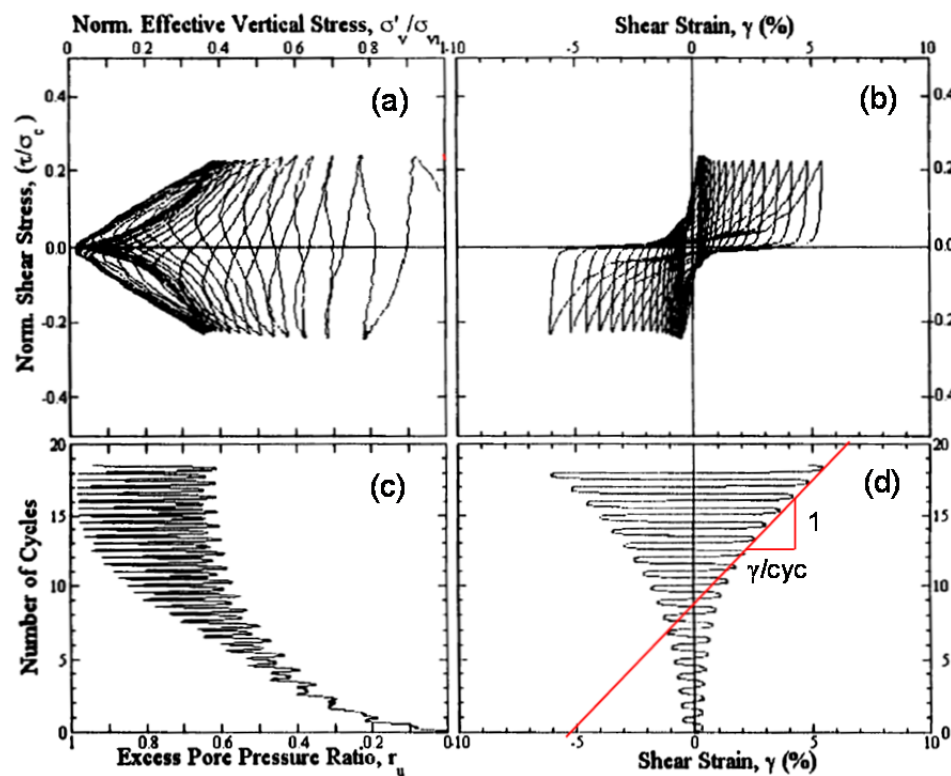


Figure 5-16: Example of undrained cyclic simple shear test result of Monterey 0/30 Sand at $D_r = 60\%$ from Wu (2002).

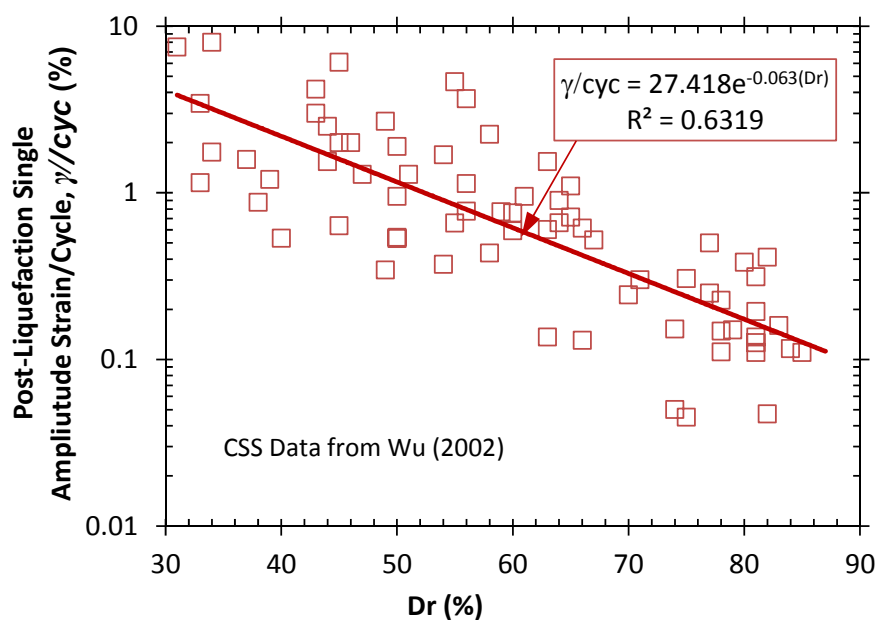


Figure 5-17: Post-liquefaction shear strain per cycle from undrained CSS test by Wu (2002).

The y_1 parameter was calibrated so that the post-liquefaction shear strain per cycle approximately matched the relationship of Eqn. (5-23) in undrained CSS simulations at 1 atmosphere vertical effective stress. Figure 5-18 presents an example of CSS simulation results showing shear stress-strain and γ/cyc versus cycle. The target γ/cyc is calculated using Eqn. (5-23) with $D_r = 60\%$. Appendix C presents γ/cyc versus loading cycle results for all of the undrained CSS simulations used for the calibration. A good match to the Wu data is achieved at $\sigma'_{v0} = 1 \text{ atm}$. The CSS simulations show a trend of increasing γ/cyc with increasing σ'_{v0} over the range of σ'_{v0} from 0.5 to 8 atm. The tests by Wu (2002) were conducted over a relatively narrow range of σ'_{v0} from 0.4 to 1.8 atm that do not provide sufficient data to evaluate the dependency of γ/cyc on σ'_{v0} .

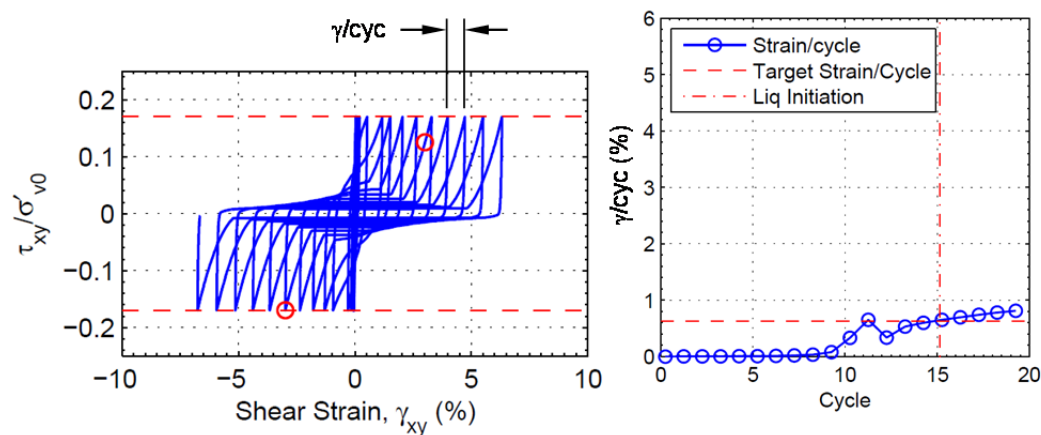


Figure 5-18: Post-liquefaction shear strain per cycle from undrained CSS simulation for $D_r = 60\%$.

5.3.3 Summary of Calibrated Parameters

A summary of the calibrated PDMY2 model parameters is presented in Table 5-1.

Table 5-1: Summary of Calibrated Model Parameters.

Parameter	Relative Density, Dr						
	30%	40%	50%	60%	70%	80%	90%
User-defined G/G_{max} versus γ	Darendeli (2000) with $PI = 0$, $OCR = 1$ and $\sigma'_o = \sigma'_{v0}$, adjusted for strength per Section 5.3.1.2						
Density, ρ [Mg/m ³]	1.70	1.77	1.83	1.90	1.97	2.03	2.10
Shear wave velocity, V_s [m/s]	$\ln(V_s) = 4.045 + 0.096 \ln(N_{60}) + 0.236 \ln(\sigma'_{v0})$ (Brandenberg et al. 2010)						
Ref. shear modulus, G_r [kPa]	$G_r = V_s^2 / \rho$						
Ref. bulk modulus, B_r [kPa]	$B_r = \frac{2G_r(1+\nu)}{3(1-2\nu)}$ (note (b))						
DSS friction angle, ϕ_{DSS} [deg]	28	31	33	36	39	41	44
Model friction angle, ϕ [deg]	23.5	25.9	28.6	31.3	34.2	37.4	40.8
Cohesion, c [kPa]	0.1						
Shear strain at failure, γ_f [-]	Irrelevant for user-defined G/G_{max} vs. γ						
Ref. pressure, p'_r [kPa]	set to σ'_{v0}						
Pressure dependent coef., d [-]	0.5						
Phase trans. angle, ϕ_{PT} [deg.]	20						
Contraction parameter 1, c_1 [-]	0.135	0.110	0.080	0.060	0.046	0.035	0.32
Contraction parameter 2, c_2 [-]	1.00	1.60	2.20	3.00	4.00	5.20	6.40
Contraction parameter 3, c_3 [-]	0.120	0.156	0.180	0.195	0.198	0.200	0.175
Dilation parameter 1, d_1 [-]	0.440	0.250	0.160	0.085	0.060	0.045	0.040
Dilation parameter 2, d_2 [-]	3.00	3.00	3.00	2.50	2.00	1.50	1.00
Dilation parameter 3, d_3 [-]	0.00	0.00	0.00	0.00	0.00	-0.17	-0.34
PT strain parameter, y_1 [-]	1.00	1.00	1.00	0.53	0.28	0.10	0.00

Table Notes:

(a) Refer to Appendix B for an explanation of the parameters.

(b) For the initial consolidation phase, ν is set to 0.499 so that a $K_0 = 1$ condition is achieved. For the dynamic loading phase, ν is set to 0.

The parameters in Table 5-1 have been calibrated for use as a complete set. The complete set of parameters must be used for the calibration to be valid. For instance, if one changes V_s or ϕ to fit some site-specific value, the liquefaction triggering calibration may be lost and re-calibration by adjustment of other parameters would be necessary.

5.3.4 One-Dimensional Non-Linear Site Response Modeling Protocols

One-dimensional (1-D) nonlinear site response analyses were performed using a single column of four-node quadrilateral u-p finite elements in plane strain. The two nodes at each elevation were constrained to the same displacement using the OpenSees “*equalDOF*” command, such that deformation was restricted to shear-beam type deformation.

In the validation analyses where down-hole accelerograms were used as input, a rigid base condition was used with the motion assigned as an acceleration time history applied at the base nodes.

For site response analyses where the rock outcrop input motions were used as input, compliant base boundary conditions (Lysmer and Kuhlemeyer 1969) were used as recommended by Stewart et al. (2008). With these conditions, a viscous dashpot is affixed to one of the base nodes with a value of the damping coefficient, $c_{L-K} = x_{el} z_{el} V_{S,Base} \rho_{Base}$, where $V_{S,Base}$ and ρ_{Base} are the shear wave velocity and density of a theoretical elastic half-space underlying the model. This Lysmer-Kuhlemeyer boundary allows downward propagating waves that have been reflected off the surface or at impedance contrasts within the

profile to be partially absorbed by the dashpot, thereby emulating continued downward propagation of the wave into the half-space. The input motion was applied at the base as a shear stress wave ($\tau_{xy}(t)$), which is evenly distributed as a force to the two base nodes ($F(t)$) according to:

$$F(t) = \tau_{xy}(t) x_{el} z_{el} / 2 \quad (5-24)$$

$$\tau_{xy}(t) = \frac{2v(t)}{2} V_{S,Base} \rho_{Base} \quad (5-25)$$

where $v(t)$ is a time history of the rock outcrop input motion. The factor of $\frac{1}{2}$ in Eqn. (5-25) accounts for the fact that outcrop motions are doubled due to the reflection at free surface, so that the upward propagating wave is half of the outcrop motion (Mejia and Dawson 2006). The factor of 2 in Eqn. (5-25) offsets the effect of $\frac{1}{2}$ of the input force being absorbed by the viscous dashpot (Mejia and Dawson 2006, Kwok et al. 2007). While these two factors cancel each other out mathematically, they have been retained in Eqn. (5-25) for the purpose of clarity and explicitness. This combination of boundary condition and input motion specification prevents artificial trapping of waves and energy within the column that can lead to spurious high frequency noise (Mejia and Dawson 2006).

Figure 5-19 presents a schematic diagram of the finite element mesh and boundary conditions used in this study. Figure 5-19 (a) shows compliant base conditions and Figure 5-19 (b) shows rigid base conditions.

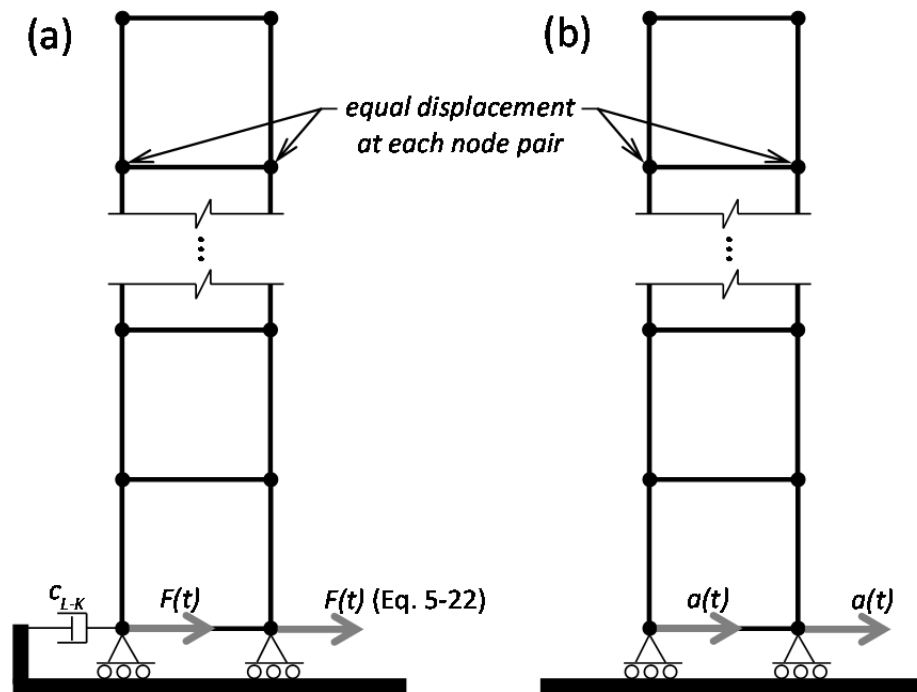


Figure 5-19: Schematic diagram of finite element mesh and boundary conditions for (a) compliant base conditions and (b) rigid base condition.

At very low strains, the PDMY2 and PIMY models behave elastically without hysteresis and associated damping. Cyclic laboratory tests generally show that a small amount of soil damping is present even when the stress-strain behavior is essentially elastic (e.g., Darendeli 2001). The analyses performed for this study used Rayleigh damping to account for small strain soil damping. Full Rayleigh damping was used, with the target frequencies set at $f'_1 = f_1/3$ and $f'_2 = 5f'_1$, where $f_1 = \overline{V_s}/(4H)$ is the small strain fundamental site frequency, H is the height of the soil column and the average shear wave velocity in soil column is:

$$\overline{V_s} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \left(\frac{d_i}{V_{si}} \right)} \quad (5-26)$$

where d_i is the thickness of sublayer i , V_{si} is the shear wave velocity of sublayer i , and n is the total number of sublayers. The use of $f'_1 = f_1/3$ and $f'_2 = 5f'_1$ is a variation from the values of f_1 and $5f_1$ that were recommended by Stewart et al. (2008). The use of f'_1 and $5f'_1$ resulted in improved results for the validation analyses for liquefied sites (Section 5.5), and thus was adopted for the parametric site response analyses performed for this study.

5.4 Model Validation for Non-Liquefaction Conditions

5.4.1 Stewart et al. (2008) Validation

OpenSees with the PIMY soil model was one of five nonlinear site response codes used in the Stewart et al. (2008). In use of the modeling protocols described by Stewart et al. (2008) (which were adopted for this study as described in Section 5.3.4), they found good agreement with exact solutions for linear cases. In validating model predictions against downhole array data, they found the models produced reasonable results, with the exception of some overdamping at high frequencies and overestimation of site amplification at the resonant frequency of the model.

5.4.2 Validation in PRENOLIN Program

As a representative of UCSD, the author of this dissertation is a participant of the PRENOLIN (Improvement of **P**rediction of **Nonlinear** effects caused by strong seismic motion) research program which was on-going at the time this work was being prepared. The objective of the PRENOLIN program is to verify and validate nonlinear codes in simple 1-D conditions; assess epistemic uncertainties in nonlinear response analyses; and develop guidelines for deterministic physics-based nonlinear analyses for use in seismic hazard analyses (Régnier et al. 2014a). The same site response software and methodologies were used in the analyses performed for the PRENOLIN program as were in this study. The first phase, which was completed by the time of this writing, consisted of verification exercises performed for idealized soil profiles under linear and nonlinear conditions. The OpenSees site response models provide results that are in good agreement with exact solutions for cases where

they are available. For cases where exact solutions are not available, the OpenSees site response models provide results that are in reasonable agreement with the majority of the other nonlinear codes used in the PRENOLIN study. A paper providing details of these comparisons was in preparation at the time of this writing (Régner et al. 2014a).

5.5 Model Validation Using Liquefied Site Case Histories

To validate the site response model under liquefaction conditions, analyses were performed for two downhole array sites where that liquefied during strong ground shaking: Wildlife Liquefaction Array near El Centro, California, and the Port Island Array in Kobe, Japan.

5.5.1 Wildlife Liquefaction Array

The Wildlife Liquefaction Array (WLA) site is located within the Salton Sea Wildlife area, approximately 6 km north of Brawly, California (33.09738° north latitude, 115.53045° west longitude). The WLA was originally characterized and instrumented by the USGS in 1982 (Bennett et al. 1984, Youd and Holzer 1994) with one surface and one downhole (at a depth of 7.5 m) accelerometer, and six piezometers. This set of equipment recorded the 23 November 1987 M 6.2 Elmore Ranch and 24 November 1987 M 6.5 Superstition Hills earthquakes. Additional geotechnical site exploration was performed and instrumentation was installed in 2003 and 2004 (Youd et al. 2004 and 2007). The site exploration consisted of 24 CPTs and 24 exploratory borings. The additional instrumentation consisted of 6 downhole accelerometers, three surface accelerometers, eight

piezometers, five slope inclinometer casings, three flexible casings and a network of 30 survey benchmarks. Subsurface characterization data from WLA is available on the internet (NEES 2014).

The site is located along the west bank of the Alamo River and is underlain by soft to medium stiff lean clay (USCS symbol CL) extending to a depth of 2.75 m, then medium dense silty sand (SM) extending to a depth of 6.75 m, then medium stiff lean clay (CL) extending to 12 m. Below 12 m, the site is underlain by interbedded sand, silt and clay sediments, which extend to the maximum depth explored by Youd et al. (2004) of 32 m. The site is within the Salton Trough, a deep sedimentary basin formed by the San Andreas transform fault system. The basin soils are expected to extend from several tens to several hundreds of meters below the site.

Figure 5-20 presents stratigraphy, CPT tip resistance, SPT blow counts, shear wave velocity, fines content and relative density for the WLA profile. The red circles represent V_s and Dr values used in site response simulations of the profile. For the clay from 0 to 2.75 m and from 6.75 to 7.5 m, the PIMY model was used with Darendeli (2000) $G/G_{\max}$ vs. γ curves with $PI = 15$ and $OCR = 1$, and modified for strength compatibility per Section 5.3.1.2 with $S_u/\sigma'_{v0} = 0.4$ and $S_u \geq 7$ kPa. For the silty sand layer from 2.75 to 6.75 m, the parameters for $Dr = 50\%$ as summarized in Table 5-1 were used except that a value of $y_1 = 0.5$ (rather than $y_1 = 1.0$) was used since it produced surface response spectra and time series that provided a slightly better match to the recorded values. Hydraulic

conductivities of $1\text{e-}4$ and $1\text{e-}6$ cm/s were used for the sand and clay layers, respectively. Table 5-2 presents a summary of the parameters used in the analysis. The modeling protocols for meshing, boundary conditions and Rayleigh damping (set to 1%) described in Section 5.3.4 were followed. The analyses employed the 4-node quadrilateral u-p element.

Site response analyses were performed using the 7.5 m deep acceleration time history from the Elmore Ranch and Superstition Hills earthquakes as input at the base of the model. For each earthquake, analyses were performed separately for the north-south and east-west components of input motion. Figure 5-21 and Figure 5-23 show the acceleration, velocity and displacement time histories and response spectra for the computed and recorded ground surface motion. These figures also show the ratio of the surface response spectrum to the input ground motion response spectrum ("Amp Ratio"). Generally good agreement is observed in the time histories and response spectra. Figure 5-22 and Figure 5-24 present plots of normalized shear stress versus shear strain, normalized shear stress versus normalized vertical effective stress, and r_u for depths of 0.5, 3, 6 and 7.25 m below ground surface. The model computes values of r_u of up to about 30%, which is generally consistent with Holzer et al. (1989) observation that that minimal excess porewater pressure was generated by the Elmore Range earthquake.

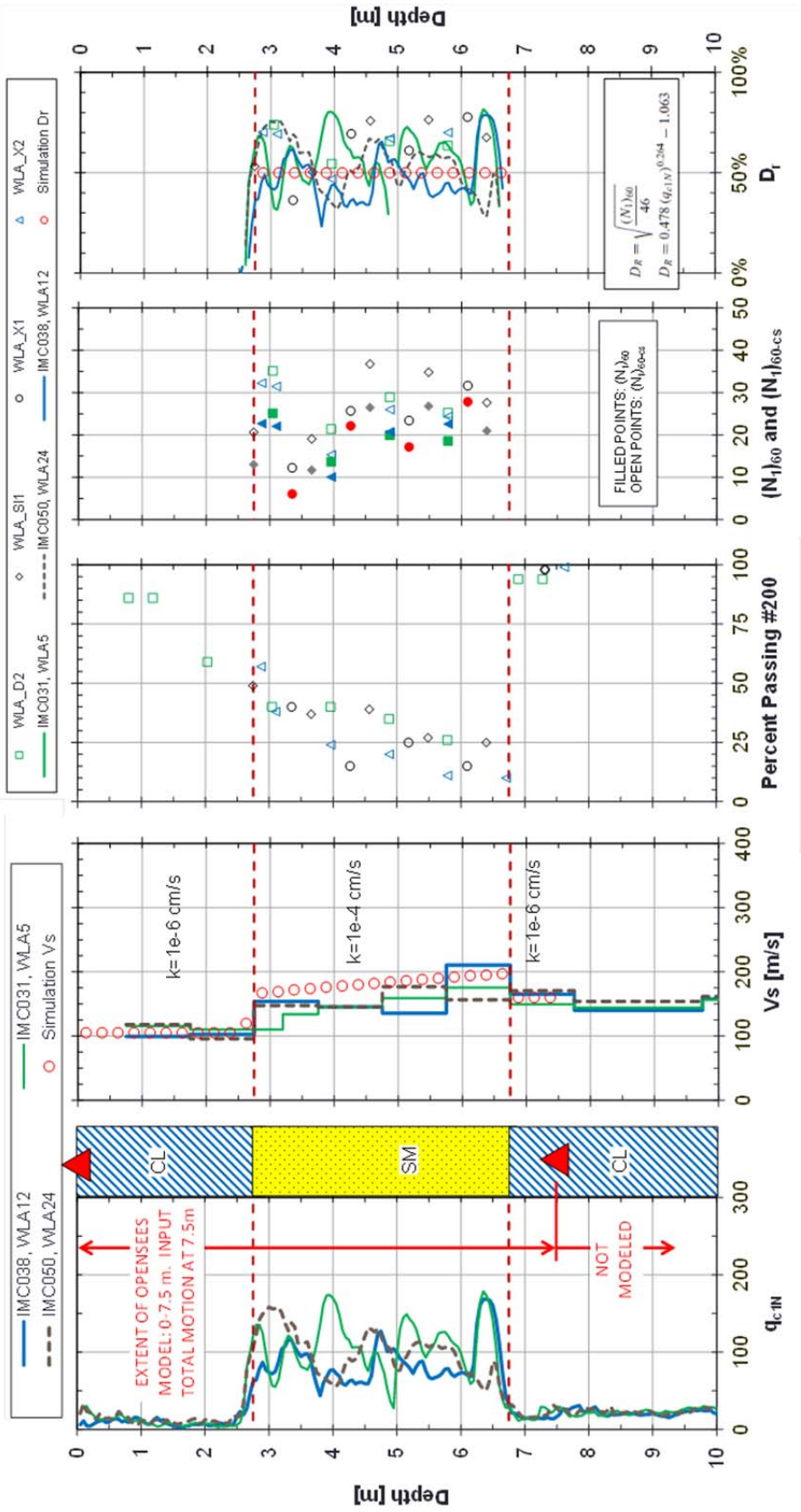


Figure 5-20: WLA geotechnical data used V_s and D_r values used for site response simulations (compiled from Youd et al. 2004 and 2007).

Table 5-2: Parameters Used in WLA Site Response Analyses

Layer No.	USCS Soil Type	Depth to Bottom (m)	Model	ρ (Mg/m ³)	Gr (kPa)	ν	ϕ (deg)	ϕ_{PT} (deg)	c (kPa)	c_1, c_2, c_3	d_1, d_2, d_3, γ_1	Hyd Cond (cm/s)
1	CL	0.25	PI	1.95	2.15E+04	0.49/0	0	na	146	na, na, na	na, na, na, na	1E-06
2	CL	0.50	PI	1.95	2.15E+04	0.49/0	0	na	7	na, na, na	na, na, na, na	1E-06
3	CL	0.75	PI	1.95	2.15E+04	0.49/0	0	na	7	na, na, na	na, na, na, na	1E-06
4	CL	1.00	PI	1.95	2.15E+04	0.49/0	0	na	7	na, na, na	na, na, na, na	1E-06
5	CL	1.25	PI	1.95	2.15E+04	0.49/0	0	na	9	na, na, na	na, na, na, na	1E-06
6	CL	1.50	PI	1.95	2.15E+04	0.49/0	0	na	11	na, na, na	na, na, na, na	1E-06
7	CL	1.75	PI	1.95	2.15E+04	0.49/0	0	na	12	na, na, na	na, na, na, na	1E-06
8	CL	2.00	PI	1.95	2.15E+04	0.49/0	0	na	13	na, na, na	na, na, na, na	1E-06
9	CL	2.25	PI	1.95	2.15E+04	0.49/0	0	na	14	na, na, na	na, na, na, na	1E-06
10	CL	2.50	PI	1.95	2.15E+04	0.49/0	0	na	15	na, na, na	na, na, na, na	1E-06
11	CL	2.75	PI	1.95	2.81E+04	0.49/0	0	na	16	na, na, na	na, na, na, na	1E-06
12	SM	3.00	PD2	1.90	5.30E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
13	SM	3.25	PD2	1.90	5.43E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
14	SM	3.50	PD2	1.90	5.62E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
15	SM	3.75	PD2	1.90	5.75E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
16	SM	4.00	PD2	1.90	5.89E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
17	SM	4.25	PD2	1.90	6.02E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
18	SM	4.50	PD2	1.90	6.16E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
19	SM	4.75	PD2	1.90	6.29E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
20	SM	5.00	PD2	1.90	6.43E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
21	SM	5.25	PD2	1.90	6.57E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
22	SM	5.50	PD2	1.90	6.72E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
23	SM	5.75	PD2	1.90	6.86E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
24	SM	6.00	PD2	1.90	7.00E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
25	SM	6.25	PD2	1.90	7.15E+04	0.49/0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04

Table 5-2: Parameters Used in WLA Site Response Analyses , Continued

Layer No.	USCS Soil Type	Depth to Bottom (m)	Model	ρ (Mg/m ³)	Gr (kPa)	ν	ϕ (deg)	ϕ_{PT} (deg)	c (kPa)	c_1, c_2, c_3	d_1, d_2, d_3, y_1	Hyd. Cond. (cm/s)
26	SM	6.50	PD2	1.90	7.22E+04	0.49 / 0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
27	SM	6.75	PD2	1.90	7.37E+04	0.49 / 0	28.6	20.0	0	0.08, 2.2, 0.18	0.16, 3, 0, 0.5	1E-04
28	CL	7.00	PI	2.00	5.12E+04	0.49 / 0	0	na	31	na, na, na	na, na, na, na	1E-06
29	CL	7.25	PI	2.00	5.12E+04	0.49 / 0	0	na	32	na, na, na	na, na, na, na	1E-06
30	CL	7.50	PI	2.00	5.12E+04	0.49 / 0	0	na	33	na, na, na	na, na, na, na	1E-06

Time histories and spectra for the Superstition Hills earthquake are presented in Figure 5-25 and Figure 5-27. The computed times series plots show a generally good match with the recorded motions, though the post-liquefaction amplitudes of the computed values are noticeably lower than the recorded values. For periods greater than about 0.4 s for the 360 degree direction and 1 s for the 90 degree direction, the spectral accelerations for the recorded motion are greater than for the computed values. This could be the result of surface wave effects that are not captured by the 1-D site response analysis. Computed normalized shear stress-strain, normalized shear stress-vertical effective stress and r_u plots for various depths are provided in Figure 5-26 and Figure 5-27.

Figure 5-27 compares the recorded average shear stress-strain response in the 360 degree direction over the interval from the surface to a depth of 7.5 m (left diagram) to the shear stress-strain response computed from a depth of 2.5 to 3m (right diagram). Since these two plots are for significantly different depth intervals, they should not be expected to produce the same results. None-the-less, there is good agreement between the two plots in terms of the shape and amplitude of the hysteresis loops.

Recorded and computed excess porewater pressure generation plots are compared in Figure 5-27. The rate of r_u accumulation in the simulation results is higher than the recorded rate. Dips in the r_u plots that corresponding to episodes of dilation occur at consistent times between the simulated results and recorded data.

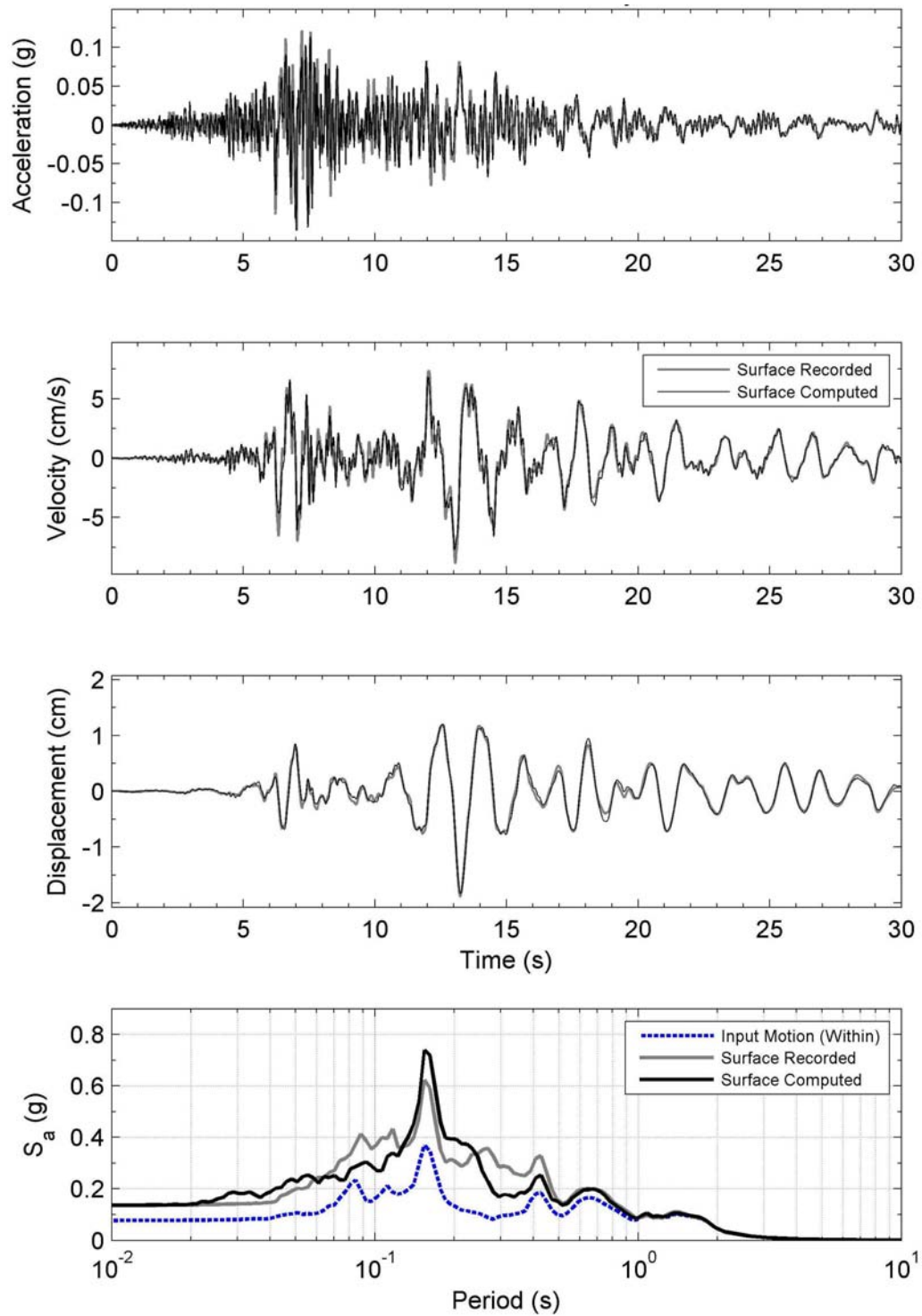


Figure 5-21: Comparison of simulated and recorded ground motions for WLA 360 component Elmore Ranch earthquake.

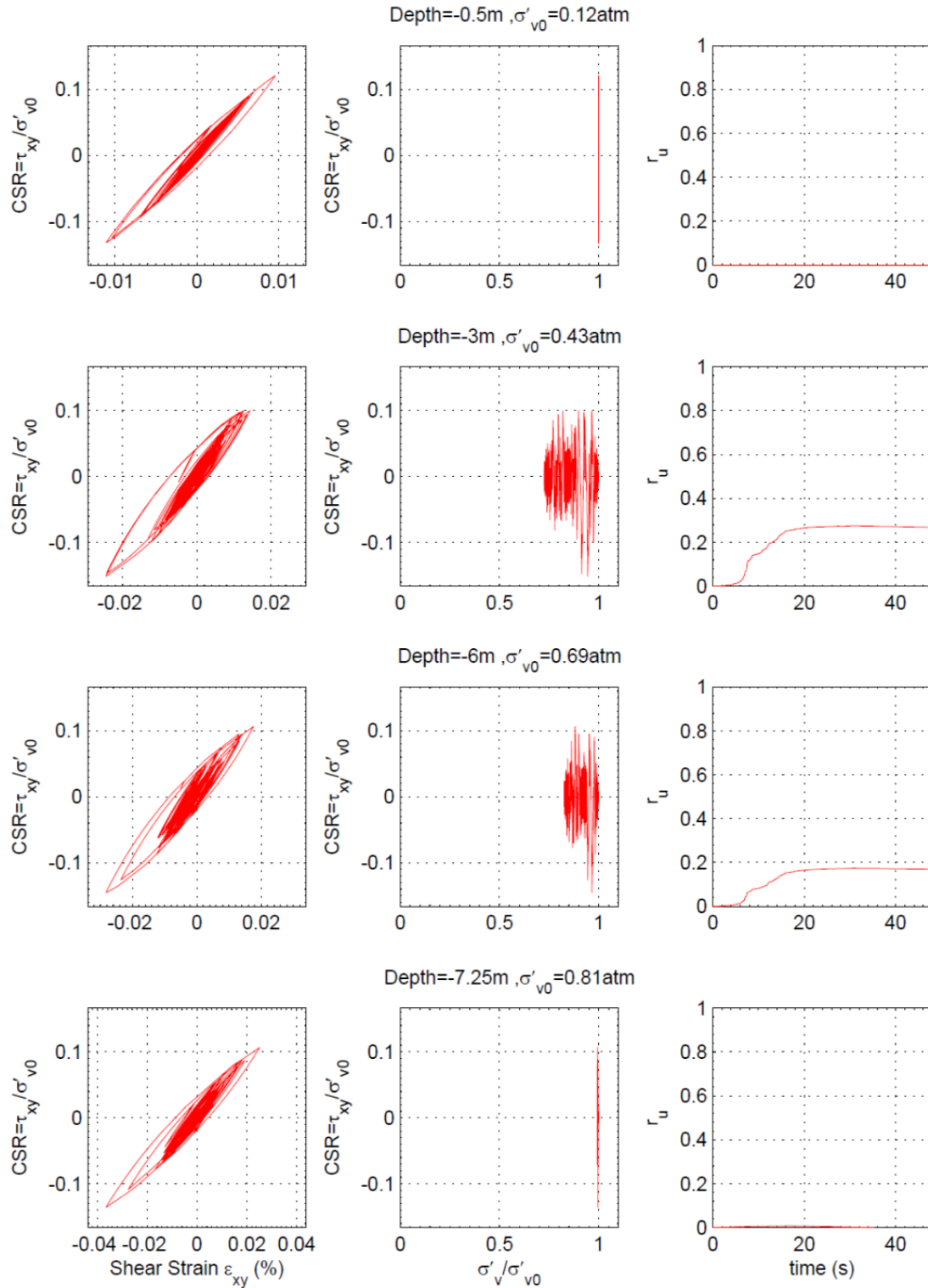


Figure 5-22: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 360 component Elmore Ranch earthquake.

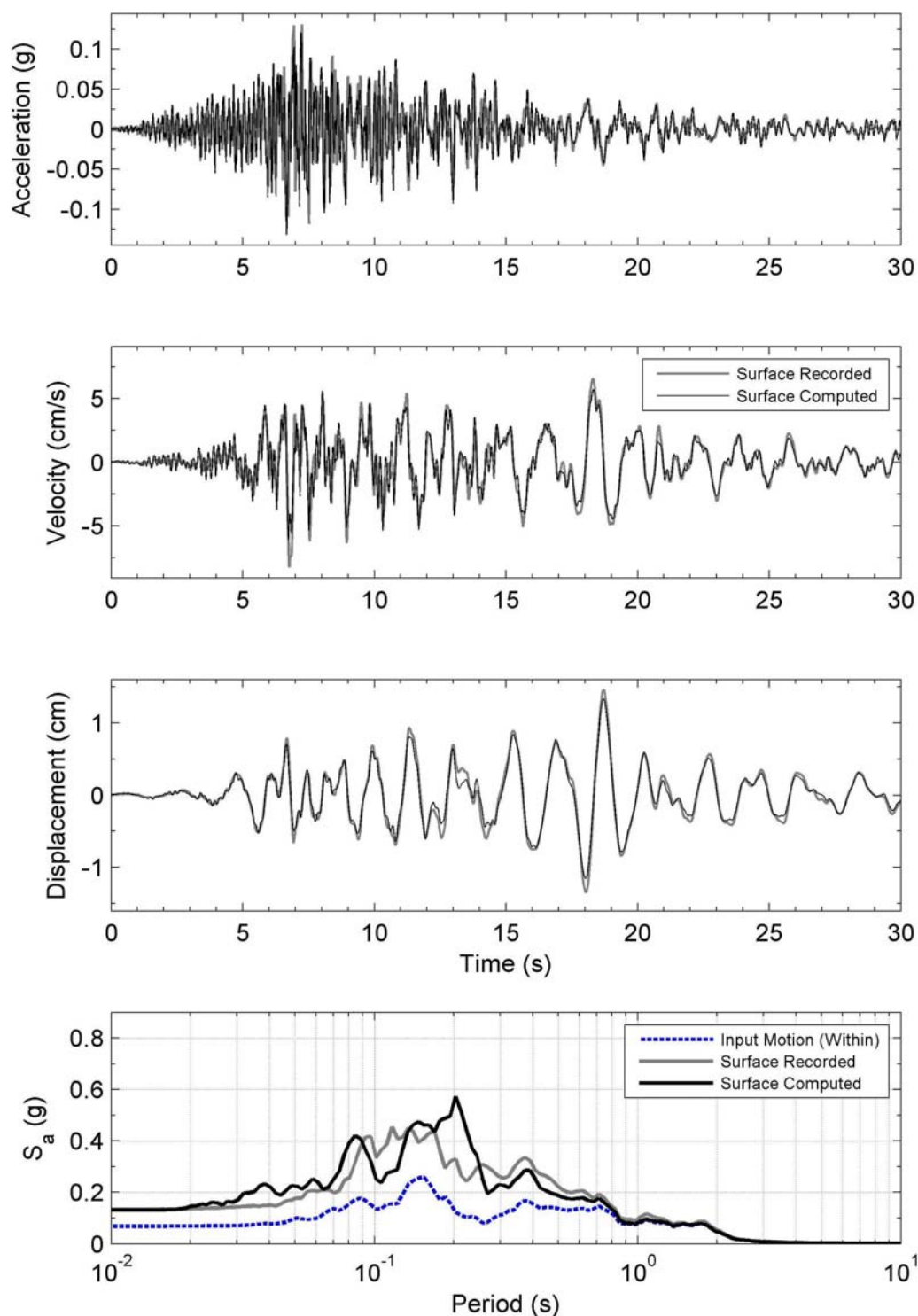


Figure 5-23: Comparison of simulated and recorded ground motions for WLA 90 component Elmore Ranch earthquake.

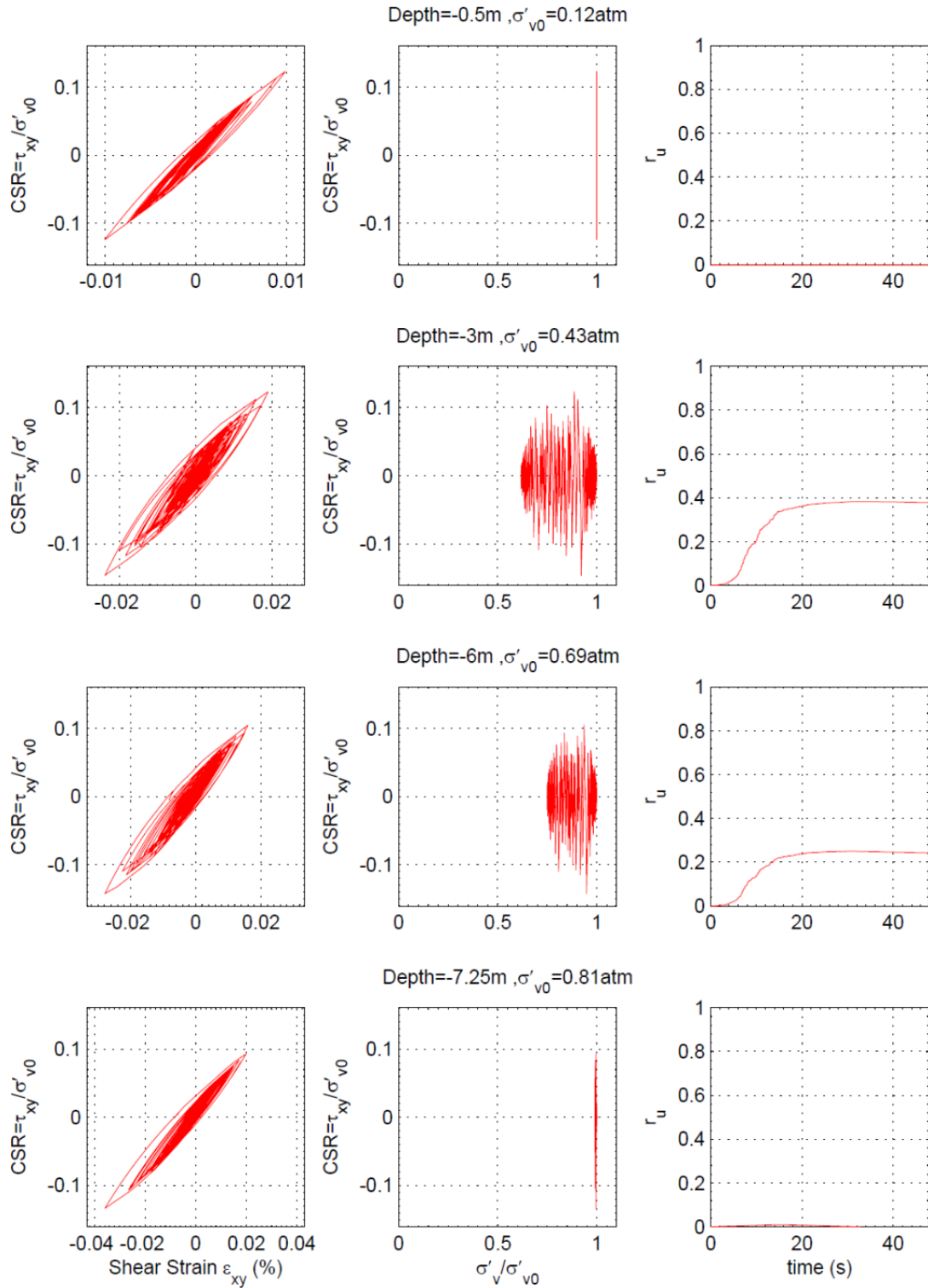


Figure 5-24: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 90 component Elmore Ranch earthquake.

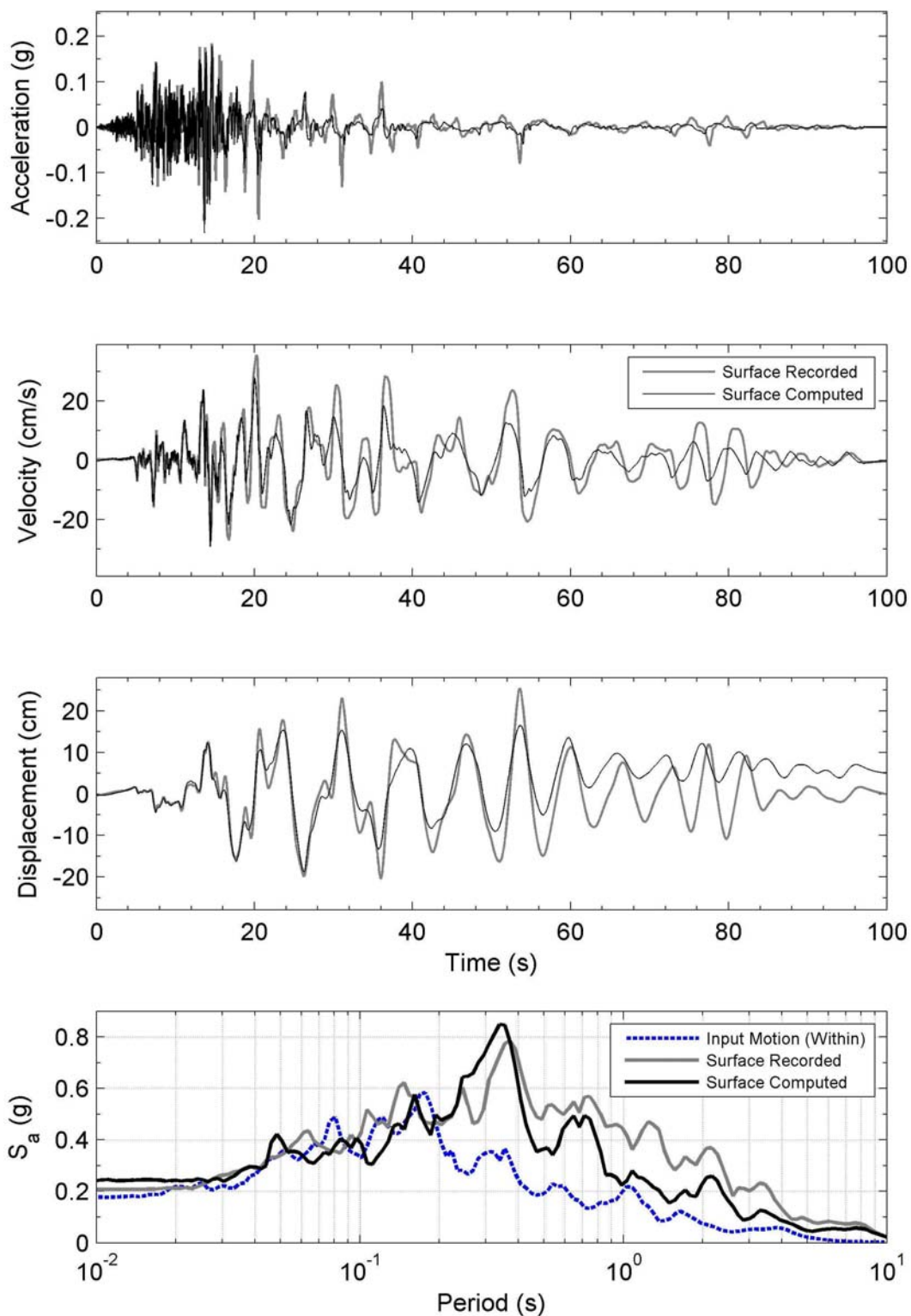


Figure 5-25: Comparison of simulated and recorded ground motions for WLA 360 component Superstition Hills earthquake.

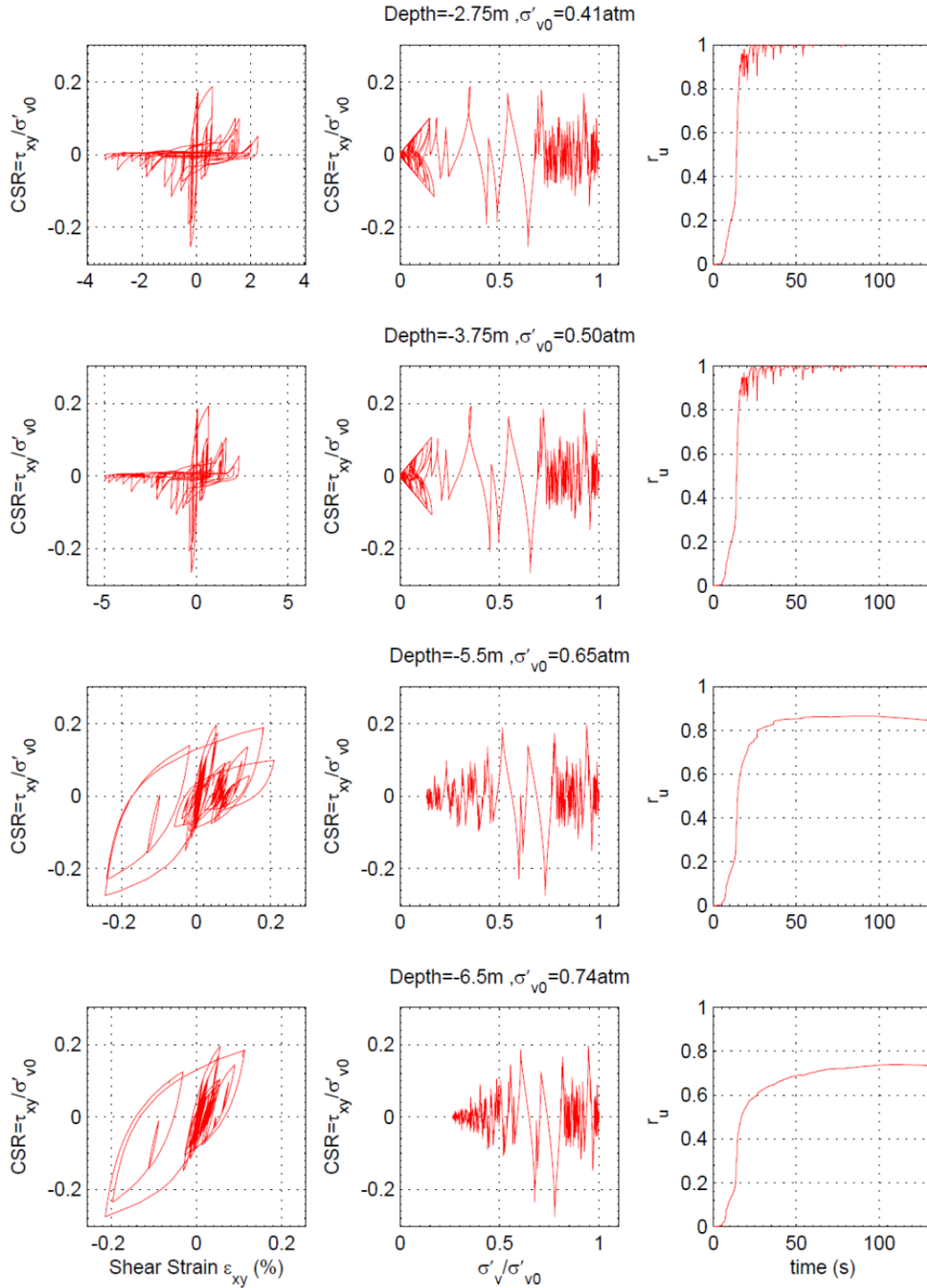


Figure 5-26: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 360 component Superstition Hills earthquake.

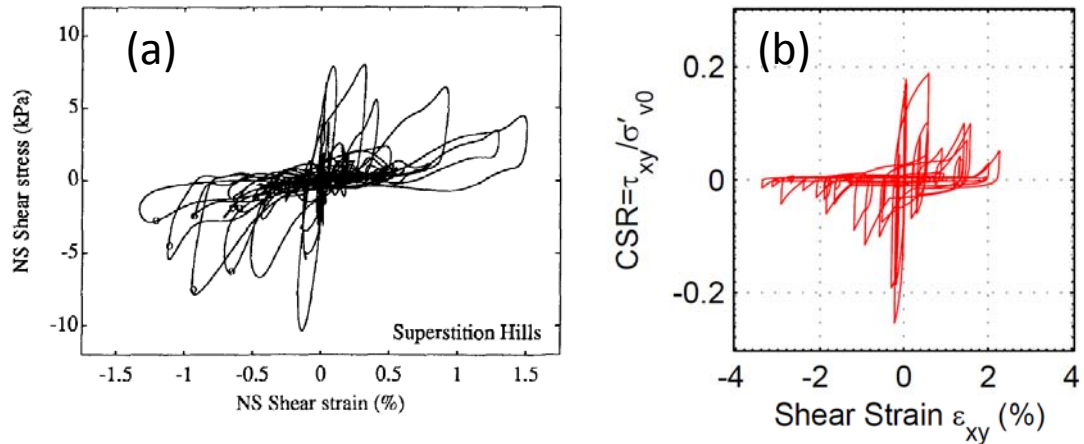


Figure 5-27: Comparison of shear stress-strain for 360 component Superstition Hills event from (a) Zhegal and Elgamal (1994) (average over upper 7.5 m), and (b) computed for element from 2.5 to 3 m.

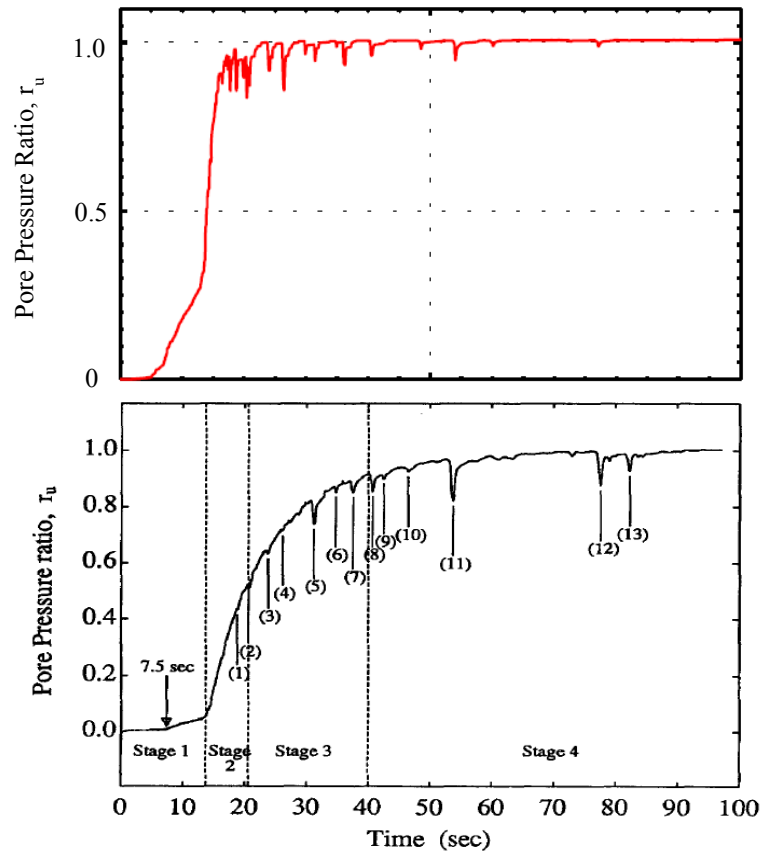


Figure 5-28: Comparison of excess porewater pressure vs. time for Superstition Hills event (a) computed at depth of 2.75 m, and (b) measured at 2.9 m (Zeghal and Elgamal 1994).

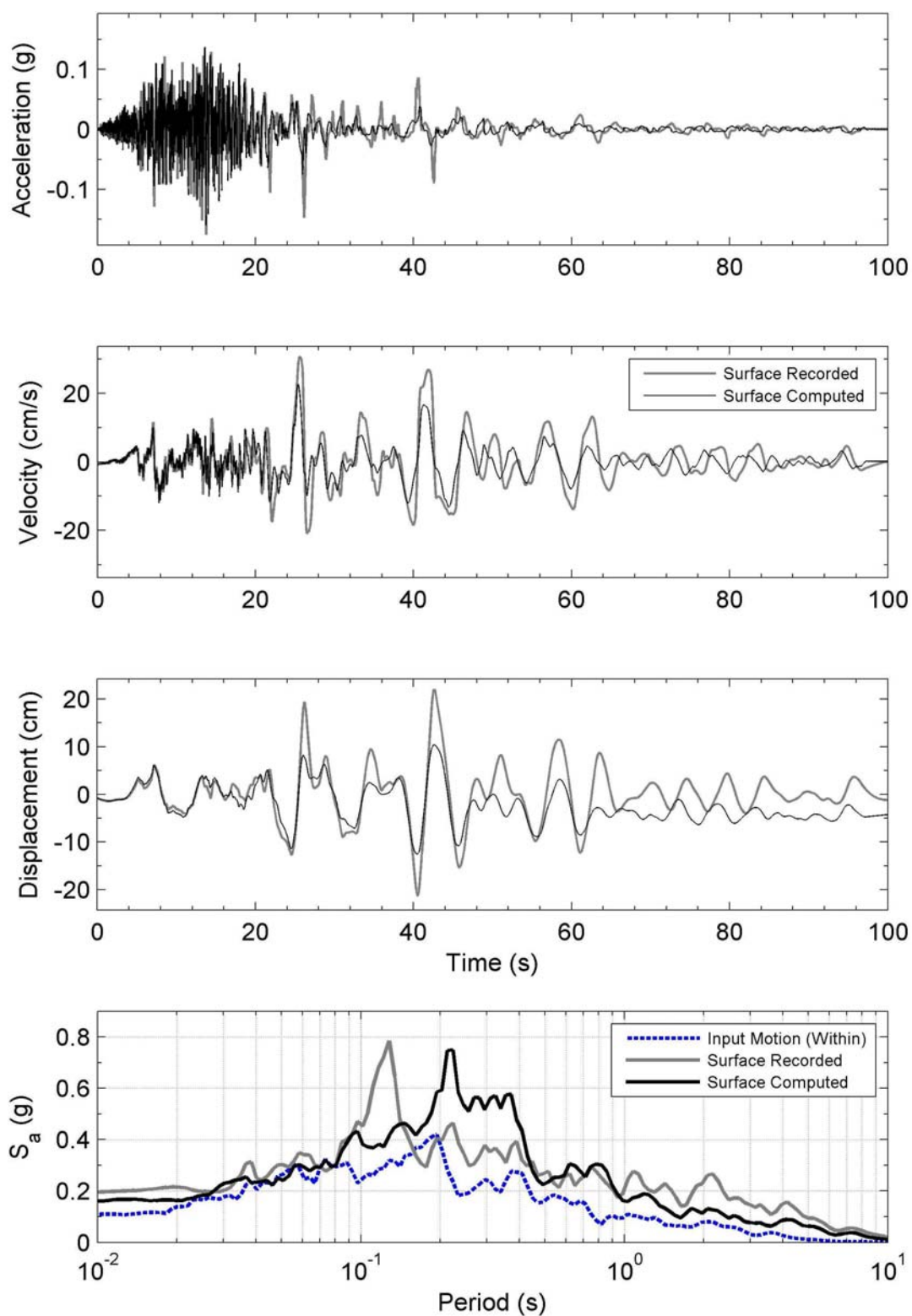


Figure 5-29: Comparison of simulated and recorded ground motions for WLA 90 component Superstition Hills earthquake.

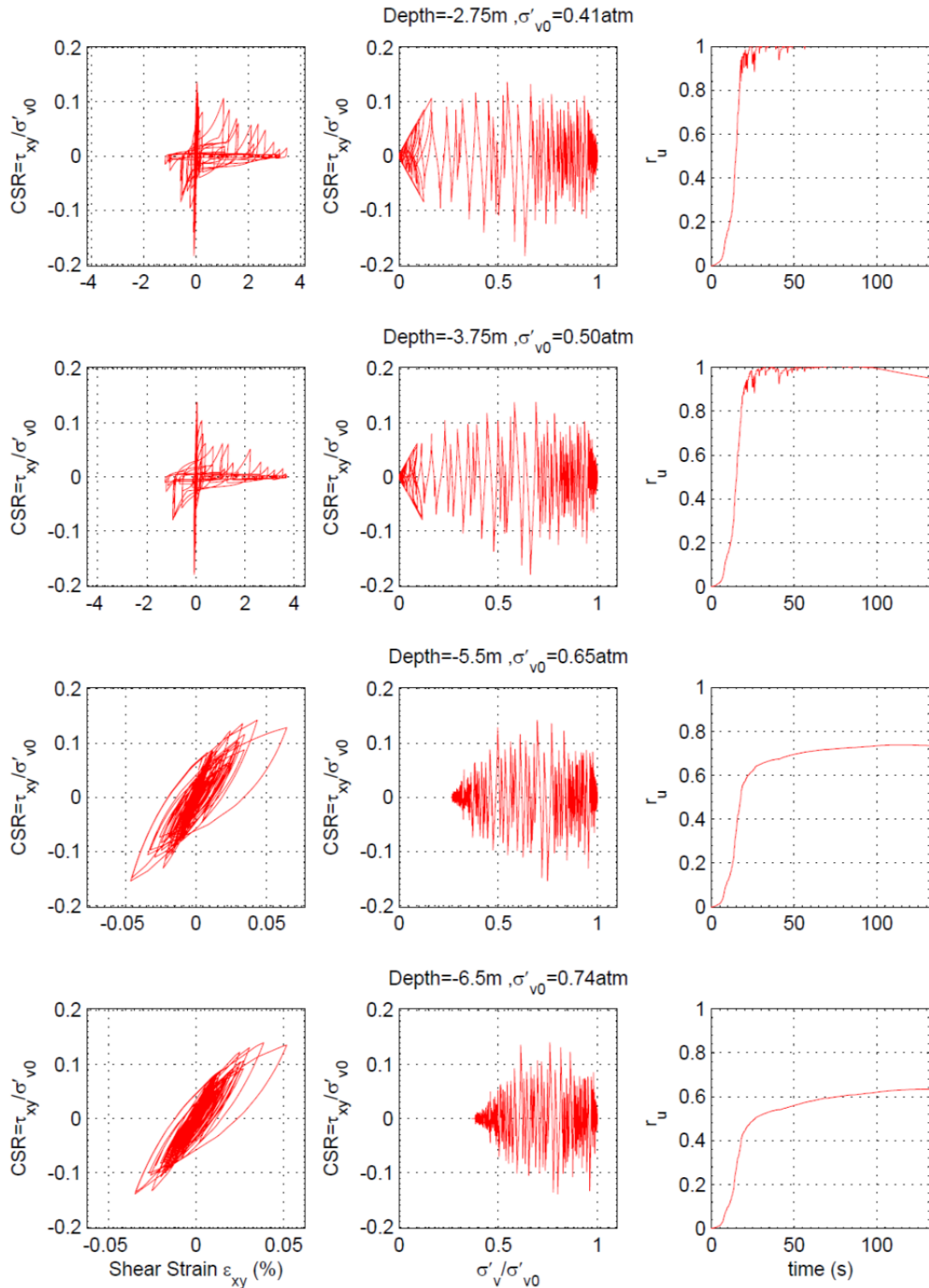


Figure 5-30: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for WLA 90 component Superstition Hills earthquake.

5.5.2 Kobe Port Island

The Port Island recording station in Kobe, Japan was located at 34.670° north latitude and 135.208° east longitude. Subsurface soil parameters used in the site response analysis were compiled from Nakakita and Watanabe (1981), Iwasaki (2009), Elgamal et al. (1996), and Cubrinovski (1996). Port Island was constructed as reclaimed land. The profile used in the site response analyses is presented in Figure 5-31. Table 5-3 presents a numerical summary of the parameters used in the analysis. A vertical array of four accelerometers at depths of 0 m, 16 m, 32 m, and 83 m recorded the $M_w = 6.9$ Hyogoken-Nanbu, Kobe earthquake on 17 January 1995. Only the upper 32 m of the soil profile was modeled, and the downhole acceleration recording at 32 m depth was used as the input excitation. Widespread liquefaction was observed in post-earthquake reconnaissance on Port Island (Bardet et al. 1995, Comartin et al. 1995, O'Rourke 1995).

For the sandy Masado fill (decomposed granite), the parameters for $D_r = 55\%$ in Table 5-1 (parameters were interpolated between $D_r = 50\%$ and $D_r = 60\%$) were used, except that the site-specific V_s values shown in Figure 5-31 and were used, and the c_1 contraction parameter was set to 0.038 instead of 0.07 to compensate from the deviation of V_s from that in Table 5-1. The clays underlying the Masado fill to a depth of 32 m were modeled using the PIMY model with Darendeli $G/G_{\max}$ vs. γ curve with $PI = 30$, $OCR = 2$, and with modification for shear strength compatibility per Section 5.3.1.2 with $S_u/\sigma'_{v0} = 0.4$

. Hydraulic conductivities of $1\text{e-}4$ and $1\text{e-}6$ cm/s were used for the sand and clay layers, respectively. The modeling protocols for meshing, boundary conditions and Rayleigh damping (set to 1%) described in Section 5.3.4 were followed.

Recorded and computed time histories and spectra are presented in Figure 5-32 and Figure 5-35 for the 00 and 90 degree azimuth directions, respectively. The computed times series plots show a good match with the recorded motions. Computed normalized shear stress-strain, normalized shear stress-vertical effective stress and r_u plots are provided in Figure 5-33, 5-34, 5-36 and 5-37. These plots show that the model predicts the occurrence of liquefaction within the Masado fill layer.

Figure 5-38 shows the recorded average shear stress-strain response in the N44W and N46E directions over the interval from the surface to a depth of 16 m. These can be compared to the computed shear stress-strain plots at a depth of 12 m in Figures 5-33 and 5-36. The recorded shear stress-strain plots do not show the relatively higher pre-liquefaction initial stiffness observable in the computed plots. Nor do the recorded plots show obvious signs of dilation phases which are apparent in the plots of calculated response. These differences could be explained by the fact that the recorded data was low-pass filtered (which could have removed the stiffer high-frequency response), and that a coarse 16 m depth interval between accelerometers was used, which cannot capture the shorter wave lengths of higher frequencies.

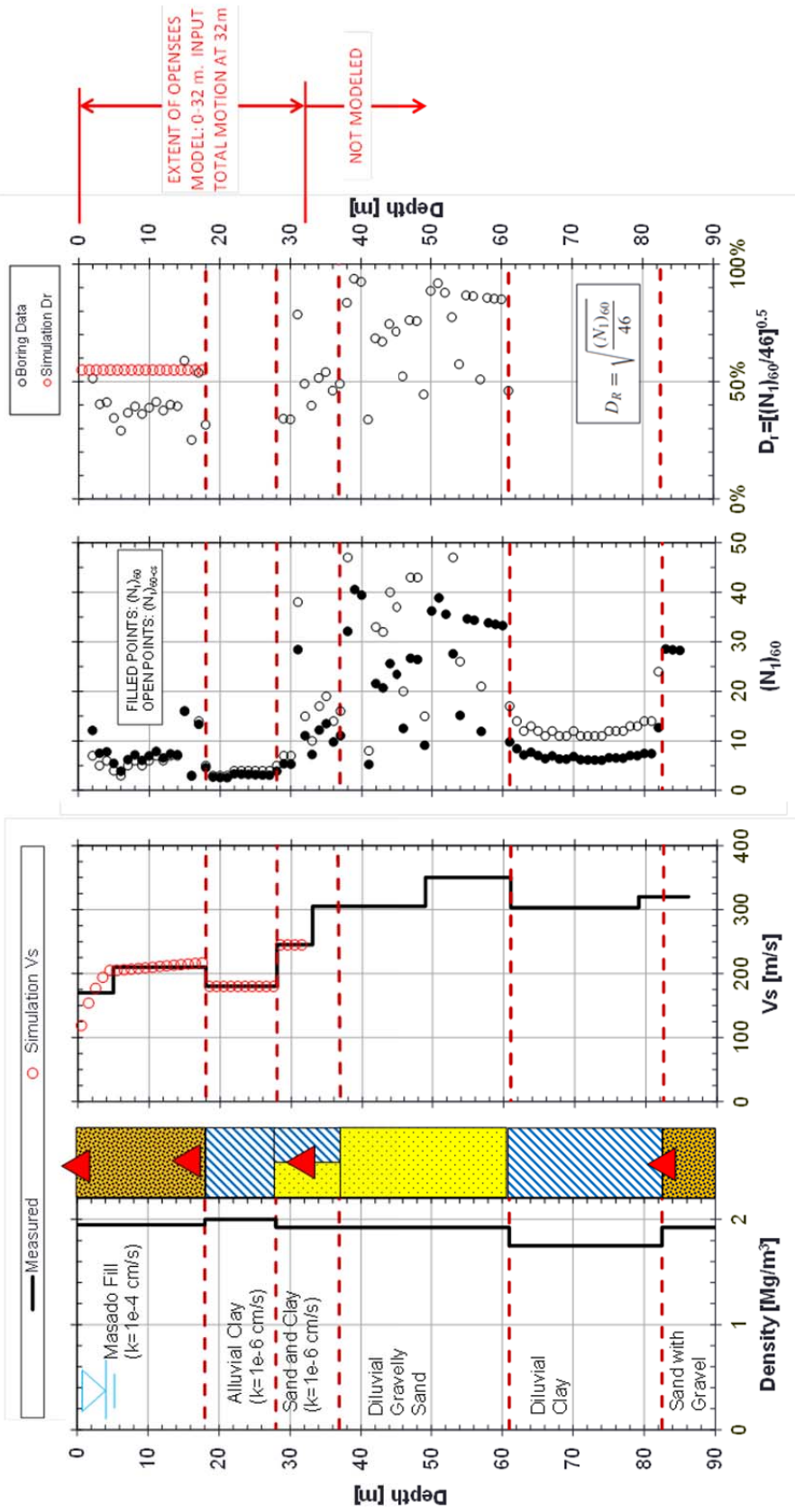


Figure 5-31: Kobe Port Island used V_s and D_r values used for site response simulations (after Nakakita and Watanabe, 1981; Iwasaki; 2009; Elgamal et al., 1996; and Cubrinovski, 1996).

Table 5-3: Parameters Used in Port Island Site Response Analyses

Layer No.	USCS Soil Type	Depth to Bottom (m)	Model	ρ (Mg/m ³)	Gr (kPa)	ν	ϕ (deg)	ϕ_{PT} (deg)	c (kPa)	c_1, c_2, c_3	d_1, d_2, d_3, γ_1	Hyd. Cond. (cm/s)
1	Fill (SP to SM)	1	PD2	1.95	2.76E+04	0.49 / 0	29.95	20	0.1	0.07, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
2	Fill (SP to SM)	2	PD2	1.95	4.62E+04	0.49 / 0	29.95	20	0.1	0.07, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
3	Fill (SP to SM)	3	PD2	1.95	6.11E+04	0.49 / 0	29.95	20	0.1	0.07, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
4	Fill (SP to SM)	4	PD2	1.95	7.34E+04	0.49 / 0	29.95	20	0.1	0.07, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
5	Fill (SP to SM)	5	PD2	1.95	8.19E+04	0.49 / 0	29.95	20	0.1	0.07, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
6	Fill (SP to SM)	6	PD2	1.95	8.19E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
7	Fill (SP to SM)	7	PD2	1.95	8.28E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
8	Fill (SP to SM)	8	PD2	1.95	8.36E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
9	Fill (SP to SM)	9	PD2	1.95	8.44E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
10	Fill (SP to SM)	10	PD2	1.95	8.52E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
11	Fill (SP to SM)	11	PD2	1.95	8.60E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
12	Fill (SP to SM)	12	PD2	1.95	8.68E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
13	Fill (SP to SM)	13	PD2	1.95	8.76E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
14	Fill (SP to SM)	14	PD2	1.95	8.85E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
15	Fill (SP to SM)	15	PD2	1.95	8.93E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
16	Fill (SP to SM)	16	PD2	1.95	9.01E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
17	Fill (SP to SM)	17	PD2	1.95	9.10E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
18	Fill (SP to SM)	18	PD2	1.95	9.18E+04	0.49 / 0	29.95	20	0.1	0.038, 2.6, 0.1875	0.1225, 2.75, 0, 0.765	1E-04
19	Alluvial Clay	19	PI	2.00	6.48E+04	0.49 / 0	0	na	84.8	na, na, na	na, na, na, na	1E-06
20	Alluvial Clay	20	PI	2.00	6.48E+04	0.49 / 0	0	na	88.7	na, na, na	na, na, na, na	1E-06
21	Alluvial Clay	21	PI	2.00	6.48E+04	0.49 / 0	0	na	92.6	na, na, na	na, na, na, na	1E-06
22	Alluvial Clay	22	PI	2.00	6.48E+04	0.49 / 0	0	na	96.5	na, na, na	na, na, na, na	1E-06
23	Alluvial Clay	23	PI	2.00	6.48E+04	0.49 / 0	0	na	100.4	na, na, na	na, na, na, na	1E-06
24	Alluvial Clay	24	PI	2.00	6.48E+04	0.49 / 0	0	na	104.4	na, na, na	na, na, na, na	1E-06
25	Alluvial Clay	25	PI	2.00	6.48E+04	0.49 / 0	0	na	108.3	na, na, na	na, na, na, na	1E-06

Table 5-3: Parameters Used in Port Island Site Response Analyses , Continued

Layer No.	USCS Soil Type	Depth to Bottom (m)	Model	ρ (Mg/m ³)	G_r (kPa)	ν	ϕ (deg)	ϕ_{PT} (deg)	c (kPa)	c_1, c_2, c_3	d_1, d_2, d_3, y_1	Hyd. Cond. (cm/s)
26	Alluvial Clay	26	PI	2.00	6.48E+04	0.49 / 0	0	na	112.2	na, na, na	na, na, na, na	1E-06
27	Alluvial Clay	27	PI	2.00	6.48E+04	0.49 / 0	0	na	116.1	na, na, na	na, na, na, na	1E-06
28	Alluvial Clay	28	PI	2.00	6.48E+04	0.49 / 0	0	na	120.1	na, na, na	na, na, na, na	1E-06
29	Sand and Clay ^(a)	29	PI	1.93	1.16E+05	0.49 / 0	0	na	123.9	na, na, na	na, na, na, na	1E-06
30	Sand and Clay	30	PI	1.93	1.16E+05	0.49 / 0	0	na	127.5	na, na, na	na, na, na, na	1E-06
31	Sand and Clay	31	PI	1.93	1.16E+05	0.49 / 1	0	na	131.1	na, na, na	na, na, na, na	1E-06
32	Sand and Clay	32	PI	1.93	1.16E+05	0.49 / 2	0	na	134.7	na, na, na	na, na, na, na	1E-06

Notes:

(a) Interbedded sand and clay modeled as clay.

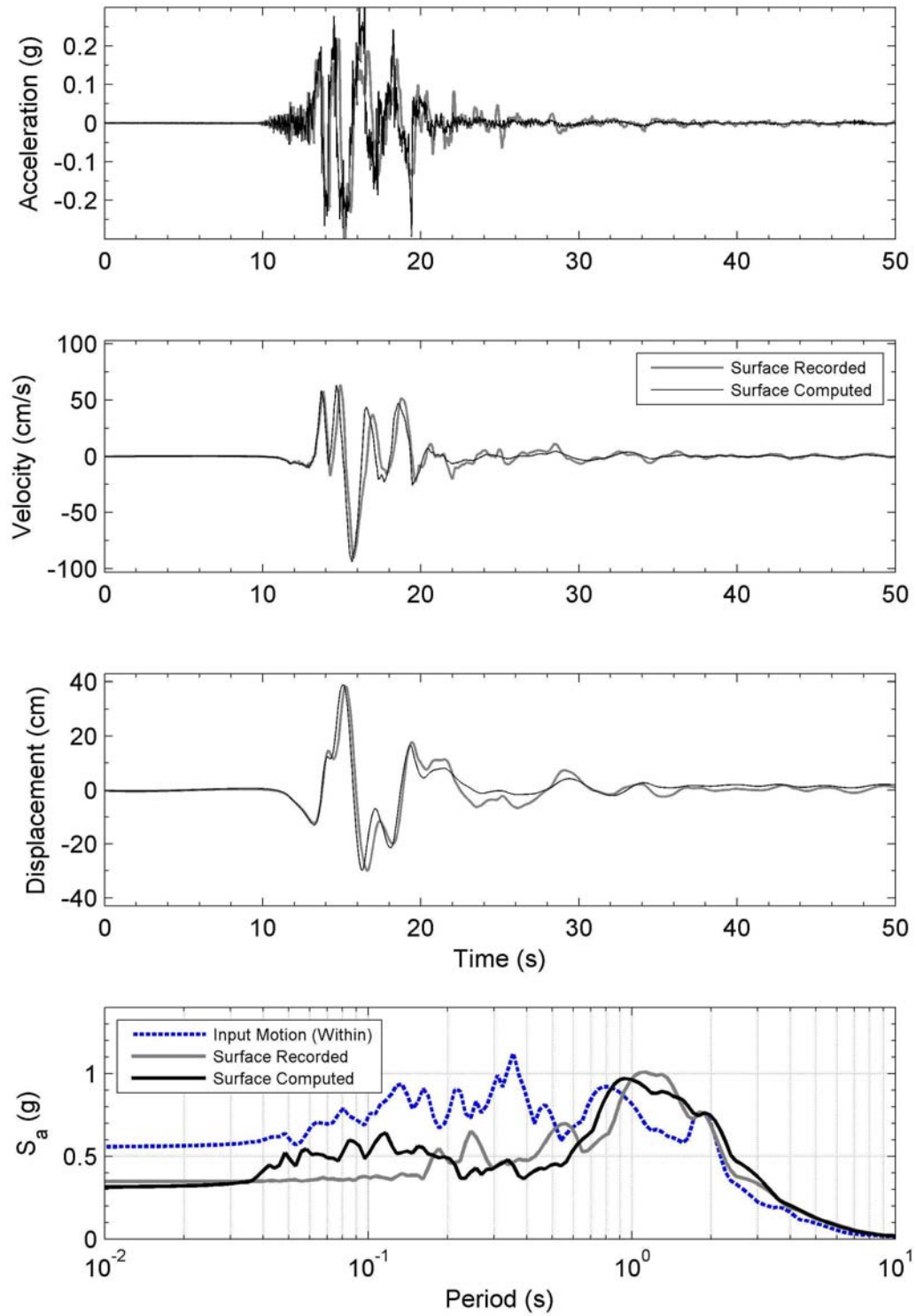


Figure 5-32: Comparison of simulated and recorded ground motions for Port Island 00 component Kobe earthquake.

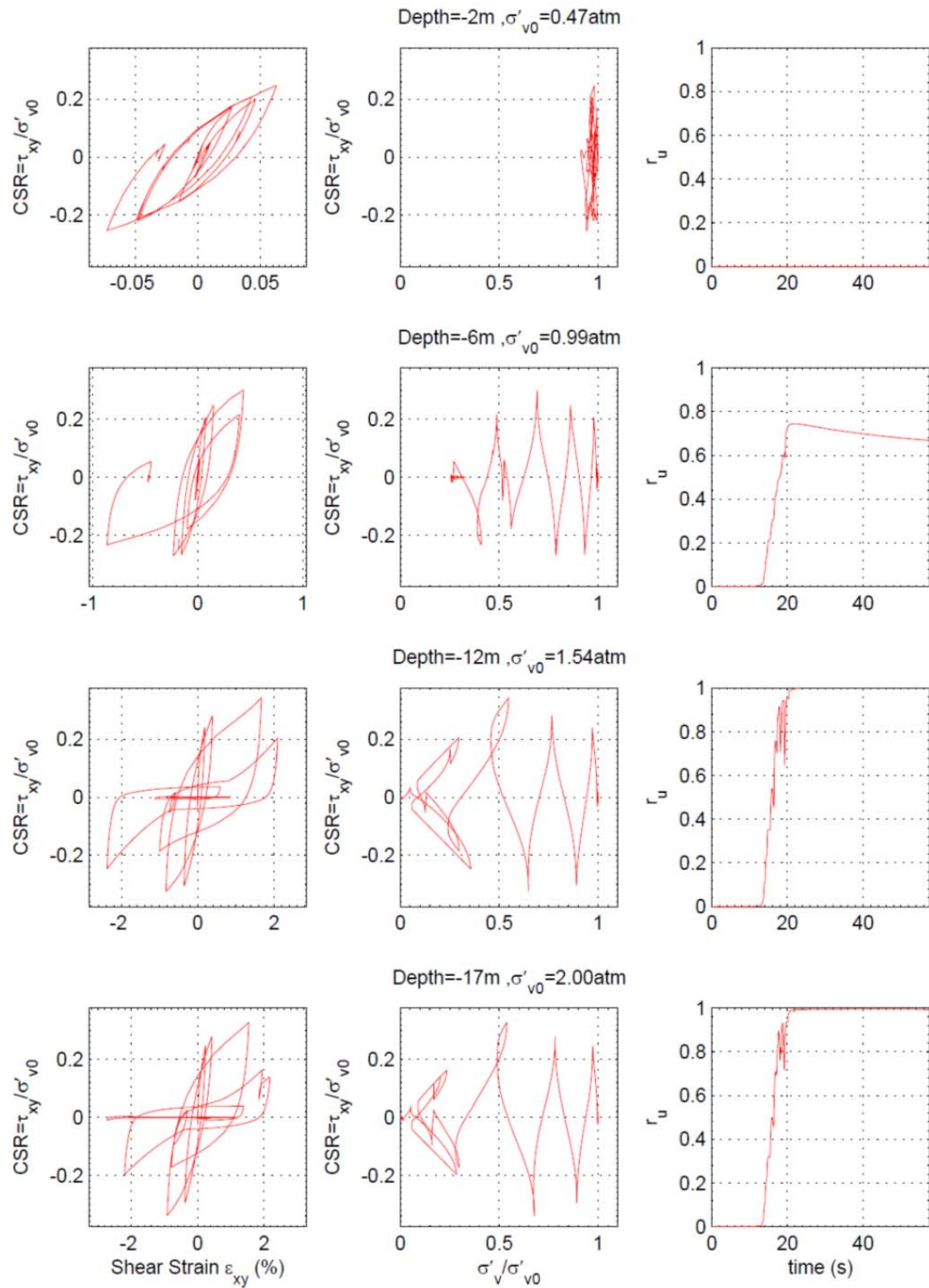


Figure 5-33: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 00 component Kobe earthquake.

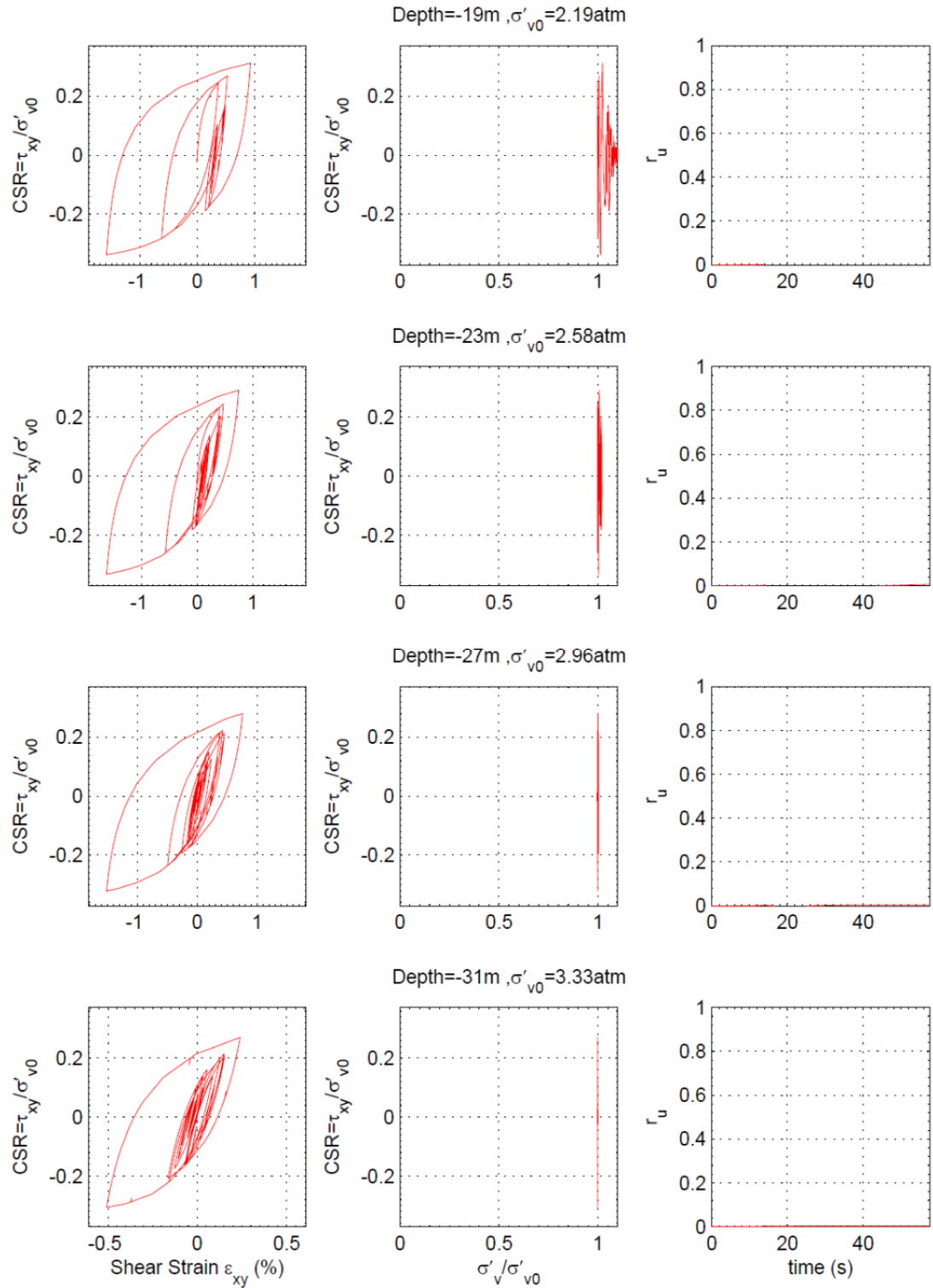


Figure 5-34: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 00 component Kobe earthquake.

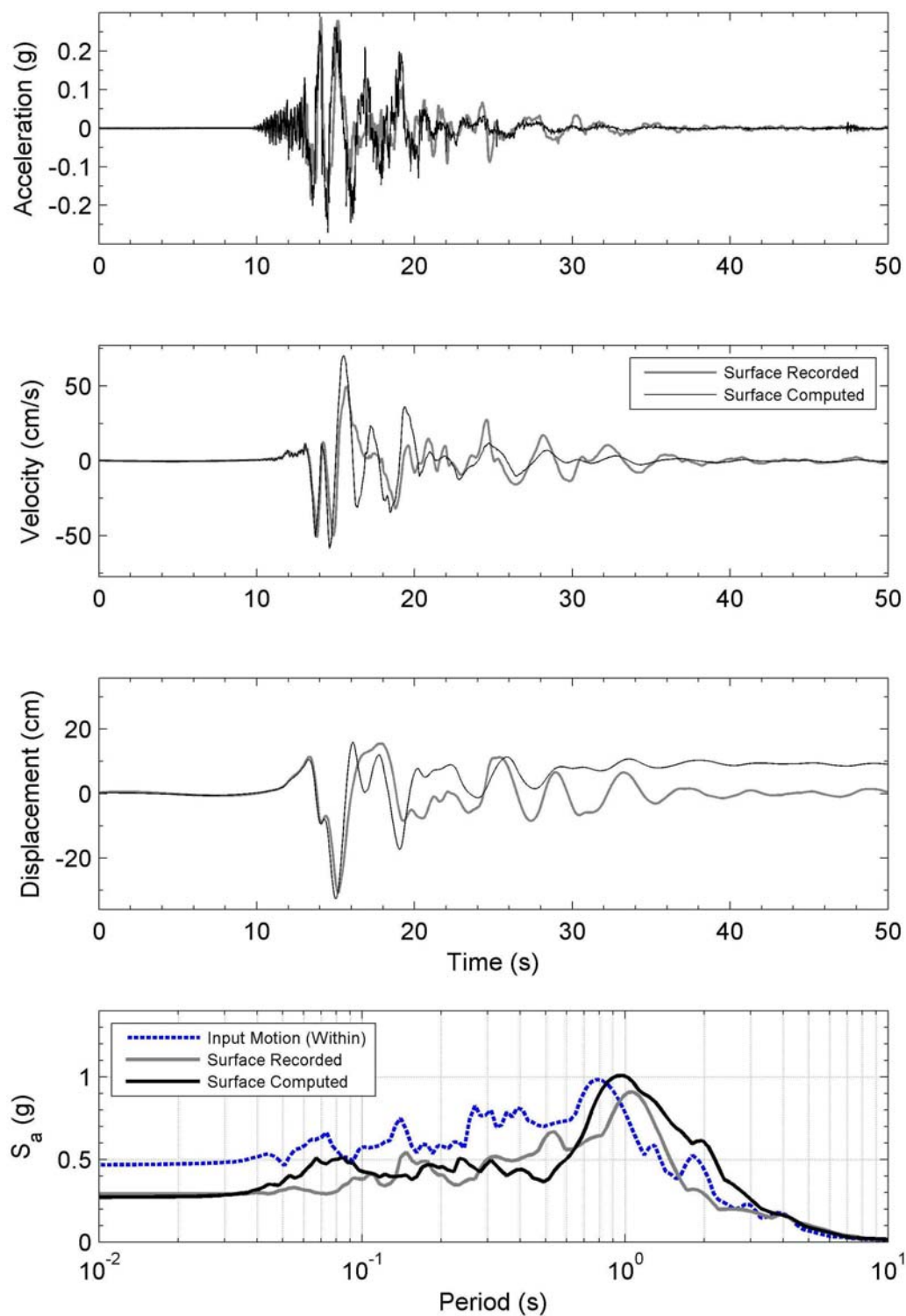


Figure 5-35: Comparison of simulated and recorded ground motions for Port Island 90 component Kobe earthquake.

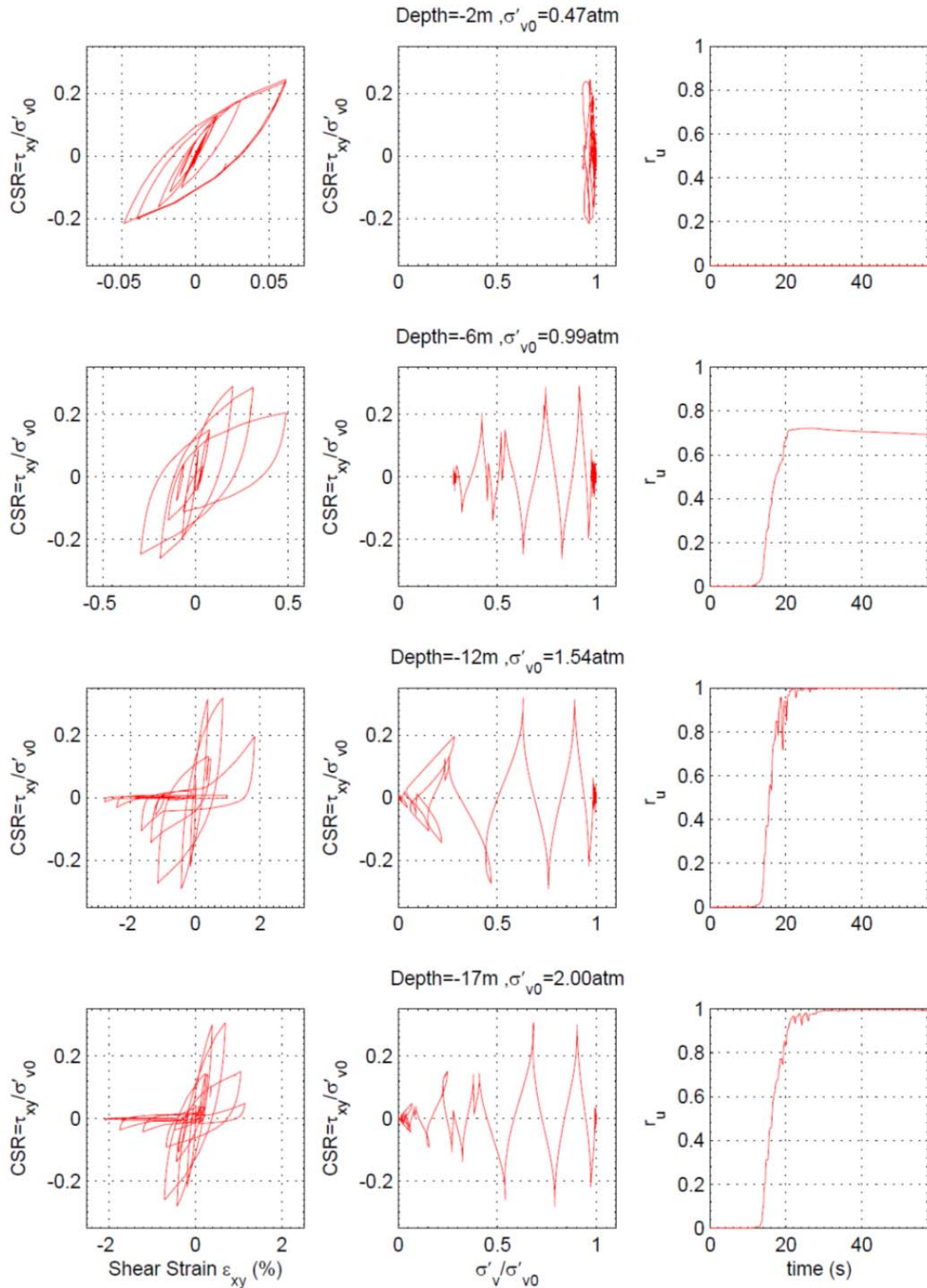


Figure 5-36: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 90 component Kobe earthquake.

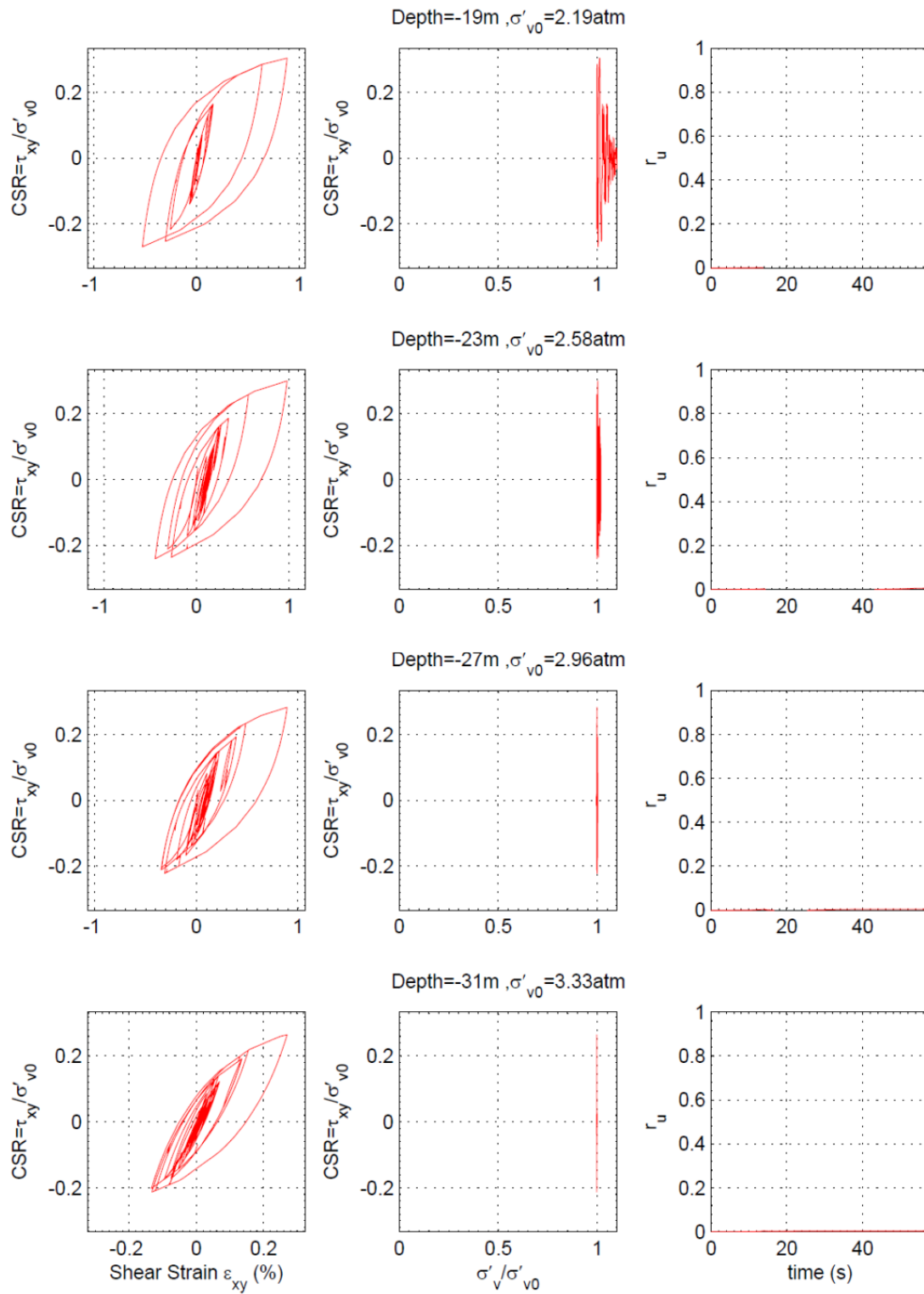


Figure 5-37: Normalized shear stress-strain, normalized shear vs. effective stress and excess porewater pressure ratio vs. time for Port Island 90 component Kobe earthquake.

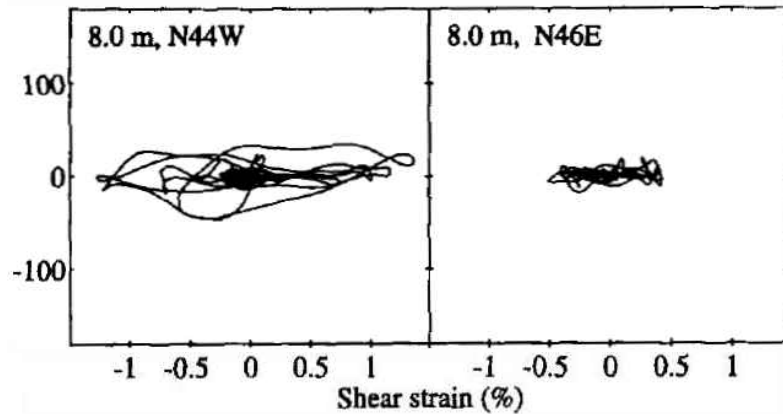


Figure 5-38: Average shear stress vs. shear strain for the depth interval of 0 to 16 m calculated from the recorded PI motions by Elgamal et al. (1996).

5.6 Summary and Conclusions

A rigorous calibration of the UCSD PDMY2 model was performed with focus on capturing salient soil behavior features for 1-D site response analyses with and without porewater pressure generation and liquefaction. A set of model parameters were developed for a typical clean sand within a range of relative densities. The parameters were calibrated based on single element cyclic simple shear test simulations which were performed for a range of confining pressures that represent depth ranges common in 1-D site response analysis of liquefiable soils. The model was calibrated for consistency with the semi-empirical liquefaction triggering criteria of Idriss and Boulanger (2006) and post-liquefaction shear strain accumulation rates from Wu (2002). The calibrated model shows generally good agreement with the K_{σ} values of Boulanger and Idriss (2004), with the MSF values of Youd et al. (2001) and Idriss and Boulanger

(2008), and with post-liquefaction shear strain per cycle reported in CSS tests by Wu (2002).

The calibrated model was validated by performing 1-D site response simulations of two well-known vertical array sites that recorded earthquakes where soil liquefaction occurred: Kobe Port Island and the WLA. For the WLA site, the validation was performed for the Elmore Ranch earthquake in which liquefaction did not occur and the Superstition Hills earthquake, where liquefaction did occur. Computed surface time histories of acceleration, velocity and displacement, and response spectra were generally in good agreement with recorded data. The good agreement was achieved using the calibrated model parameters, with only minor site-specific adjustments to shear wave velocity and the contraction parameter (c_1).

5.7 References

- Ambraseys, N. N. (1988). "Engineering seismology." *Earthquake Engrg. and Struct. Dynamics*, 17, pp 1–105.
- Andrus, R. D., and Stokoe, K. H., II (2000). "Liquefaction resistance of soils from shear-wave velocity." *J. Geotech. and Geoenviron. Engrg.*, ASCE, 126(11), pp 1015–1025.
- Arango, I. (1996). "Magnitude scaling factors for soil liquefaction evaluations." *J. Geotech. Engrg.*, ASCE, 122(11), pp 929–936.
- Bardet, J.P., Oka, F., Sugito, M. and Yashima, A. (1995). "The great Hanshin Earthquake disaster," *Prelim. Investigation Rep.*, Univ. of Southern California, Los Angeles, Calif.
- Bennett, M.J., McLaughlin, P.V., Sarmiento, John, and Youd, T.L. "Geotechnical investigation of liquefaction sites, Imperial Valley, California," U.S. Geological Survey Open File Report, 84-252, 103 p.

- Boulanger, R.W. and Idriss, I. M., (2004). "State normalization of penetration resistances and the effect of overburden stress on liquefaction resistance," in *Proceedings, 11th Intrn. Conf. on Soil Dyn. and Eq. Engr., and 3rd Intrn. Conf. on Eq. Geotech. Engr.*, D. Doolin et al., eds., Stallion Press, Vol. 2, pp. 484–91.
- Brandenberg, S.J., Bellana, N. and Shantz, T. (2010). "Shear wave velocity as function of standard penetration test resistance and vertical effective stress at California bridge sites", *Soil Dyn.and Eq. Engr.*, 30(2010), pp 1026-1035.
- Comartin, C.D., Greene, M. and Tubbesing, S.K., eds. (1995). "The Hyogo-Ken Nanbu Earthquake, great Hanshin Earthquake disaster, January 17, 1995," *Preliminary Reconnaissance Report*, EERI, Oakland, Calif.
- Cubrinovski, M., Ishihara, K. and Tanizawa, F. (1996). "Numerical simulation of the Kobe Port Island Liquefaction," *Proc. 11th World Conf. on Eq. Engrg.*, Paper No. 330.
- Darendeli, M. (2001). "A New Family of Normalized Modulus Reduction and Material Damping Curves for Equivalent Linear Analysis," PhD Dissertation, University of Texas, Austin.
- Electrical Power Research Institute (EPRI 1993). "Guidelines for Determining Design Basis Ground Motions," Final Report, EPRI TR-102293, Palo Alto, CA, November.
- Elgamal, A.-W., Zhegal, M., Parra, E. (1996). "Liquefaction of reclaimed island in Kobe, Japan," *J. of Geotech. Engr.*, 122 (1), pp 39-49.
- Elgamal, A., Yang, Z. and Parra, E. (2002). "Computational modeling of cyclic mobility and post-liquefaction site response," *Soil Dynamics and Earthquake Engineering*, 22, pp. 259-271.
- Elgamal, A., Yang, Z., Parra, E. and Ragheb, A. (2003). "Modeling of Cyclic Mobility in Saturated Cohesionless Soils," *Int. J. Plasticity*, 19(6), pp 883-905.
- Gingery, J. R., and Elgamal, A. (2013). "Shear Stress-Strain Curves Based on the $G/G_{\max}$ Logic: A Procedure for Strength Compatibility," In IACGE 2013, ASCE, Challenges and Recent Advances in Geotechnical and Seismic Research and Practices, pp. 721-729.
- Hardin, B. O. and Drnevich, V. P. (1972). Shear modulus and damping in soils: design equation and curves. *J. Soil Mech. Found. Engng. Div., ASCE* **98**, No. 7, pp 667–691.

- Hashash, Y.M.A. and Park, D. (2001). "Non-linear one-dimensional seismic ground motion propagation in the Mississippi embayment," *Eng. Geol.*, 62(1-3), pp 185-206.
- Hatanaka, M. and Uchida, A. (1996). "Empirical correlation between penetration resistance and friction angle of sandy soils," *Soils and Foundations*, Vol. 36, No. 4.
- Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall, 733 pp.
- Holtzer, T.L., Youd, T.L. and Hanks, T.C. (1989). "Dynamics of liquefaction during the Superstition Hills Earthquake (M = 6.5) of November 24, 1987," *Science*, 244, pp 56-59.
- Idriss, I.M. and Boulanger, R.W. (2006). "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," *J. Soil Dynamics and Earthquake Eng.* 26, pp 115–30.
- Idriss, I.M. and Boulanger, R.W. (2008). *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12.
- Ishihara, K. (1993). "Liquefaction and flow failure during earthquakes," 33rd Rankine Lecture, *Geotechnique*, Vol. 43, No. 3, pp 351-415.
- Ishihara, K. (1996). *Soil Behavior in Earthquake Geotechnics*, Clarendon Press, Oxford, 250 p.
- Iwasaki, Y. (2009). "Vertical array recordings during the 1995 Hyogoken-Nambu earthquake by Kobe city, KEPCO and other organizations," *Eq. Geotech. Case Hist. for Performance-Based Design*.
- Kulhawy, F.H. and Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation Design*, Electrical Power Research Institute.
- Kwok, A.O.L., Stewart, J.P., Hashash, Y.M.A., Matasovic, N., Pyke, R., Wang, Z. and Yang, Z. (2007). "Use of exact solutions of wave propagation problems to guide implementation of nonlinear seismic ground response analysis procedures," *J. of Geotech. and Geoenvr. Engr.*, 133 (11), pp 1385-1398.
- Ladd, C.C. and Foote, R. (1974). "A new design procedure for stability of soft clays," *J. of the Geotechnical Engr. Div.*, ASCE, Vol. 100, No GT7, pp 763-786.
- Ladd, C.C. and DeGroot, D.J. (2003). Invited Paper: "Recommended practice for soft ground site characterization." The Arthur Casagrande Lecture, *Proc.*

of the 12th Panamerican Conf. on Soil Mechanics and Geotechnical Engr., Boston, MA, 3-57.

- Lysmer, J. and Kuhlemeyer, R.L. (1969). "Finite Difference Model for Infinite Media," *J. Eng. Mech.*, 95 (EMR), pp 859-877.
- Masing, G. (1926). "Eigenspannungen and verfertigung beim messing," *Proc., 2nd Int. Congress on Applied Mech.*, Zurich, Switzerland.
- Matasovic, N. (1993). "Seismic response of composite horizontally-layered soil deposits," PhD Dissertation, University of California, Los Angeles.
- Mazzoni, S., McKenna, F., Fenves, G.L. (2010). OpenSees Online Documentation, http://opensees.berkeley.edu/wiki/index.php/Main_Page, Pacific Earthquake Engineering (PEER) Center.
- Mejia, L.H. and Dawson, E.M. (2006). "Earthquake deconvolution for FLAC," 4th *Int. FLAC Symp. On Num. Modeling in Geomech*, 9 p.
- Menq, F-Y (2003). "Dynamic properties of sandy and gravelly soil," PhD Dissertation, University of Texas, Austin.
- Mroz, Z. (1967). "On the Description of Anisotropic Work Hardening," *J. Mechanics and Physics of Solids*, 15, 163-175.
- Nakakita, Y. and Watanabe, Y. (1981). "Soil stabilization by preloading in Kobe Port Island," *Proc., 9th Int. COnf. on Soil Mech. And Found. Engrg.*, Japanese Soc. Of Soil. Mech and Found. Engrg., Tokyo, Japan, pp 611-622.
- George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) (2014). Wildlife Liquefaction Array website: <http://www.nees.ucsb.edu/facilities/wla>.
- O'Rourke, T.D. (1995). "Geotechnical effects," *Preliminary Rep. from the Hyogoken-Nanbu Earthquake of January 17, 1995; Nat. Ctr. for Earthquake Engrg. Res. Bull.*, SUNY, Buffalo, N.Y., 9(1).
- Prevost, J.H. (1985). "A Simple Plasticity Theory for Frictional Cohesionless Soils," *Soil Dyn. and Eq. Engr.*, 4(1), 9-17.
- Régnier, J., Bonilla, L. F., Bard, P. Y., Bertrand, E., Hollender, F., Kawase, H., Sicilia, D., the PRENOLIN participants: D. Assimaki, D. Boldini, S. Iai, S. Kramer, E. Foerster, C. Gélis, G. Gazetas, J. Gingery, Y. Hashash, P. Moczo, S. Foti, G. Lanzo, F. Lopez-Caballero, D. Roten, K. Pitilakis, F. DeMartin, B. Jeremic, A. Nieto-Ferro, M.P. Santisi, and D. Mercerat (2014a). "PRENOLIN Project: A Benchmark on Numerical Simulation of 1-

- D Non-Linear Site Effect. 2 Preliminary Results from the Verification Phase on Idealistic Cases,” Abstract and presentation, Seismological Society of America Annual Meeting.
- Régnier, J., Bonilla, L. F., Bard, P. Y., Bertrand, E., Hollender, F., Kawase, H., Sicilia, D., the PRENOLIN participants: D. Assimaki, D. Boldini, S. Iai, S. Kramer, E. Foerster, C. Gélis, G. Gazetas, J. Gingery, Y. Hashash, P. Moczo, S. Foti, G. Lanzo, F. Lopez-Caballero, D. Roten, K. Pitilakis, F. DeMartin, B. Jeremic, A. Nieto-Ferro, M. P. Santisi, and D. Mercierat (2014b). “Quantitative comparison of multiple numerical solutions for 1-D nonlinear site response analysis. Part 1: verification phase on canonical cases,” manuscript under preparation.
- Roblee, C. and Chiou, B. (2004). “A Proposed Geindex Model for Design Selection of Non-Linear Properties for Site Response Analyses,” *International Workshop on Uncertainties in Nonlinear Soil Properties and Their Impact on Modeling Dynamic Soil Response*, PEER Headquarters, UC Berkeley.
- Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., Zettler, T.E. (2002). Evaluation of Soil and Rock Properties, FHWA Geotechnical Engineering Circular No. 5, Report No. FHWA-IF-02-034.
- Schnabel, P.B., Lysmer, J. and Seed, H.B. (1972). “SHAKE: A computer program for earthquake response analysis of horizontally layered sites,” Report No. UCB/EERC-72/12, University of California, Berkeley, December, 102p.
- Seed, H. B. (1983). “Earthquake-resistant design of earth dams.” *Proc., Symp. Seismic Des. of Earth Dams and Caverns*, ASCE, New York, 41–64.
- Seed, H.B. and Idriss, I.M., (1971). “Simplified procedure for evaluating soil liquefaction potential,” *J. of the Geotech. Engr. Div.*, ASCE, 97(9), pp 1249-1273.
- Seed, R. B., and Harder, L. F., Jr. (1990). “SPT-based analysis of cyclic pore pressure generation and undrained residual strength.” *Proc., H. Bolton Seed Memorial Symp.*, BiTech Publishers Ltd., Vancouver, pp 351–376.
- Stewart, J.P., Kowk, A., Hashash, Y.M.A., Matasovic, N., Pyke, R., Wang, Z. and Yang, Z. (2008). “Benchmarking of Nonlinear Geotechnical Ground Response Analysis Procedures,” PEER Report 2008/04.
- Weiler, W.A. (1988). “Small strain shear modulus of clay,” *Proc. Of ASCE Conference on Earthquake Engr. and Soil Dyn. II: Recent Advances in Ground Motion Evaluation*, GSP 20, ASCE, New York, pp 331-345.

- Wu, J. (2002). "Liquefaction triggering and post-liquefaction deformation of Monterey 0/30 sand under uni-directional cyclic simple shear loading," PhD Dissertation, U.C. Berkeley.
- Yang, Z. and Elgamal, A. (2008). "Calibration of a sand model for predicting liquefaction triggering and liquefaction-induced lateral spreading," unpublished draft manuscript.
- Yang, Z., Elgamal, A., Parra, E. (2003). "Computational model for cyclic mobility and associated shear deformation," *J. of Geotech. and Geoenvr. Engr.*, 129 (2), pp 1119-1127.
- Yang, Z., Lu, J., Elgamal, A. (2008). OpenSees Soil Models and Solid-Fluid Coupled Elements, User's Manual, Version 1.0, UCSD Dept. of Structural Engr.
- Youd, T. L., and Perkins, M. (1978). "Mapping liquefaction-induced ground failure potential," *J. Geotech. Engr. Div.*, ASCE 104(GT4), pp 433-46.
- Youd, T.L. and Holzer, T.L. (1994). "Piezometer performance at the Wildlife liquefaction site," *J. of Geotech. Engr.*, ASCE, 120(6), pp 975-995.
- Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., 2001. "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils," *J. Geotech. and Geoenvr. Eng.*, ASCE 127(10), pp 817-33.
- Youd, T.L., Bartholomew, H.A.J., Proctor, J.S. (2004). "Geotechnical logs and data from permanently instrumented field sites: Garner Valley Downhole Array (GVDA) and Wildlife Liquefaction Array (WLA)," Brigham Young University report to NEES.
- Youd, T.L., Steidl, J.H. and Steller, R.A. (2007). "Instrumentation of the Wildlife Liquefaction Array," *4th Intr. Conf. on Earthquake Geotech. Engr.*, Thessaloniki, Greece, Paper No. 1251.
- Zeghal, M. and Elgamal, A. (1994). "Analysis of site liquefaction using earthquake records," *J. Geotech. and Geoenvr. Engr.*, ASCE, 120(6), pp. 996-1017.

Chapter 6 Parametric Site Response Analyses

6.1 Introduction

There are several limitations in the empirical data analysis and model presented in Sections 2 and 4. The database is limited to only 19 ground motion recordings from seven earthquakes. If/when available, larger dataset would be preferable in order to provide improved statistical robustness. Moreover, limited geotechnical data is available for these sites. As such, the empirical data is not adequate for evaluation of factors that could affect liquefied site response such as the liquefiable layer thickness, its depth, and the soil relative density. To address the limitations of the empirical data, a series of parametric 1-D site response analyses were performed to provide a much larger synthetic dataset, and to permit a better understanding of how geotechnical conditions could affect the site response and resulting response spectra.

This chapter describes the soil profiles, input ground motions and analysis procedures used in this parametric study. Excess porewater pressure results are used to evaluate whether the element-level calibration liquefaction triggering calibration still holds true when employed in the more complex site response analysis system. Liquefaction amplification factors are calculated for the parametric site response results in a way analogous to the empirically calculated liquefaction amplification factors. The site response and empirical liquefaction amplification factors are compared. The effect of various parameters are

evaluated with respect to their significance to the site response liquefaction amplification factors.

6.2 Soil Profiles

The site response analyses were performed using Opensees (Version 2.4.0) and the calibrated model parameters and modeling protocols described in Chapter 5. The analyses were performed for all combinations of three depths to the top of the liquefiable layer ($Z_{liq} = \{1,3,10\}$ m), three liquefiable layer thicknesses ($T_{liq} = \{1,3,10\}$ m) and four liquefiable layer relative densities ($D_r = \{30,50,70,80\}$ %), for a total of $3 \times 3 \times 4 = 36$ soil profiles. The groundwater level was assumed at 1 m depth below the surface in all cases. Figure 6-1 presents a diagram of the soil profiles considered in the parametric analyses. The compliant base at the bottom of the profile was modeled as bedrock with a density of 2.16 Mg/m^3 and V_s of 560 m/s, or firm soil a density of 2.16 Mg/m^3 and V_s of 335 m/s. The hydraulic conductivity of all of the soils was set to $1 \times 10^{-3} \text{ cm/s}$, a value that is typical of silt sand to sand (Power 1992). Additional explanation of the two model base conditions is provided in Section 5.3.4.

The variation of the site response model profile was designed represent a wide range of conditions encountered in practice. The profile variations are not intended to be inclusive of all possible real-world site conditions. Instead, the variations of parameters are intended to be sufficient enough for the results to show the influence of these selected parameters.

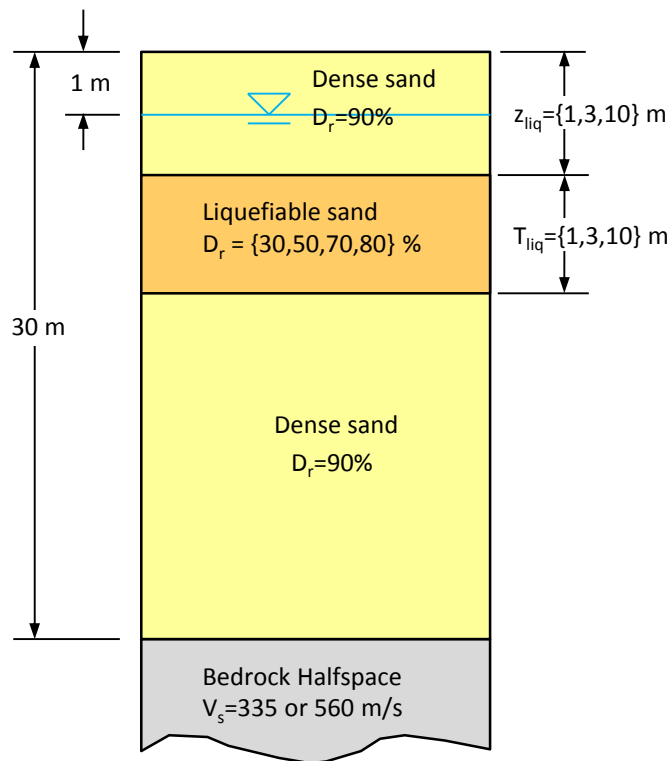


Figure 6-1: Soil profiles used in the conducted site response analyses.

6.3 Input Ground Motions

A suite of ground motions was selected to span ranges of magnitude and distance, faulting mechanism, and spectral shape and amplitude. Since the focus of these analyses is on modeling of liquefaction, the distance and magnitude pairings were limited to those that have been shown by Ambrayeses (1988) to produce liquefaction (i.e., those to the left of the black line in the upper left chart in Figure 6-2).

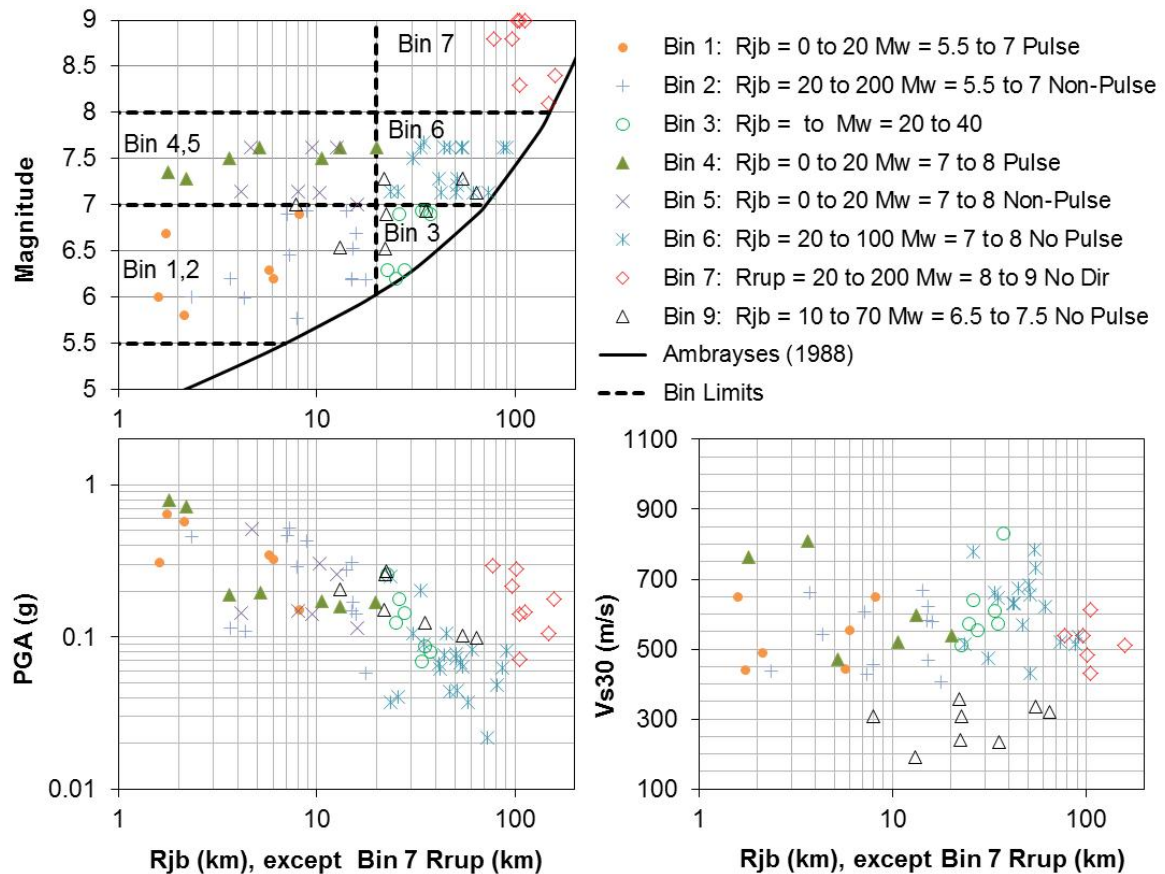


Figure 6-2: Plots showing characteristics of ground motions used as input in site response analyses.

A total of 83 ground motions grouped into 8 magnitude-distance- V_{s30} bins were used (Table 6-1). Table 6-2 presents a summary of the input ground motions and associated metadata. The shallow crustal earthquake recordings in Bins 1-6 and 9 were obtained from the PEER NGA West 2 ground motion database (Seyhan et al. 2014). Bin 7 consists of 8 interplate subduction zone earthquake recordings that were obtained from the database compiled by Carlton (2014). Most of the bins were populated with 8 to 13 ground motions, while Bin 6 was populated with 22 recordings since these magnitude-distance ranges represent a common

design scenario in active tectonic regions subject to shallow crustal earthquakes. A previously considered Bin 8 was eliminated during the course of the study, but the Bin 9 designation was retained because it had already been used in numerous computer codes and changing it to Bin 8 could have created confusion.

Table 6-1: Summary of magnitude and distance bins and the number of ground motions in each bin (in brackets).

Distance Magnitude (V_{s30})	Near Fault $R_{jb} < 20$ km		More Distant
	Non-Pulse	Pulse	$R_{jb} > 20$ km
$M_w \sim 9$ ($V_{s30} \geq 400$ m/s)	Not used		Bin 7 [8]
$M_w = 7-8$ ($V_{s30} \geq 400$ m/s)	Bin 4 [8]	Bin 5 [8]	Bin 6 [22]
$M_w = 6.5-7.5$ ($180 \leq V_{s30} \leq 360$ m/s)	Bin 9, $R_{jb}=10-70$ km, non-pulse [9]		
$M_w = 6-7$ ($V_{s30} \geq 400$ m/s)	Bin 1 [8]	Bin 2 [13]	Bin 3 [7]

Note: The 'Bin 8' designation was not used.

The motions were divided into near-fault and more distant earthquake scenarios, with Joyner-Boore site-source distances (R_{jb} , which is the closest horizontal distance from the site to the surface projection of the fault rupture) less than and more than 20 km, respectively. The near-fault motions were further subdivided into motions that met the criteria of Baker (2007) for pulse-like, and those that did not.

Initially, recordings only from rock sites with $V_{s30} \geq 400$ m/s were considered (Bins 1-7), and a site response model bedrock V_s of 560 m/s was

used. The ground motions and model bedrock conditions were selected to provide similarity to the rock reference condition used in the 2008 NGA GMPEs (i.e., 1100 m/s for AS08 and CB08, 760 m/s for BA08, and 1130 m/s for CY08). The ground motions in Bins 1-7 have a geometric mean V_{s30} of 571 m/s, which is consistent with the model bedrock V_s of 560 m/s.

Bin 9 was added after initial results with Bin 1-7 as input showed less amplification of long period spectral accelerations at liquefaction sites than expected compared to the empirical data. Bin 9 consists of ground motions from stiff soil sites with V_{s30} values of 180 to 360 m/s, M_w of 6.5 to 7.5, site-source distances of 10 to 70 km, and no pulse. The Bin 9 addition was made in recognition of the fact that bedrock is encountered at depths greater than 30 m at many liquefaction sites, and thus firm soil (rather than rock) conditions could be present at the base of the profile for many liquefiable soil sites. The Bin 9 input was also used to evaluate whether the longer period energy of the firm ground site recordings could affect liquefaction site response differently than the bedrock site recordings.

Table 6-2: Summary Site Response Input Ground Motions

Bin No.	Seq. No. ^(a)	Earthquake	Date	Station	Az. (deg)	Mom. Mag.	R _{rup}	V _{s30} (m/s)	T _{max} (s)	PGA (g)	Mech. ^(b)
1	150	Coyote Lake	8/6/1979	Gitroy Array #6	246	5.74	3.1	663	11.4	0.38	SS
	285	Irpina, Italy-01	11/23/1980	Bagnoli Irpinio	72	6.9	8.2	650	10.0	0.15	N
	568	San Salvador	10/10/1986	Geotech Investig Center	121	5.8	6.3	489	8.0	0.58	SS
	1086	Northridge-01	1/17/1994	Sylmar - Olive View Med FF	20	6.69	5.3	441	8.3	0.64	RO
	2734	Chi-Chi, Taiwan-04	9/20/1999	CHY074	355	6.2	6.2	553	12.5	0.32	SS
	3473	Chi-Chi, Taiwan-06	9/25/1999	TCU078	288	6.3	11.5	443	25.0	0.35	RO
	4040	Bam, Iran	12/26/2003	Bam	277	6.6	1.7	487	16.0	0.74	SS
	4097	Parkfield-02, CA	9/28/2004	Slack Canyon	0	6	3.0	648	8.0	0.31	SS
	28	Parkfield	6/28/1966	Cholame - Shandon Array #12	50	6.19	17.6	409	8.0	0.06	SS
	164	Imperial Valley-06	10/15/1979	Cerro Prieto	147	6.53	15.2	472	8.9	0.17	SS
2	410	Coalinga-05	7/22/1983	Palmer Ave	270	5.77	12.1	458	16.0	0.29	RO
	550	Chalfant Valley-02	7/21/1986	Bishop - Paradise Lodge	70	6.19	18.3	585	8.0	0.15	SS
	594	Whittier Narrows-01	10/1/1987	Baldwin Park - N Holly	180	5.99	16.7	545	10.0	0.11	O
	765	Loma Prieta	10/18/1989	Gitroy Array #1	0	6.93	9.6	1428	13.3	0.43	O
	801	Loma Prieta	10/18/1989	San Jose - Santa Teresa Hills	225	6.93	14.7	672	26.3	0.28	O
	901	Big Bear-01	6/28/1992	Big Bear Lake - Civic Center	0	6.46	8.3	430	8.3	0.52	SS
	957	Northridge-01	1/17/1994	Burbank - Howard Rd.	60	6.69	16.9	582	10.0	0.14	RO
	1111	Kobe, Japan	1/16/1995	Nishi-Akashi	0	6.9	7.1	609	8.0	0.47	SS
	2622	Chi-Chi, Taiwan-03	9/20/1999	TCU071	0	6.2	16.5	625	25.0	0.31	RO
	2632	Chi-Chi, Taiwan-03	9/20/1999	TCU084	0	6.2	9.3	665	25.0	0.12	RO
	4133	Parkfield-02, CA	9/28/2004	Parkfield - Vineyard Cany 2W	90	6	3.5	439	8.0	0.46	SS
3	751	Loma Prieta	10/18/1989	Calaveras Reservoir	90	6.93	35.5	572	11.9	0.09	O
	791	Loma Prieta	10/18/1989	SAGO South - Surface	261	6.93	34.3	609	20.0	0.07	O
	3507	Chi-Chi, Taiwan-06	9/25/1999	TCU129	0	6.3	24.8	511	22.2	0.26	RO
	2709	Chi-Chi, Taiwan-04	9/20/1999	CHY035	0	6.2	25.1	573	100	0.12	SS
	3300	Chi-Chi, Taiwan-06	9/25/1999	CHY074	0	6.3	29.3	553	33.3	0.14	RO
	5483	Iwate	6/13/2008	AKTH05	0	6.9	39.4	829	26.7	0.08	RO
	5474	Iwate	6/13/2008	AKT019	0	6.9	28.8	640	40.0	0.18	RO

Table 6-2: Summary Site Response Input Ground Motions , Continued

Bin No.	Seq. No. ^(a)	Earthquake	Date	Station	Az. (deg)	Mom. Mag.	R _{rup}	V _{s30} (m/s)	T _{max} (s)	PGA (g)	Mech. ^(b)
4	143	Tabas, Iran	9/16/1978	Tabas	0	7.35	2.1	767	8.0	0.81	RO
	828	Cape Mendocino	4/25/1992	Petrolia	287	7.01	8.2	422	14.3	0.62	RO
	879	Landers	6/28/1992	Lucerne	270	7.28	2.2	1369	100	0.73	SS
	1148	Kocaeli, Turkey	8/17/1999	Arcelik	80	7.51	13.5	523	11.4	0.17	SS
	1165	Kocaeli, Turkey	8/17/1999	Izmit	265	7.51	7.2	811	8.0	0.19	SS
	1482	Chi-Chi, Taiwan	9/20/1999	TCU039	94	7.62	19.9	541	40.0	0.17	O
	1515	Chi-Chi, Taiwan	9/20/1999	TCU082	85	7.62	5.2	473	40.0	0.20	O
	1548	Chi-Chi, Taiwan	9/20/1999	TCU128	84	7.62	13.1	600	40.0	0.16	O
5	827	Cape Mendocino	4/25/1992	Fortuna - Fortuna Blvd	0	7.01	20.0	457	14.3	0.12	RO
	1202	Chi-Chi, Taiwan	9/20/1999	CHY035	90	7.62	12.7	573	20.0	0.26	O
	1490	Chi-Chi, Taiwan	9/20/1999	TCU050	90	7.62	9.5	542	26.7	0.14	O
	1520	Chi-Chi, Taiwan	9/20/1999	TCU088	0	7.62	18.2	665	20.0	0.52	O
	1611	Duzce, Turkey	11/12/1999	Lamont 1058	0	7.14	0.2	529	13.3	0.10	SS
	1612	Duzce, Turkey	11/12/1999	Lamont 1059	0	7.14	4.2	551	13.3	0.15	SS
	1618	Duzce, Turkey	11/12/1999	Lamont 531	0	7.14	8.0	638	13.3	0.15	SS
	1787	Hector Mine	10/16/1999	Hector	0	7.13	11.7	726	40.0	0.31	SS
	891	Landers	6/28/1992	Silent Valley - Poppet Flat	0	7.28	50.9	659	8.3	0.05	SS
	897	Landers	6/28/1992	Twentynine Palms	0	7.28	41.4	635	8.3	0.07	SS
6	1166	Kocaeli, Turkey	8/17/1999	Iznik	180	7.51	30.7	477	8.0	0.11	SS
	1230	Chi-Chi, Taiwan	9/20/1999	CHY079	90	7.62	47.5	573	40.0	0.04	O
	1256	Chi-Chi, Taiwan	9/20/1999	HWA002	0	7.62	56.9	789	13.3	0.07	O
	1284	Chi-Chi, Taiwan	9/20/1999	HWA035	90	7.62	48.4	677	40.0	0.08	O
	1350	Chi-Chi, Taiwan	9/20/1999	ILA067	90	7.62	38.8	665	26.7	0.21	O
	1437	Chi-Chi, Taiwan	9/20/1999	TAP053	90	7.62	92.4	539	40.0	0.08	O
	1463	Chi-Chi, Taiwan	9/20/1999	TCU003	90	7.62	86.6	517	26.7	0.06	O
	1582	Chi-Chi, Taiwan	9/20/1999	TTN032	90	7.62	57.7	734	26.7	0.07	O
	1613	Duzce, Turkey	11/12/1999	Lamont 1060	0	7.14	25.9	782	13.3	0.04	SS
	1616	Duzce, Turkey	11/12/1999	Lamont 362	0	7.14	23.4	517	13.3	0.04	SS
	1626	Sitka, Alaska	7/30/1972	Sitka Observatory	90	7.68	34.6	650	12.5	0.09	SS

Table 6-2: Summary Site Response Input Ground Motions , Continued

Bin No.	Seq. No. ^(a)	Earthquake	Date	Station	Az. (deg)	Mom. Mag.	R _{rup}	V _{s30} (m/s)	T _{max} (s)	PGA (g)	Mech. ^(b)
6	1627	Caldiran, Turkey	11/24/1976	Maku	41	7.21	50.8	433	100	0.07	SS
	1786	Hector Mine	10/16/1999	Heart Bar State Park	90	7.13	61.2	625	33.4	0.08	SS
	1795	Hector Mine	10/16/1999	Joshua Tree N.M. - Keys View	90	7.13	50.4	686	38.5	0.08	SS
	1833	Hector Mine	10/16/1999	Snow Creek	90	7.13	72.9	524	35.0	0.02	SS
	1836	Hector Mine	10/16/1999	Twentynine Palms	90	7.13	42.1	635	14.3	0.06	SS
	3750	Cape Mendocino	4/25/1992	Loleta Fire Station	270	7.01	25.9	516	16.0	0.25	RO
	3752	Landers	6/28/1992	Forest Falls Post Office	210	7.28	45.3	436	8.0	0.11	SS
	5842	El Mayor-Cuapah	4/4/2010	Anza Borrego S.P. - Tierra Blan	0	7.2	58.0	585	40.0	0.04	SS
	5862	El Mayor-Cuapah	4/4/2010	Bombay Beach - Bertram	90	7.2	81.6	491	13.3	0.05	SS
	B32	Michoacan, Mexico	9/19/1985	Atoyac	(c)	8.1	147	Rock	(c)	0.11	Sub
7	B40	Maule, Chile	2/27/2010	Cerro Santa Lucia	(c)	8.8	77.4	540	(c)	0.29	Sub
	B42	Maule, Chile	2/27/2010	Santiago La Florida	(c)	8.8	96.1	540	(c)	0.22	Sub
	B52	South Peru	6/23/2001	Ponochile	(c)	8.4	158	511	(c)	0.18	Sub
	B54	Tohoku	3/11/2011	Tsubakidai	(c)	9	105	430	(c)	0.14	Sub
	B58	Tohoku	3/11/2011	Kuroiso	(c)	9	102	482	(c)	0.28	Sub
	B64	Cascadia1Synth	synth	Victoria	(c)	9	112	B/C	(c)	0.15	Sub
	B65	Tokachi-oki	9/26/2003	Tomuraushi	(c)	8.3	105	611	(c)	0.07	Sub
	169	Imperial Valley-06	10/15/1979	Delta	262	6.53	22.0	242	11.4	0.26	SS
	728	Superstition Hills-02	11/24/1987	Westmorland Fire Sta	90	6.54	13.0	194	11.4	0.21	SS
	757	Loma Prieta	10/18/1989	Dumbarton Bridge W. End FF	267	6.93	35.5	238	8.0	0.13	O
9	829	Cape Mendocino	4/25/1992	Rio Dell Overpass - FF	270	7.01	14.3	312	14.3	na	RO
	850	Landers	6/28/1992	Desert Hot Springs	0	7.28	21.8	359	14.3	0.15	SS
	862	Landers	6/28/1992	Indio - Coachella Canal	0	7.28	54.3	339	10.0	0.10	SS
	959	Northridge-01	1/17/1994	Canoga Park - Topanga Can	106	6.69	14.7	267	8.0	0.36	RO
	1107	Kobe, Japan	1/16/1995	Kakogawa	0	6.9	22.5	312	8.0	0.28	SS
	1766	Hector Mine	10/16/1999	Baker Fire Station	140	7.13	64.8	325	14.3	0.10	SS

Notes:

(a) Indicates NGA sequence number, except for Bin 7 ground motions which are assigned a sequence number consistent with Carlton (2014).

(b) SS = strike slip, N = normal, RO = reverse-oblique, O = oblique, Sub = subduction zone,

(c) Not available.

Response spectra for all of the ground motions are presented in Figure 6-3, and response spectra for each of the bins is presented in Figure 6-4. Appendix D presents plots of acceleration-, velocity-, and displacement-time, as well as response spectra for two horizontal components of motion for each of the recordings. One of the two components was selected for use in the site response analyses: for pulse like motions the pulse axis was used; for non-pulse motions one of the two axes was selected arbitrarily. The selected axis of motion is indicated in the Azimuth column of Table 6-2.

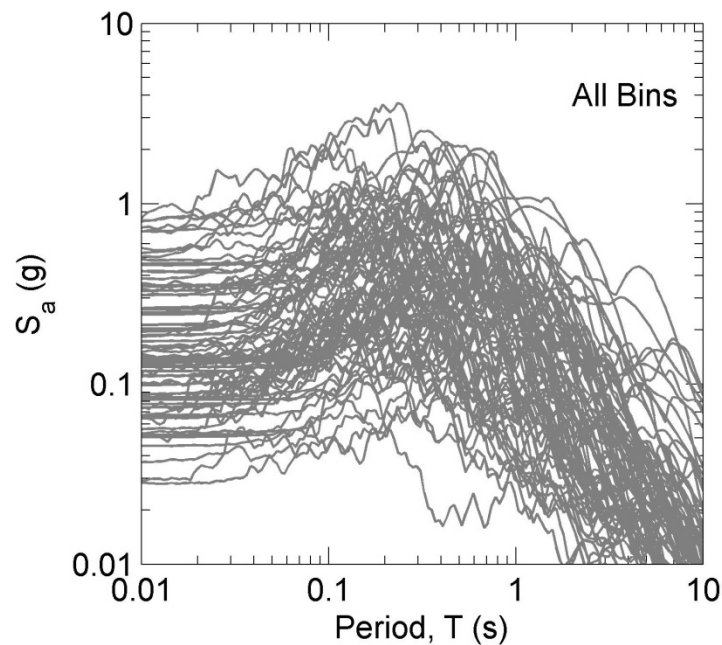


Figure 6-3: Response spectra for all the ground motions used in the site response analyses.

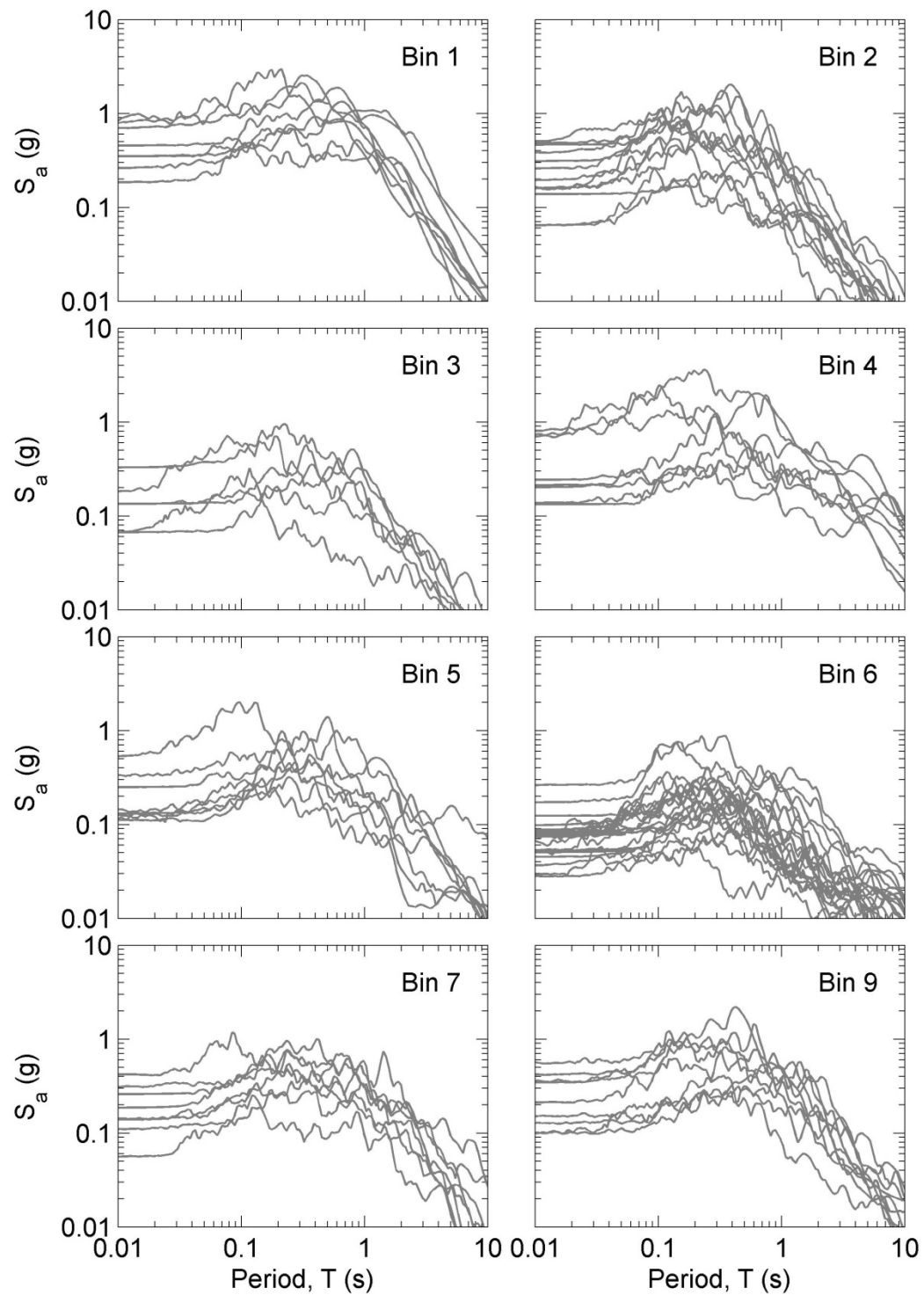


Figure 6-4: Response spectra for each ground motion bin used in the site response analyses.

For each bin, the spectral acceleration values vary by roughly an order of magnitude. This variation is reasonable considering that the 95.4 percent confidence interval (median $\pm$ 2 standard deviations) of modern GMPEs ranges by about an order of magnitude (Abrahamson et al. 2008): with a typical standard deviation of about 0.6 natural log units, the variation in spectral acceleration is $\exp(2 \cdot 0.6)/\exp(-2 \cdot 0.6) = 11$.

6.4 Analysis Procedures

Section 5.3.4 describes the general protocols for the 1-D nonlinear site response analyses. Additional procedures specific the parametric site response analyses are described below.

Each of the 36 soil profiles and 83 input ground motions were analyzed in two ways: with and without porewater pressure generation. Thus, $36 \times 83 = 2988$ run pairs or $36 \times 2 \times 83 = 5976$ total runs were made. Each run pair is analogous to the empirical pairs presented in Section 4, which consist of a recorded ground motion at a liquefaction site and a GMPE-based non-liquefaction estimate of the ground motion. The simulation without porewater pressure generation is analogous to the GMPE ground motion estimate, which does not account for liquefaction. The simulation with porewater pressure (at least for cases where excess porewater pressure ratios near unity are generated in the simulation) is analogous to the liquefied site ground motion recordings. The ratio of the response spectra from the liquefaction and non-liquefaction runs are then analogous to the AF_{liq} parameter from Chapter 4.

For the cases without porewater pressure generation, the PIMY model was used for all of the soil layers in the model based on its simplicity and ease of implementation for cases where porewater pressure generation is not modeled.

For cases with porewater pressure generation, the PDMY2 model was used for all elements. Consideration was given to use of the PIMY model for elements above and below the potentially liquefiable layer (i.e., the $D_r = 90\%$ layers). Use of the PIMY model for the $D_r = 90\%$ layers would have meant that the layers outside of the potentially liquefiable layer behaved in a fully drained manner consistent with the drained cases. However, a major drawback of using the PIMY model would have been that layers adjacent to the potentially liquefiable layer would not have been subject to softening caused by porewater pressure migration from the potentially liquefiable layer. Since porewater pressure migration is a realistic and well recognized phenomenon, it was desirable to include this behavior for the porewater pressure generation cases and thus the PDMY2 model was used for the $D_r = 90\%$ layers.

The porewater pressure degree of freedom in the finite elements was set as fixed at the nodes at and above the phreatic surface (i.e., dictating zero excess pore pressure throughout), and free at the nodes below the phreatic surface (including the base nodes). Under these fixity conditions, during the initial gravity consolidation phase a hydrostatic condition develops from the phreatic surface downward, whilst porewater pressure above the phreatic surface are zero. During dynamic loading in the porewater pressure generation case, porewater pressures are allowed to change below the phreatic surface as a

result of soil contraction and dilation (during shear loading). Free drainage is allowed at the phreatic surface, and the base of the model acts as a no-flow boundary (i.e., as if an impermeable underlying layer is present).

6.5 Results and Conclusions

6.5.1 Fidelity of Models' Liquefaction Triggering Predictions

As described in Section 5.3.2, model parameters were calibrated against the Idriss and Boulanger (2006) semi-empirical liquefaction triggering relationship at the *element* level. The full site response model represents a much more complex system than the single element simulations. Therefore it is important to show that the full site response models also provide reasonable and unbiased predictions of liquefaction triggering relative to the Idriss and Boulanger (2006) relationship. To this end, this section presents a comparison of maximum excess porewater pressure ratios (r_u) generated in the site response analyses against liquefaction triggering factors of safety calculated using the semi-empirical model of Idriss & Boulanger (2006).

There are several features of the site response model that differ from the single element simulations and thus could potentially lead to differences in response. Groundwater flow and drainage is permitted in the site response model whereas no drainage occurred in the element simulations. The groundwater flow could lead to partial drainage during the earthquake loading, and could lead to porewater pressure migration from zones of liquefaction to adjacent zones with lower contraction-induced porewater pressures. The element

simulations used uniform sinusoidal loading whereas the site response simulations used actual earthquake time histories as input.

The initial and minimum vertical effective stresses were recorded for each element for each of the site response analyses with porewater pressure generation. Then excess porewater pressure ratios were calculated according to

$$r_u = (\Delta u)_{\max} / \sigma'_{v0} = (\sigma'_{v0} - \sigma'_{v,\min}) / \sigma'_{v0} .$$

Factors of safety against liquefaction triggering were calculated for points at the top, middle and bottom of the potentially liquefiable layer using the Idriss and Boulanger (2006, 2008) method. These factors of safety (FS_{IB08}) were calculated using the surface PGA from the site response analysis without porewater pressure generation and the magnitude of the input ground motion earthquake. The stress reduction factors were based on the simplified magnitude-dependent relationship provided by Idriss and Boulanger (2006) for consistency with practice where cyclic stress values calculated by site response analysis are frequently not available. The factor of safety thus computed is indifferent to porewater pressure migration, layering and ground motion variability.

Figure 6-5 and 6-6 each present three examples of effective stress and excess porewater pressure ratio profiles and calculated FS_{IB08} . These examples show generally good agreement between r_u and FS_{IB08} . The plots also show how porewater pressure migration can occur in the model. The very dense soil below the liquefiable layer typically generates minimal excess porewater pressure, as can be observed in at the bottom of the profile in the left and middle plots in

Figure 6-5. Migration of excess porewater pressure generated in the liquefiable layer into the underlying dense layer can be observed in all three of the cases presented in Figure 6-5. In the right-most case, 6 meters of dense soil below the liquefiable layer achieved an r_u value of 1. Migration into the overlying dense layer can be observed in Figure 6-6. The excess porewater pressure migration effect may be important in site response since it tends to soften (and in some cases liquefy) otherwise non-liquefiable layers, and further soften the overall site response.

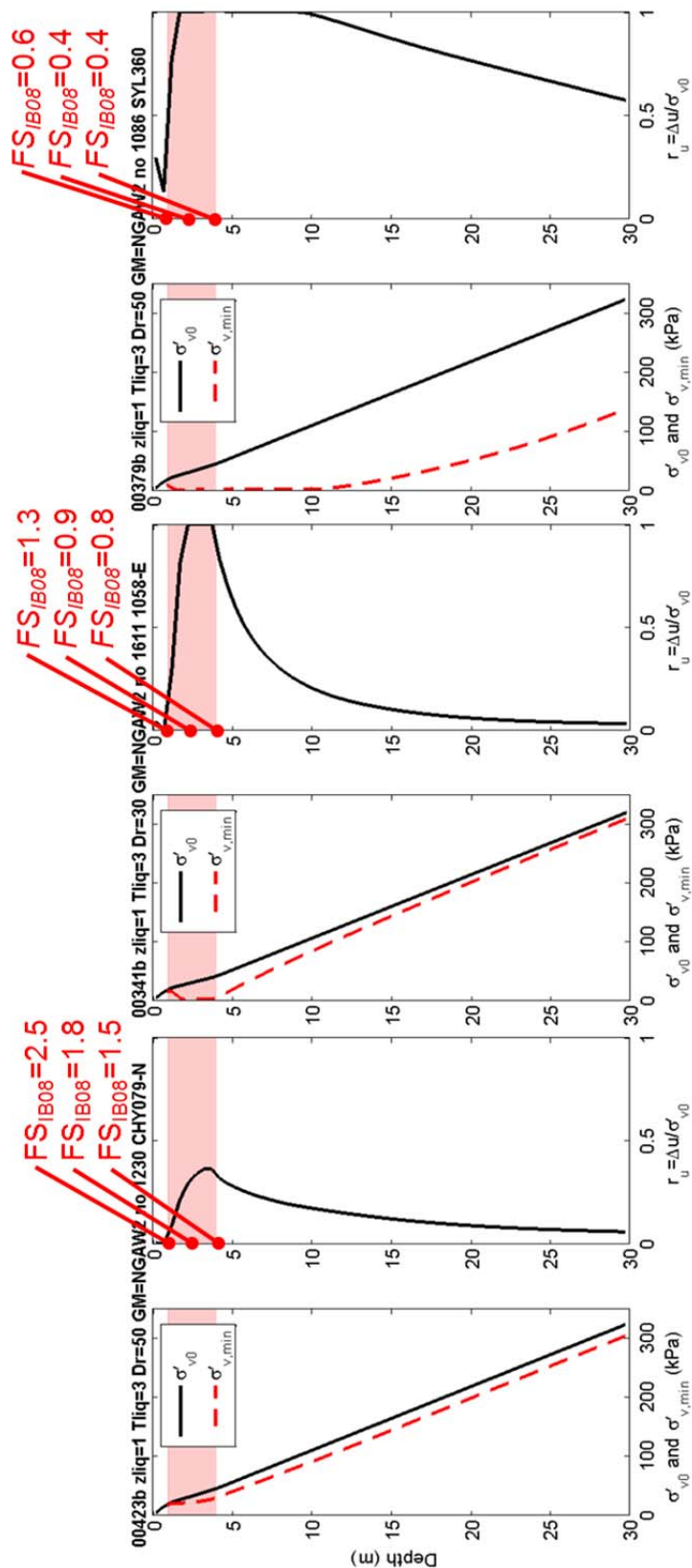


Figure 6-5: Three examples (cases 432b, 341b and 379b) of initial and minimum vertical effective stress and maximum excess porewater pressure ratio profiles, and calculated factors of safety. The red-shaded zone is the potentially liquefiable layer. All have $z_{liq} = 3\text{m}$, $T_{liq} = 1\text{m}$ and groundwater is at 1 m, but differing D_r and input ground motions as noted at top of each figure.

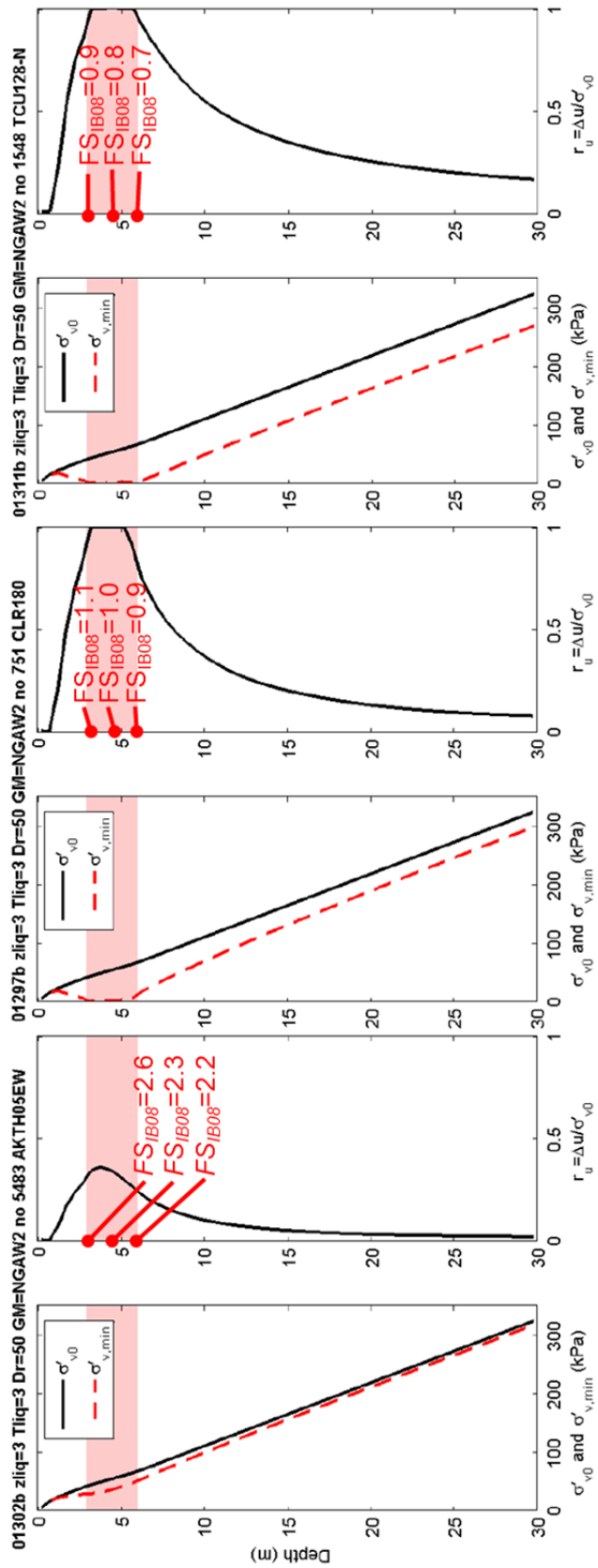


Figure 6-6: Three examples (cases 1302b, 1297b and 1297b) of initial and minimum vertical effective stress and maximum excess porewater pressure ratio profiles, and calculated factors of safety. The red-shaded zone is the potentially liquefiable layer. All have $D_r = 50\%$, $Z_{liq} = 3\text{ m}$, $T_{liq} = 3\text{ m}$ and groundwater is at 1 m, but input ground motions are different as noted at top of figure.

Figure 6-7 shows r_u plotted versus the reciprocal of FS_{IB08} , with the data color-coded by relative density. The shaded area is taken from a similar plot by Tokimatsu and Seed (1987). For r_u less than 1, the figure shows increasing $1/FS_{liq}$ with decreasing relative density. In other words, for sites that don't reach full liquefaction, higher relative densities produce higher r_u values for a given factor of safety. This is expected based on the element calibration test results as exemplified in Figure 6-8. The more dilatant $D_r = 70\%$ sample experiences several cycles of cyclic mobility with elevated (but less than unity) maximum values of r_u prior to reaching liquefaction (6% double amplitude shear strain). In contrast, the looser $D_r = 30\%$ sample is less dilatant, and thus cyclic mobility does not occur until just before liquefaction is reached. At N/N_{liq} less than about 0.95, which is the same domain as $1/FS_{IB08} < 1$, the r_u of the $D_r = 70\%$ test is higher than that of the $D_r = 30\%$ test, and this is consistent with the data in Figure 6-7.

The disagreement with Tokimatsu and Seed's range could stem from several sources. First, their data were generated from laboratory tests, and rather than an empirical liquefaction FS reciprocal they calculated $(\tau/\sigma'_0)/(\tau/\sigma'_0)_{liq}$, where (τ/σ'_0) is the laboratory test shear stress ratio and $(\tau/\sigma'_0)_{liq}$ is the shear stress ratio causing liquefaction. While this ratio is analogous to the semi-empirical factor of safety, it may not account for factors like magnitude and overburden stress in the same way or at all. Secondly, semi-empirical

liquefaction triggering methods necessarily make simplifying assumptions about dynamic site response. Most significantly, the stress

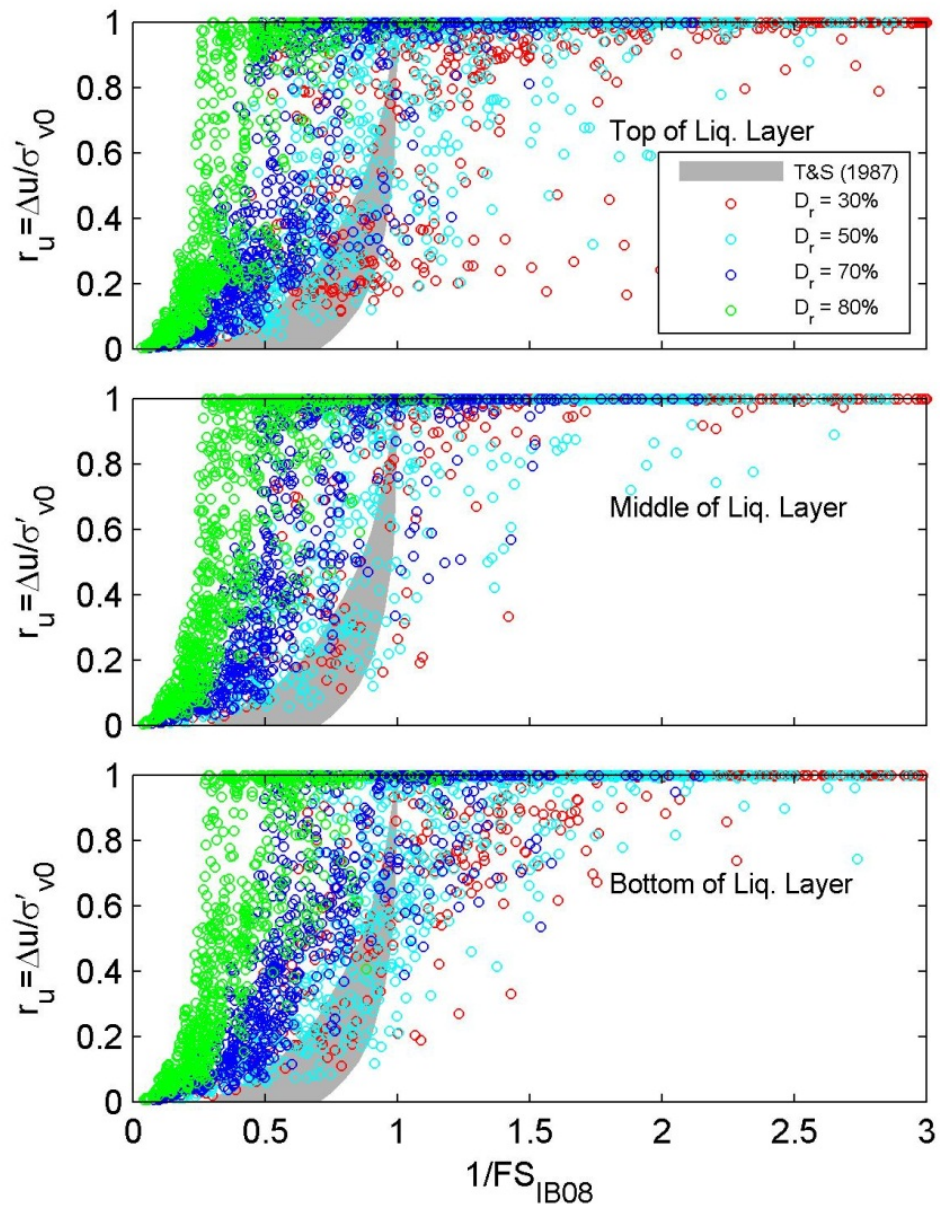


Figure 6-7: Plots of excess porewater pressure ratio versus reciprocal of liquefaction factor of safety.

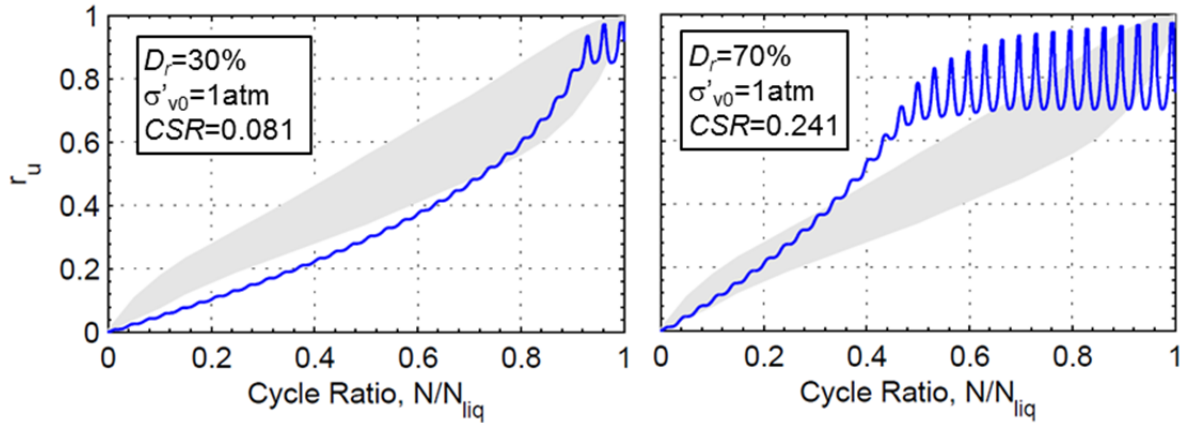


Figure 6-8: Examples of rates of r_u generation from element calibration CSS simulations.

reduction factors (r_d) used are simplified averages and actual site response can vary significantly from the values. The magnitude scaling factor is used in semi-empirical triggering methods as a proxy for equivalent number of cycles, and uncertainty in the correlation between these parameters can contribute to deviations of expected trends between r_u and liquefaction factor of safety. Thirdly, porewater pressure migration from adjacent layers could affect r_u values in the site response analysis, whereas porewater pressure migration is not significant in laboratory specimens.

Figure 6-9 presents plots of r_u versus FS_{IB08} in log-log space. Plotting the data in this manner reveals that the relationship between these parameters is linear in log-log space for $r_u < 1$.

Figure 6-7 is useful to examine trends for sites that do not fully liquefy, but the large amount of data clustered at $r_u = 1$ require other modes of examination. Figure 6-10 and Figure 6-11 present a liquefaction factor of safety histogram and cumulative probability density function (CDF), respectively, for cases with $r_u = 1$.

The plots are made for a dataset that includes the top, middle and bottom points of the liquefiable layer. Figure 6-11 shows that about 80 percent of the runs with $r_u = 1$ had liquefaction factors of safety of 1 or less. Based on these results, the site response model predicts liquefaction occurrence in a manner that is generally consistent with Idriss & Boulanger (2006) factors of safety.

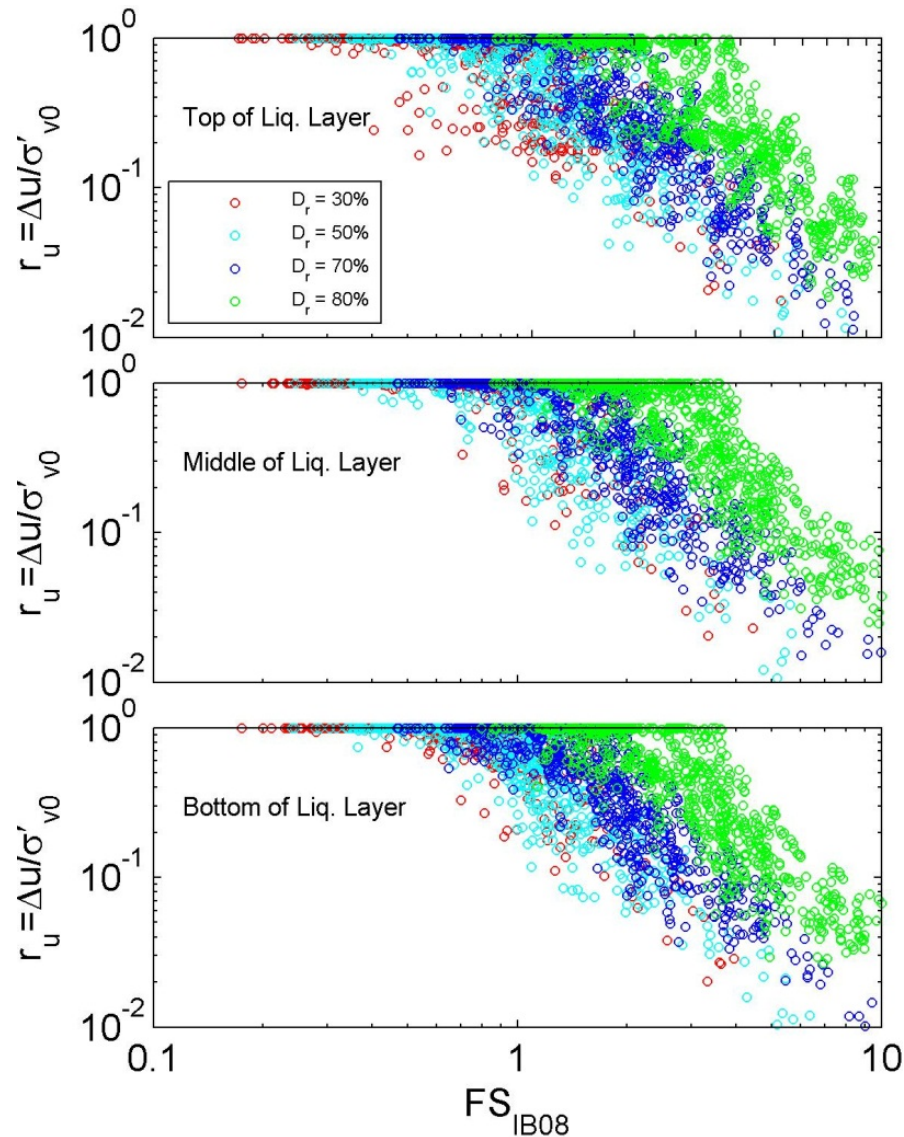


Figure 6-9: Plots of excess porewater pressure ratio versus liquefaction factor of safety.

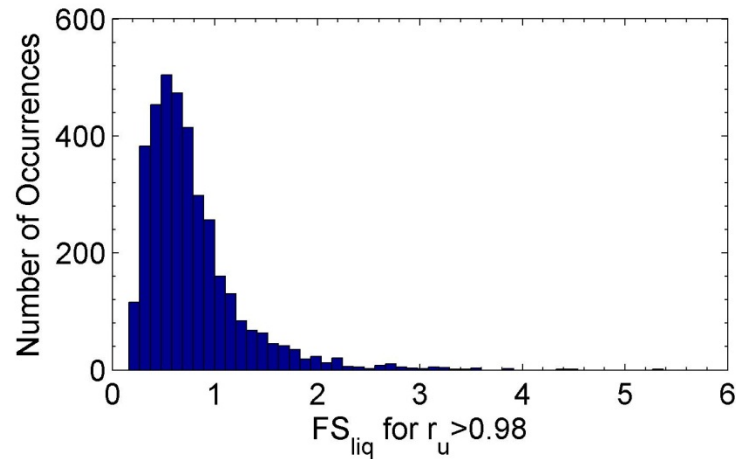


Figure 6-10: Histogram of Idriss & Boulanger (2006) liquefaction factor of safety for cases with $r_u \geq 1$.

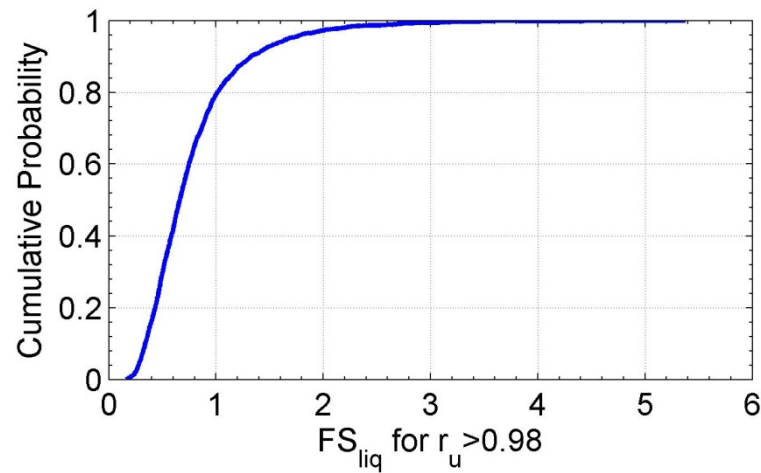


Figure 6-11: CDF of Idriss & Boulanger (2006) liquefaction factor of safety for cases with $r_u = 1$.

6.5.2 Example Time Histories

Similar to the empirical data processing described in Chapter 4, an amplification factor (AF_{liq}) is calculated for each liquefaction case pair for a range of periods:

$$AF_{liq} = Sa_{Liq} / Sa_{NonLiq} \quad (6-1)$$

where Sa_{Liq} and Sa_{NonLiq} are the period-dependent spectral ordinates for the site response cases with and without porewater pressure generation, respectively. In this calculation, the response spectra are calculated for the ground surface based on 5 percent of critical damping.

Figures 6-12 through 6-14 present examples of acceleration time histories and response spectra at the surface of the model for the with- and without-porewater pressure generation cases, and AF_{liq} for a range of periods. The three plots all show the same soil profile ($D_r = 50\%$, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$), but shaken by three different ground motions with progressively increasing intensity levels. These time histories correspond to the same three cases presented in the r_u plots of Figure 6-6.

Figure 6-12 shows a case where the maximum r_u reached was 0.34 and the minimum FS_{IB08} was 2.2 in the liquefiable layer (liquefaction was not triggered). For this case it can be observed that the acceleration-time histories and response spectra are nearly identical for the with- and without-porewater pressure cases. The moderate level of excess porewater pressure increase has little effect on the surface accelerations and the AF_{liq} values are close to unity for all periods.

Figure 6-13 shows a case where liquefaction was triggered (maximum value of $r_u = 1.0$ and the minimum $FS_{IB08} = 0.9$ in the liquefiable layer). This plot shows a marked difference in the acceleration-time history starting at about 10 seconds and a diminished and lower frequency response thereafter. The value of

AF_{liq} are generally greater than unity at periods greater than about 1 second and less than unity below $T = 1$ second.

A case with maximum value of $r_u = 1.0$ and minimum $FS_{IB08} = 0.7$ is presented in Figure 6-14. There is a dramatic difference in the earthquake time histories after about 38 seconds of shaking. After this time, run with porewater pressure shows a marked decrease in high frequency content, and dilation-related acceleration spikes develop. There are two peaks in the AF_{liq} plot at periods of 0.08 and 0.28 seconds with values of 1.6. At periods between 0.1 and 0.65 seconds AF_{liq} values are less than unity. Below $T = 0.1$ seconds, AF_{liq} are greater than unity, and $AF_{liq}(PGA) = 1.2$. It is evident by examining Figure 6-14 that the PGA in the run with porewater pressure generation occurs at a dilation-related acceleration spike at $t = 44$ seconds.

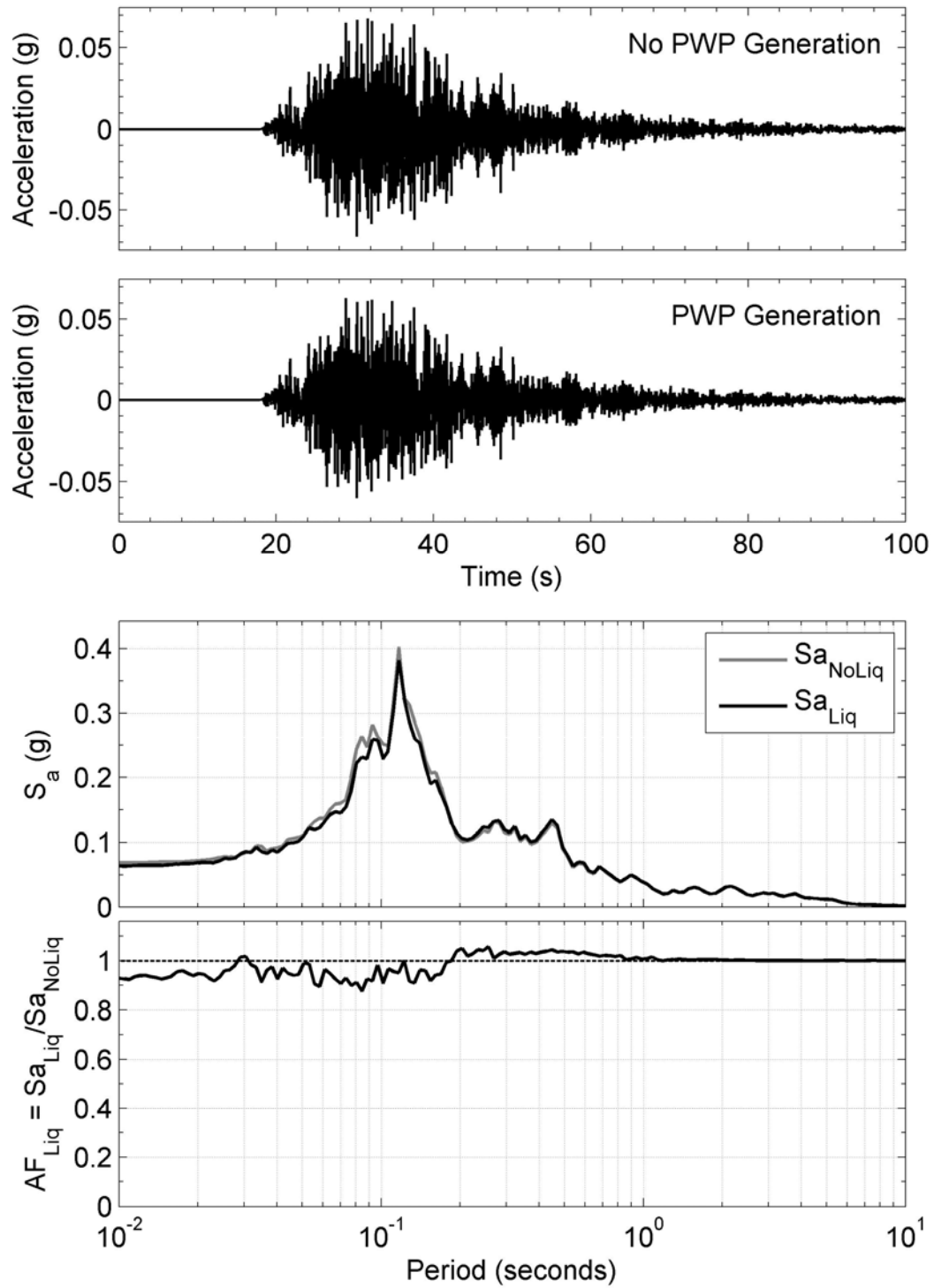


Figure 6-12: Site response results for case 1302 with $D_r = 50\%$, NGA No. 5483, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $r_{u,max} = 0.34$, $FS_{IB08} = 2.2$.

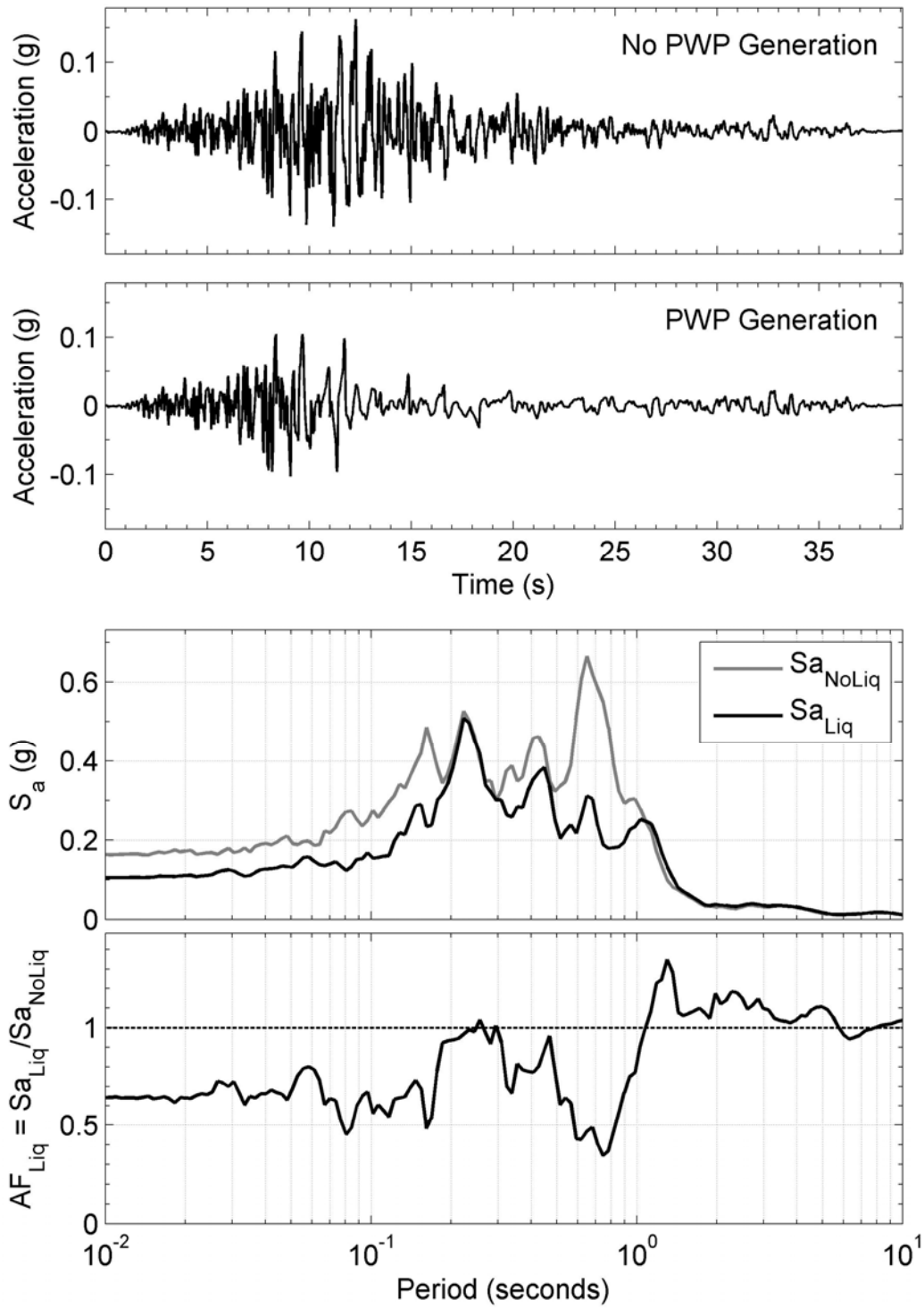


Figure 6-13: Site response results for case 1297 with $D_r = 50\%$, NGA No. 751, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $r_{u,max} = 1.0$, $FS_{B08} = 0.9$.

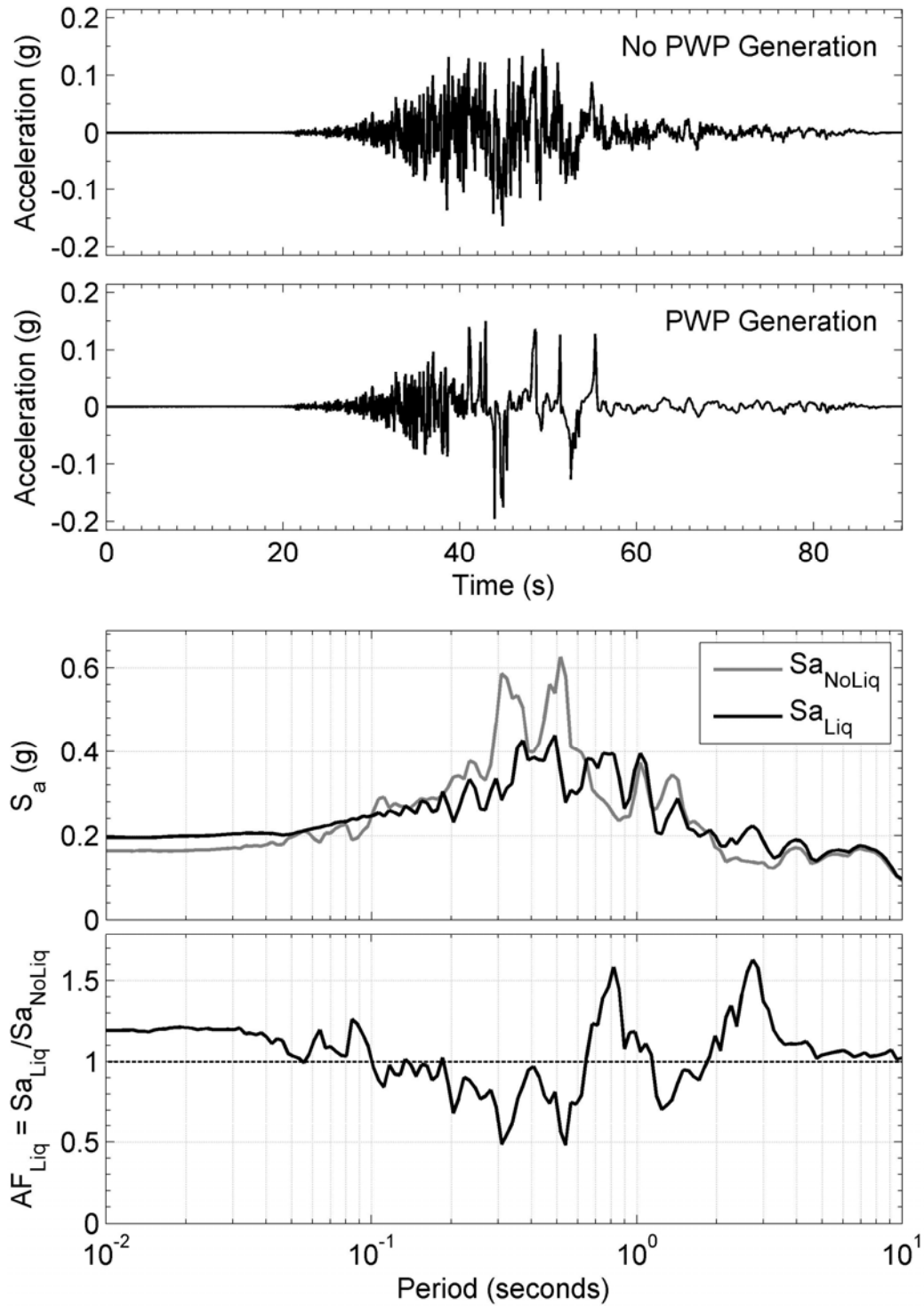


Figure 6-14: Site response results for case 1311 with $D_r = 50\%$, NGA No. 1548, $z_{liq} = 3\text{m}$, $T_{liq} = 3\text{m}$, $r_{u,max} = 1.0$, $FS_{IB08} = 0.74$, pulse-like.

6.5.3 Overall Amplification Factors and r_u Dataset Filtering

Figure 6-15 presents AF_{liq} data for all of the site response run pairs (n_{pairs}), except for porewater pressure generation cases with PGA greater than 1.3. The PGA data filter was applied to exclude a small set of cases ($n_{pairs}=68$, or just 2 percent of the runs) with unrealistically high PGA values. The high PGA values are considered unrealistic because the PGA values observed in the empirical dataset were all less than 0.75g. The unrealistically high PGA values were likely generated because the model does not account for water cavitation that would occur at high negative porewater pressure. Cavitation would reduce the effective stress and stiffness and preclude such high shear stress waves from being transmitted. This data subset includes cases where liquefaction was triggered in the porewater pressure generation run along with cases where it was not. For this same data subset, Figure 6-16 presents plots of AF_{liq} versus $r_{u,max}$ for various periods. In these plots, $r_{u,max}$ is the maximum value of r_u achieved in any of the elements within the potentially liquefiable layer during the site response analysis. These data show that $AF_{liq} \approx 1$ for $r_{u,max} < 0.95$. For cases where $r_{u,max} \geq 0.95$ the scatter about $AF_{liq} = 1$ increases dramatically. The widest scatter about $AF_{liq} = 1$ occurs when $r_{u,max} \approx 1$. These data show that there appears to be a binary separation of AF_{liq} results between cases where liquefaction is and is not triggered. This suggests significant differences in site response between a drained and undrained condition do not occur unless one or more layers fully liquefy (that is, they achieve $r_u \approx 1$).

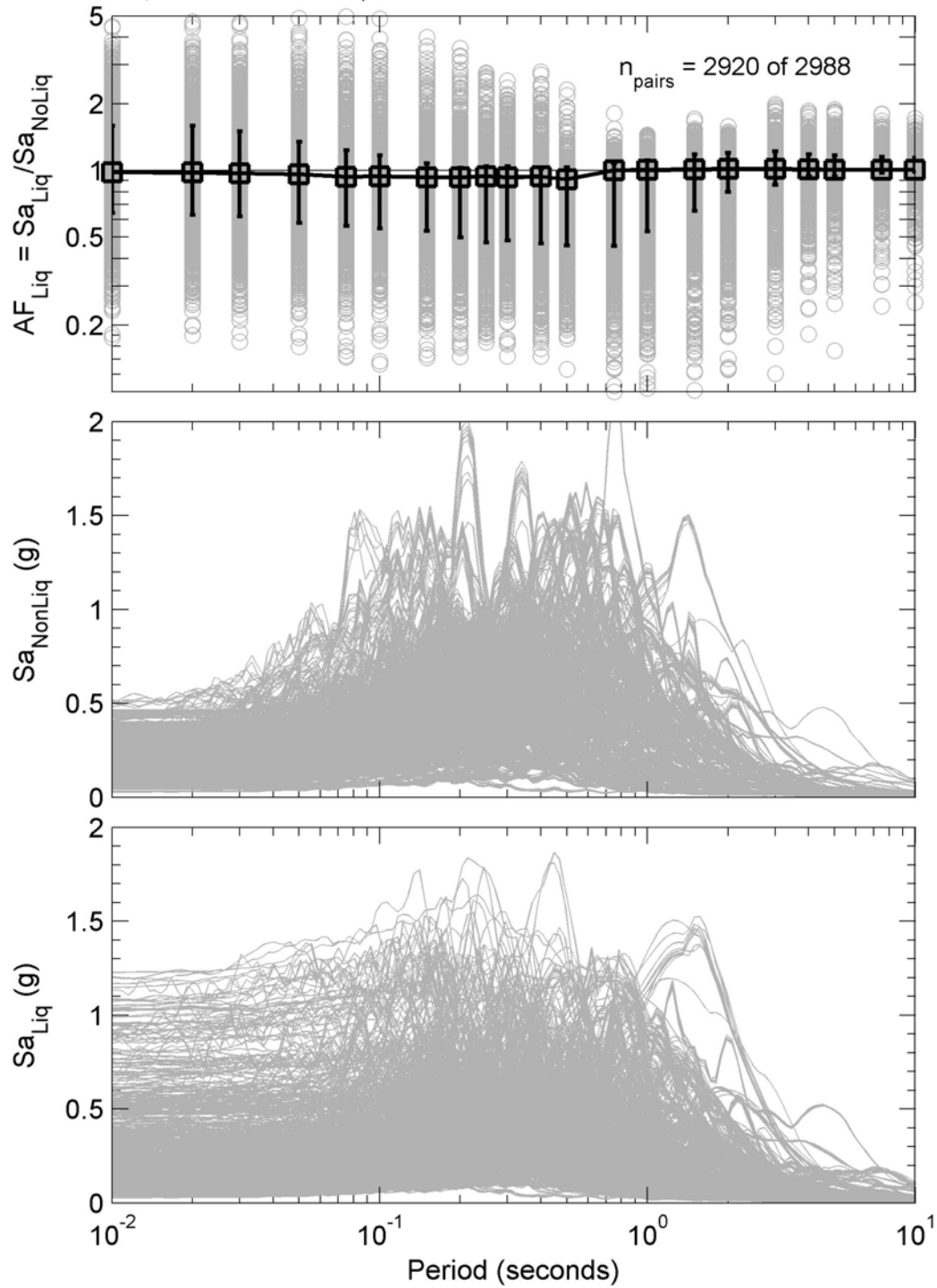


Figure 6-15: Response spectra and AF_{liq} for liquefied and non-liquefied cases. Black squares are median values and error bars are 0.1 and 0.9 quantiles.

For sites that do not fully liquefy, the differences between drained and undrained response are much less significant. Therefore, in practice, the extra effort of conducting effective stress site response analyses with porewater pressure generation does not appear warranted unless liquefaction is expected at the site (i.e., unless liquefaction triggering factors of safety are less than unity).

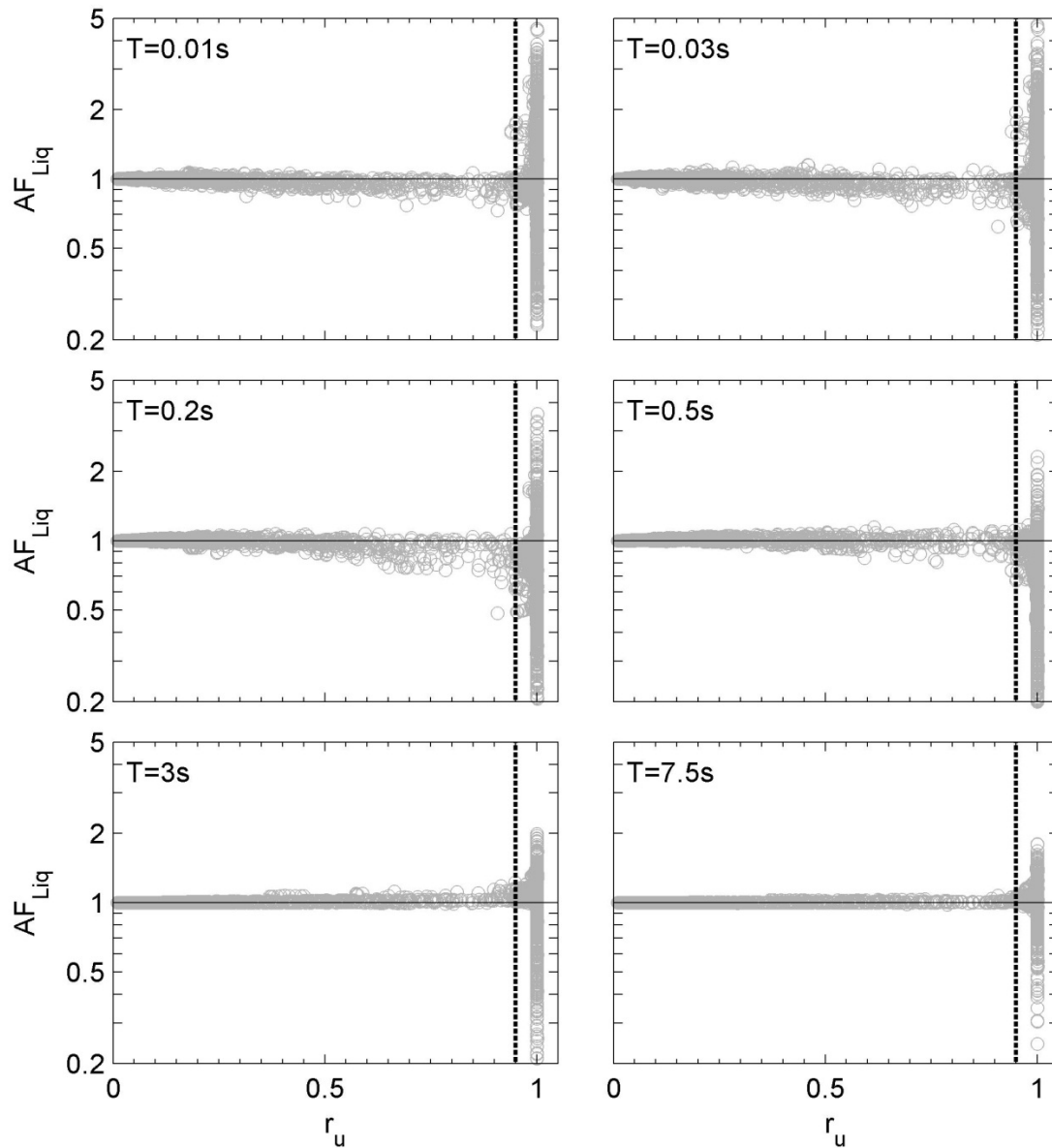


Figure 6-16: Dependency of AF_{liq} on $r_{u,max}$. Vertical dashed line is at $r_{u,max} = 0.95$.

Since the interest here is in evaluating the effects of liquefaction on ground motion, attention is focused on the site response cases where liquefaction was triggered, as defined by achieving $r_{u,max} \geq 1$. For ease of reference, site response analysis pairs where the porewater pressure generation run achieved $r_u \geq 1$ will be referred to herein as “liquefaction case” pairs (versus a “non-liquefaction” case pair where $r_{u,max} < 1$). Consideration of only site response cases that liquefied also facilitates comparison of this data to the empirical data from Chapter 3.

6.5.4 Comparison of Site Response and Empirical Data

Figure 6-17 presents liquefaction case response spectra for the with- and without-porewater pressure pairs, and the corresponding AF_{liq} values. The upper plot in this figure provides a sense of the dispersion and central tendency of the AF_{liq} data, but the data are too numerous and overlapping to observe their distribution. Figure 6-18 presents histograms of the AF_{liq} data for “short periods (T=0.01 [PGA] and 0.03 seconds)”, “intermediate periods” (T=0.2 and 0.5 seconds), and “long periods” (T=3 and 7.5 seconds).

The AF_{liq} values from the site response simulations are compared with the empirically calculated values from Chapter 3 in Figure 6-19. The median values of AF_{liq} for the empirical and site response cases follow a similar shape that is depressed at intermediate periods and higher at long and short periods. However, the amplitudes of the median values are considerably greater at all periods for the empirical data compared to the site response data.

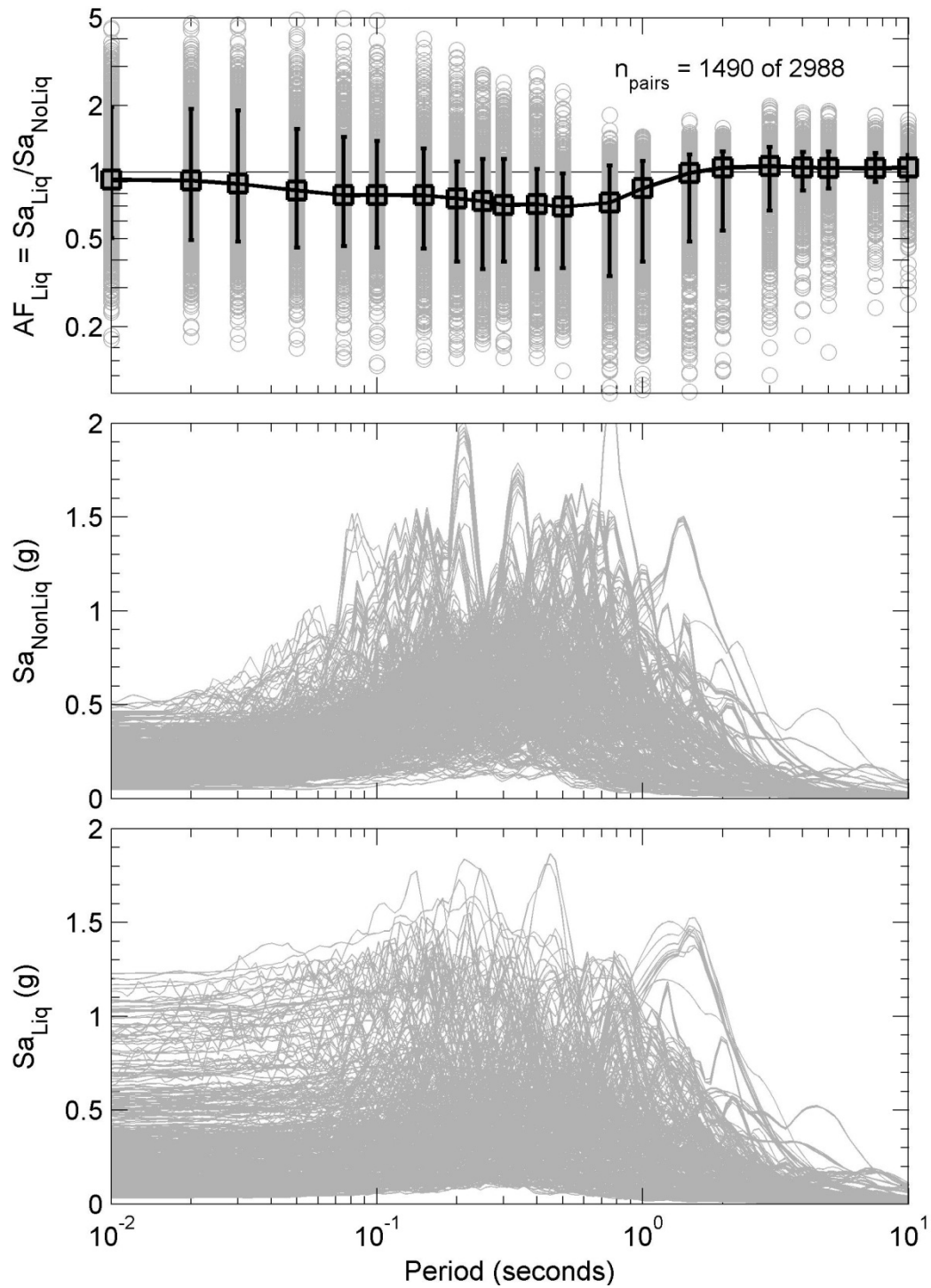


Figure 6-17: Response spectra and AF_{liq} for liquefied and non-liquefied cases, (only cases with $r_u = 1$). Black squares are medians and error bars are 0.1 and 0.9 quantiles.

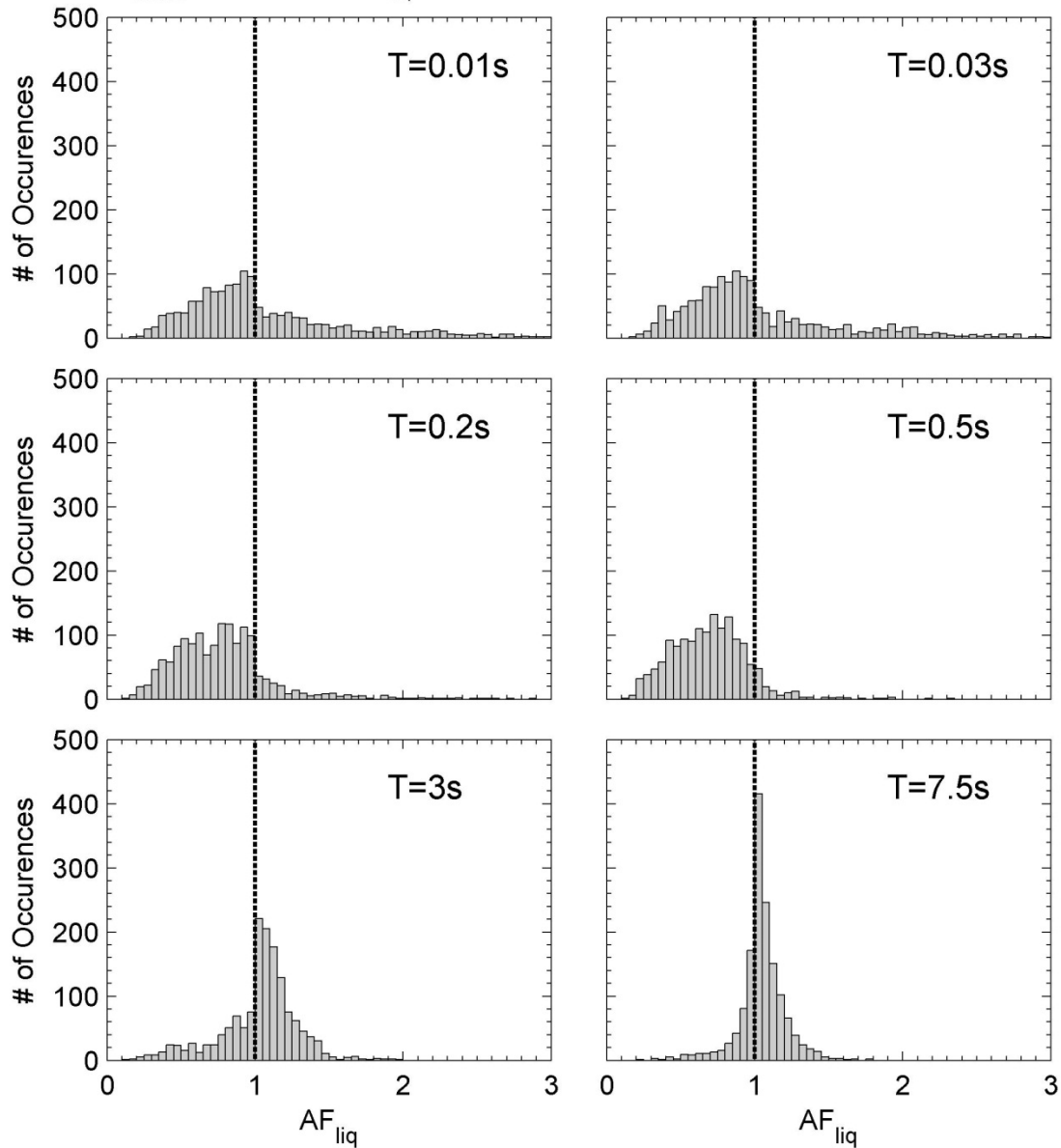


Figure 6-18: Histograms of AF_{liq} for various periods (only $r_{u,max} = 1$).

Differences between the empirical and site response AF_{liq} values are most pronounced at long periods, where in some cases the median empirical values exceed even the maxima of the site response values. The least difference is present at intermediate periods, especially for the BA08 and CY08 GMPEs, where there is a good match to results at periods of about 0.1 to 0.2 seconds. At

short and intermediate periods, the empirical data set fall within the range of site response values.

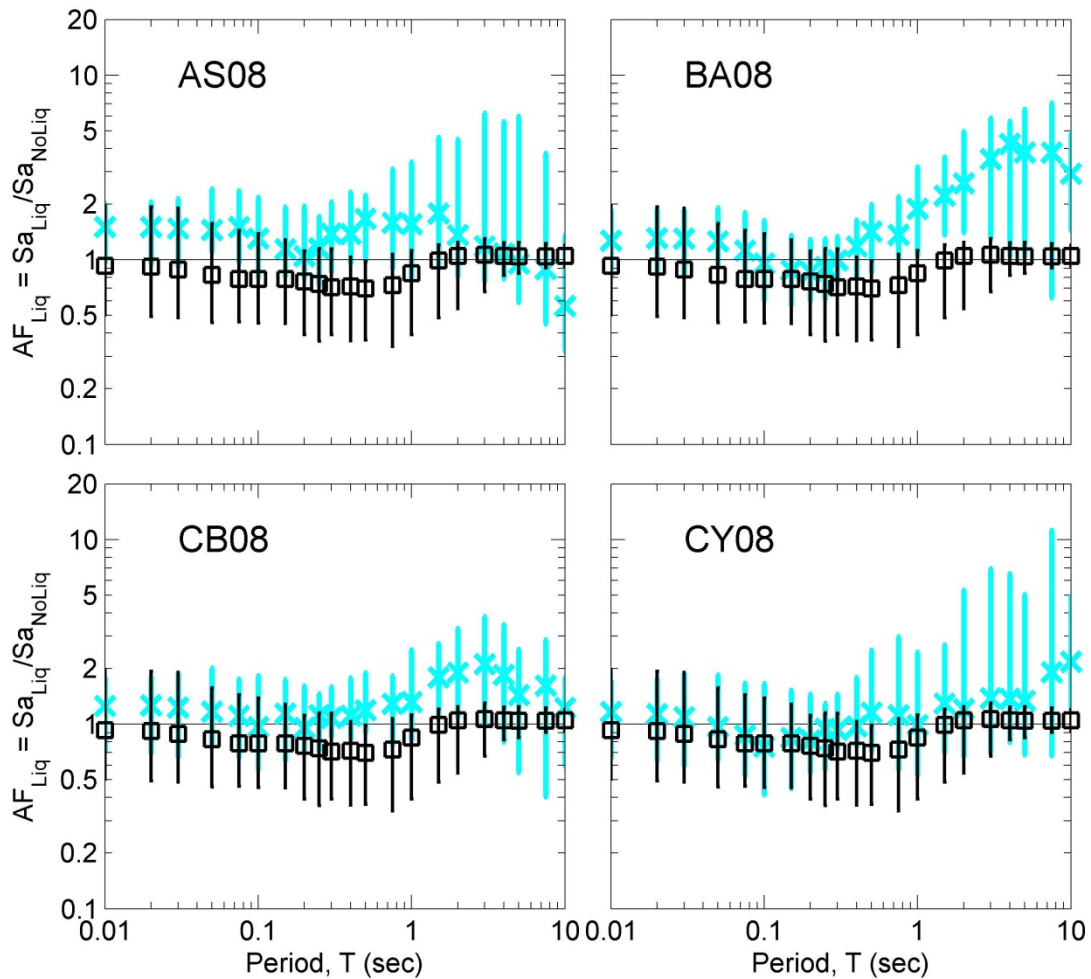


Figure 6-19: Comparison of empirical (cyan) and site response (black) AF_{liq} values in terms of median ('x' and square markers) and 0.1 and 0.9 quantiles (error bars).

There are several potential reasons for the disparity between the empirical and site response results. The site response analyses were conducted as 1-D, with only vertically propagating shear waves considered. The 1-D analyses do not capture surface wave or basin effects, which are known to amplify the longer

periods. It is possible that constructive interplay between surface waves and liquefiable soil response lead to higher long period spectral acceleration. It is also possible, despite their sophistication and rigorous calibration, that shortcomings exist in the site response model. In any case, additional research is needed to explore reasons for differences in empirical and site response data.

Limited metadata on site conditions are available for the empirical ground motion recordings. But given the relatively small sample size of the empirical data, it is likely that the site response data represent a broader range of conditions in terms of parameters such as T_{liq} , z_{liq} , D_r , and input ground motion. In the subsequent sections, data analysis is performed to evaluate the influence of various site response analysis parameters on the resulting AF_{liq} values.

6.5.5 Examination of Parameter Dependence

This section presents and discusses a series of plots of AF_{liq} versus various parameters. The plots are summarized for the same samples of short, intermediate and long periods described Section 6.5.4. For cases where the parameters are more continuously distributed, ordinary least-squares linear regression is performed (either log-log or log-linear, depending upon the parameter). The plots present the linear regression as a solid black line, and plus and minus one standard deviation (a lognormal distribution is assumed) is shown by dashed black lines. The 95 percent confidence interval of the regression line is shown by red dash-dot lines. For cases where the parameters are distributed in discrete bins, statistics are calculated for each bin. The plots show median

values as black squares with error bars representing the 0.1 and 0.9 quantile values.

Figure 6-20 presents plots of AF_{liq} versus M_w for periods various periods which all show a weak negative trend. By comparison, the empirical residual data presented in Figure 4-8 (recall that the empirical form of $AF_{liq} = \exp(\varepsilon_{ij})$) show variations of positive to negative trend with M_w . Given the sparseness of the empirical data and the frequent lack of statistical significance in M_w in the empirical model described in Chapter 4 (e.g., confidence levels shown in Table 4-5 are typically much lower than 95%), along with the weak trend expressed in Figure 6-20, earthquake magnitude does not appear to play a significant role in controlling AF_{liq} factors.

Plots of AF_{liq} versus peak ground acceleration of the no porewater pressure generation case (PGA_{nonliq}) are presented in Figure 6-21. These data show a strong positive trend at short periods and a moderate positive trend at moderate and long periods. The positive trend at short periods is likely attributable to stronger input motions causing higher shear stress demands, which in turn produce stronger dilation phases and acceleration spikes.

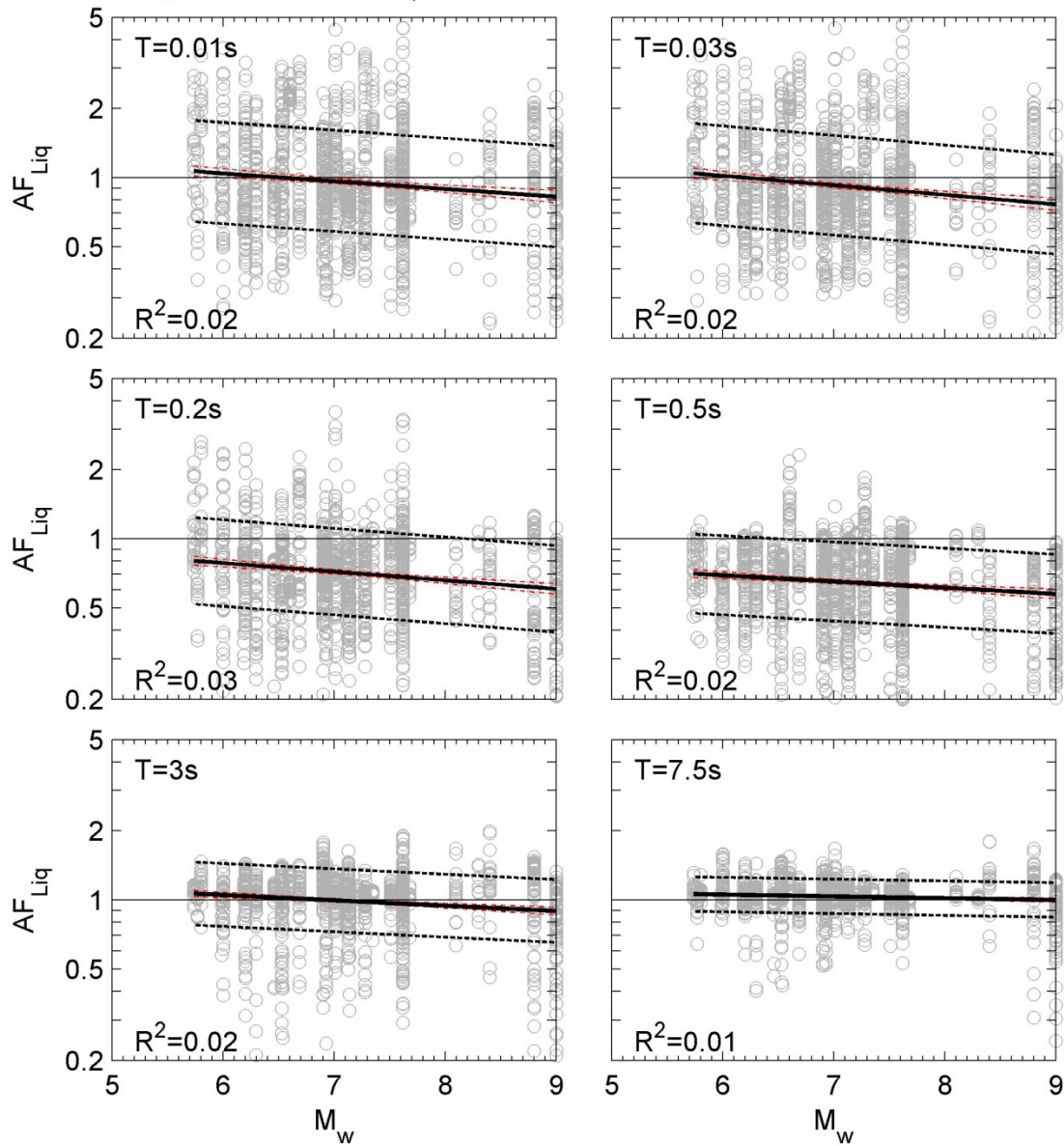


Figure 6-20: Dependency of AF_{liq} on M_w (only $r_{u,max} = 1$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.

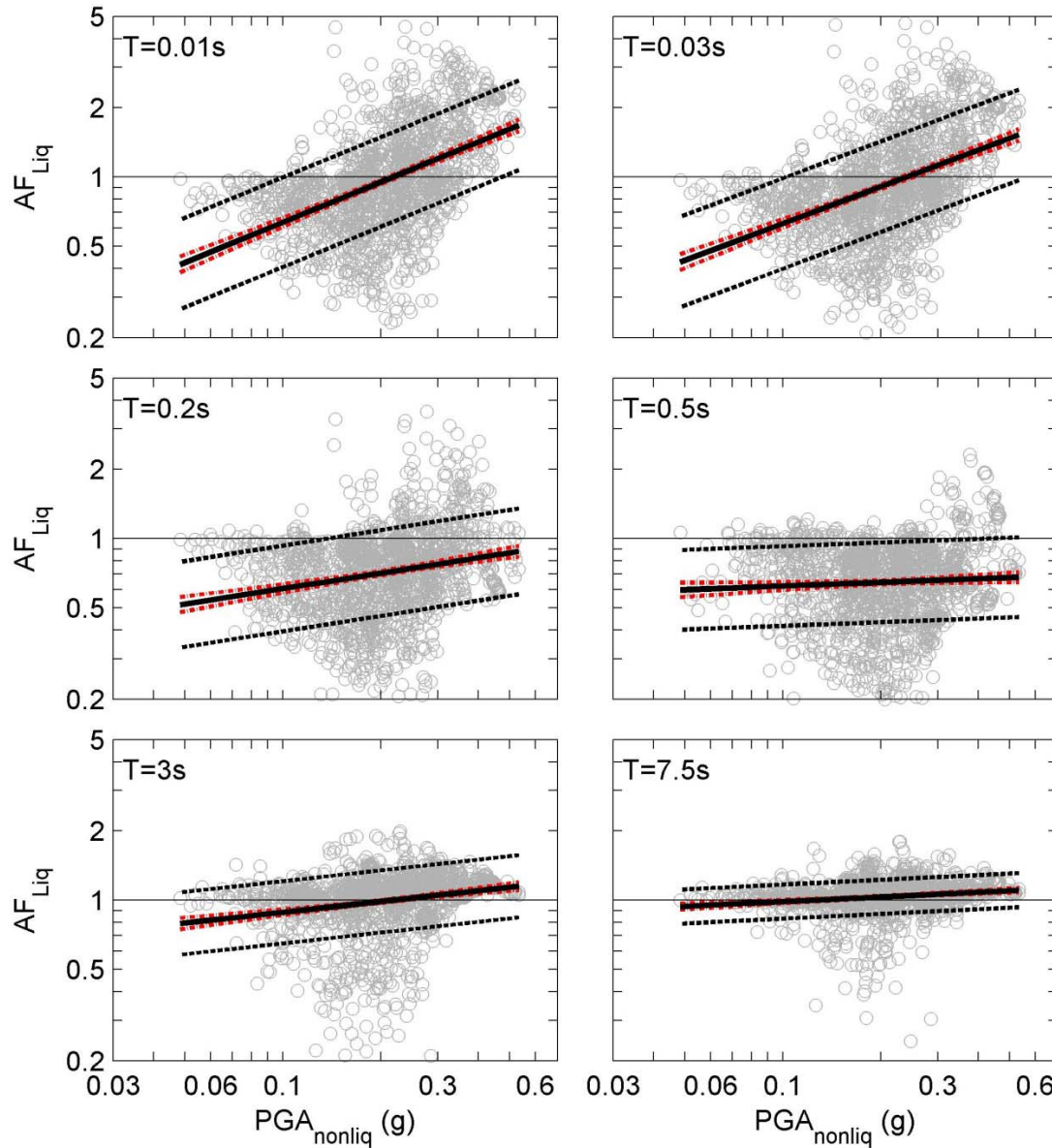


Figure 6-21: Dependency of AF_{liq} on PGA_{nonliq} (only $r_{u,max} = 1$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.

The site response data for correlation with PGA_{nonliq} lies in contrast with the empirical residual data presented in Figure 4-6, which generally show a weak to moderate negative trend. The reason(s) for the inconsistency between the empirical and site response data is not apparent.

The cumulative absolute velocity (CAV) has been shown by Kramer and Mitchell (2006) to be an effective predictive parameter for excess porewater pressure generation. The CAV is the definite integral of the absolute value of the acceleration-time history, evaluated from the beginning to the end of the time history. Kramer and Mitchell defined the CAV_5 variation as follows:

$$CAV_5 = \int_0^{t_{max}} \langle \chi \rangle |a(t)| dt \quad (6-2)$$

where t_{max} is the time at the end of the acceleration-time history recording and $\langle \chi \rangle$ is 0 when $|a(t)| < 5 \text{ cm/s}$ and 1 when $|a(t)| \geq 5 \text{ cm/s}$.

Figure 6-22 presents plots of AF_{liq} versus CAV_5 . At short periods there is a moderate positive trend, at moderate periods there is a weak negative trend and at long periods there is virtually no trend. Because the scatter in the data is significant, particularly at short to moderate periods, CAV_5 does not appear to be a critical parameter for prediction of AF_{liq} .

Figure 6-23 shows the dependency of AF_{liq} on D_r . At short period, the median values show a generally increasing trend with D_r . These differences can be explained by the decreased dilatancy in the model at lower relative densities, which results in fewer acceleration spikes. At intermediate periods, there is a general trend of increasing deamplification ($AF_{liq} < 1$) with decrease in D_r . At long periods median values of AF_{liq} generally increase slightly with D_r . This may occur because increased damping in hysteresis loops without dilation, which occur more frequently in $D_r=30$ percent cases. Across all period ranges, AF_{liq} values are lower for the case of $D_r=30$ percent. This suggests that very loose soils with

little to no dilative behavior tend to deamplify ground motions. At all period ranges, the dispersion of the data tend to decrease with increasing relative density.

Figure 6-24 presents the dependency of AF_{liq} on the depth to the top of the liquefiable layer, z_{liq} . A weak trend of decreasing AF_{liq} with z_{liq} is observed at short periods. At intermediate and long periods there is essentially no trend in the data. AF_{liq} values do not appear to be particularly sensitive to the depth at which liquefaction occurs.

The dependency of AF_{liq} on the thickness of the liquefiable layer (T_{liq}) is presented in Figure 6-25. All periods show a generally weak trend of decreasing AF_{liq} at increasing T_{liq} . This suggests that even relatively thin liquefiable layers are sufficient to modify ground motions. However, dispersion tends to increase with increasing T_{liq} , and some of the highest AF_{liq} values occur for the thickest liquefiable layer case of $T_{liq}=10\text{m}$. So depending on specific site and input ground motion conditions, thicker liquefiable layers do not always translate into lower ground motion levels.

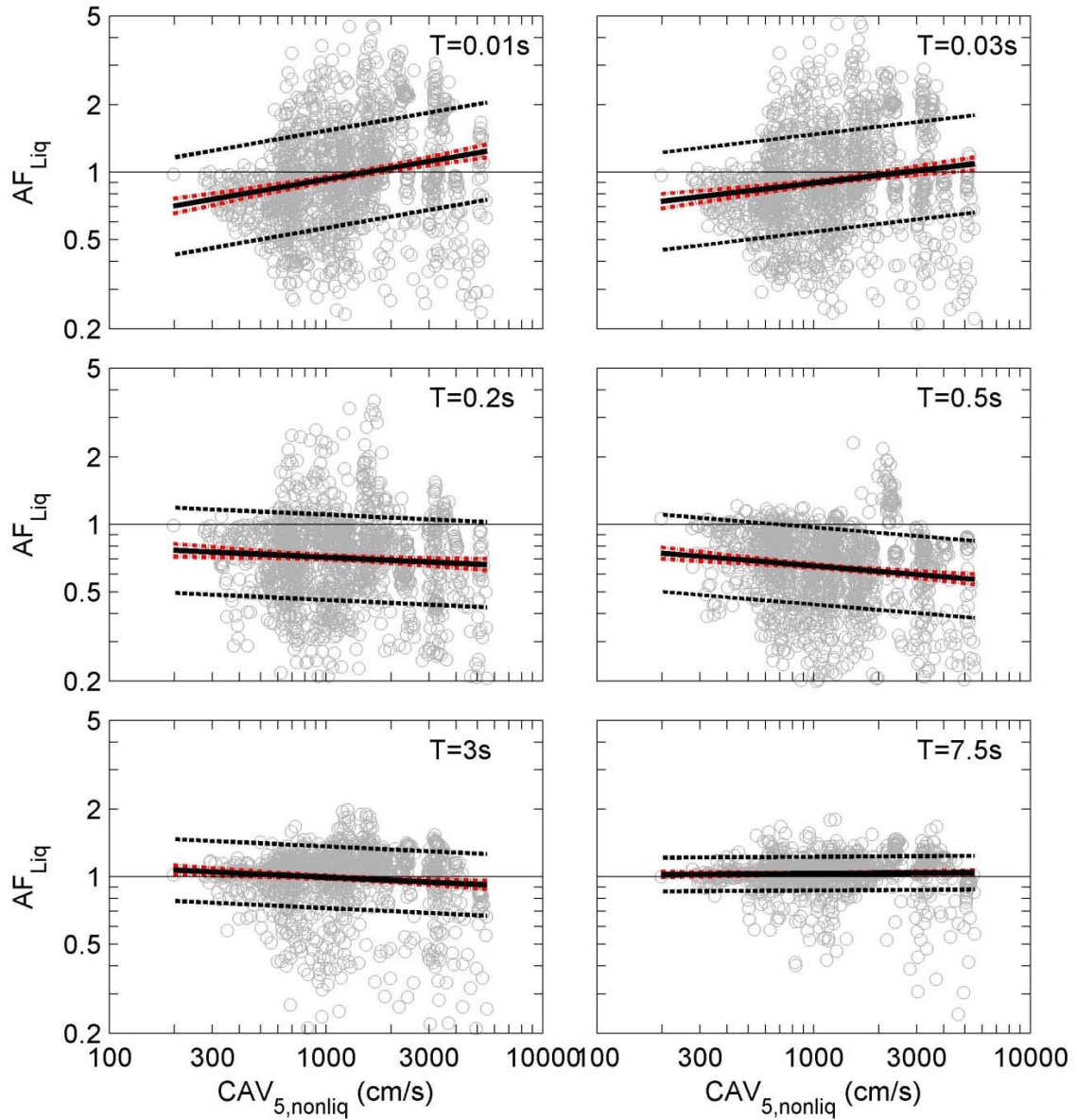


Figure 6-22: Dependency of AF_{liq} on CAV_5 (only $r_{u,max} = 1$). Solid and dashed black lines show linear regression and $\pm$ one standard deviation. Red dash-dot lines show 95 percent confidence interval of the regression.

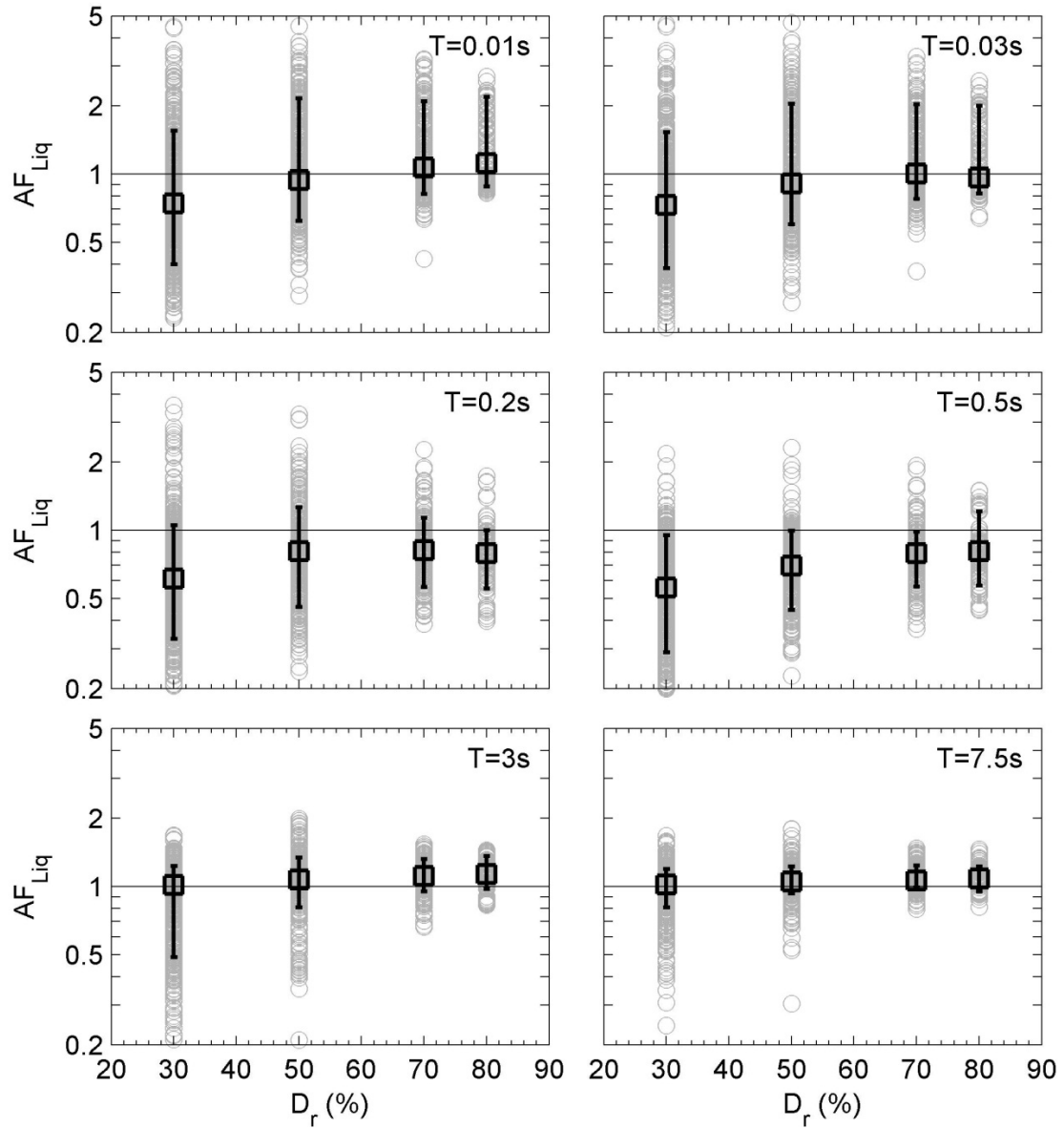


Figure 6-23: Dependency of AF_{liq} on D_r (only $r_{u,max} = 1$). Black squares show median values and error bars show 0.1 and 0.9 quantiles.

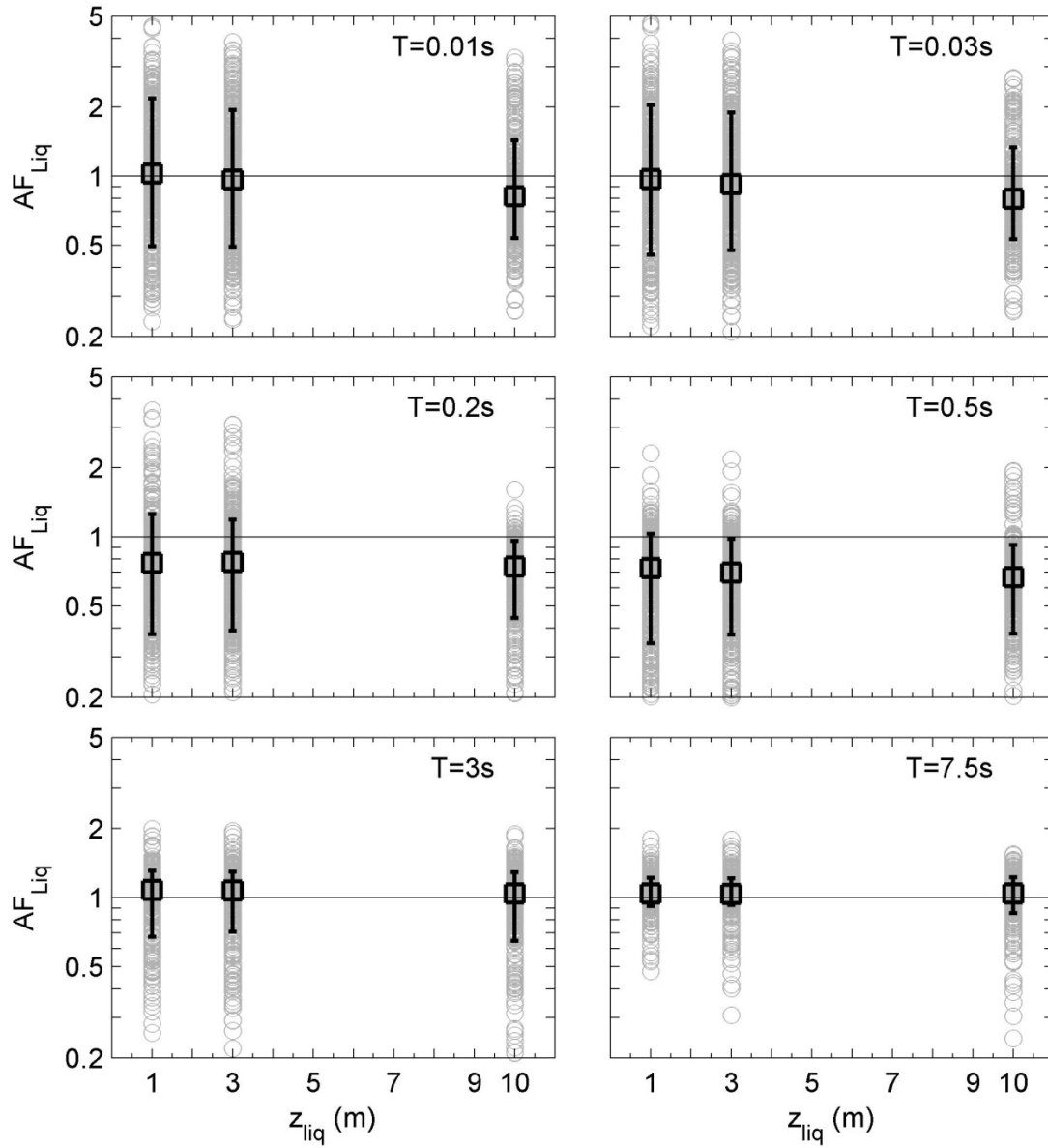


Figure 6-24: Dependency of AF_{liq} on z_{liq} (only $r_{u,max} = 1$). Black squares show median values and error bars show 0.1 and 0.9 quantiles.

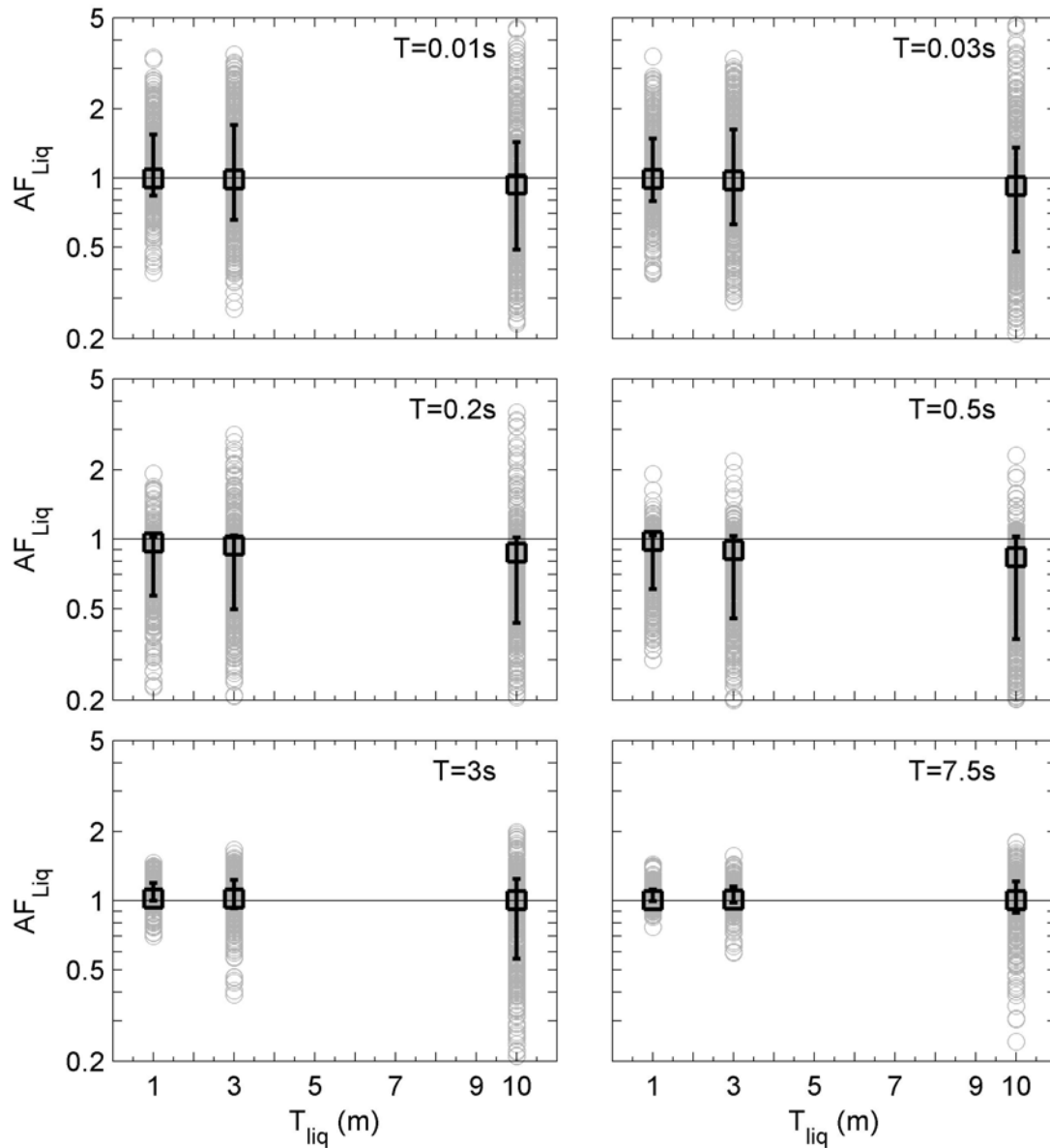


Figure 6-25: Dependency of AF_{liq} on T_{liq} (only $r_{u,max} = 1$).

Of the 83 input ground motions 21 were classified as pulse-like (Bins 1 and 4) according to Baker (2007) and 62 were non-pulse like. Figure 6-26 (a) presents AF_{liq} results for pulse-like runs using pulse-like ground motions and (b) shows non-pulse-like motion runs. These data show that AF_{liq} values are higher

at all periods for the pulse-like input motions compared to the non-pulse-like input motions. These data suggest that the nature of the input motion plays a significant role in the site response of liquefiable soils, and that pulse-like motions tend to enhance ground motion amplification at liquefaction sites.

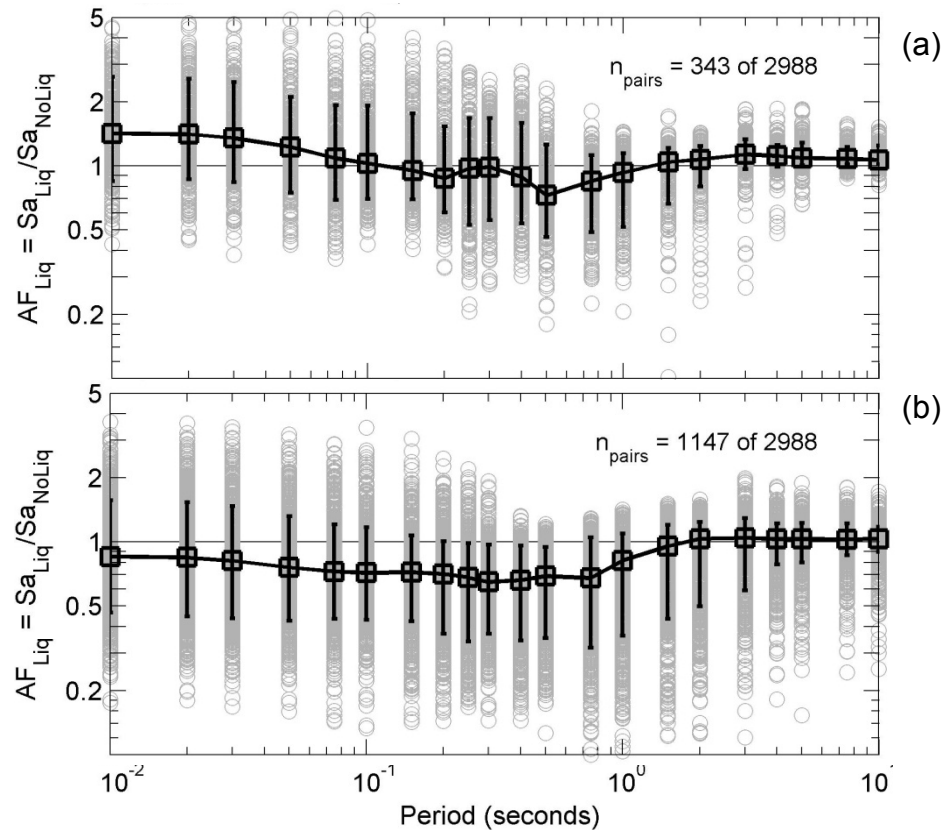


Figure 6-26: Comparison of cases with $r_{u,max} = 1$ and (a) pulse-like input ground motion only, (b) non-pulse-like. Black squares show median values and error bars show 0.1 and 0.9 quantiles

In Figure 6-27 a comparison is made between input ground motions recorded at soil sites with $180 \leq V_{s30} \leq 360$ m/s, and rock sites with $V_{s30} > 360$ m/s. The differences between AF_{liq} are minimal, suggesting that bedrock

condition at a liquefiable site has little influence on the resulting surface ground motion.

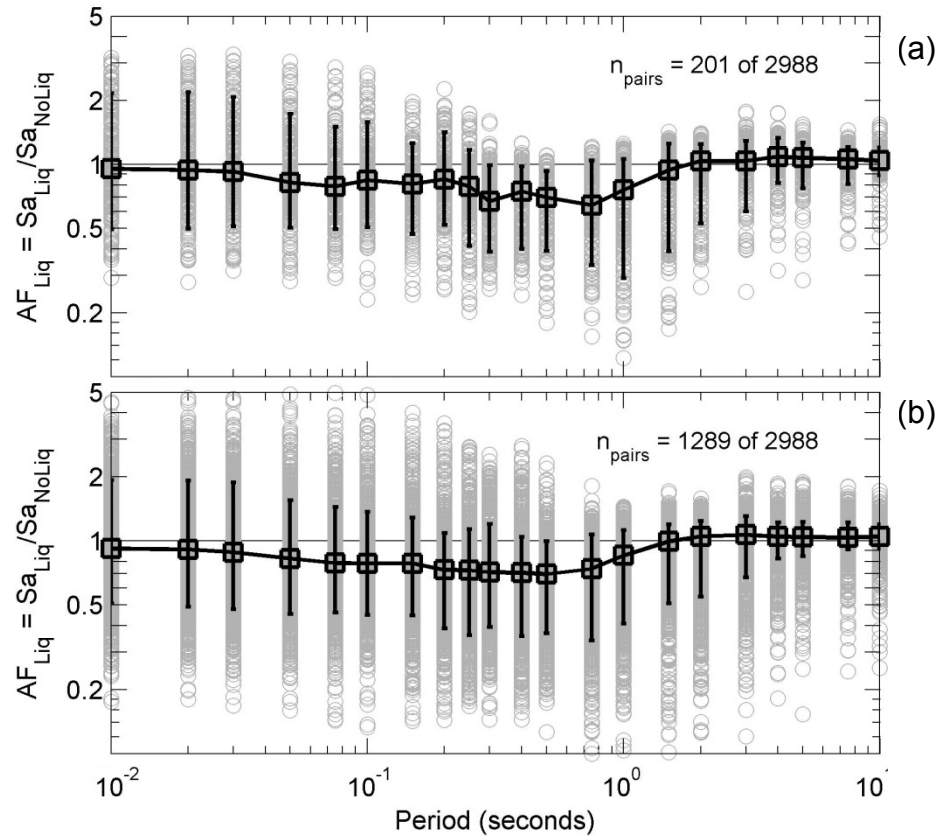


Figure 6-27: Comparison of cases with $r_{u,max} = 1$ and (a) $180 \leq V_{s30} \leq 360$ m/s, (b) $V_{s30} > 400$ m/s. Black squares show median values and error bars show 0.1 and 0.9 quantiles

6.6 Summary and Conclusions

Liquefaction amplification factors calculated by 1-D site response analyses were systematically lower than those observed in empirical data. Additional research is needed to evaluate whether this could be the result of surface wave or basin effects that are not captured by the 1-D model, limitations in the model's ability to replicate all salient aspects of shear wave propagation through liquefied soils, or other factors. Despite their difference from the empirical data, the site

response data provided an opportunity to evaluate potential dependency of AF_{liq} values on parameters that are generally not available for the empirical data set, including T_{liq} , Z_{liq} , D_r , PGA , CAV_5 , input motion forward directivity and V_s of the input motion recording station. Of these parameters, PGA and forward directivity input motions have the most effect on AF_{liq} values.

While the focus of this study has been on evaluating difference between site response with and without porewater pressure generation and liquefaction, other uses of the dataset are possible. These uses could include evaluation of stress reduction factors (r_d) used in triggering analyses, the timing of liquefaction triggering, induced shear strain and sequencing of liquefaction triggering for thick liquefiable deposits. In this sense, the synthetic dataset generated in these analyses serves as a resource for use in future research.

6.7 References

- Abrahamson, N.A., Atkinson, G., Boore, D., Bozorgina, Y., Campbell, K., Chiou, B., Idriss, I., Silva, W. and Youngs, R. (2008). "Comparison of the NGA ground motion relationships," *Earthquake Spectra*, 24(1), pp 45-66.
- Ambraseys, N.N. (1988). "Engineering seismology," *Earthquake Engineering and Structural Dynamics*, (17) pp 1-105.
- Baker, J.W. (2007). "Quantitative classification of near-fault ground motions using wavelet analysis," *Bulletin of the Seismological Society of America*, 97(5), pp. 1486-1501.
- Carlton, B. (2014). "An Improved Description of the Seismic Response of Sites with High Plasticity Soils, Organic Clays, and Deep Soft Soil Deposits." PhD dissertation, University of California, Berkeley.
- Idriss, I.M. and Boulanger, R.W. (2006). "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," *J. Soil Dynamics and Earthquake Eng.* 26, pp 115–30.

- Idriss, I.M. and Boulanger, R.W. (2008). *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12.
- Power, J. P. (1992). *Construction Dewatering, New methods and Applications*, Second Edition, John Wiley & Sons, Inc., New York.
- Seyhan, E., Stewart, J.P., Ancheta, T.D., Darragh, R.B. and Graves, R.W. (2014). "NGA-West 2 Site Database," in press *Earthquake Spectra*, EERI.
- Tokimatsu, K. and Seed, H.B. (1987). "Evaluation of settlements in sands due to earthquake shaking," *J. Geotech. Engrg.*, ASCE, 113 (8), pp 861-878.

Chapter 7 Summary and Conclusions

7.1 Summary

Strength loss and permanent ground deformation resulting from earthquake-induced soil liquefaction has been studied by several researcher groups over the years. By contrast, limited research has been conducted on the effects of liquefaction on ground motions. A better understanding of ground motions at liquefaction sites is needed so that structures situated on potentially liquefiable ground can be designed more reliably. To address this need, a two-part study was undertaken to evaluate the effects of liquefaction on earthquake ground motions.

In the first part, an empirical dataset was compiled of ground motions recorded at sites where surface manifestations of liquefaction were observed. The dataset was focused on recordings from shallow crustal earthquakes to facilitate comparisons with response spectra calculated using NGA GMPEs. The dataset consisted of 19 free-field recordings from seven earthquakes.

An analysis of the temporal and frequency characteristics of these ground motions was performed. The mean instantaneous Fourier frequency (MIF) was calculated over a moving time window of 2.56 seconds. The time-dependent MIF was compared to timing of spectral accelerations to evaluate whether spectral accelerations at various periods occurred more frequently before or after liquefaction was triggered.

Response spectra were calculated using the NGA GMPEs for each of the liquefaction recording sites based on the site conditions, site-source distances and faulting mechanisms. These response spectra represent ground motions estimates in the absence of liquefaction. Intra-event residuals were calculated for the recorded liquefaction spectra relative to the non-liquefaction NGA spectra. The residuals were used in a regression analysis to develop a preliminary model to adjust NGA spectra to account for the effects of liquefaction.

The second part of this study involved parametric site response analyses that permitted consideration of a much broader array of site conditions and earthquake excitation than were represented in the empirical data. The Opensees finite element program was used in conjunction with constitutive models capable of representing salient features of liquefaction, including contraction, dilation, and cyclic mobility. As part of the calibration activities, a new method for adjusting the tail of G/G_{max} vs. γ backbone curves for strength compatibility was developed. The liquefaction model was rigorously calibrated to a widely-used semi-empirical liquefaction triggering relationship (Boulanger and Idriss 2006). Post-liquefaction cycle-by-cycle shear strain accumulation was calibrated to cyclic direct simple shear test results for Monterey 0/30 sand.

Using the calibrated model, a total of 2988 site response analysis pairs were calculated. For each pair, one run was performed without porewater pressure generation and the other run was performed with porewater pressure generation. Surface spectra generated for these pairs are analogous to the empirical dataset case of NGA and recorded motion spectra. Using a subset of

the site response results where liquefaction was triggered, the ratio of spectra with porewater pressure generation to without porewater pressure generation was calculated, $AF_{liq}(T)$. These period-dependent amplification factors were examined to evaluate potential dependency on the following factors: liquefiable layer thickness, depth and relative density; earthquake magnitude; shaking amplitude; and input motion directivity.

7.2 Conclusions

7.2.1 Empirical Data Studies

The following conclusions are drawn relative to the empirical data studies:

1. The temporal and frequency analysis of the empirical liquefaction ground motion data showed that the majority of longer-period spectral accelerations (T greater than about 3 seconds) occurred after the estimated onset of liquefaction. This is consistent with the notion that long periods are amplified during liquefaction because the softened soil condition increases the site period. Conversely, shorter period spectral accelerations (T less than about 1 second) occur more frequently prior to the estimated onset of liquefaction. Approximately 20 percent of the PGA values occurred after liquefaction was triggered, which suggests that liquefaction may not always filter out all high frequency ground motions.

2. The empirical ground motion study showed that mean intra-event residuals calculated from NGA GMPEs are positive at long periods ($T > 0.4$ seconds). The maximum mean amplification factors at $T > 0.4$ seconds are 1.8 to

3.1 for the four NGA GMPEs studied. This result is generally consistent with previous studies (Youd and Carter 2005; Hartvigsen 2007) that indicate higher amplitude spectral accelerations at long periods. At short periods ($T \leq 0.05$ seconds), slightly positive mean residuals were observed. This suggests that short period ground motions may also be amplified by liquefaction because of dilation phase-related acceleration spikes. At intermediate periods ($0.05 \leq T \leq 0.4$ seconds), a trough-like trend is present in the residual data, which is consistent with the expected shift of predominate site period to longer periods.

3. The preliminary liquefaction amplification model can be used to adjust 2008 NGA GMPE spectra for the effects of liquefaction. The model is a function of M , PGA and V_{s30} . Bias and trend are essentially eliminated in residuals calculated from spectra adjusted with the model. The model can be incorporated into DSHA and PSHA ground motion calculations. Since the model is based on a limited dataset it should be used with caution and in conjunction with site-specific nonlinear effective stress site response analyses. The model should be used within the bounds of the empirical dataset: shallow crustal earthquakes in active tectonic regions; NGA-estimated PGA of $0.1\ g$ to $0.6\ g$; V_{s30} within $150\ m/s$ to $260\ m/s$; and M_w between 6.3 and 7.7 .

7.2.2 Site Response Modeling

The following conclusions are drawn relative to the conducted site response modeling:

1. Calibration of the PIMY and PDMY2 models was performed for a generic clean sand with specific emphasis on application to 1-D site response analyses. The model was calibrated for consistency with the semi-empirical liquefaction triggering criteria of Idriss and Boulanger (2006) and post-liquefaction shear strain accumulation rates from Wu (2002). Results of single element direct simple shear tests showed that the model provides reasonable results over a wide range of earthquake magnitudes, overburden stress levels and soil relative density. The calibrated model was implemented in 1-D site response analyses of the Port Island and Wildlife Liquefaction Array sites, and showed good agreement with recorded responses.

2. Examination of $r_{u,max}$ data from the parametric site response analyses showed good agreement between the models' prediction of liquefaction triggering, and predictions based on the Idriss and Boulanger (2006) semi-empirical method. This indicates calibration of the constitutive model performed at the element level still holds in the more complicated 1-D site response model.

3. For cases where $r_{u,max} < 0.95$, values of $AF_{liq} \approx 1$ whereas scatter about $AF_{liq} = 1$ increases dramatically for $r_{u,max} \geq 0.95$. This indicates that porewater pressure generation did not play a significant role in site response unless liquefaction was triggered.

4. Of the 2988 parametric site response run pairs, a subset of 1782 resulted in liquefaction (as judged by the conditions of $r_{u,max} \geq 0.98$ in at least one of the finite elements within the potentially liquefiable layer). The AF_{liq} values

from the site response results showed mean values that were systematically larger across all periods compared to the empirical data. The difference was most pronounced at long periods, where the mean empirical AF_{liq} values were larger than even the maxima of the site response AF_{liq} values. The differences might be attributable to the inability of the 1-D site response analyses to capture 2-D and 3-D effects such as surface waves and basin effects, or possibly shortcomings in the 1-D model's ability to faithfully represent all salient aspects of 1-D wave propagation under liquefaction conditions. In any case, the discrepancy raises the question of whether 1-D effective stress site response analyses can accurately represent site response at liquefaction sites, even when a sophisticated model that is rigorously calibrated is used.

5. Values of AF_{liq} were plotted against the following parameters: M , PGA_{nonliq} , $CAV_{5,nonliq}$, D_r , z_{liq} and T_{liq} . The plots were evaluated for trends that could suggest dependence of AF_{liq} on the parameters. The following table presents a summary of the relative level of correlation between AF_{liq} and the parameters.

Table 7-1: Summary of correlation between and mean AF_{liq} values and various parameters.

Parameter	Short Period ($T \leq 0.05$ s)	Intermediate Period ($0.05 \leq T \leq 0.4$ s)	Long Period ($T \geq 4$ s)
M_w	-	-	-
PGA_{nonliq}	++	+	+
CAV_5	+	-	0
D_r	0 ^(b)	+	0 ^(b)
Z_{liq}	-	0	0
T_{liq}	- ^(c)	- ^(c)	- ^(c)
Pulse	++	++	+
Bedrock & Ground Motion Vs	0	0	0

Notes:

- (a) -- = strong negative correlation; - = weak negative correlation; 0 = little to no correlation; + = weak positive correlation; ++ = strong positive correlation.
- (b) No significant trend across $D_r = 50$ to 70 percent. However, mean AF_{liq} values are markedly less for $D_r = 30$ percent.
- (c) While mean values of AF_{liq} decrease with increasing T_{liq} , greater dispersion also occurs, and the largest values of AF_{liq} occur at $T_{liq} = 10$ m.

6. Based on the relative correlation results presented above, some conclusions can be drawn regarding which parameters appear to affect the liquefaction site response results the most. The level of shaking in terms of PGA_{nonliq} played a significant role, particularly at short periods. The strong positive trend at short periods is likely attributable to higher levels of shaking causing stronger dilation-related acceleration spikes. When pulse-like ground motions are used as input, AF_{liq} values tend to be systematically higher at all periods. This could be the result of the long pulse-period resonating with the liquefied site period.

7. It is interesting to note that there is little dependency of AF_{liq} on Z_{liq} and T_{liq} in these results. This indicates that the thin, deeply embedded liquefiable

layers altered ground motions in a similar fashion to the shallower and thicker liquefiable layers. Considering this results, thinner and more deeply embedded liquefiable layers should not automatically be dismissed with respect to their potential impact on ground motion alteration.

7.3 Recommendations for Future Research

The empirical data analyses conducted for this study excluded subduction zone earthquakes because large epistemic uncertainty exists in the currently available subduction zone GMPEs. Several recent large subduction zone earthquakes have provided significant new data, and revisions to GMPEs are underway. Once revised GMPEs with lower epistemic uncertainty are available, residual analysis of subduction zone liquefaction sites should be performed.

The empirical model should be updated for the NGA West 2 GMPEs, which became available after the work was completed.

Additional research is needed to elucidate the reasons for differences in amplification factors between the empirical dataset and the site response data. It is suspected that interplay between surface waves (possibly basin-generated) and liquefiable soil response, which cannot be replicated in 1-D analyses, may play a role. Therefore, additional research should be performed that employs 2-D and possibly 3-D site response analysis with domains large enough to incorporate surface wave propagation and basin effects.

Despite the rigorous model calibration performed for this study, it is possible that characteristics or limitations of the liquefaction constitutive model or

modeling protocols used could affect the results. Additional analyses using alternative constitutive models, numerical platforms, and modeling protocols could provide additional insight.

7.4 References

- Hartvigsen, A.J. (2007). Influence of pore pressures in liquefiable soils on elastic response spectra. Master's thesis, University of Washington.
- Idriss, I.M. and Boulanger, R.W. (2006). "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," *J. Soil Dynamics and Earthquake Eng.* 26, pp 115–30.
- Youd, T.L. and Carter, B.L. (2005). "Influence of soil softening and liquefaction on spectral acceleration," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 131(7).

Appendix A

Empirical Site and Source Information

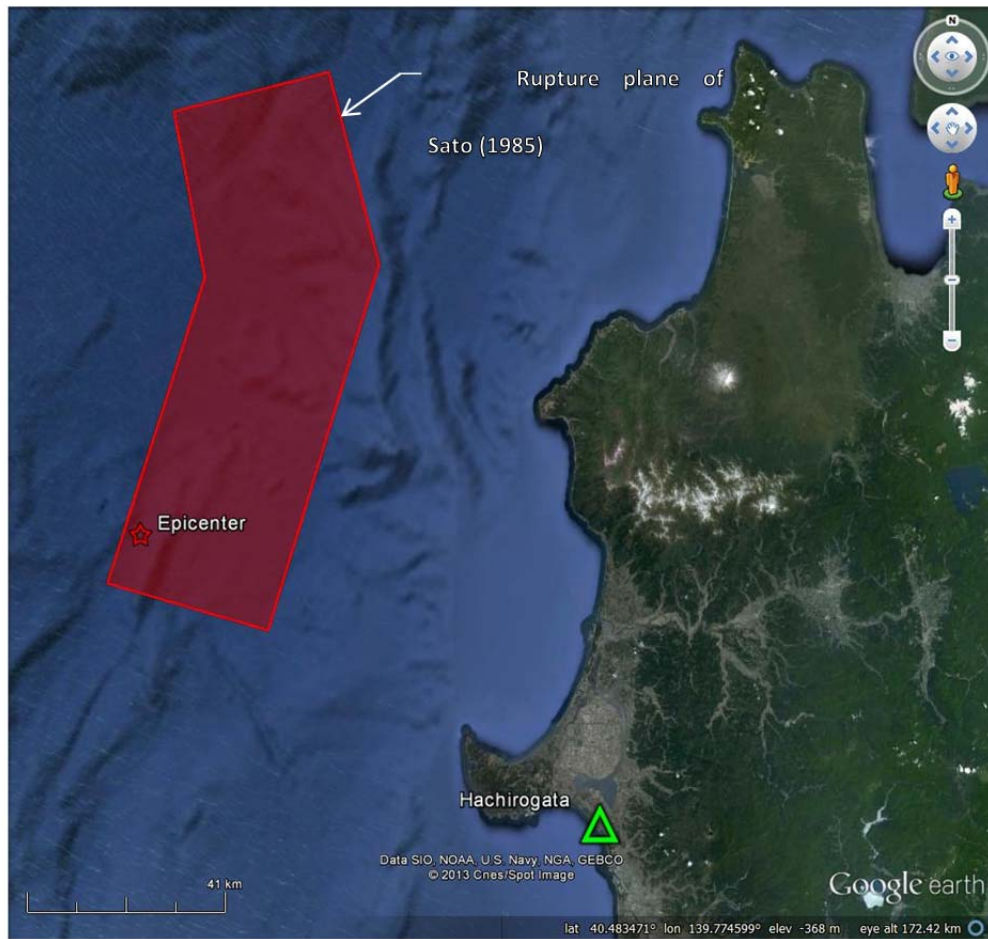


Figure A-1: Nihonkai-Chubu earthquake rupture plane and Hachirogata liquefaction site accelerometer station (triangle).

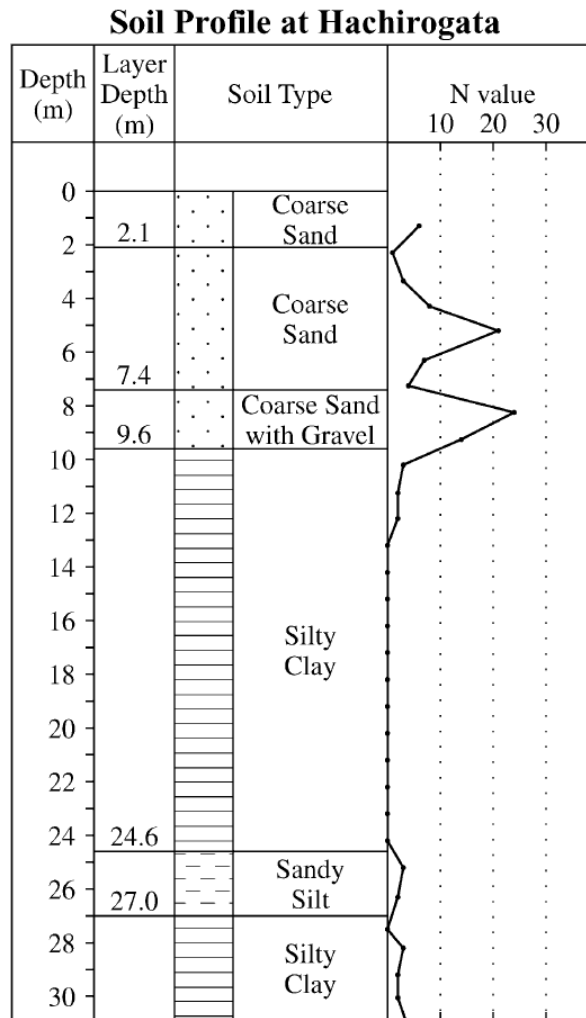


Figure A-2: Hachirogata station soil profile after Kostodinov and Towhata (2002).

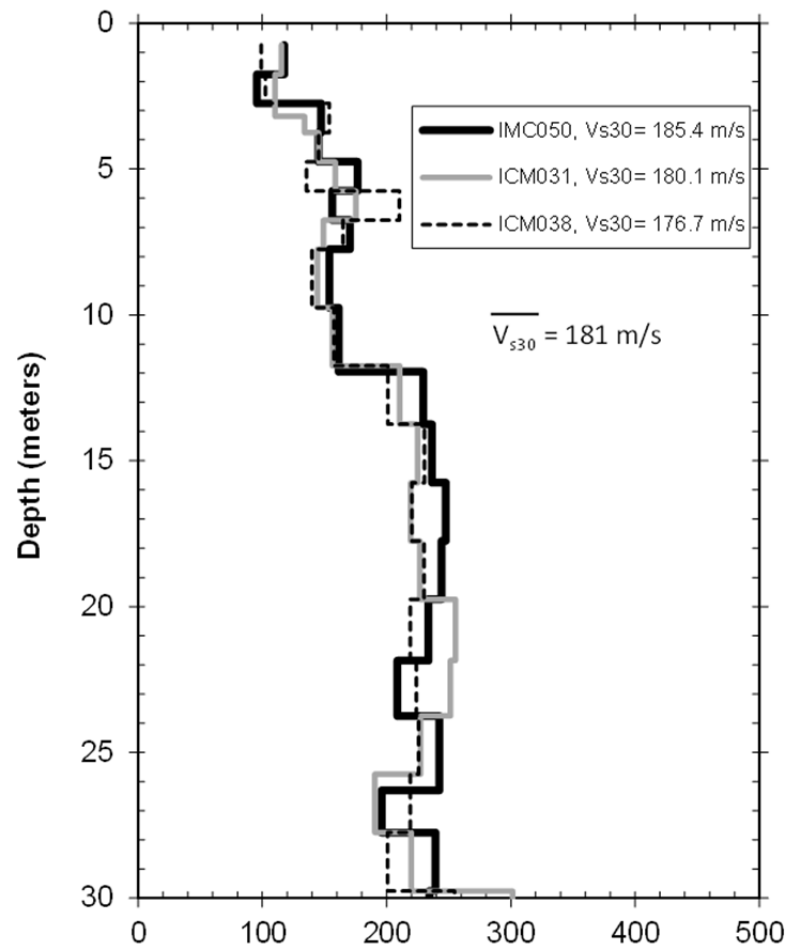


Figure A-3: Downhole Vs measurements from three seismic CPTs at Wildlife Liquefaction Array (Youd et al. 2007, NEES@UCSB 2012).

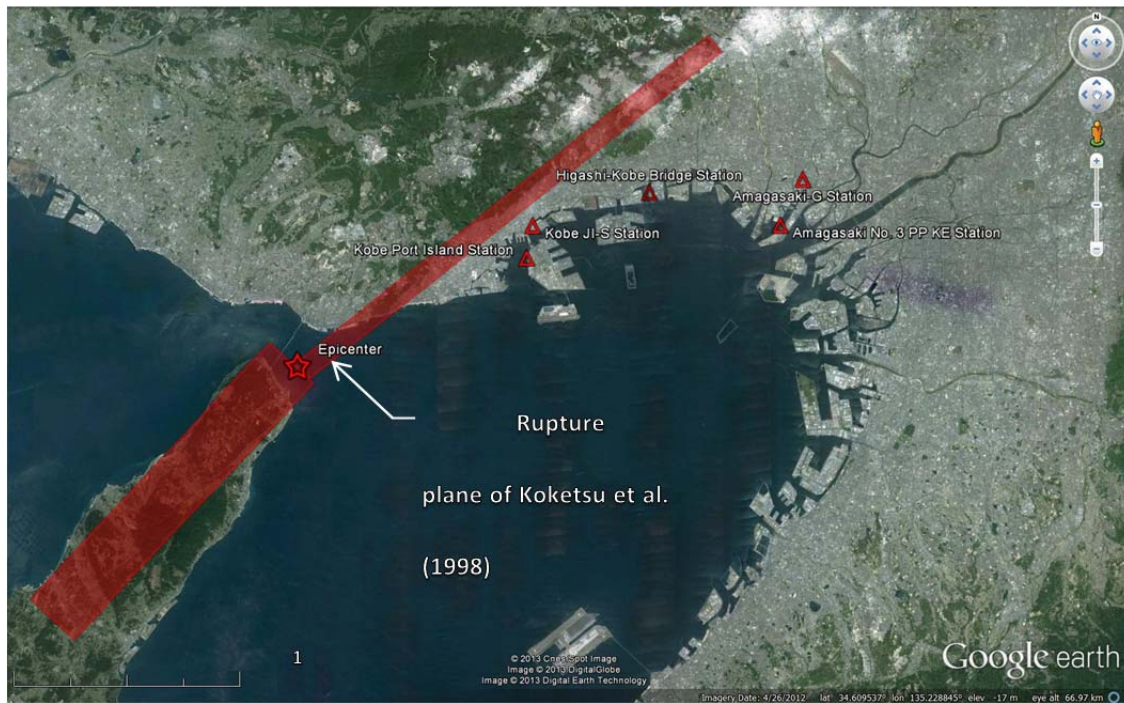


Figure A-4: Hyogoken-Nambu Kobe earthquake rupture plane and liquefaction site accelerometer stations (triangles).

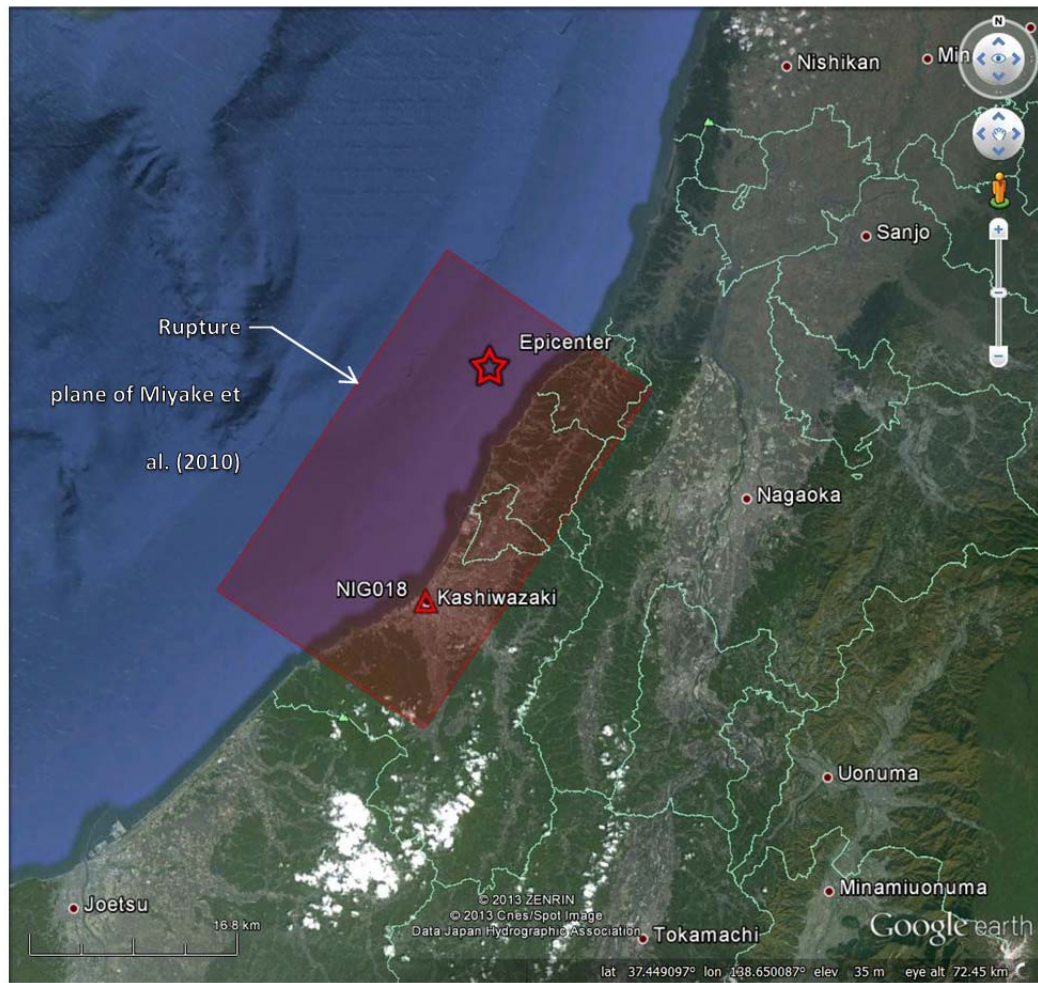


Figure A-5: Niigata-Ken Chuetsu-Oki earthquake rupture plane and NIG018 liquefaction site accelerometer station (triangle).

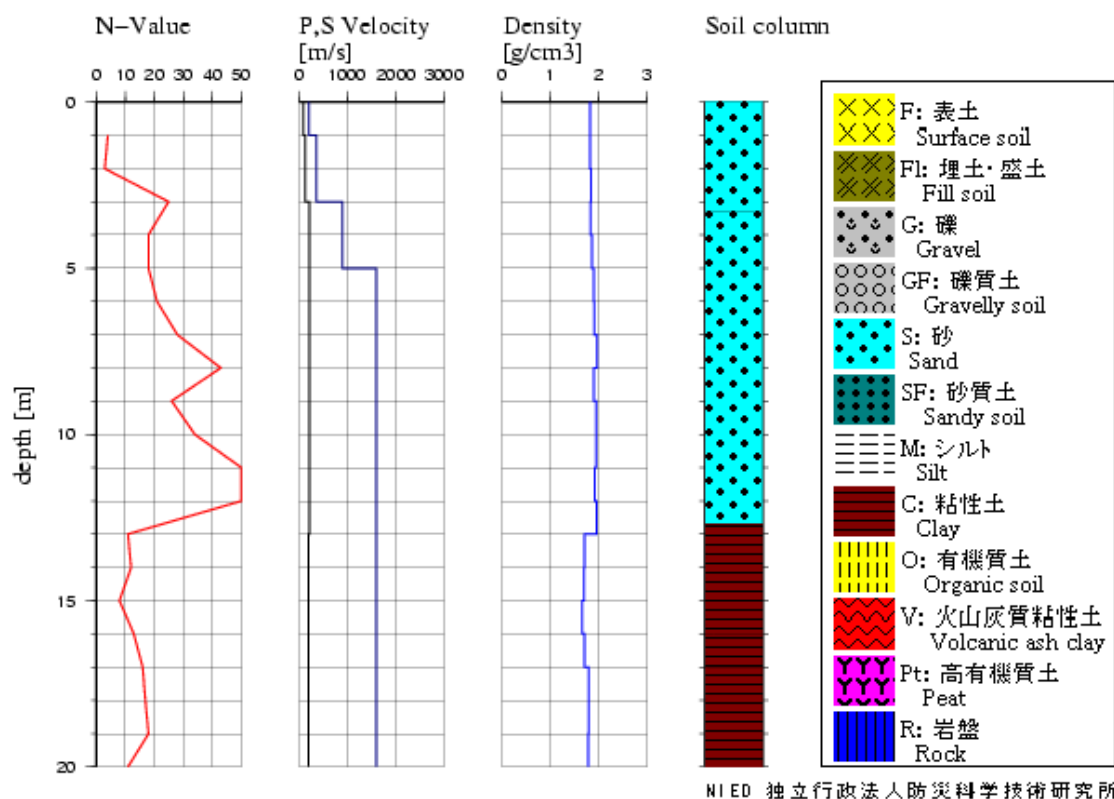


Figure A-6: Soil profile of the NIG018 K-Net strong motion station.



Figure A-7: Darfield earthquake rupture plane and acceleromometer station (triangles) used to compute inter-event residual.



Figure A-8: Darfield earthquake rupture plane, epicenter and liquefied site acceleromometer stations (triangles).

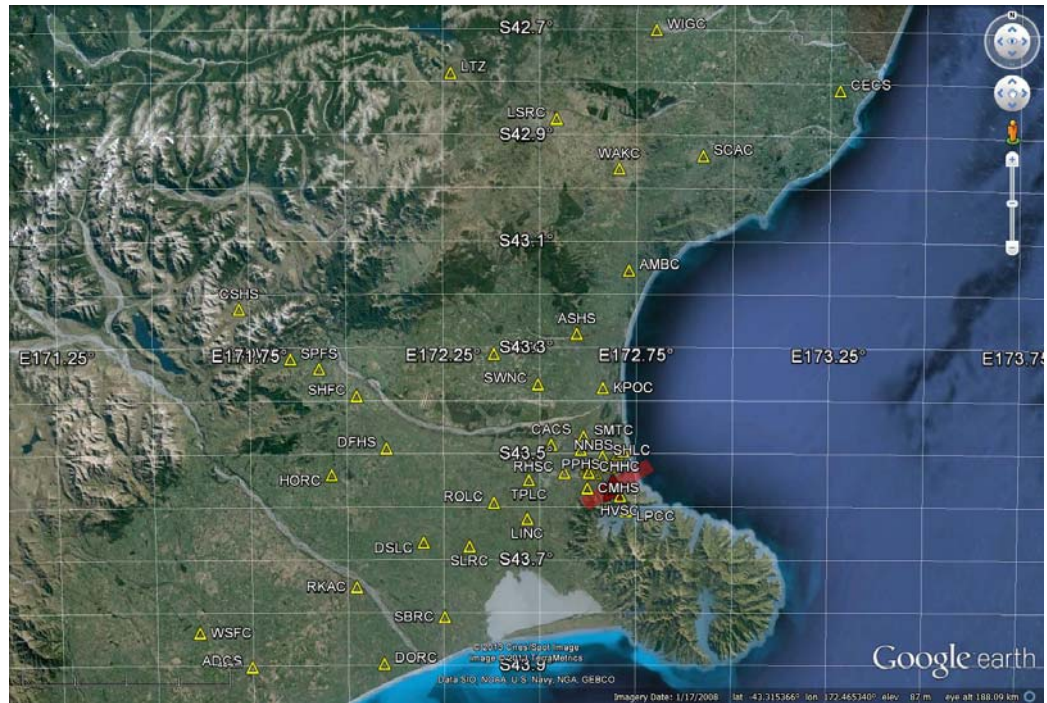


Figure A-9: Christchurch earthquake rupture plane and acceleromometer station (triangles) used to compute inter-event residual.



Figure A-10: Christchurch earthquake rupture plane, epicenter and liquefied site acceleromometer stations (triangles).

Appendix B - Summary of PD2 and PI Models

B.1 Introduction

PressureDependentMultiYield02 (PD2) is an elasto-plastic constitutive model suitable for simulating the response of frictional soils under monotonic and cyclic drained and undrained loading conditions. It can capture salient response characteristics including shear-induced contraction and dilation, excess porewater pressure development, and cyclic mobility. PressureIndependentMultiYield (PI) is an elasto-plastic model is intended for simulation of monotonic and cyclic response of soils whose confinement is insensitive to changes in mean effective stress, such as the undrained response of clays or the drained response of sands. These models have been implemented in the finite element program Opensees (Mazzoni et al. 2010).

This appendix was derived from an unpublished manuscript by Yang and Elgamal (2005) and provides an overview of the PD2 model formulation. The PD2 model was modified from the earlier PressureDependentMultiYield (PD) model, which has been described by Yang et al. (2003, 2008) and Elgamal et al. (2003). While Yang et al. (2008) describes the parameters needed for the PD2 model, details of the differences between the PD and PD2 model have not yet been detailed in the literature. Sections B.2 through B.4 provide a general description of the PD2 model, with focus on modifications to the flow rule and the strain-space mechanism that deviate from the PD model described by (Yang et al. 2003).

The PI is essentially a simplified form of the PD2 model, which is intended for use with materials whose shear behavior is insensitive to confinement change. The Section B.5 provides descriptions of differences between the PI and PD2 model.

B.2 Yield function

Following the standard convention, it is assumed that material elasticity is linear and isotropic, and that nonlinearity and anisotropy result from plasticity (Hill 1950). The yield function (Fig. B.1) is selected as a conical surface in principal stress space (Prevost 1985, Lacy 1986):

$$f = \frac{3}{2}(\mathbf{s} - (p' + p'_0)\boldsymbol{\alpha}) : (\mathbf{s} - (p' + p'_0)\boldsymbol{\alpha}) - M^2(p' + p'_0)^2 = 0 \quad (\text{B.1})$$

in the domain $p' \geq 0$, where $\mathbf{s} = \boldsymbol{\sigma}' - p' \boldsymbol{\delta}$ is the deviatoric stress tensor ($\boldsymbol{\sigma}'$ =effective Cauchy stress tensor, $\boldsymbol{\delta}$ =second-order identity tensor), p' is mean effective stress, p'_0 is a small positive constant (typically, 0.1 kPa) such that the yield surface size remains finite at $p' = 0$ (for numerical convenience and to avoid ambiguity in defining the yield surface normal at the yield surface apex), $\boldsymbol{\alpha}$ is a second-order deviatoric tensor defining the yield surface center in deviatoric stress subspace, M defines the yield surface size, and ":" denotes doubly contracted tensor product. In the context of multi-surface plasticity (Iwan 1967, Mroz 1967, Prevost 1985), the hardening zone is defined by a number of similar yield surfaces (Fig. B.1) with a common apex (at $-p'_0$ along the hydrostatic axis). The outmost surface is designated as the failure surface, the size of which

(M_r) is related to the friction angle ϕ (Chen and Mizuno 1990). The yield surfaces may be calibrated by matching a piecewise linear approximation of the nonlinear shear stress-strain curve (backbone curve, Yang et al. 2003). Note that Lode angle effect may also be incorporated in this model, as described in a related work (Yang and Elgamal 2005).

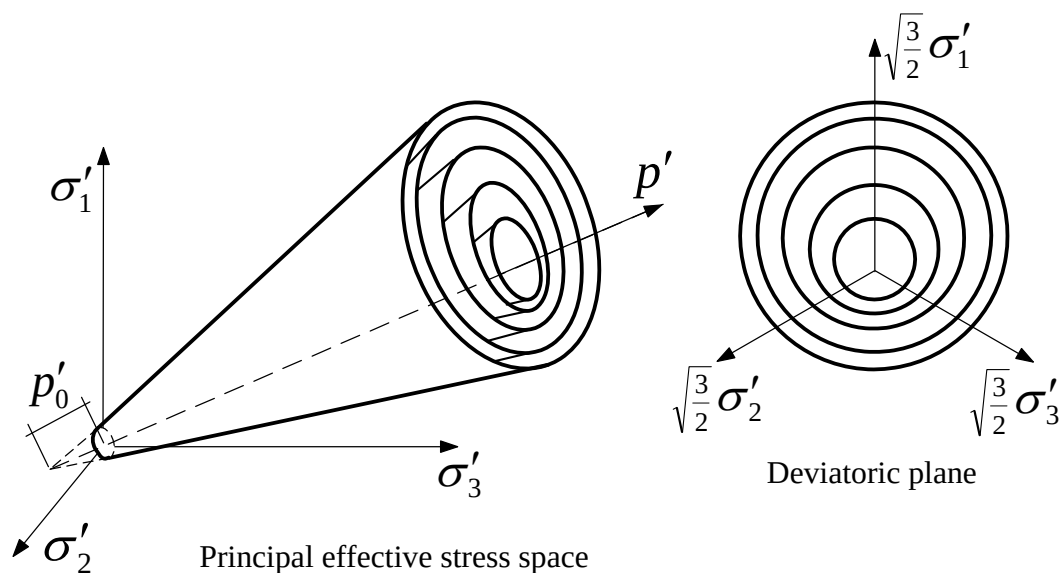


Figure B-1: Conical yield surfaces in principal stress space and deviatoric plane (after Prevost 1985, Parra 1996, and Yang 2000).

Finally, low-strain shear modulus G is assumed to vary with the mean effective stress (or “confinement”) p' such that $G = G_r \left[(p' + p'_0) / (p'_r + p'_0) \right]^d$ where G_r is reference shear modulus defined at a reference confinement p'_r , and d is a material parameter ($=0.5$ typically for sand, Kramer 1996). The bulk modulus B and tangent shear moduli were assumed to follow the same confinement dependence rule $B = B_r \left[(p' + p'_0) / (p'_r + p'_0) \right]^d$.

B.3 Hardening Rule and Shear Stress-Strain Response

Following Mroz (1967) and Prevost (1985), a purely deviatoric kinematic hardening rule was employed to generate hysteretic response (Elgamal et al. 2003). This rule maintains the Mroz (1967) concept of conjugate-points contact, with slight modification in order to enhance computational efficiency (Parra 1996, Elgamal et al. 2003).

Drained shear stress-strain response is governed by backbone curve (Kramer 1996). In the PD2 model, the backbone curve can be established internally based on the hyperbolic formulation (Konder 1963; Duncan and Chang (1970) according to the following formula:

$$\tau = \frac{G\gamma}{1 + \frac{\gamma}{\gamma_r} \left(\frac{p'_r}{p'} \right)^d} \quad (\text{B.2})$$

Where τ and γ are the octahedral shear stress and strain defined in Eqs. B.3 and B.4.

$$\gamma = \frac{2}{3} \left[(\varepsilon_{xx} - \varepsilon_{yy})^2 + (\varepsilon_{yy} - \varepsilon_{zz})^2 + (\varepsilon_{xx} - \varepsilon_{zz})^2 + 6\varepsilon_{xy}^2 + 6\varepsilon_{yz}^2 + 6\varepsilon_{xz}^2 \right]^{1/2} \quad (\text{B.3})$$

$$\tau = \frac{1}{3} \left[(\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{xx} - \sigma_{zz})^2 + 6\sigma_{xy}^2 + 6\sigma_{yz}^2 + 6\sigma_{xz}^2 \right]^{1/2} \quad (\text{B.4})$$

Note that shear strain on the xy-plane in simple shear is $\gamma_{xy} = \varepsilon_{xy} / 2$. The octahedral shear strength is defined as a function of the current effective confinement (p') according to:

$$\tau_f = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} p'$$

The reference octahedral shear strain (γ_r) is selected such that it satisfies the following:

$$\tau_f = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} p'_r = \frac{G_r \gamma_{\max}}{1 + \gamma_{\max} / \gamma_r} \quad (\text{B.5})$$

In the multi-surface plasticity framework, the hyperbolic backbone curve is replaced by a piecewise linear approximation (Fig. B.2). A linear elasto-plastic shear modulus H_m in octahedral shear stress-strain space is defined between each yield surface f_m for $m=1,2,3,\dots,\text{NYS}$, where NYS is the total number of yield surfaces. At the reference confinement, p'_r , H_m is defined by $H_m = 2(\tau_{m+1} - \tau_m) / (\gamma_{m+1} - \gamma_m)$ and the size of the yield surface is defined by:

$$M_m = 3\tau_m / \sqrt{2} (p'_r + p'_0) \quad (\text{B.6})$$

At failure, $M_{\text{NYS}} = M_f$, $\tau_{\text{NYS}} = \tau_f$ and

$$M_{\text{NYS}} = \frac{6 \sin \phi}{3 - \sin \phi} \quad (\text{B.7})$$

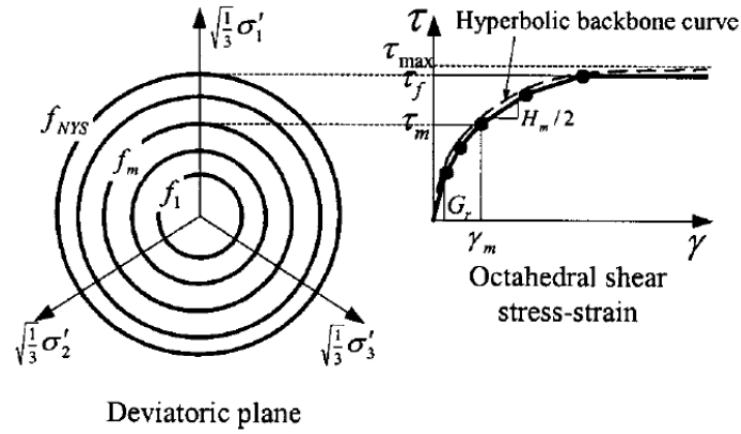


Figure B-2: Backbone curve its piecewise-linear representation though and yield surfaces and tangent shear moduli.

If a condition of $K_0 = 1$ is assumed with simple shear ($\sigma_{yz} = \sigma_{xz} = 0$ and $\varepsilon_{xz} = \varepsilon_{zy} = 0$ and $\varepsilon_{xx} = \varepsilon_{yy} = \varepsilon_{zz} \approx 0$) then Eqns. B.3 and B.4 reduce to

$$\gamma = 2\sqrt{6}/3 \varepsilon_{xy} = \sqrt{6}/3 \gamma_{xy} \quad \text{and} \quad \tau = \sqrt{6}/3 \sigma_{xy} = \sqrt{6}/3 \tau_{xy}, \quad \text{and}$$

$$G_{oct} = \tau_{oct} / \gamma_{oct} = \tau_{xy} / \gamma_{xy} = G.$$

As an alternative to the hyperbolic relationship defined by Eqs. B.2 through B.4, the PD2 model allows the user to specify a shear modulus reduction curve in order to define the backbone curve. Consistent with typical practice, it is assumed that the shear modulus reduction curves are defined for simple shear conditions with the shear strength defined as the maximum shear stress on the xy-plane, $\tau_{xy,f}$. Combining Eqns. B.6, B.7 and $\tau_f = \sqrt{6}/3 \tau_{xy,f}$ yields:

$$\sin \phi = \frac{3\sqrt{3} \tau_{xy,f} / p'_r}{6 + \sqrt{3} \tau_{xy,f} / p'_r} \quad (B.8)$$

where $\tau_{xy,f} = G_{\max} (G/G_{\max})_{NYS} \gamma_{xy,NYS}$, i.e., the product of the last pair of $(G/G_{\max})G_{\max}$ and γ_{xy} in the modulus reduction curve. If one defines the shear strength in simple shear as $\tau_{xy,f} = \sigma'_v \tan \phi_{DSS}$, and $\sigma'_v = p'_r$, and substitutes these into Eqn. B.8, then the friction angle in simple shear (ϕ_{DSS}) is related to the model friction angle (ϕ) by:

$$\phi_{DSS} = \tan^{-1} \left[\frac{2\sqrt{3} \sin \phi}{3(3 - \sin \phi)} \right] \quad (B.9)$$

and conversely

$$\phi = \tan^{-1} \left[\frac{3 \tan \phi_{DSS}}{2\sqrt{3} + \tan \phi_{DSS}} \right] \quad (B.10)$$

When exercising this option, it is important to use a $G/G_{\max}$ vs. γ curve that does not imply strain softening, which is not allowed by the model. Yield surfaces for stress states within the failure surface are established using variations of Eqs. B.5 and B.6, where the mobilized friction angle (ϕ_m), modulus reduction pair product (σ_m) and octahedral shear stress (τ) are used in place of ϕ , σ_{NYS} and τ_f , respectively.

B.4 Flow rule

We define **Q** and **P** as the outer normals to the yield surface and the plastic potential surface, respectively. These tensors may be conveniently decomposed into deviatoric and volumetric components, giving $\mathbf{Q} = \mathbf{Q}' + \mathbf{Q}''\mathbf{\delta}$ and $\mathbf{P} = \mathbf{P}' + \mathbf{P}''\mathbf{\delta}$ (Prevost 1985). Nonassociativity of the

plastic flow is restricted to its volumetric component (Prevost 1985), i.e., $Q' = P'$ and $P'' \neq Q''$.

The relative location of the stress state with respect to the phase transformation (PT) surface may be inferred (Prevost 1985) from the stress ratio $\eta \equiv \sqrt{3(\mathbf{s}:\mathbf{s})/2} / (p' + p'_0)$. Designating η_{PT} as the stress ratio along the PT surface, it follows that $\eta < \eta_{PT}$ (or $\eta > \eta_{PT}$) if the stress state is inside (or outside) the PT surface.

Depending on the value of η and the sign of $\dot{\eta}$ (time rate of η), distinct contractive/dilative (dilatancy) behaviors are reproduced by specifying appropriate expressions for P'' (Fig. B.3). In addition, a neutral phase ($P'' = 0$, Phases 4-5 and 7-8 in Fig. B.3) is introduced between the contraction ($P'' > 0$, phases 3-4 and 6-7) and the dilation ($P'' < 0$, phases 5-6 and 8-9) phases. This neutral phase conveniently allows for modeling the accumulation of highly yielded shear strain, as will be discussed below.

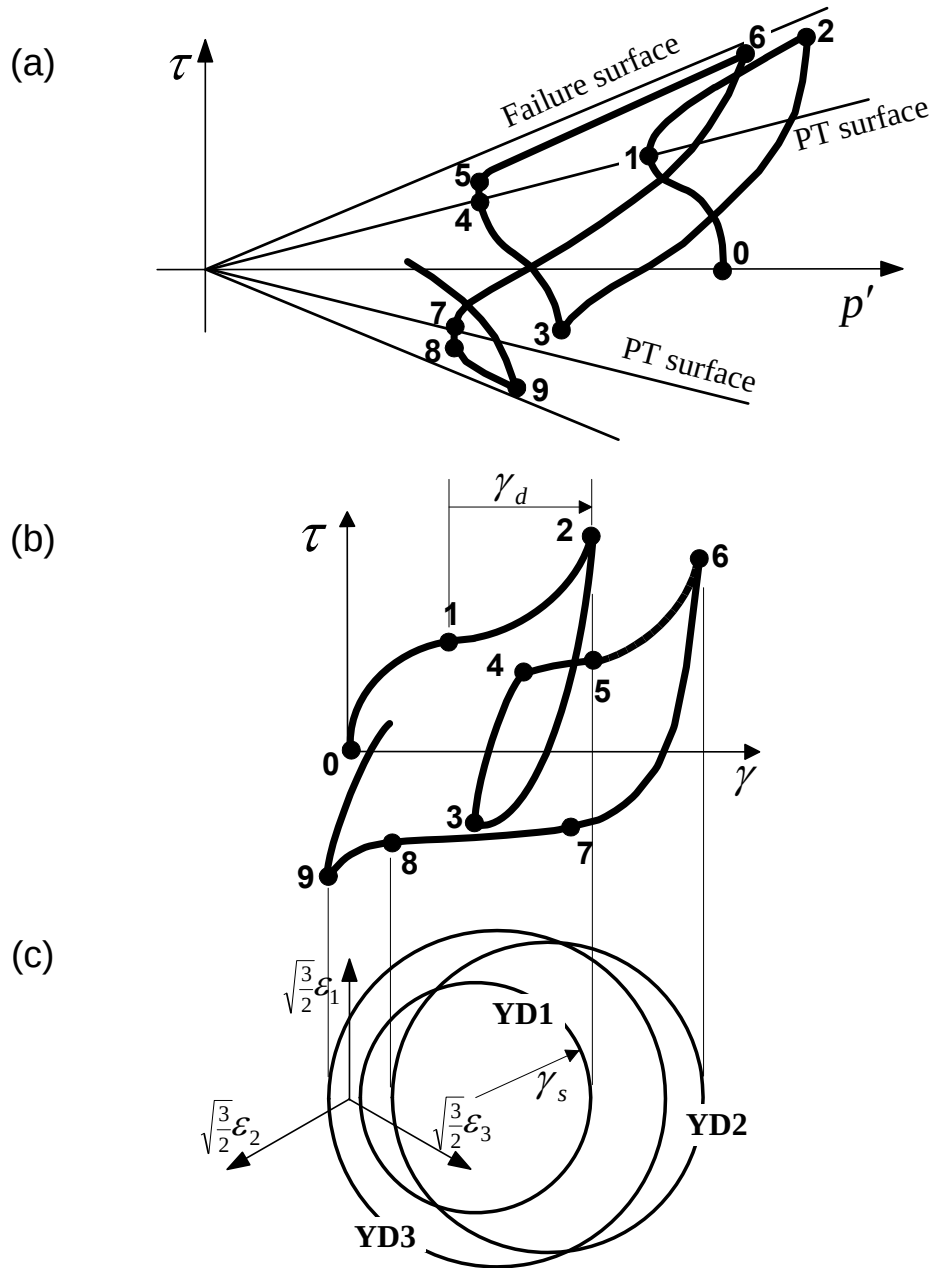


Figure B-3: Schematic of constitutive model response showing (a) octahedral stress τ - effective confinement p' response, (b) τ - octahedral strain γ response, and (c) configuration of yield domain.

B.4.1 Contractive phase (phases 0-1, 2-3, 3-4, and 6-7 in Fig. B.3)

Shear-induced contraction occurs inside the PT surface ($\eta < \eta_{PT}$), as well as outside ($\eta > \eta_{PT}$) when $\dot{\eta} < 0$. The contraction flow rule is:

$$P'' = \left(1 - \frac{\mathbf{n} \cdot \dot{\mathbf{s}}}{\|\dot{\mathbf{s}}\|} \frac{\eta}{\eta_{PT}}\right)^2 (c_1 + c_2 \gamma_d) \left(\frac{p'}{p_a}\right)^{c_3} \quad (\text{B.11})$$

where c_1 , c_2 and c_3 are non-negative calibration constants, γ_d is octahedral shear strain accumulated during previous dilation phases (more discussion below), p_a is atmospheric pressure for normalization purpose, and $\dot{\mathbf{s}}$ is stress rate. The $\mathbf{n}$ and $\dot{\mathbf{s}}$ are introduced in the equation to account for general 3D loading scenarios. As shown in Fig. B.4, $\mathbf{n}$ is the outer normal to a surface similar to the yield surfaces passing by the stress point $\mathbf{s}$ (with center on the hydrostatic axis).

The parameter c_3 in Eq. B.7 represents the dependence of pore pressure buildup on initial confinement (i.e., K_σ effect). Therefore, contraction tendency becomes stronger at higher confinements. Furthermore, the $c_2 \gamma_d$ term represents the effect of previous dilative phases on the contraction behavior during subsequent unloading. Similar response mechanisms have also been included in other notable models such as Dafalias and Manzari (1999) and Papadimitriou et al. (2001).

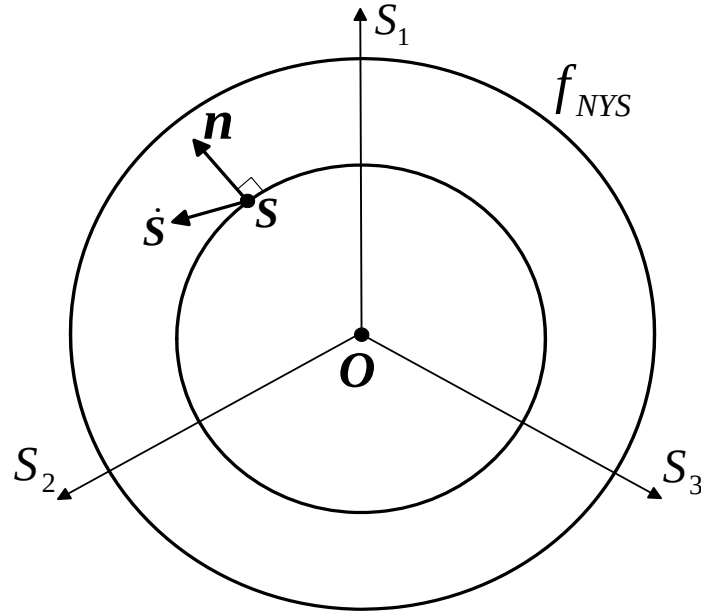


Figure B-4: Definition of vector n in the flow rules (Eqs. B.7 and B.8).

B.4.2 Dilative phase (phases 1-2, 5-6, and 8-9 in Fig. B.3)

Dilation appears only due to shear loading outside the PT surface ($\eta > \eta_{PT}$ with $\dot{\eta} > 0$), according to the following:

$$P'' = -\left(1 - \frac{\mathbf{n} \cdot \dot{\mathbf{s}}}{\|\dot{\mathbf{s}}\|} \frac{\eta}{\eta_{PT}}\right)^2 d_1 (\gamma_d)^{d_2} \left(\frac{p'}{p_a}\right)^{-d_3} \quad (\text{B.12})$$

where d_1 , d_2 and d_3 are non-negative calibration constants, and γ_d is the octahedral shear strain accumulated during all dilation phases in the same direction as long as there is no significant load reversal. In Eq. B.8, the last pressure-dependence term reflects the influence of K_σ effect on dilation tendency (i.e., less dilation at higher confinements).

B.4.5 Neutral phase (phases 4-5 and 7-8 in Fig. B.3)

As shown in Fig. B.1, when the stress state approaches the PT surface ($\eta = \eta_{PT}$) from below, a significant amount of permanent shear strain may accumulate prior to dilation, with minimal changes in shear stress and p' (implying $P'' \approx 0$). Such minimal change in the stress state is difficult to employ as a basis for modeling the associated extent of shear strain accumulation. Hence, for simplicity, $P'' \approx 0$ is maintained during this highly yielded phase (phase 4-5), until a boundary defined in deviatoric strain space is reached (Fig. B.3c), with subsequent dilation thereafter (phase 5-6). This domain will enlarge or translate (Fig. B.3c) depending on load history, as described below.

In the original formulation (Yang et al. 2003), the domain of liquefaction-induced shear deformation only exists for confinements lower than a user-prescribed value (e.g., 10 kPa). However, such a confinement level is practically difficult to define. Moreover, the size of this domain is limited to a user-defined maximal value. Once this maximum is reached, further cyclic loading will result in identical shear stress-strain loops. As observed in typical laboratory sample tests, the extent of cyclic-liquefaction permanent shear deformation can grow indefinitely, on a cycle-by-cycle basis. Such a degradation of shear stress-strain response may be considered as a damage effect to the sand skeleton due to repeated cyclic loading and loss of confinement (under undrained conditions).

In this paper, a damage parameter is introduced in the above strain-space mechanism, as a function of shear strain and effective confinement histories. This allows for continuing enlargement of the domain during cyclic loading and eliminates the need of prescribing a particular confinement to activate the strain-

space mechanism. Specifically, the yield domain is a circle in deviatoric strain space (for simplicity), with the radius γ_s (Fig. B.3c) defined as:

$$\gamma_s = \frac{y_1}{2} \left\langle \frac{p'_o - p'_n}{p'_o} \right\rangle^{0.25} \int_0^t d\gamma_d \quad (\text{B.13})$$

where p'_o is initial mean effective confinement, p'_n mean effective confinement at the beginning of current neutral phase, $\int_0^t d\gamma_d$ shear strain accumulated during all previous dilative phases, y_1 a positive calibration constant, and $\langle \rangle$ denotes MacCauley's brackets (i.e., $\langle a \rangle = \max(a, 0)$). In other words, Eq. B.9 states that the domain size grows in proportion to: 1) shear strain accumulated during previous dilative phases, and 2) reduction in mean effective confinement p' .

Fig. B.3c illustrates the evolution process of the yield domain during a schematic cyclic simple shear path. When the stress state reaches the PT surface for the first time (State 1), the yield domain size is zero (since no dilation has taken place yet) and dilation starts immediately. As the dilation proceeds (phase 1-2), the yield domain size increases and the boundary of the domain coincides with the (deviatoric) shear strain state (Yield Domain 1 or YD1). During the unloading-reloading phases 2-3-4, the yield domain remains untouched. When the stress state reaches the PT surface again at state 4, the corresponding strain state is inside the yield domain, which triggers the neutral phase 4-5. During this phase, P'' remains zero till the boundary of the yield domain is reached at state 5 (which coincides with state 2 in shear strain). Thereafter,

dilation starts and the yield domain enlarges as well as translates with the shear strain state (YD2). When the stress state reaches the PT surface on the opposite side during the unloading phase 6-7, the corresponding strain state is inside the yield domain and therefore the neutral phase takes place again. This phase continues till the boundary of the yield domain is reached on the opposite side at state 8. Thereafter, dilation starts and the yield domain enlarges and translates again (YD3).

Fig. B.5 presents example results of single element cyclic simple shear simulations performed with the PD2 model in Openses. The upper figures (a) are drained strain-controlled simulation, and the lower figures (b) are from an undrained stress controlled simulation. The undrained test exemplifies excess porewater pressure buildup and reduction of vertical effective stress with increasing cycles, and phases of contraction and dilation. The drained tests demonstrates the show no porewater pressure buildup or change in effective stress, and illustrates the shear stress-strain backbone curve.

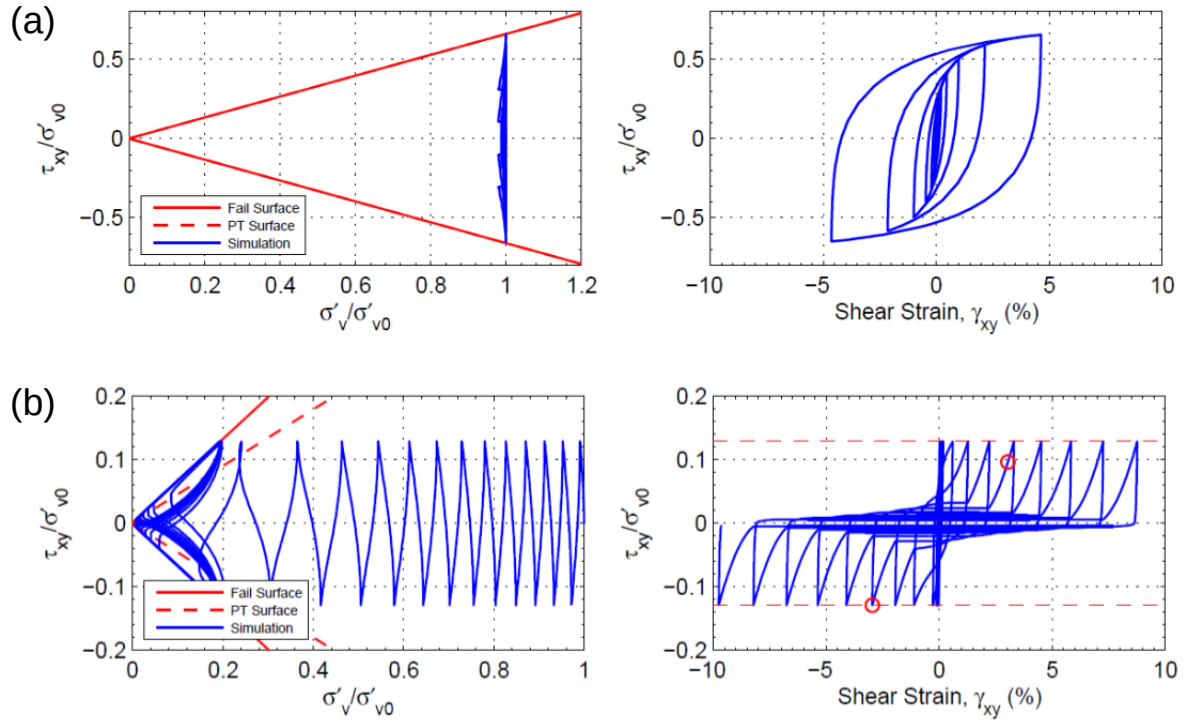


Figure B-5: Examples of (a) drained and (b) undrained response of cyclic simple shear simulations.

B.5 PI Model

The formulation of the PI model is similar to the PD2 with the exception of the differences described below. The shear strength is defined in terms of the initial confinement (p'_i), and can include both frictional and cohesive (c) components:

$$\tau_f = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} p'_i + \frac{2\sqrt{2}}{3} c \quad (\text{B.14})$$

Eq. B.2 is used for the internally-generated hyperbolic shear stress-strain curve option, but γ_r satisfies the following relationship:

$$\tau_r = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} p'_i + \frac{2\sqrt{2}}{3} c = \frac{G_r \gamma_{\max}}{1 + \gamma_{\max} / \gamma_r} \quad (\text{B.15})$$

For the case of user-defined shear modulus reduction curves with $\phi = 0$, the cohesion is defined by:

$$c = \frac{\sqrt{3} \sigma_{NYS}}{2} \quad (\text{B.16})$$

whereas for a material with $\phi > 0$ and $c = 0$, the value of ϕ is defined by Eq. B.7.

The hardening rule for the PI model is the same as that of the PD2 model. In the PI model, the volumetric response is linear-elastic and independent of the deviatoric response (associated flow rule). The contraction and dilation rules described for the PI2 model do not apply. The PI model uses the same pressure-dependent shear and bulk modulus relationships as the PD2 model.

B.6 References

- Chen, W.F. and Mizuno, E. (1990). *Nonlinear Analysis in Soil Mechanics, Theory and Implementation*, Elsevier, New York, NY.
- Dafalias, Y.F. and Manzari, T.M. (2004). "Simple Plasticity Sand Model Accounting for Fabric Change Effects," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 130(6), 622-634.
- Elgamal, A., Yang, Z., Parra, E. and Ragheb, A. (2003). "Modeling of Cyclic Mobility in Saturated Cohesionless Soils," *Int. J. Plasticity*, 19(6), 883-905.
- Hill, R. (1950). *The Mathematical Theory of Plasticity*, Oxford Univ. Press, London.

- Iwan, W.D. (1967). "On a Class of Models for the Yielding Behavior of Continuous and Composite Systems," *J. Applied Mechanics*, ASME, 34, 612-617.
- Kramer, S.L. (1996). *Geotechnical Earthquake Engineering*, Prentice Hall, NJ.
- Lacy, S. (1986). "Numerical Procedures for Nonlinear Transient Analysis of Two-phase Soil System," *Ph.D. dissertation*, Princeton University, New Jersey.
- Mazzoni, S., McKenna, F., Fenves, G.L. (2010). OpenSees Online Documentation, http://opensees.berkeley.edu/wiki/index.php/Main_Page, Pacific Earthquake Engineering (PEER) Center.
- Mroz, Z. (1967). "On the Description of Anisotropic Work Hardening," *J. Mechanics and Physics of Solids*, 15, 163-175.
- Papadimitriou, A.G., Bouckovalas, G.D., and Dafalias, Y.F. (2001). "Plasticity Model for Sand under Small and Large Cyclic Strains," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 127(11), 973-983.
- Parra, E. (1996). "Numerical Modeling of Liquefaction and Lateral Ground Deformation Including Cyclic Mobility and Dilation Response in Soil Systems," *Ph.D. Thesis*, Dept. of Civil Engineering, RPI, Troy, NY.
- Prevost, J.H. (1985). "A Simple Plasticity Theory for Frictional Cohesionless Soils," *Soil Dynamics And Earthquake Engineering*, 4(1), 9-17.
- Yang, Z., Elgamal, A. and Parra, E. (2003). "A Computational Model for Cyclic Mobility and Associated Shear Deformation," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 129(12), 1119-1127.
- Yang, Z. and Elgamal, A. (2008). "Multi-surface Cyclic Plasticity Sand Model with Lode Angle Effect." *Geotechnical and Geological Engineering*, 26(3), 335-348.
- Yang, Z. and Elgamal, A. (2005). "Calibration of a sand model for predicting liquefaction triggering and liquefaction-induced lateral spreading," *unpublished draft manuscript*.

Appendix C

Element Calibration Test Results

Table C-1 Index of Element Test Simulation Results

Relative Density (%)	Vertical Effective Stress (atm)	Figure No.	
		Drained	Undrained
30	0.5	C-1	C-2
	1	C-3	C-4
	2	C-5	C-6
	4	C-7	C-8
	8	C-9	C-10
	16	C-11	C-12
40	0.5	C-13	C-14
	1	C-15	C-16
	2	C-17	C-18
	4	C-19	C-20
	8	C-21	C-22
	16	C-23	C-24
50	0.5	C-25	C-26
	1	C-27	C-28
	2	C-29	C-30
	4	C-31	C-32
	8	C-33	C-34
	16	C-35	C-36
60	0.5	C-37	C-38
	1	C-39	C-40
	2	C-41	C-42
	4	C-43	C-44
	8	C-45	C-46
	16	C-47	C-48
70	0.5	C-49	C-50
	1	C-51	C-52
	2	C-53	C-54
	4	C-55	C-56
	8	C-57	C-58
	16	C-59	C-60
80	0.5	C-61	C-62
	1	C-63	C-64
	2	C-65	C-66
	4	C-67	C-68
	8	C-69	C-70
	16	C-71	C-72
90	0.5	C-73	C-74
	1	C-75	C-76
	2	C-77	C-78
	4	C-79	C-80
	8	C-81	C-82
	16	C-83	C-84

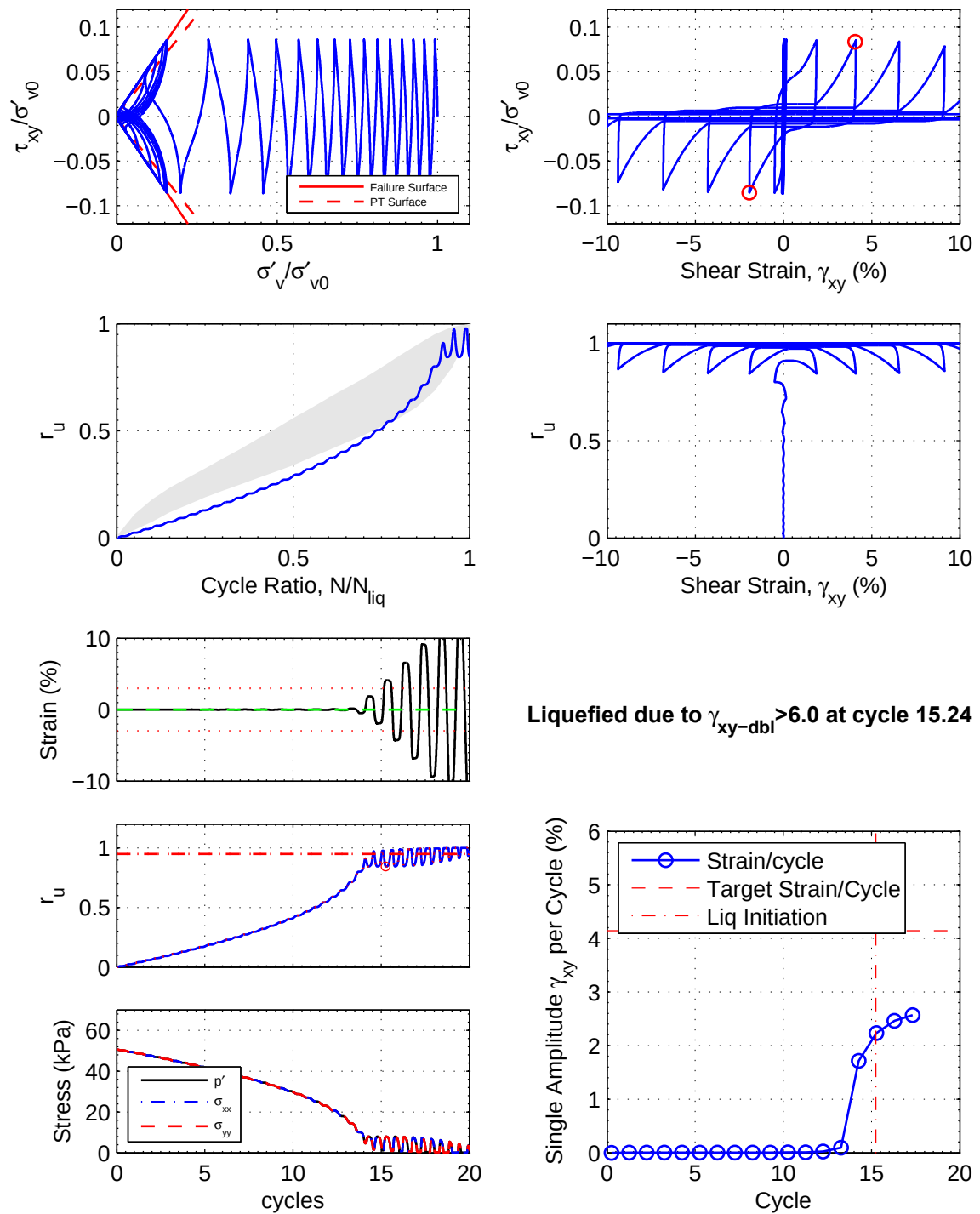


Figure C-1: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=0.5$ atm, $CSR=0.086$, $\phi=23.5$, $\phi_{PT}=20$

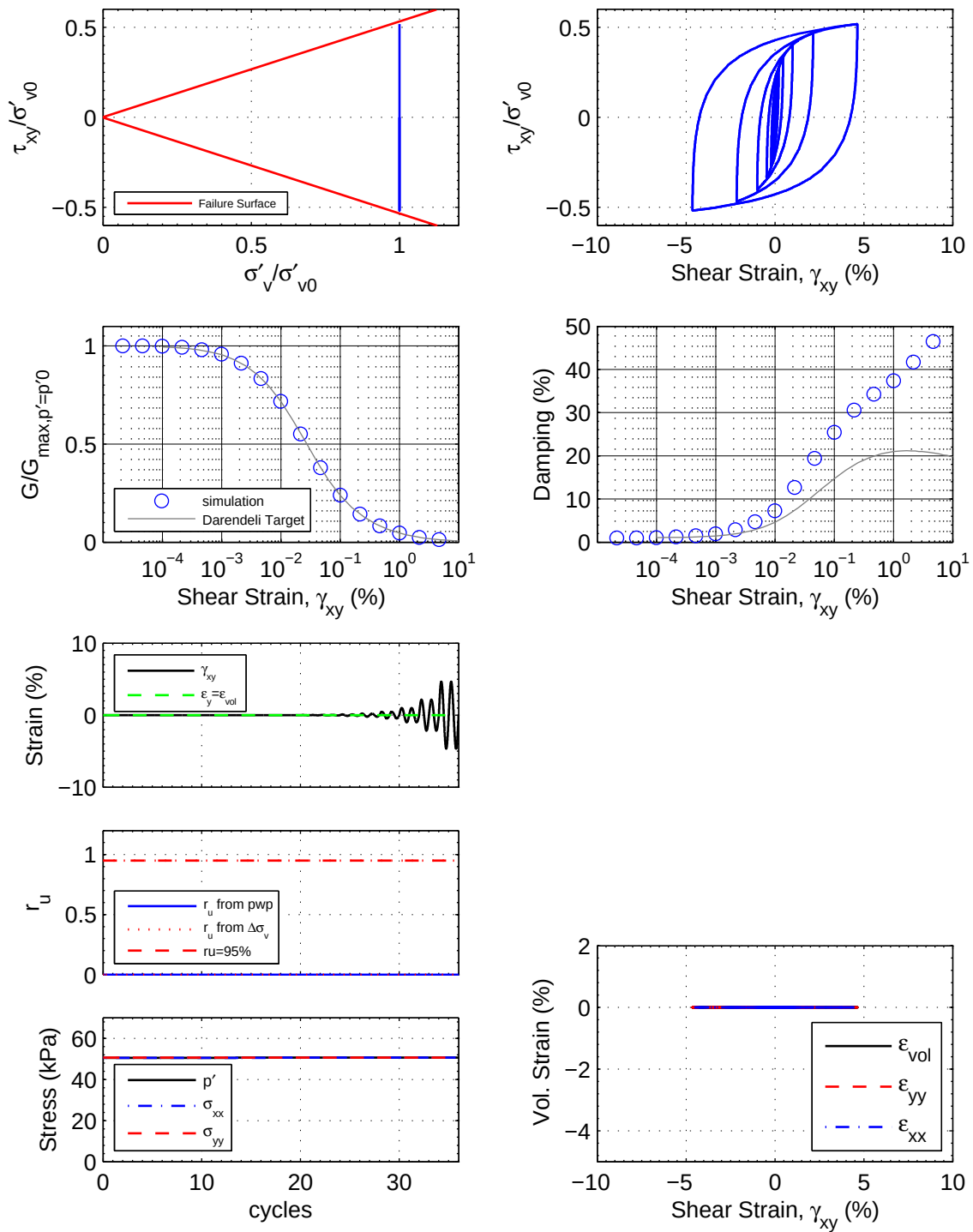


Figure C-2: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$

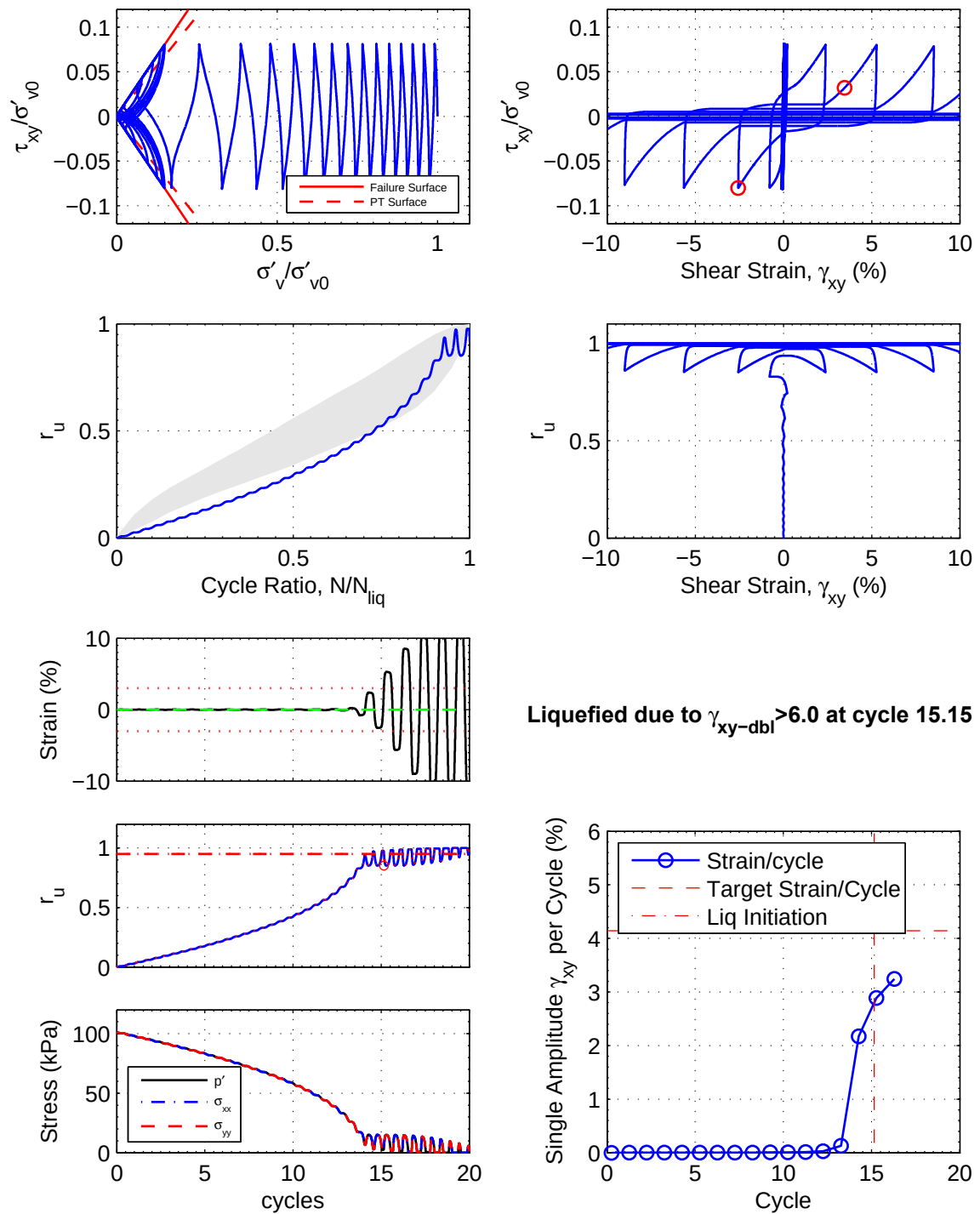


Figure C-3: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=1$ atm, $CSR=0.081$, $\phi=23.5$, $\phi_{PT}=20$

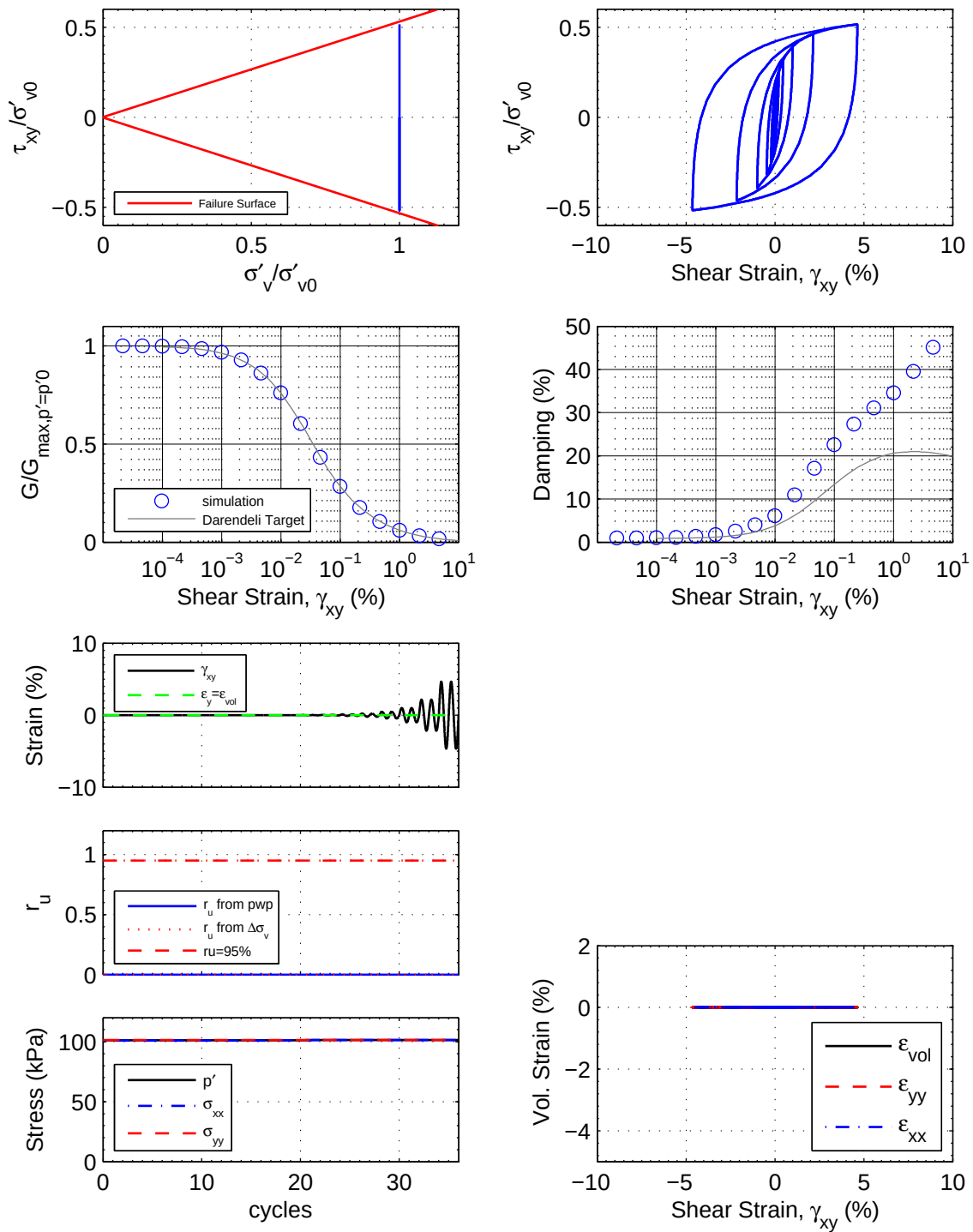


Figure C-4: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=1$ atm, $\phi=23.5$, $\phi_{PT}=na$

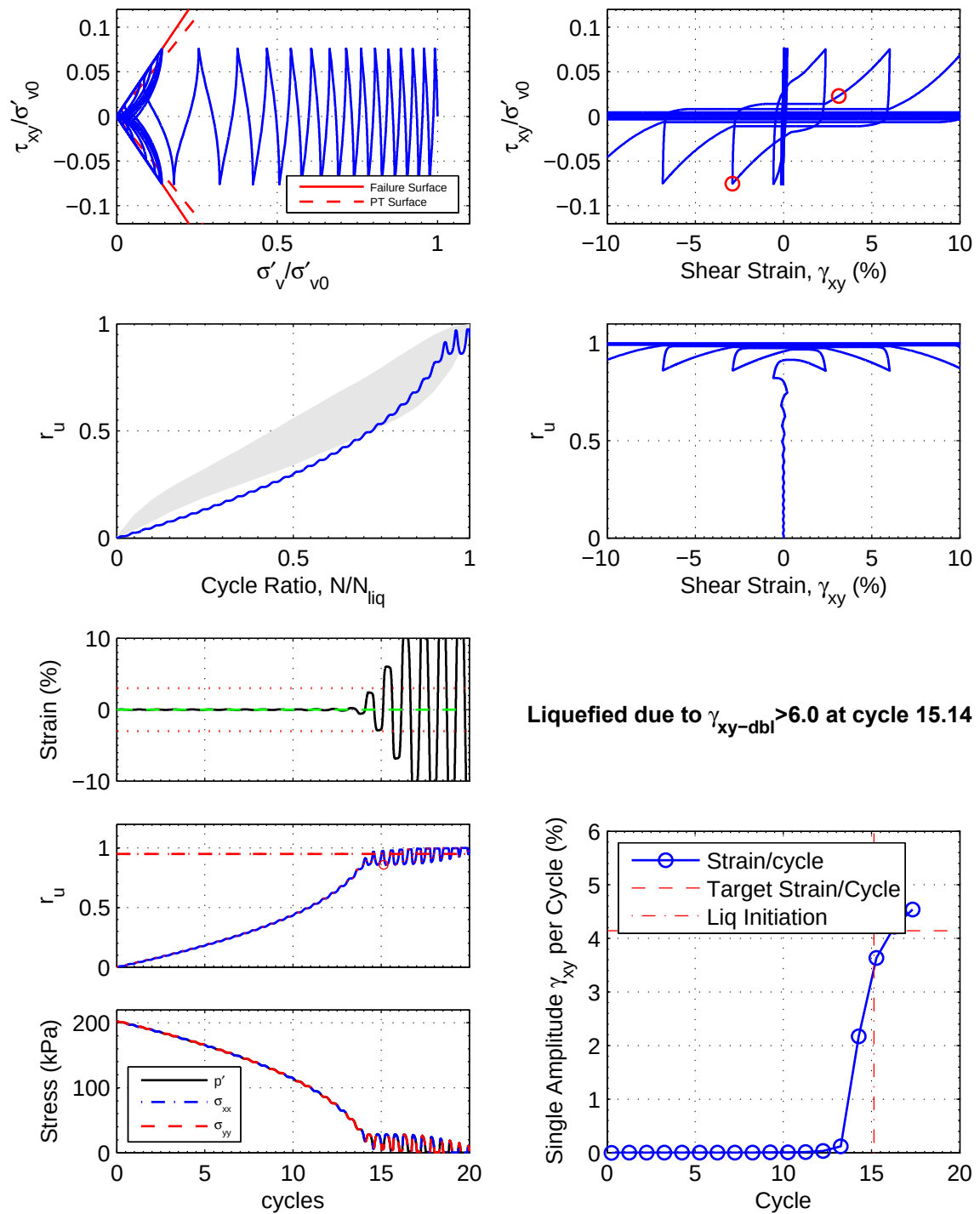


Figure C-5: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=2$ atm, $CSR=0.076$, $\phi=23.5$, $\phi_{PT}=20$

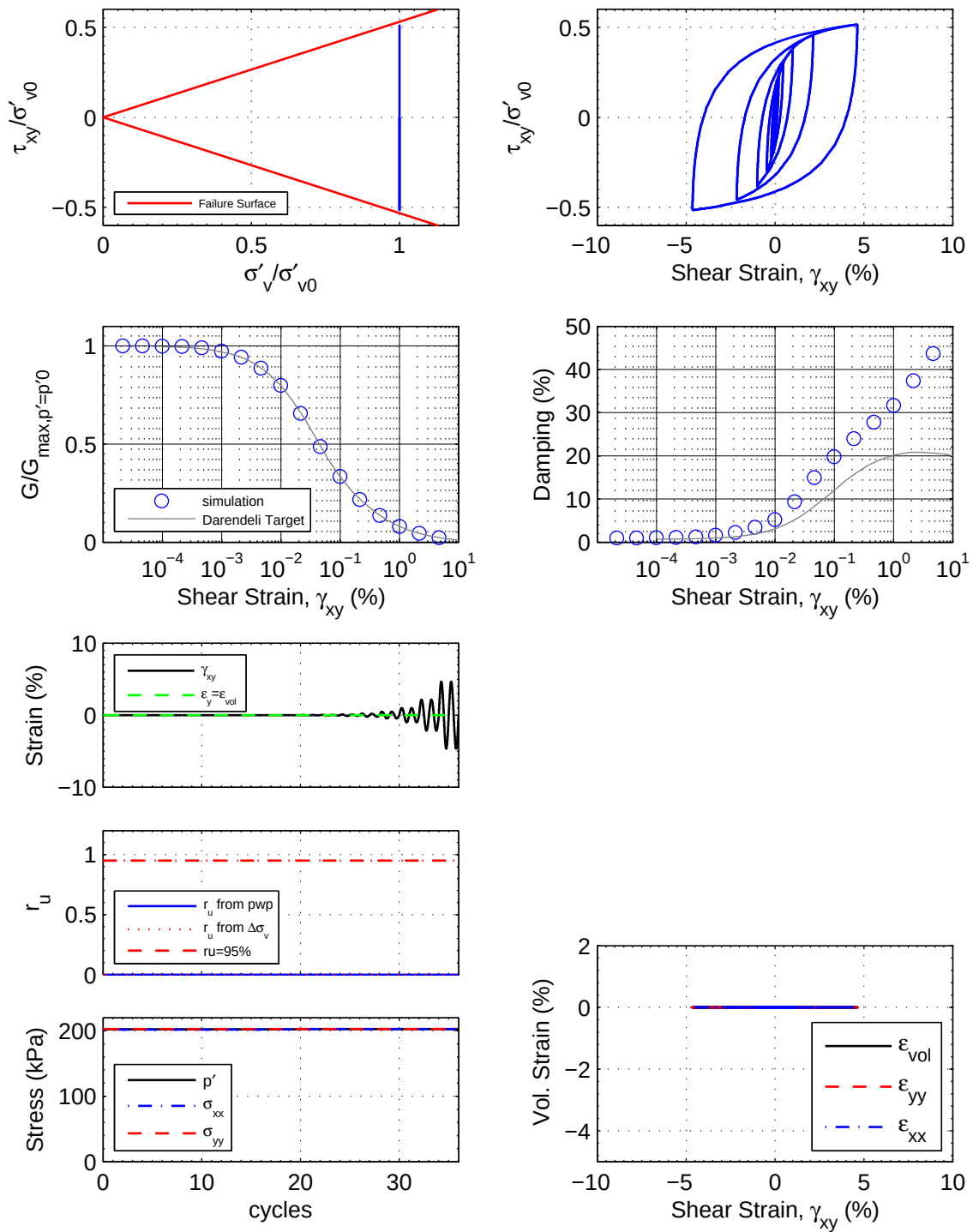


Figure C-6: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=2$ atm, $\phi=23.5$, $\phi_{PT}=na$

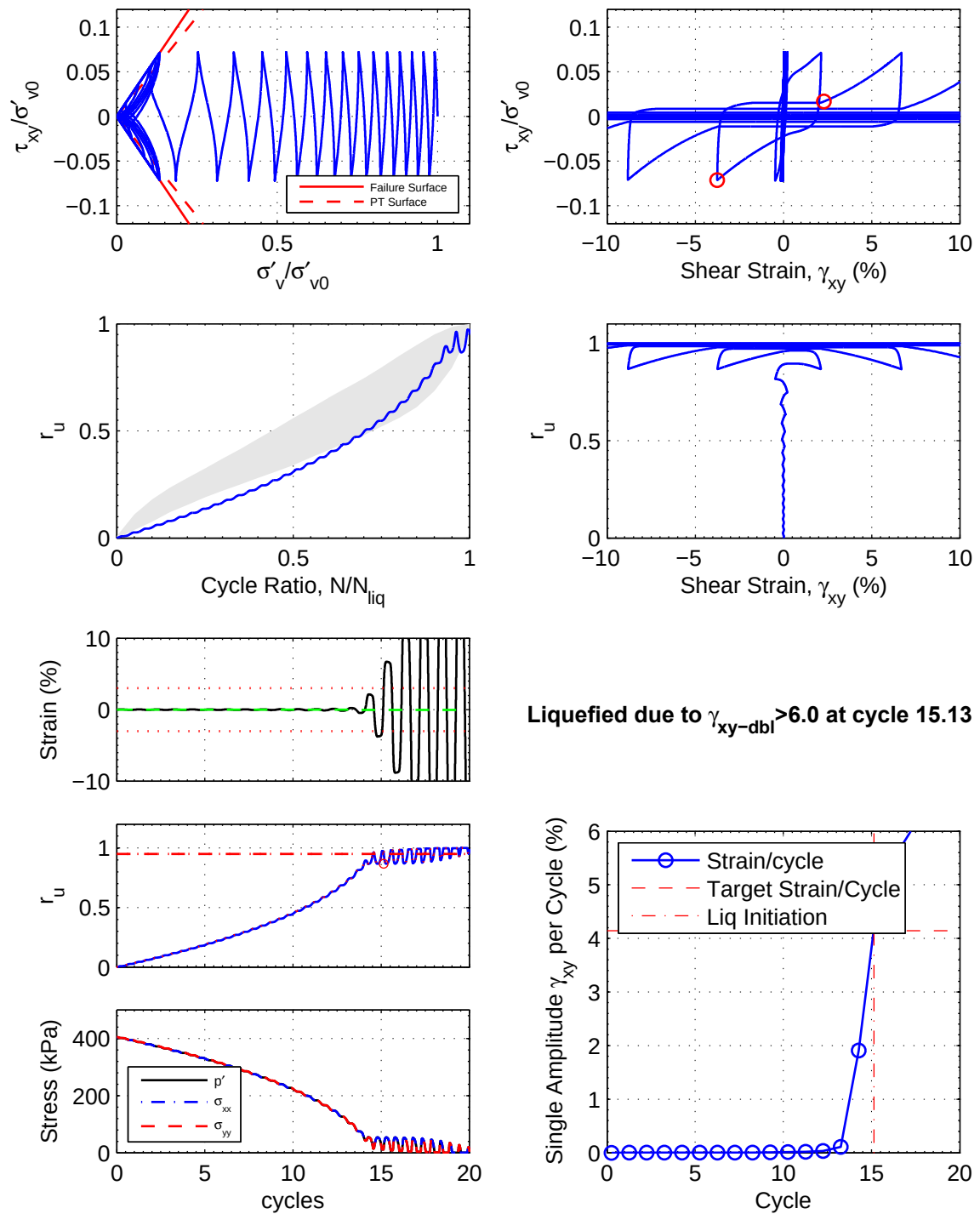


Figure C-7: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=4$ atm, $CSR=0.072$, $\phi=23.5$, $\phi_{PT}=20$

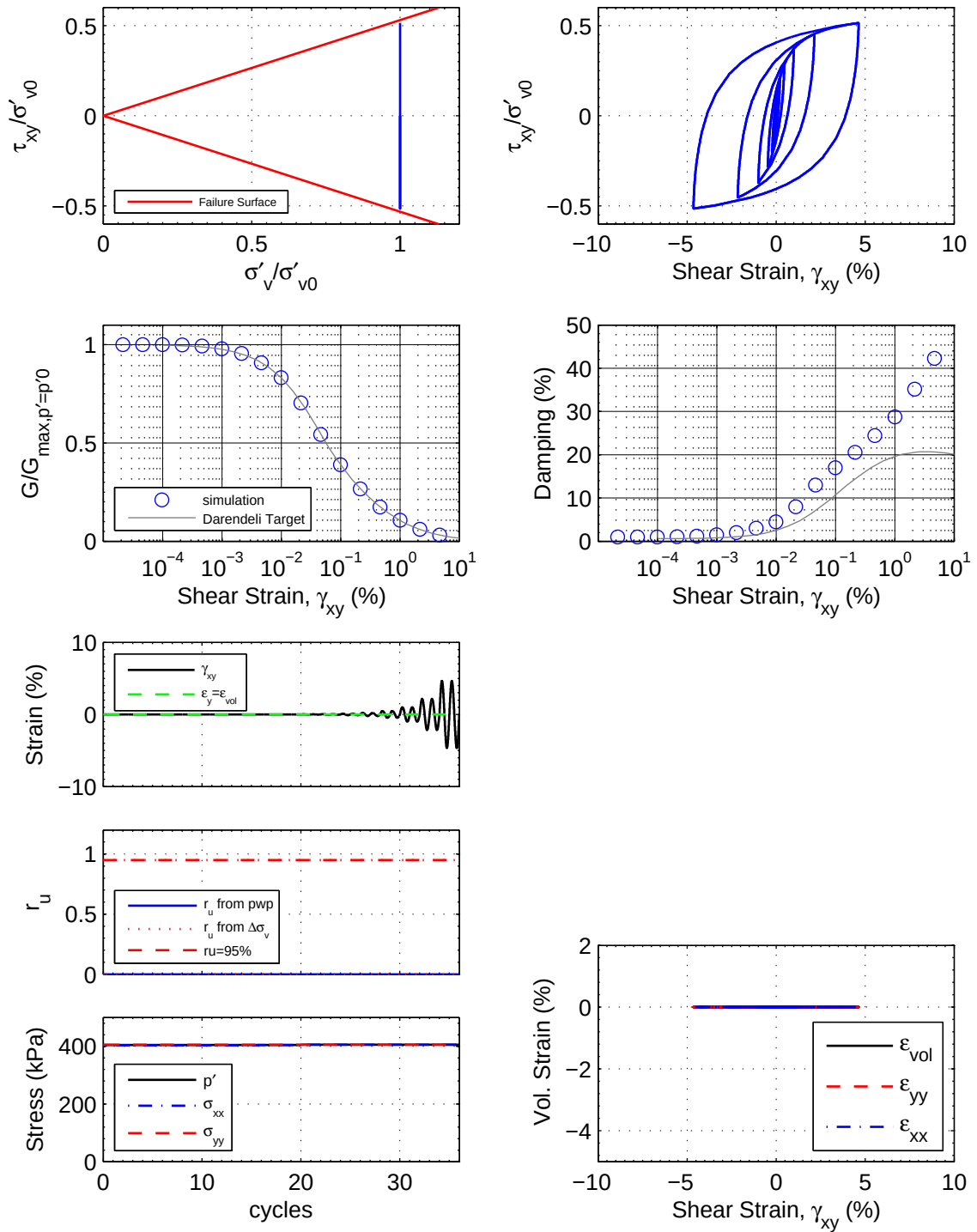


Figure C-8: Drained CSS Simulation: PIMY, $Dr=30, \sigma'_{v0}=4$ atm, CyclicStrain, $\phi=23.5, \phi_{PT}=na$

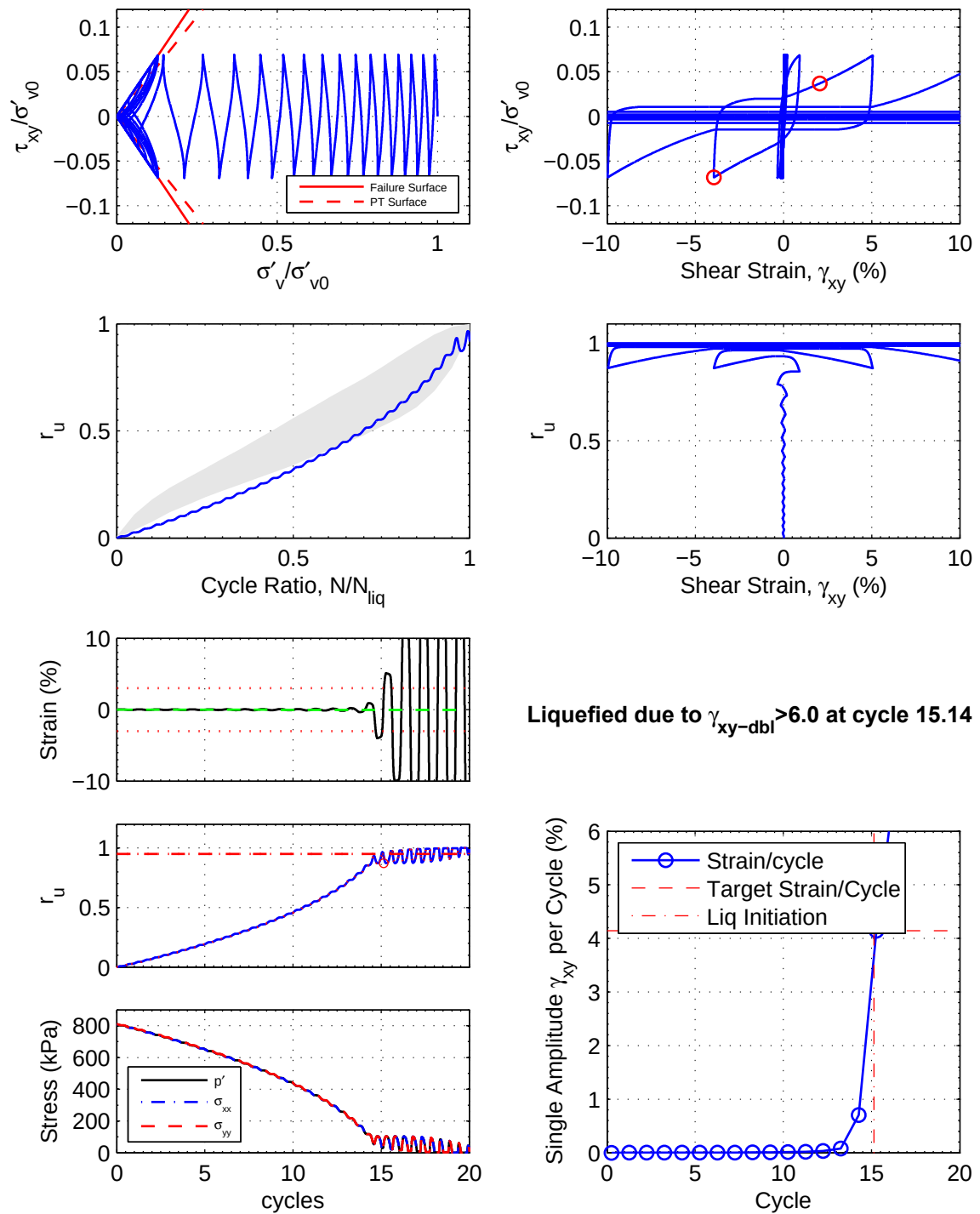


Figure C-9: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=8$ atm, $CSR=0.069$, $\phi=23.5$, $\phi_{PT}=20$

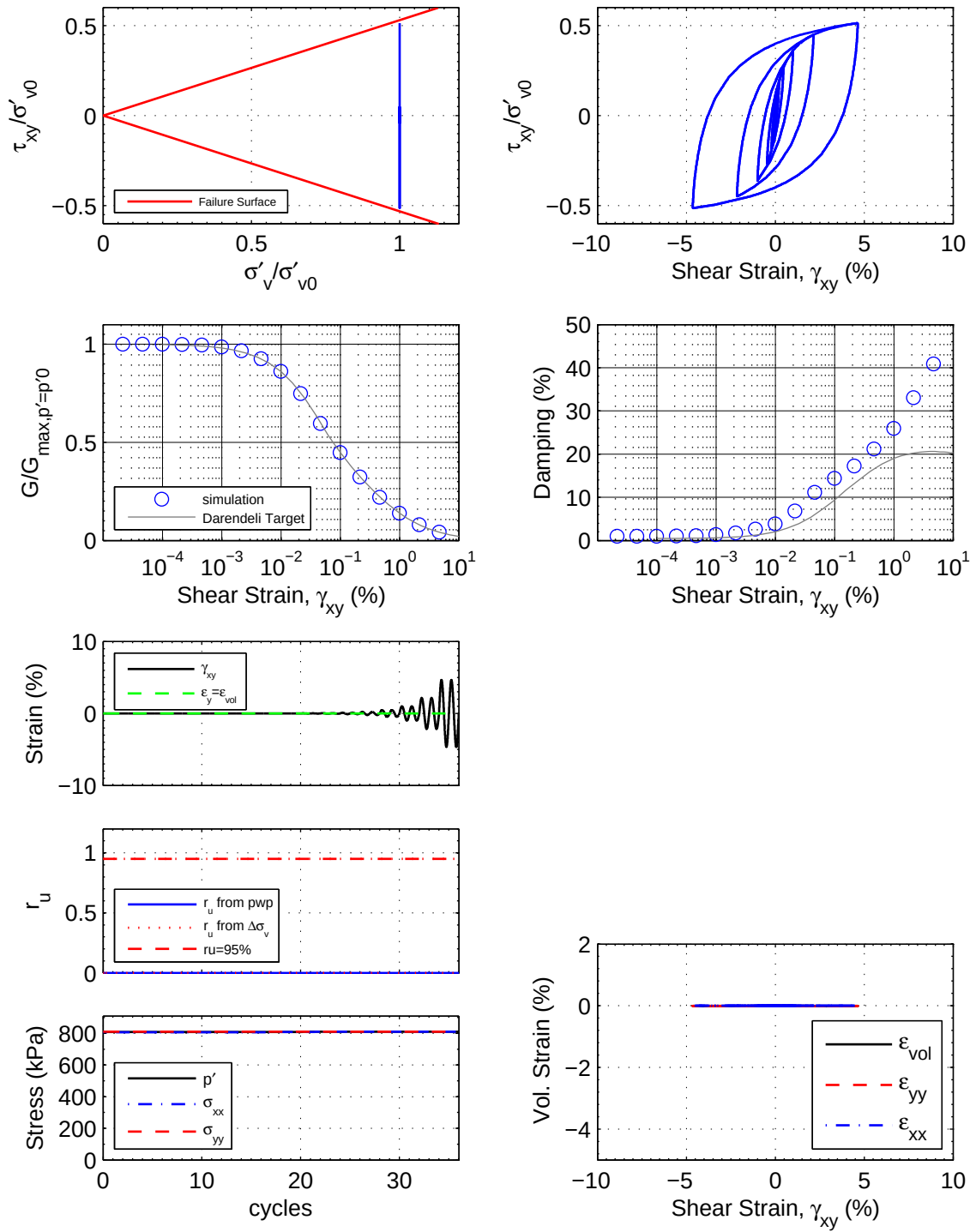


Figure C-10: Drained CSS Simulation: PIMY, $Dr=30$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=23.5$, $\phi_{PT}=na$

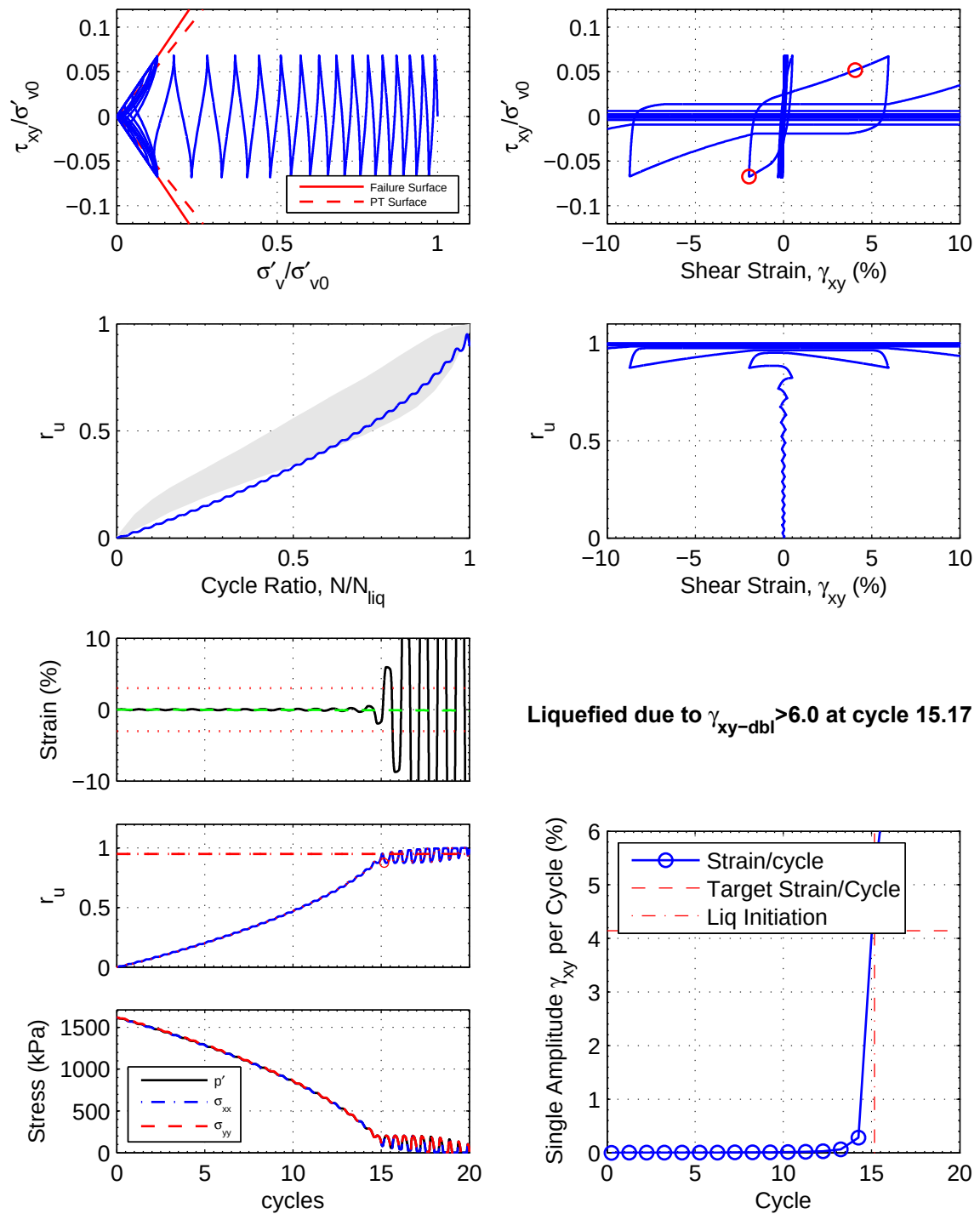


Figure C-11: Undrained CSS Simulation: PDMY2, $Dr=30$, $\sigma'_{v0}=16$ atm, $CSR=0.068$, $\phi=23.5$, $\phi_{PT}=20$

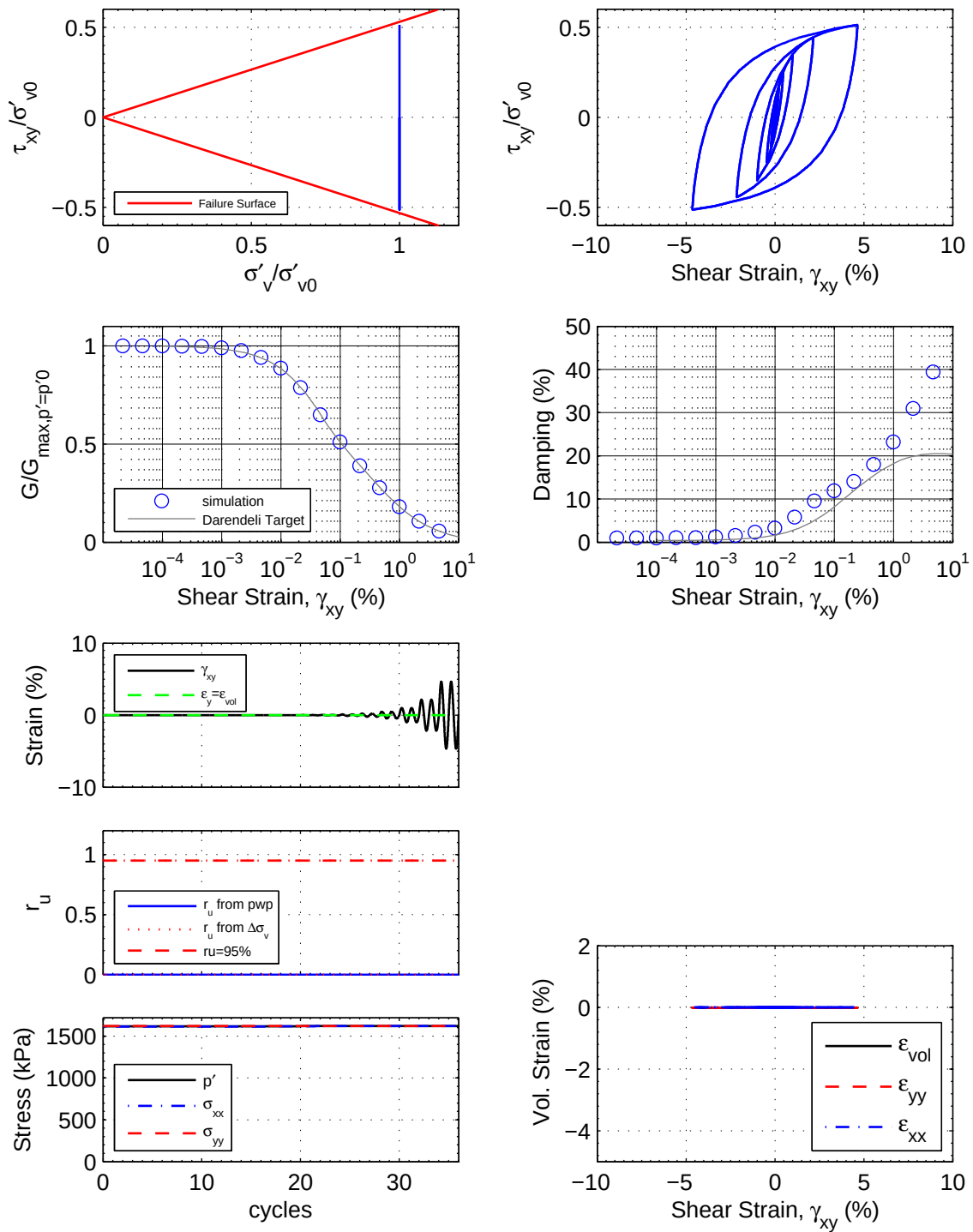


Figure C-12: Drained CSS Simulation: PIMY, $Dr=30, \sigma'_{v0}=16$ atm, CyclicStrain, $\phi=23.5, \phi_{PT}=na$

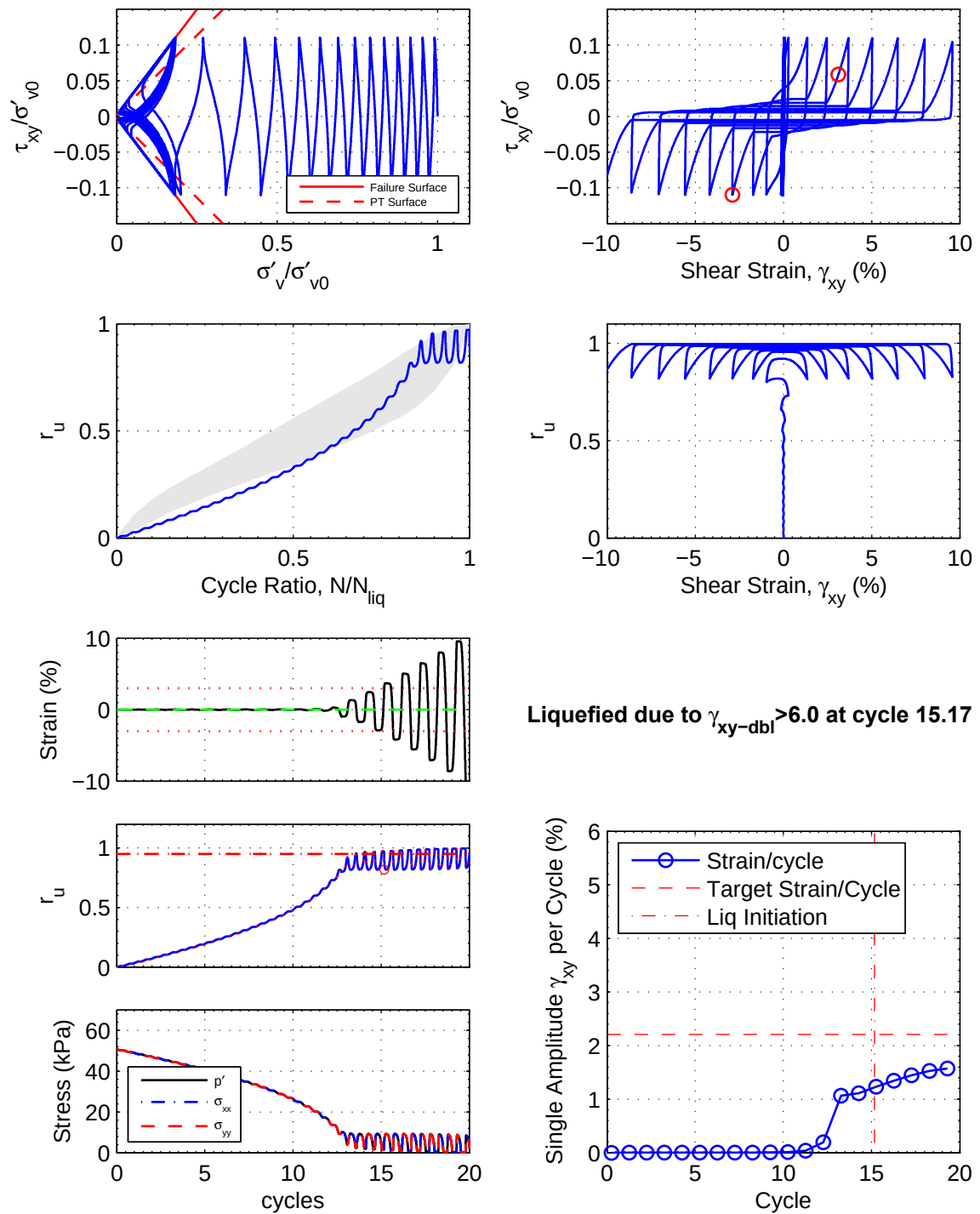


Figure C-13: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=0.5$ atm, $CSR=0.110$, $\phi=25.9$, $\phi_{PT}=20$

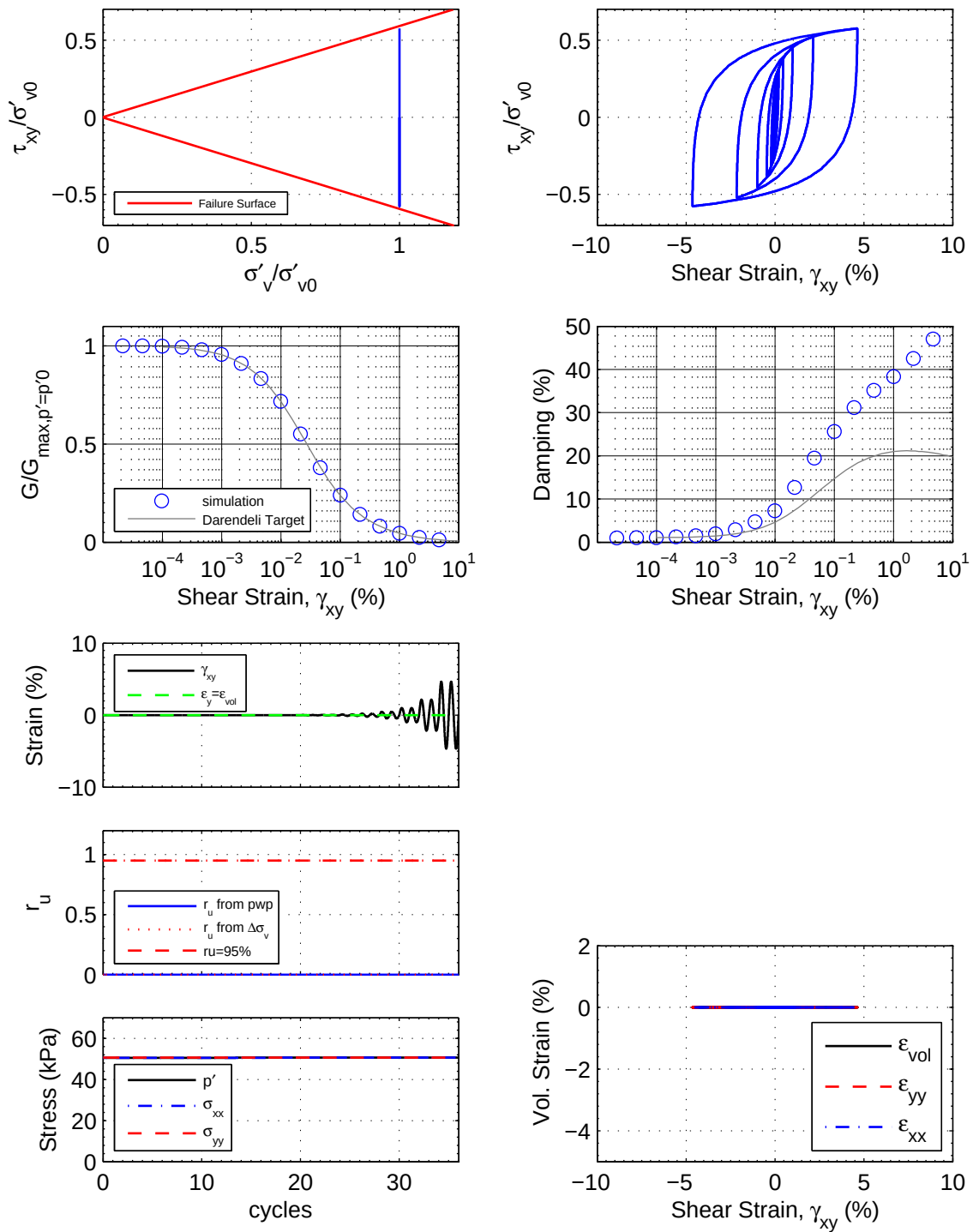


Figure C-14: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$

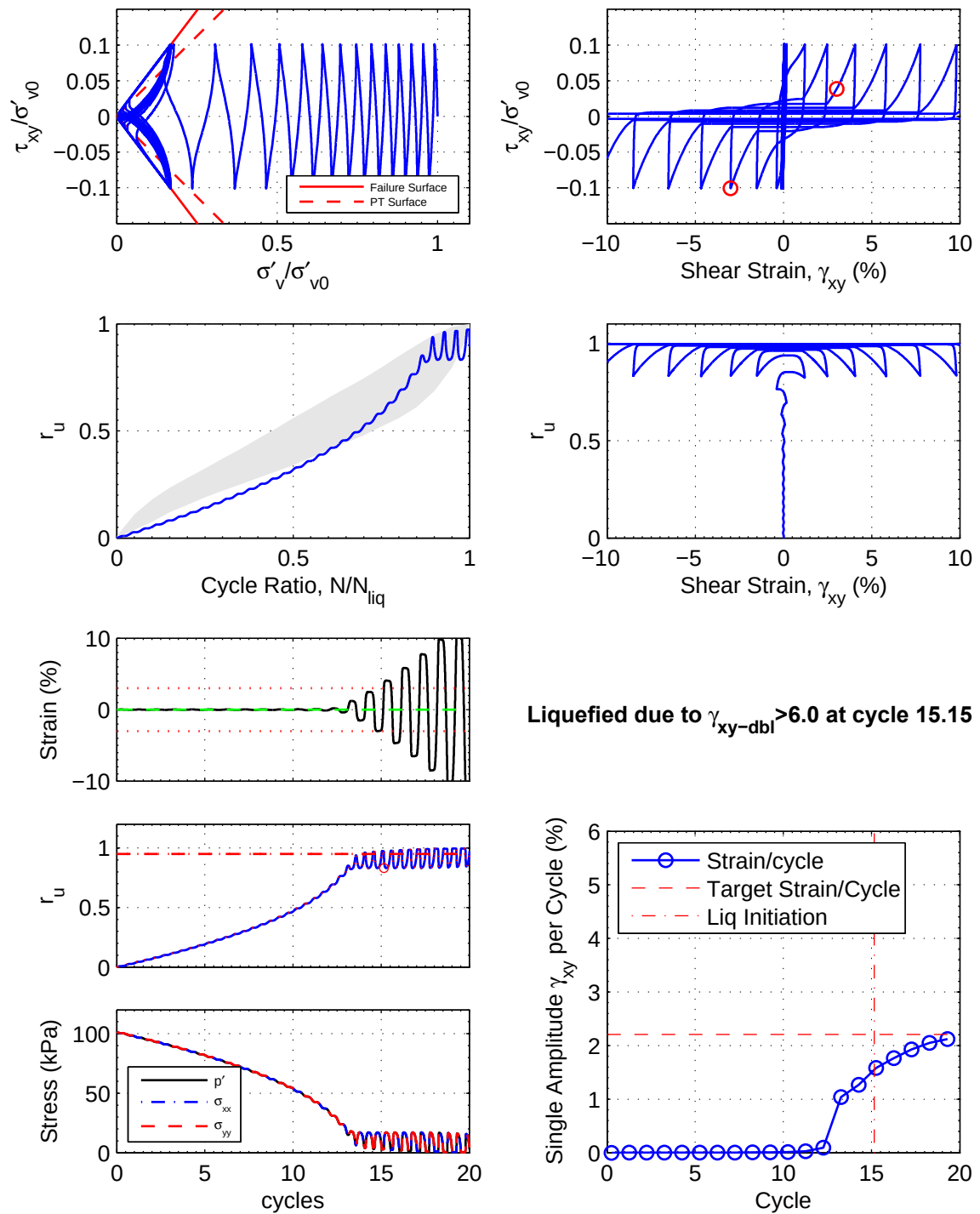


Figure C-15: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=1$ atm, $CSR=0.101$, $\phi=25.9$, $\phi_{PT}=20$

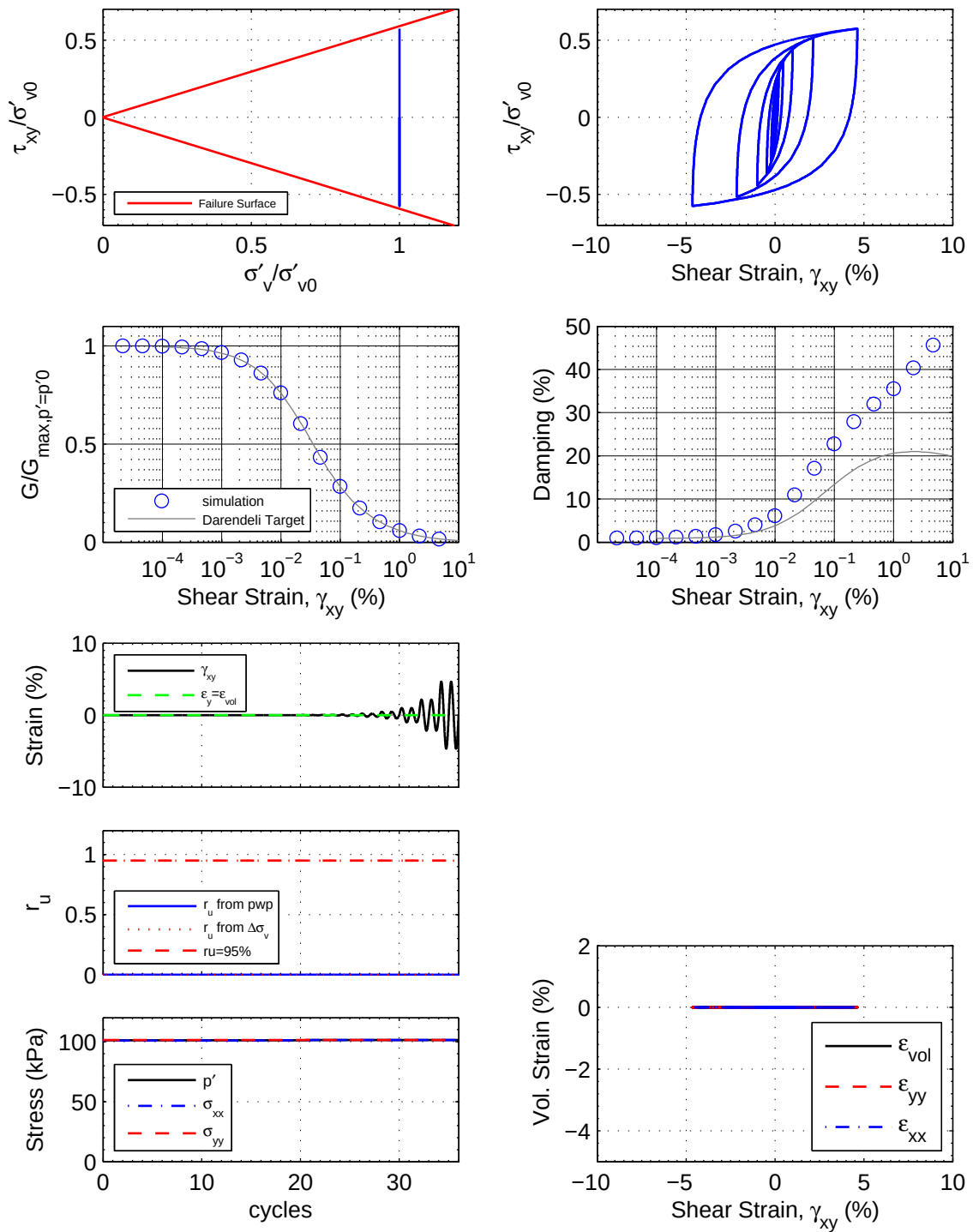


Figure C-16: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=1$ atm, $\phi=25.9$, $\phi_{PT}=na$

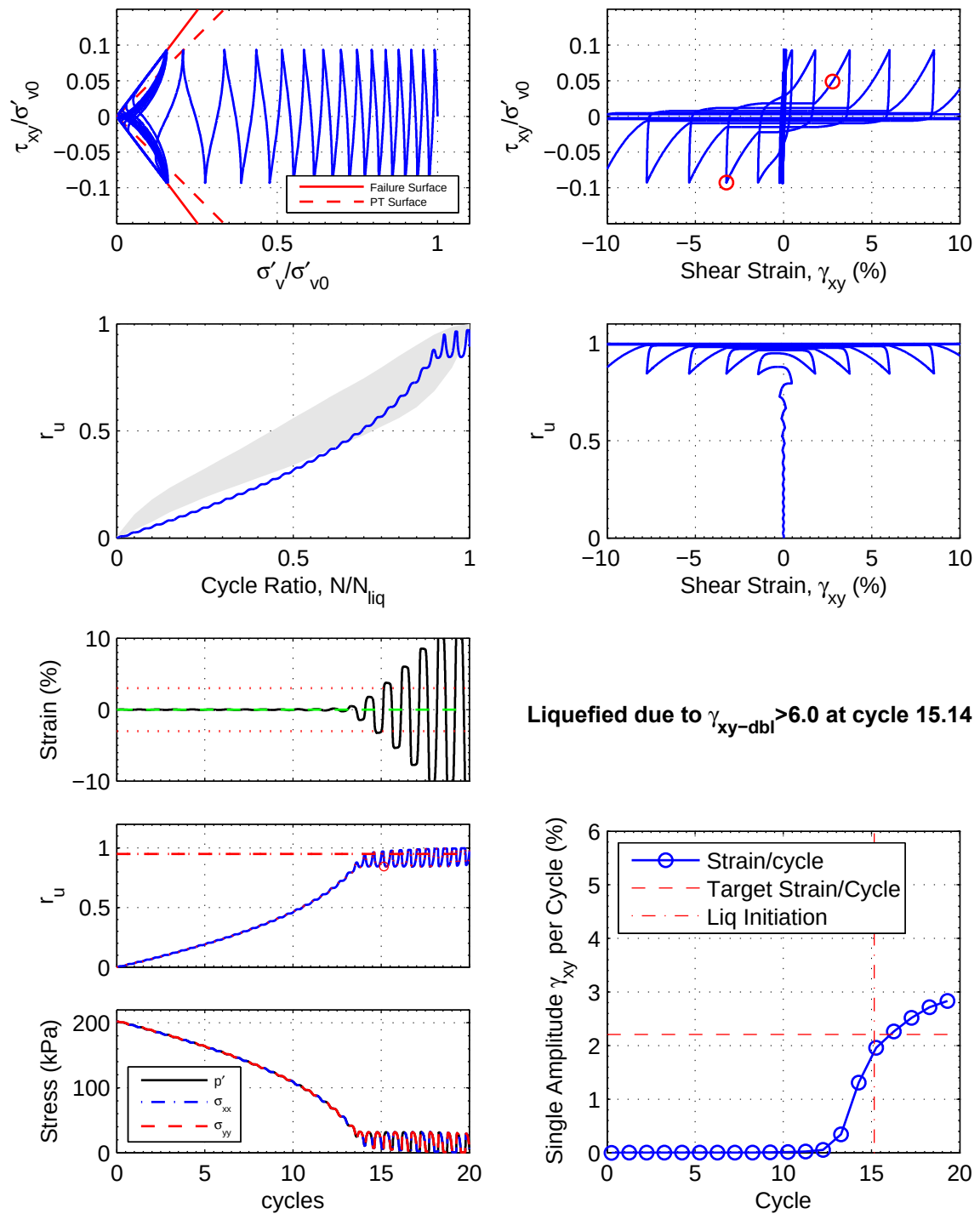


Figure C-17: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=2$ atm, $CSR=0.093$, $\phi=25.9$, $\phi_{PT}=20$

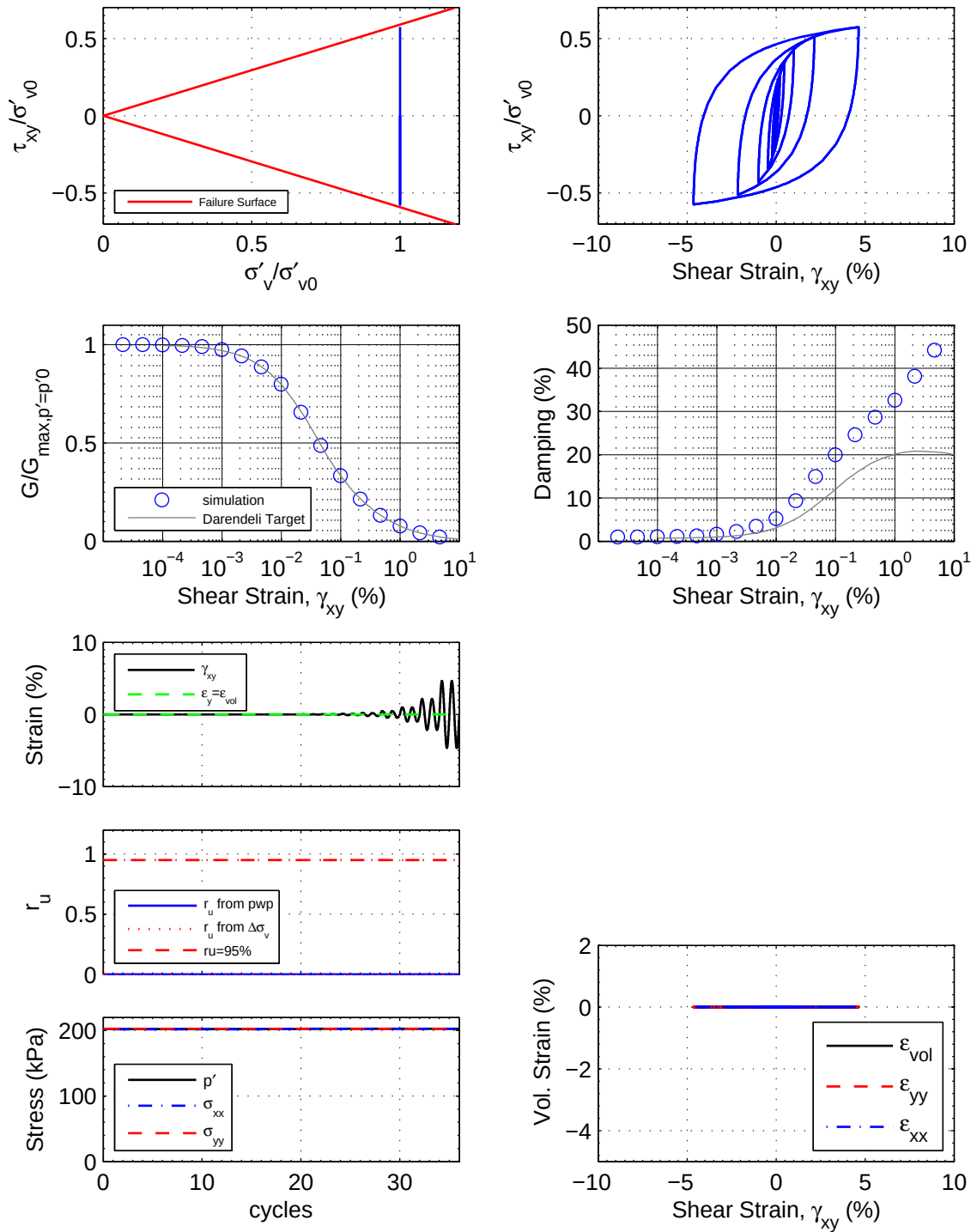


Figure C-18: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$

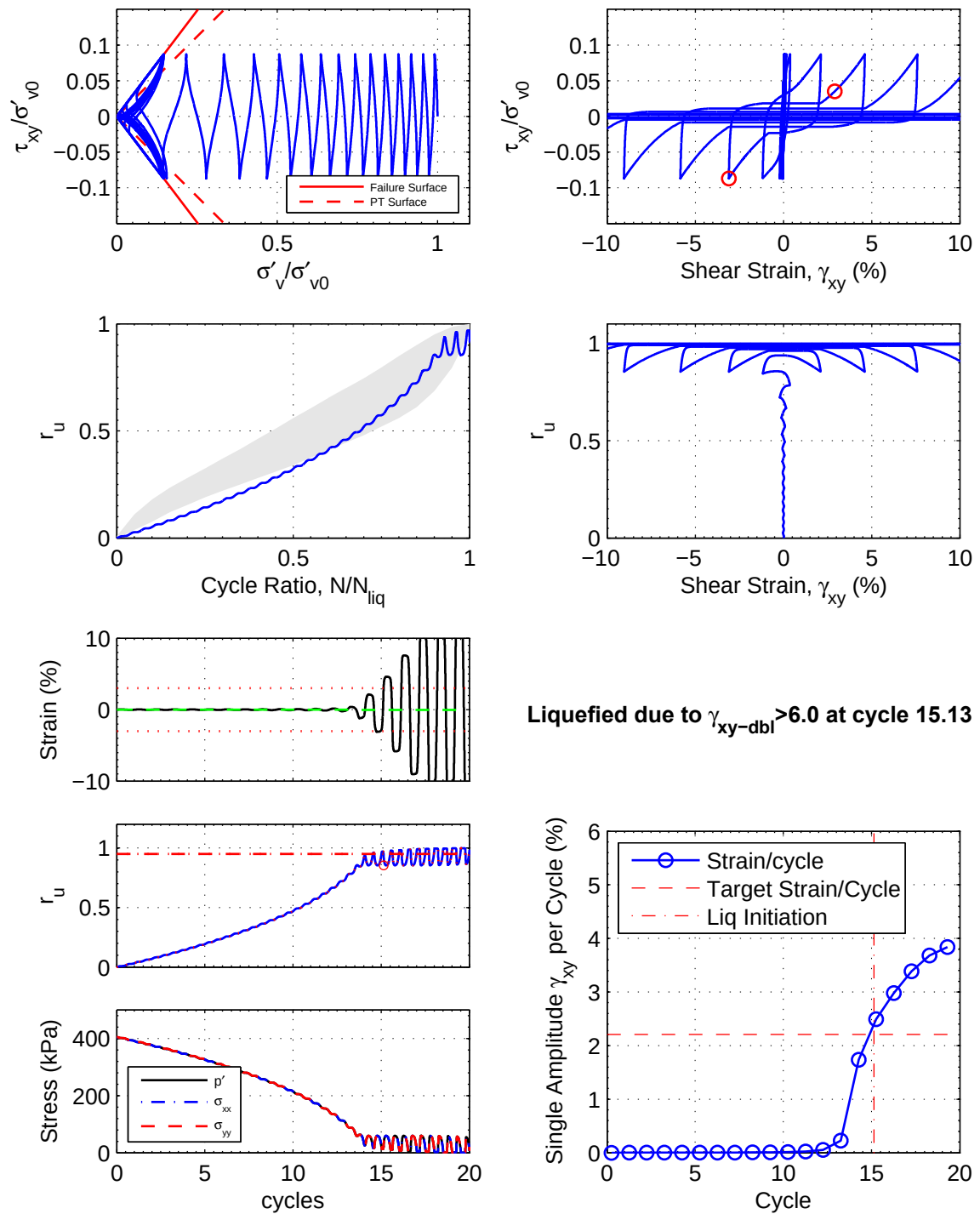


Figure C-19: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=4$ atm, $CSR=0.087$, $\phi=25.9$, $\phi_{PT}=20$

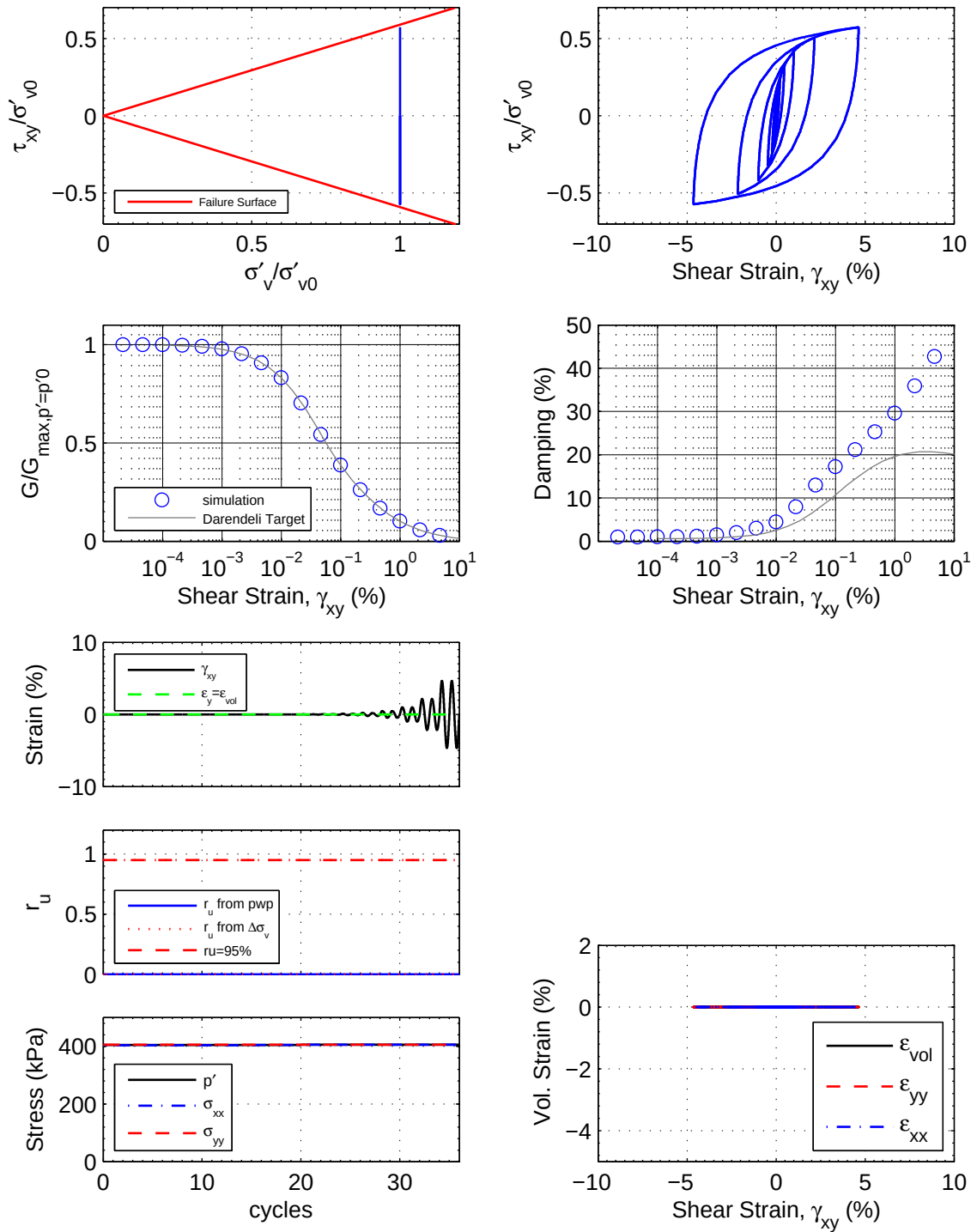


Figure C-20: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$

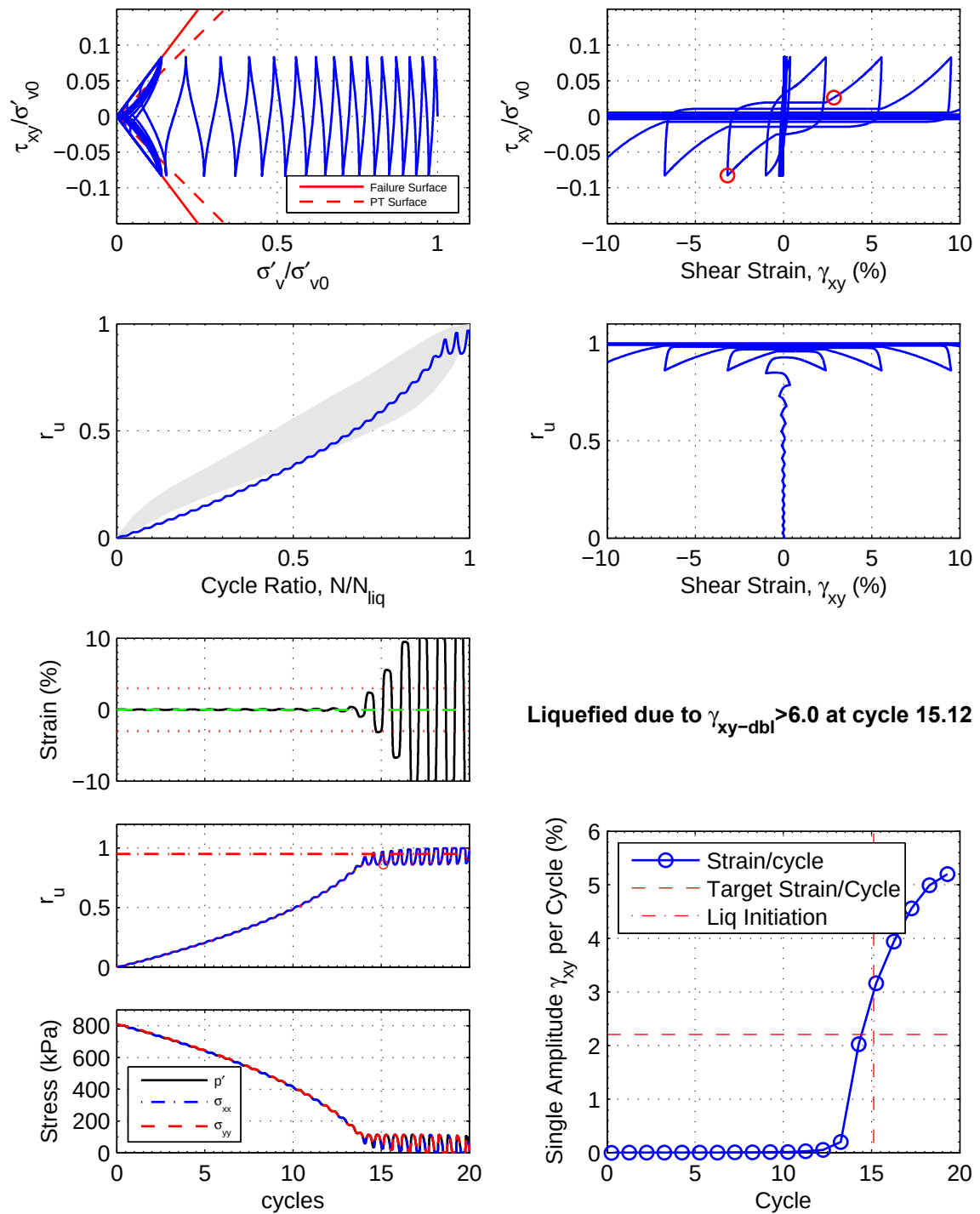


Figure C-21: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=8$ atm, $CSR=0.083$, $\phi=25.9$, $\phi_{PT}=20$

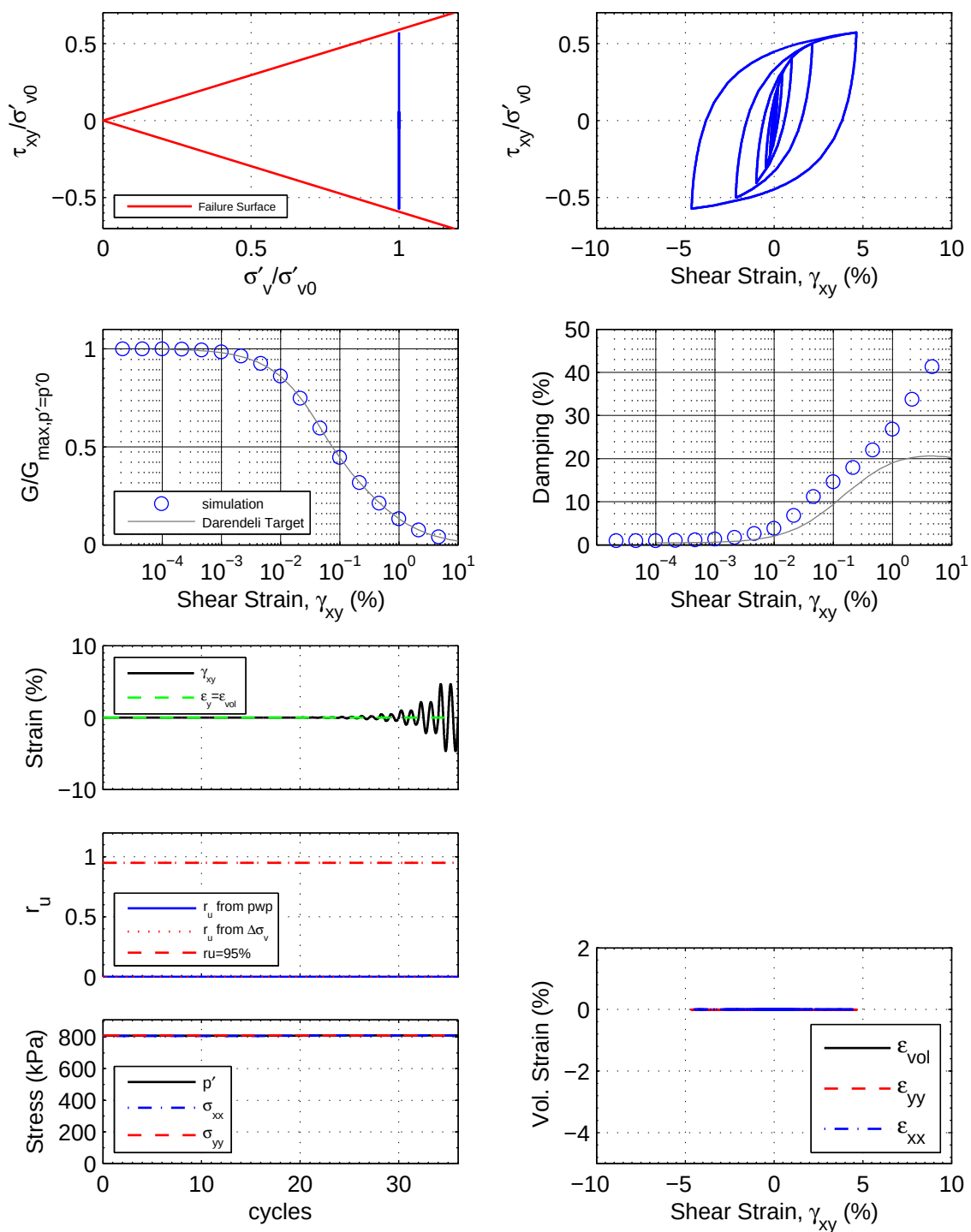


Figure C-22: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=8$ atm, $\phi=25.9$, $\phi_{PT}=na$

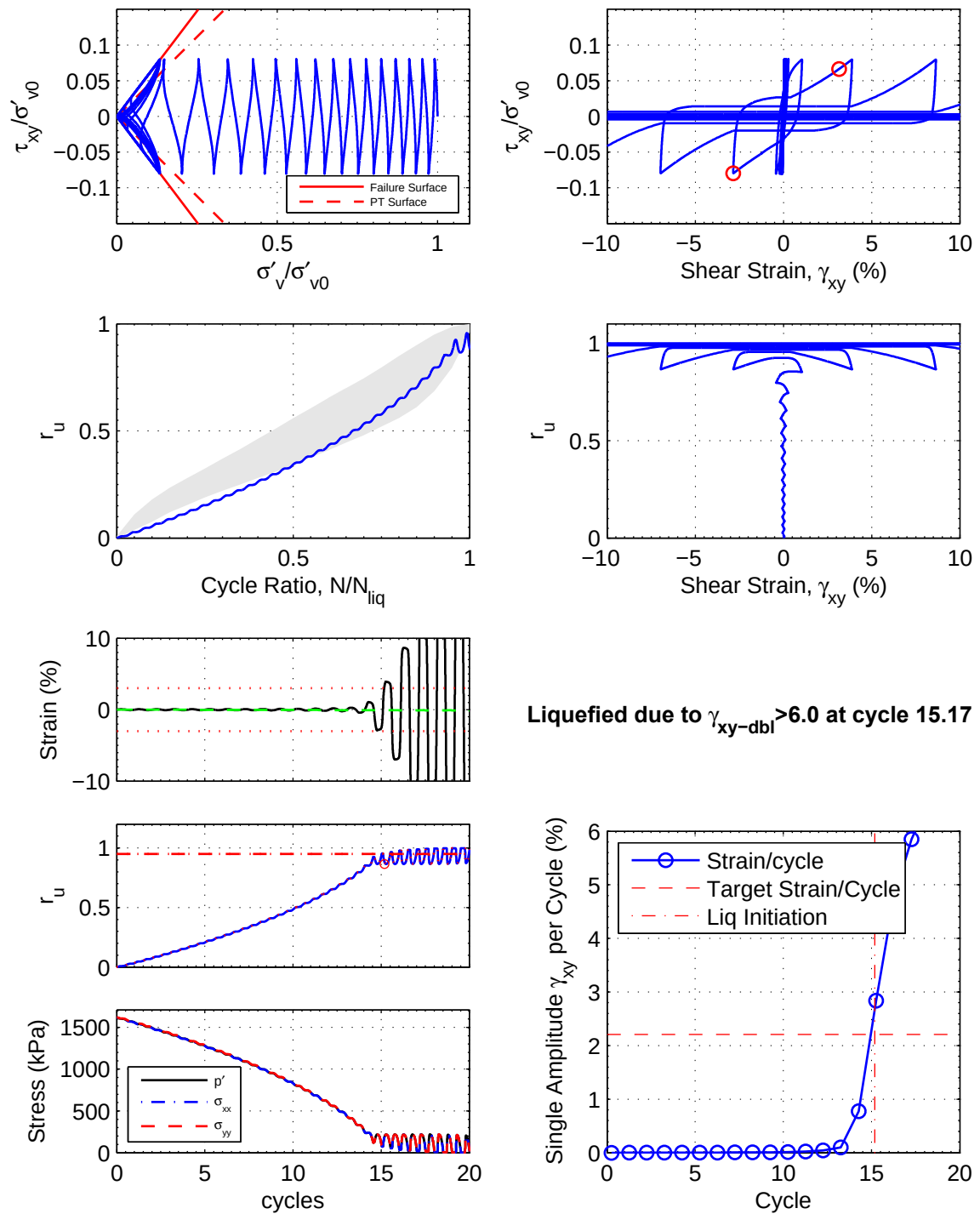


Figure C-23: Undrained CSS Simulation: PDMY2, $Dr=40$, $\sigma'_{v0}=16$ atm, $CSR=0.080$, $\phi=25.9$, $\phi_{PT}=20$

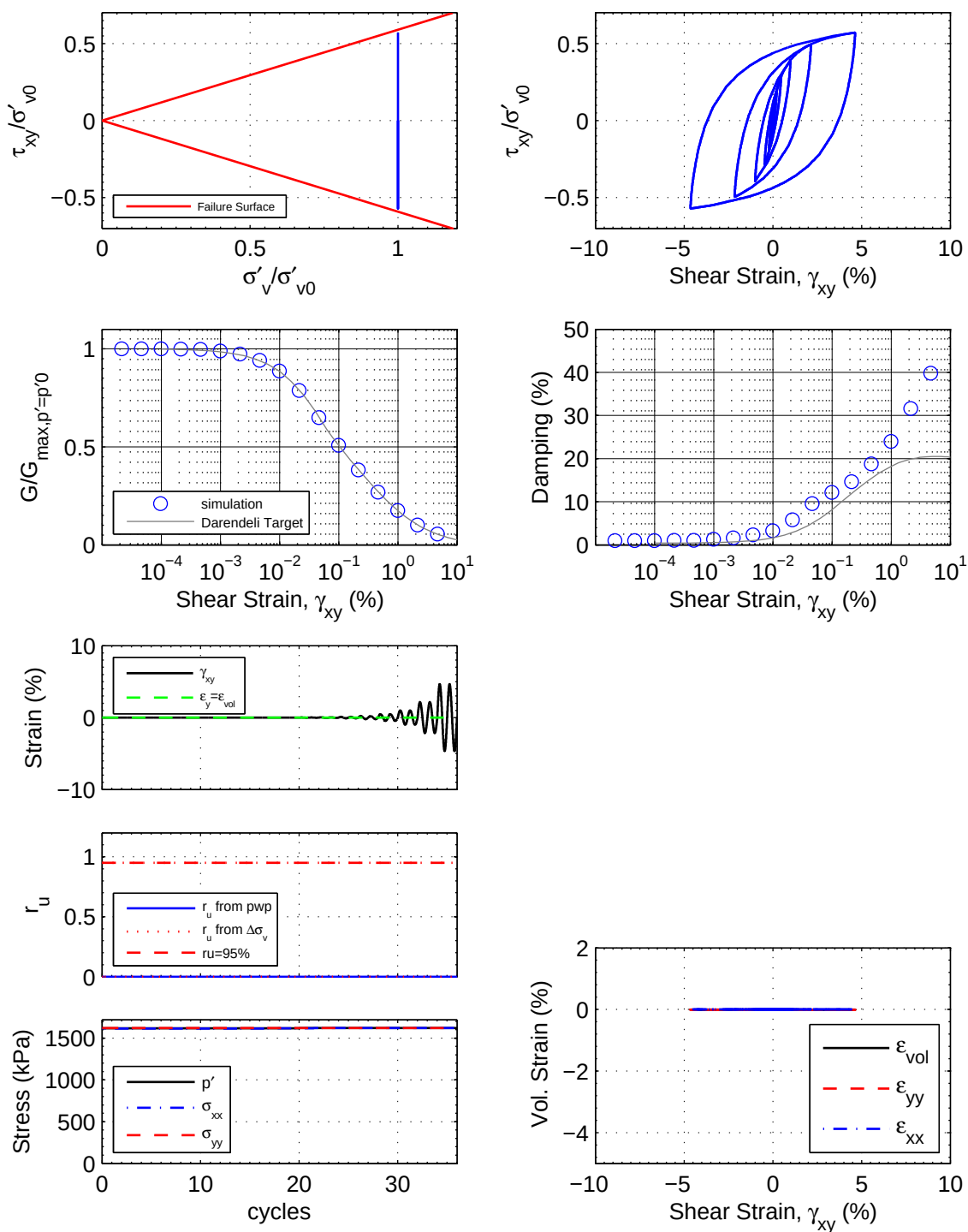


Figure C-24: Drained CSS Simulation: PIMY, $Dr=40$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=25.9$, $\phi_{PT}=na$

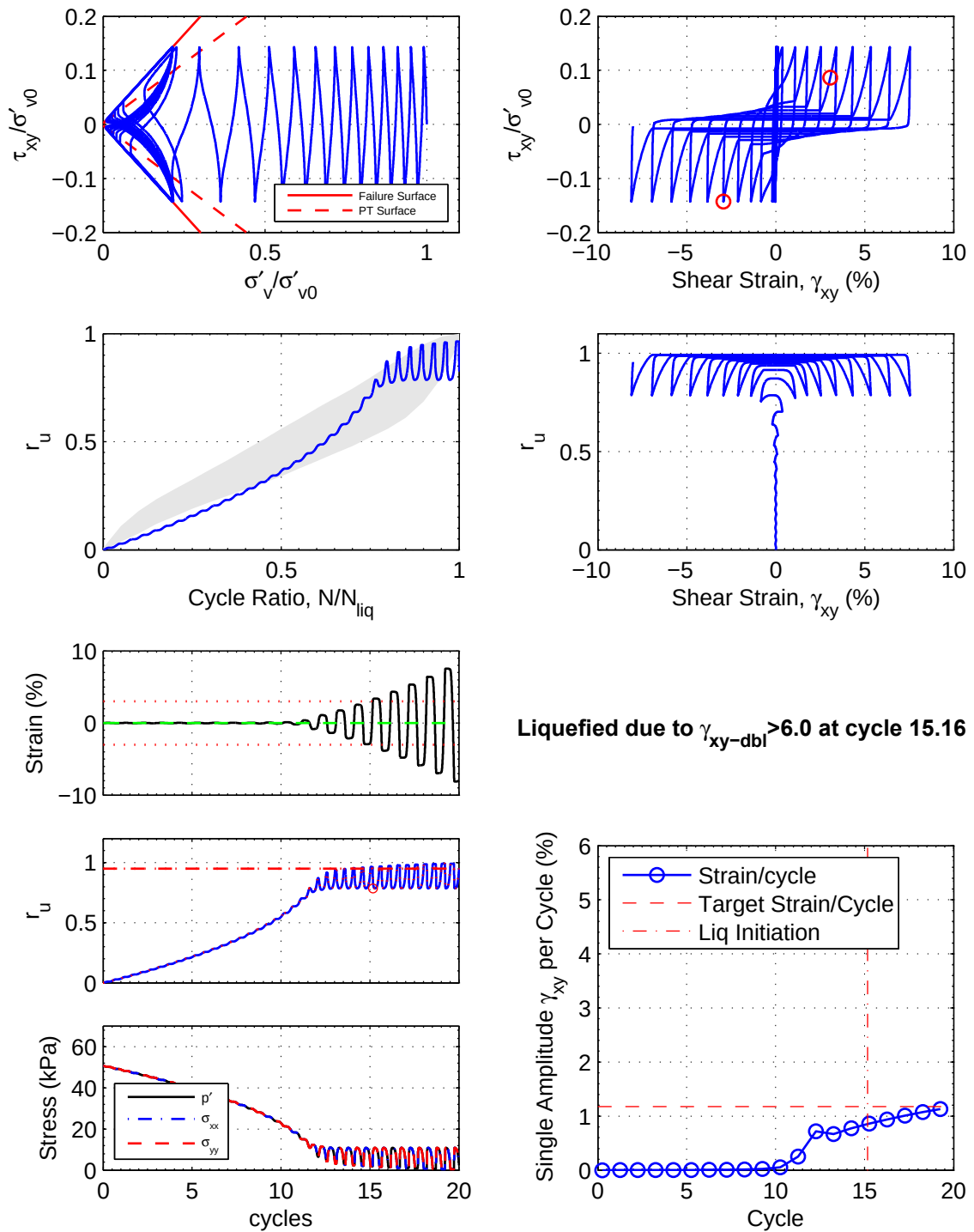


Figure C-25: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=0.5$ atm, $CSR=0.143$, $\phi=28.6$, $\phi_{PT}=20$

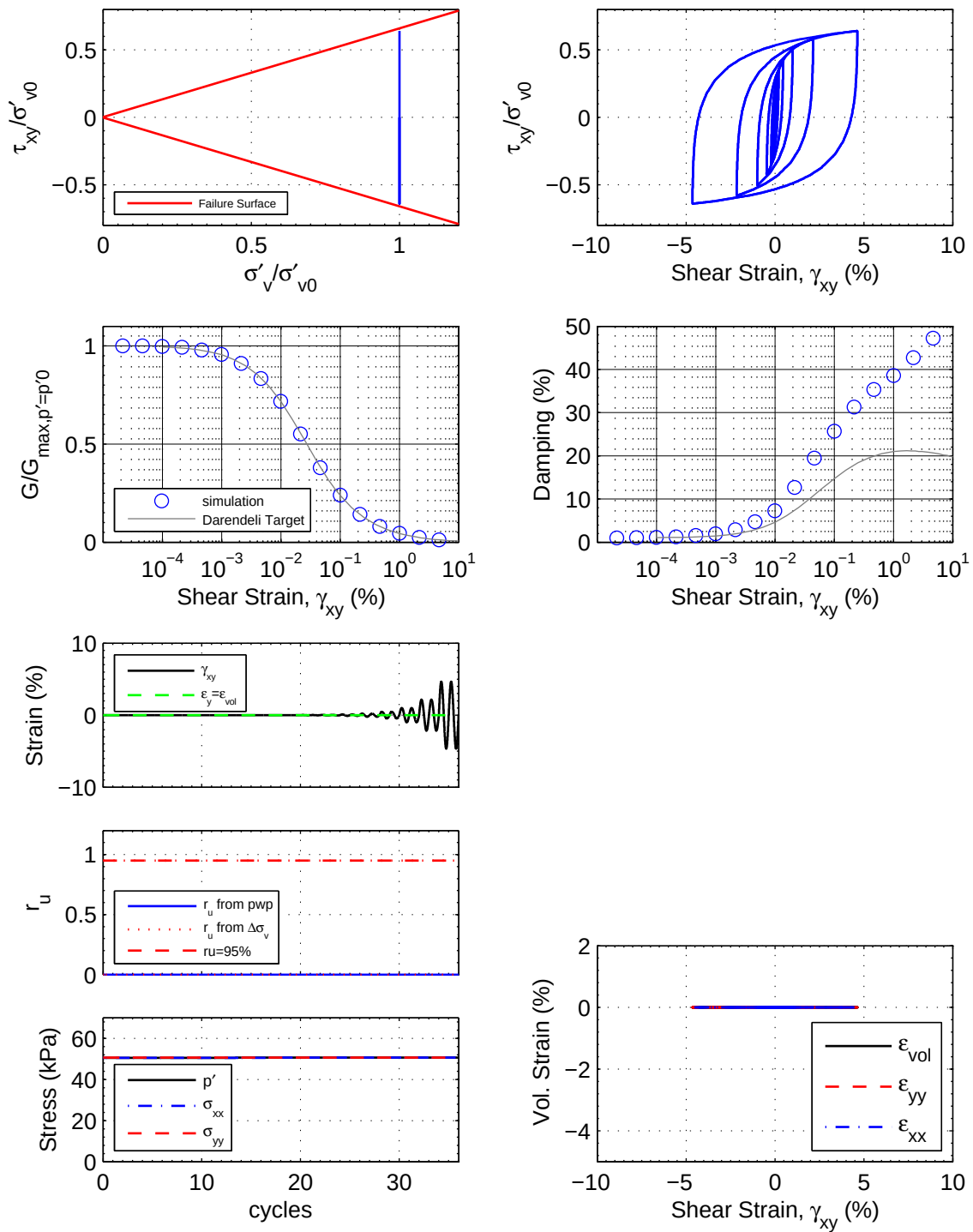


Figure C-26: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$

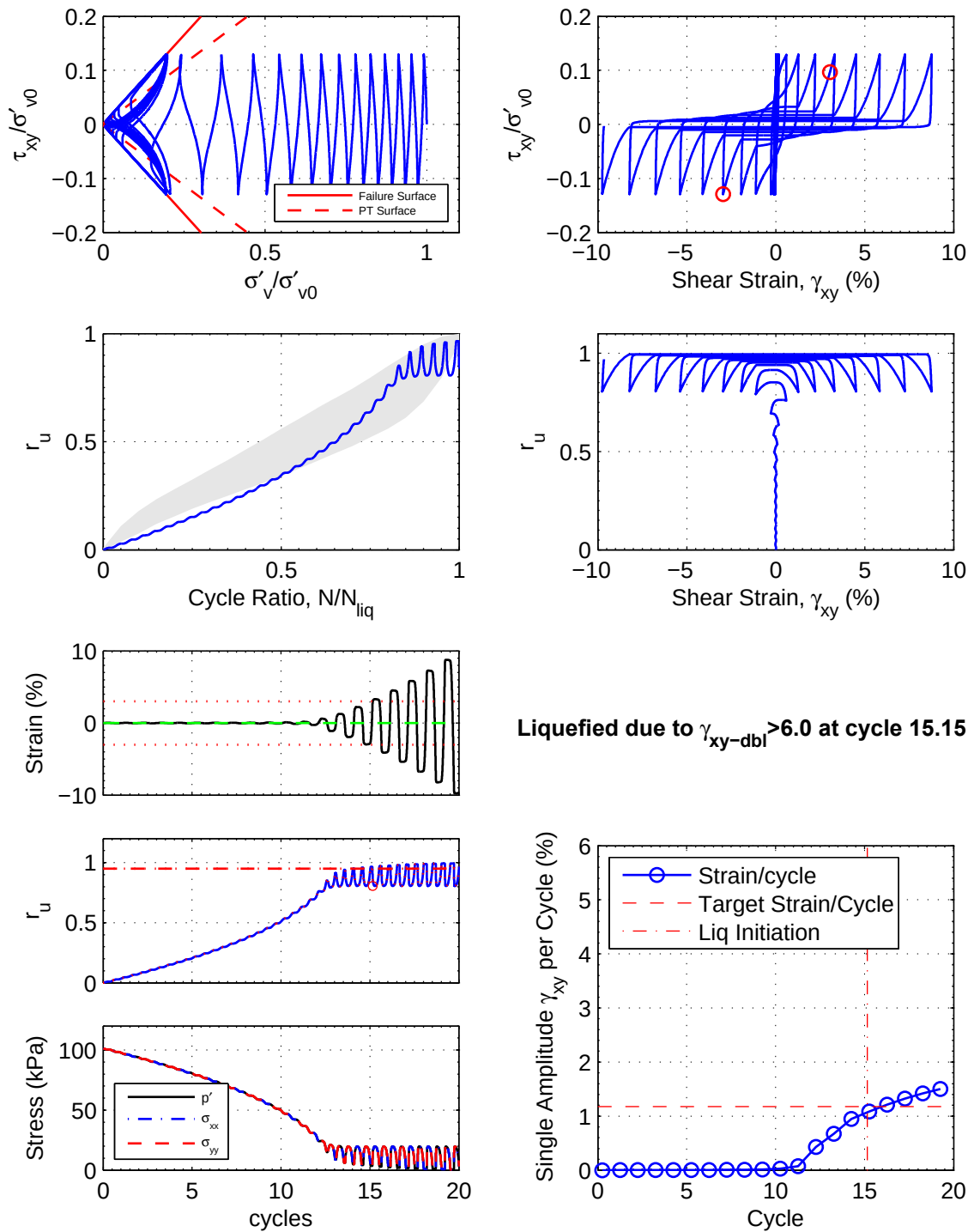


Figure C-27: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=1$ atm, $CSR=0.129$, $\phi=28.6$, $\phi_{PT}=20$

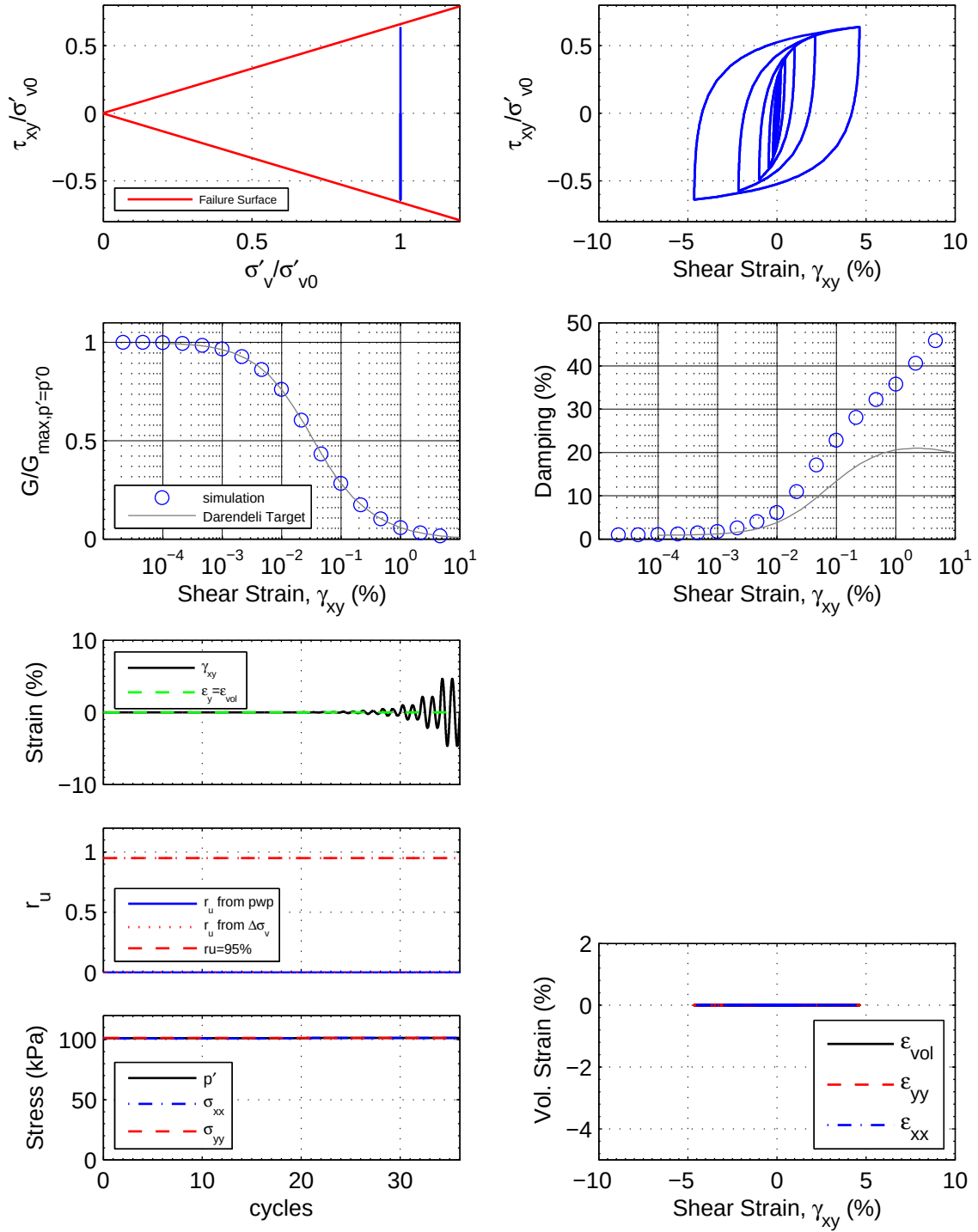


Figure C-28: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$

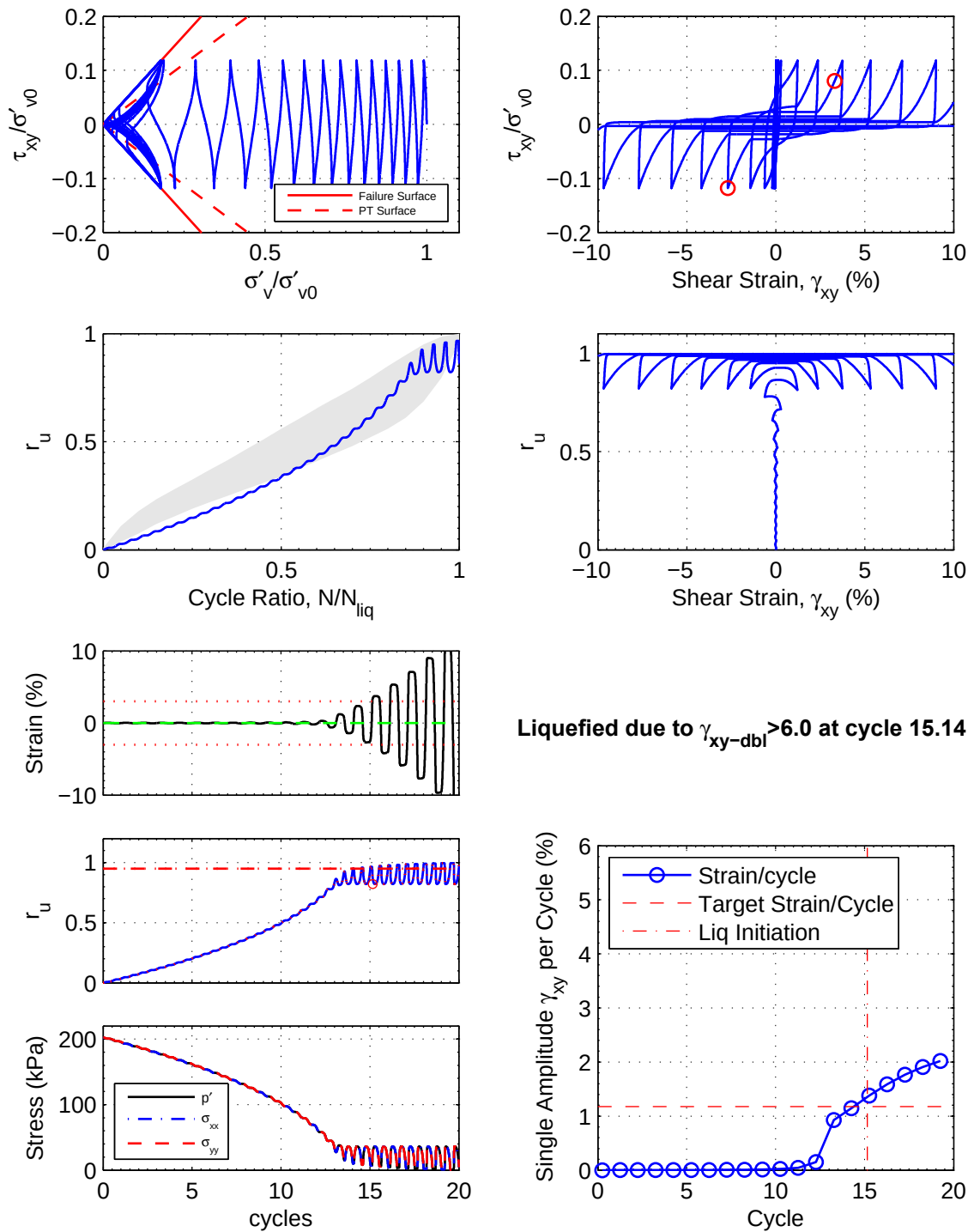


Figure C-29: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=2$ atm, $CSR=0.118$, $\phi=28.6$, $\phi_{PT}=20$

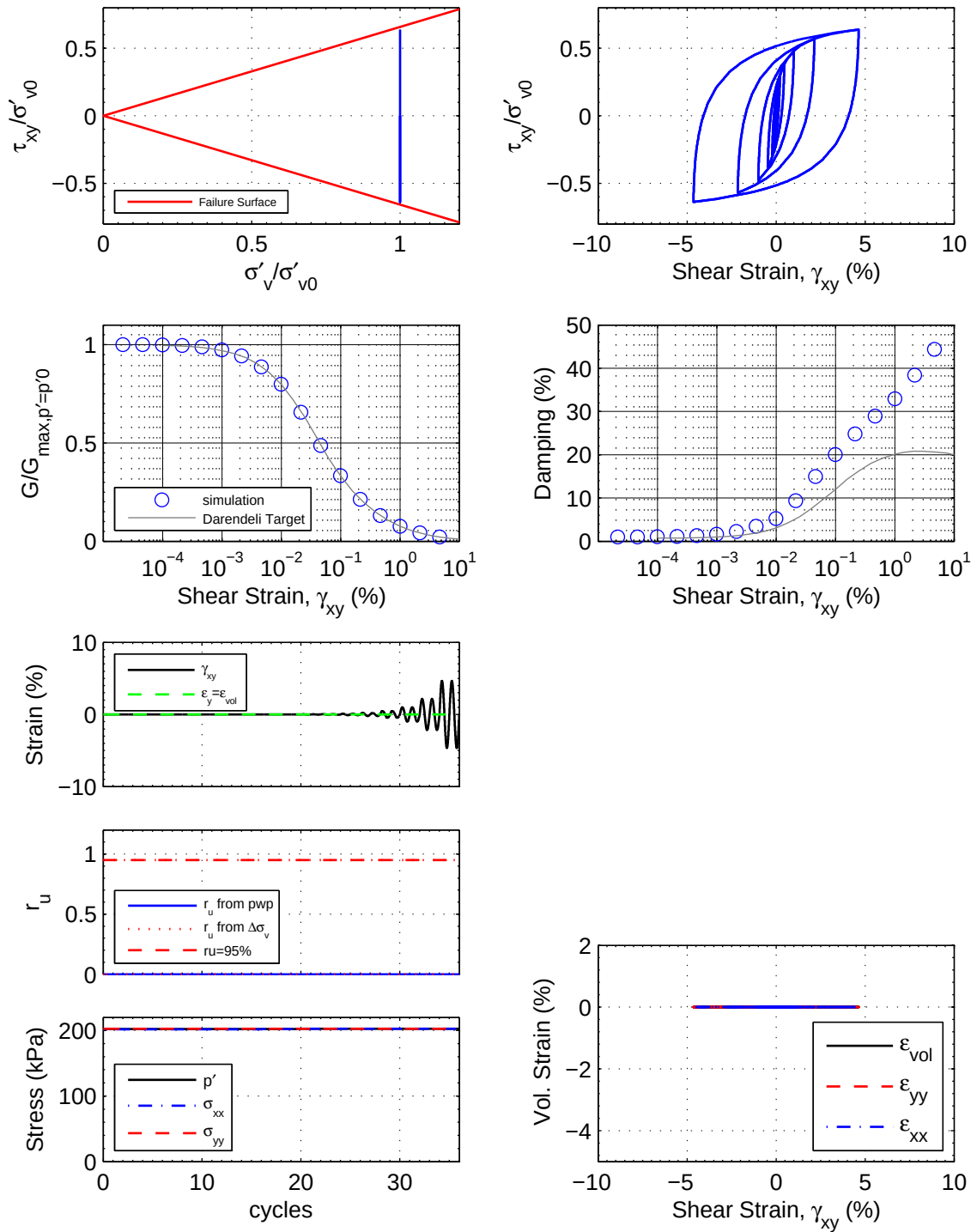


Figure C-30: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$

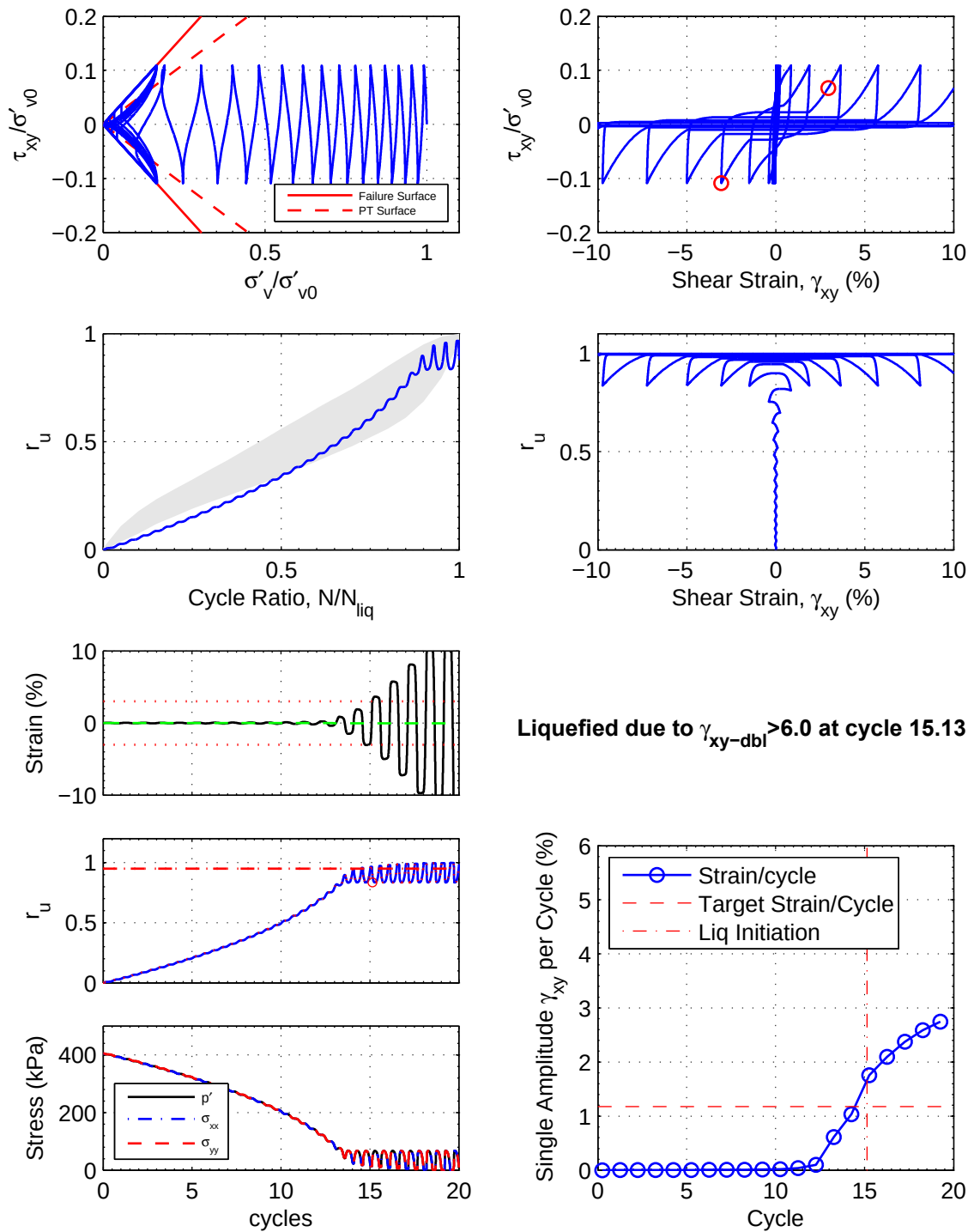


Figure C-31: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=4$ atm, $CSR=0.109$, $\phi=28.6$, $\phi_{PT}=20$

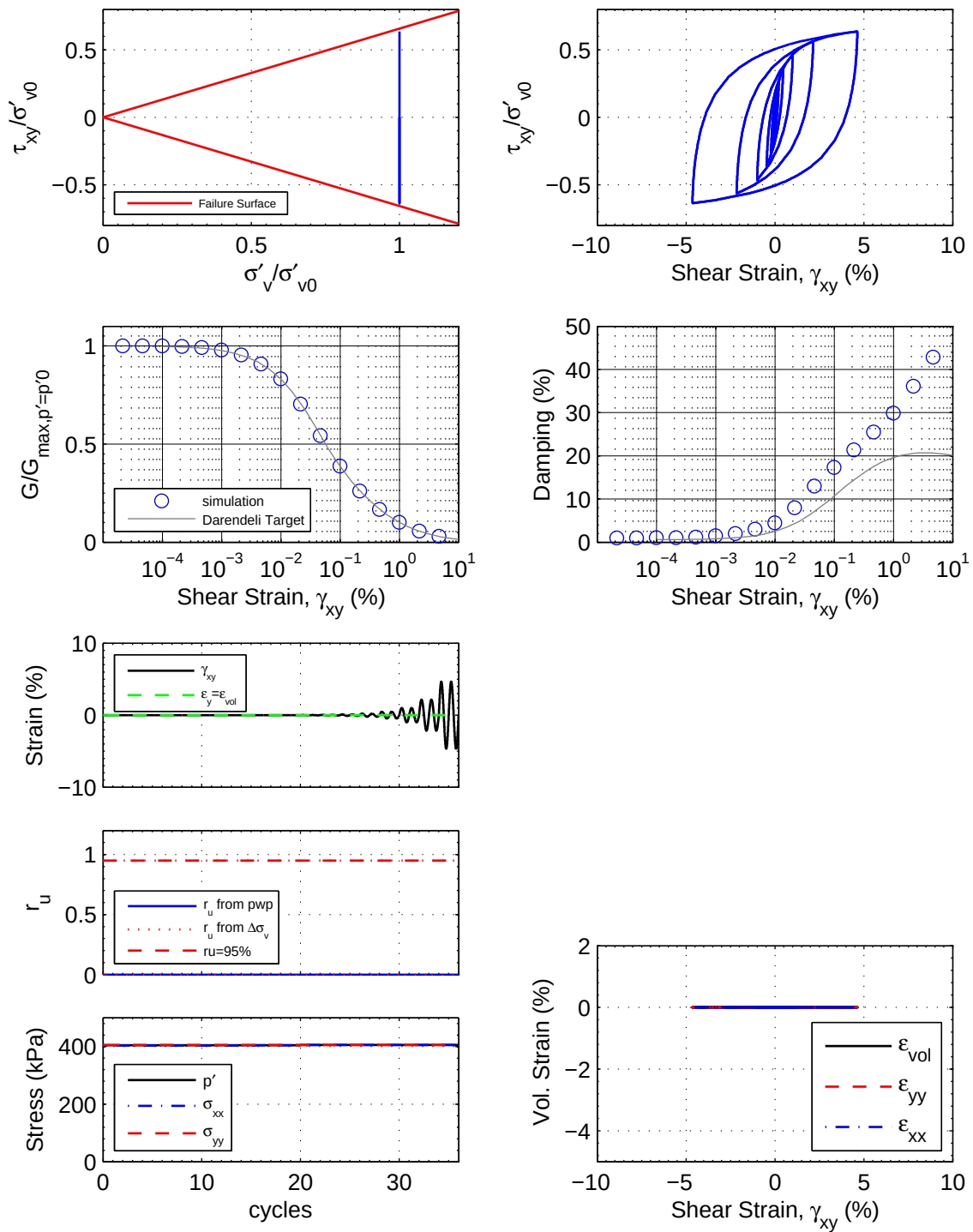


Figure C-32: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=28.6$, $\phi_{PT}=na$

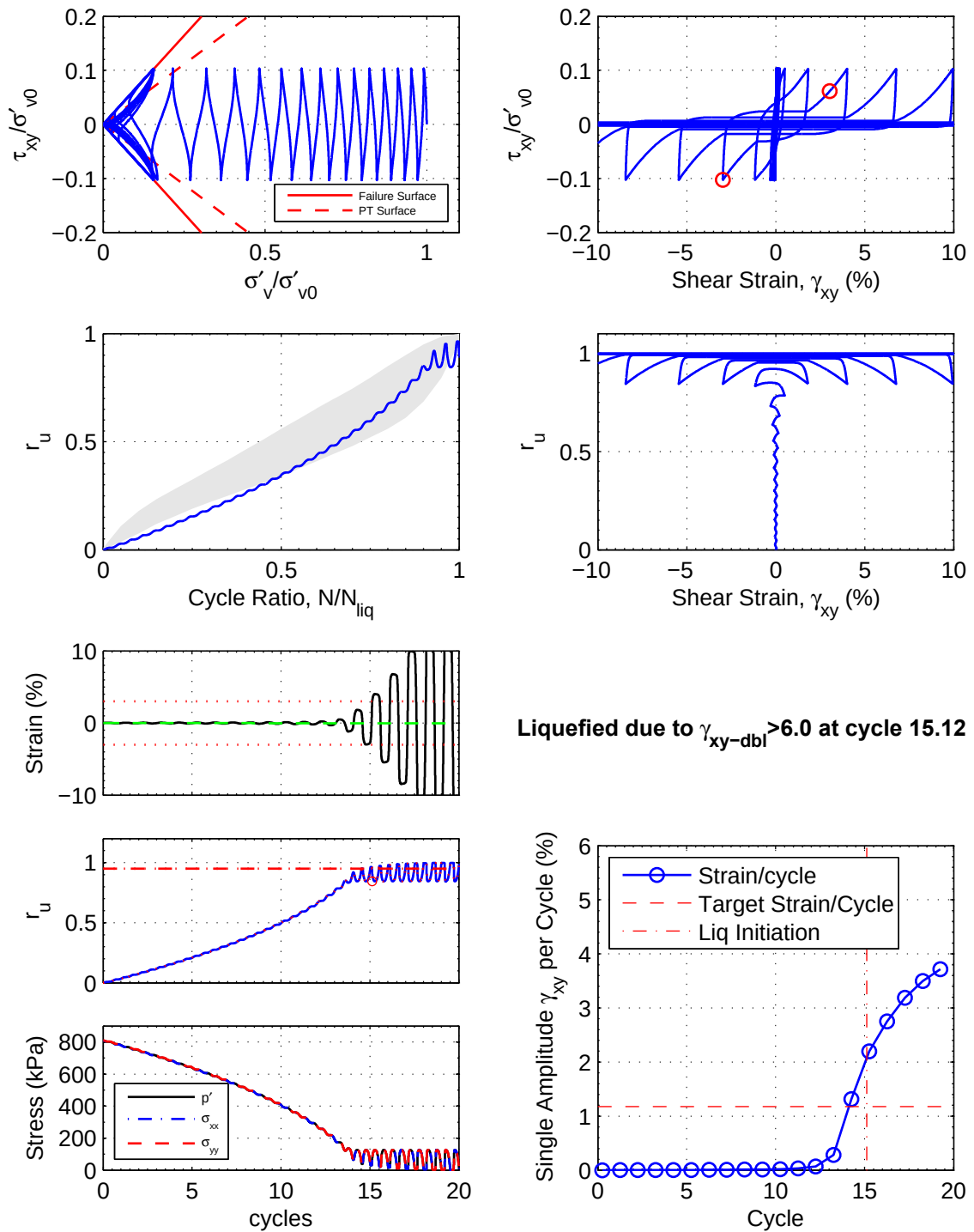


Figure C-33: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=8$ atm, $CSR=0.103$, $\phi=28.6$, $\phi_{PT}=20$

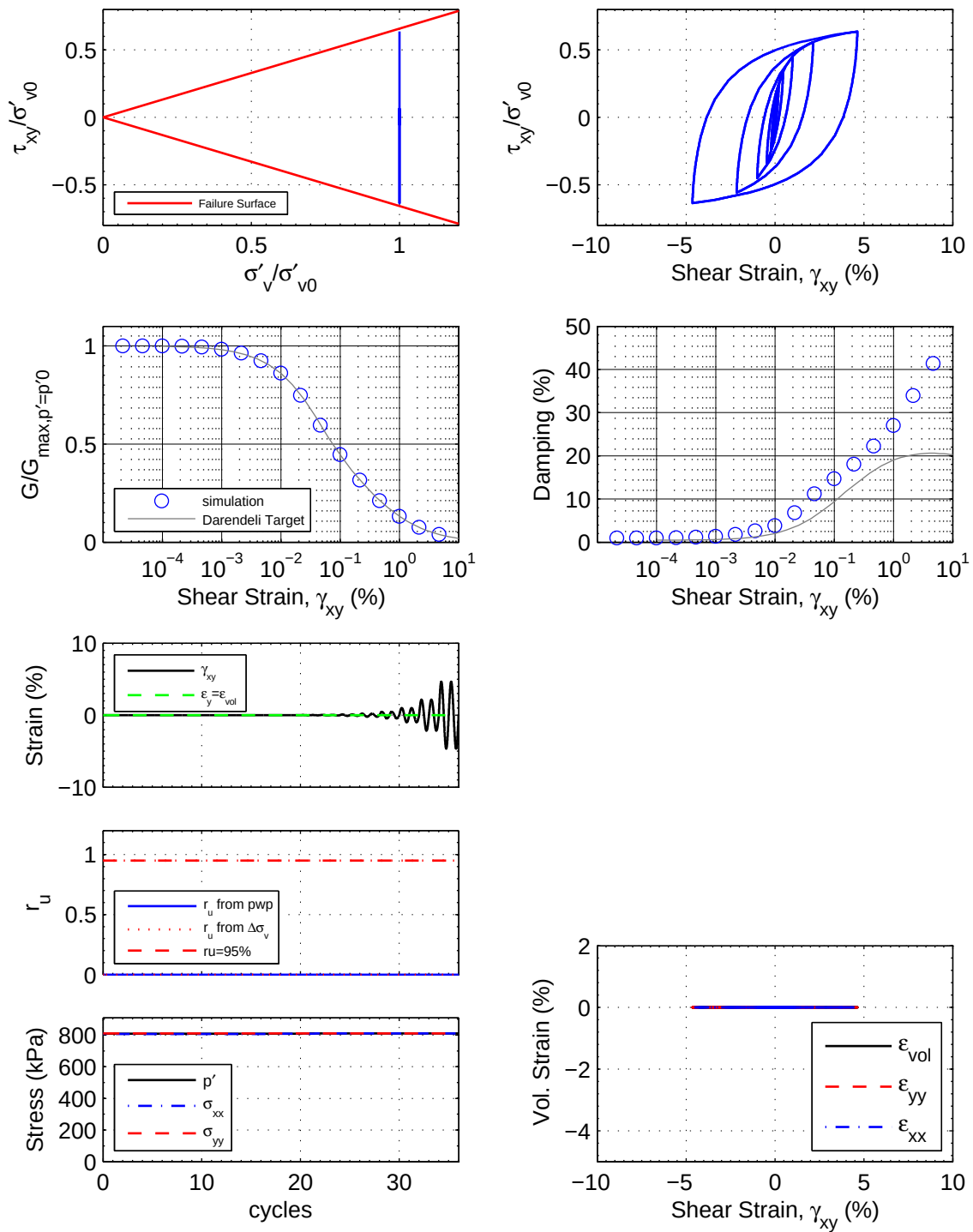


Figure C-34: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=8$ atm, $\phi=28.6$, $\phi_{PT}=na$

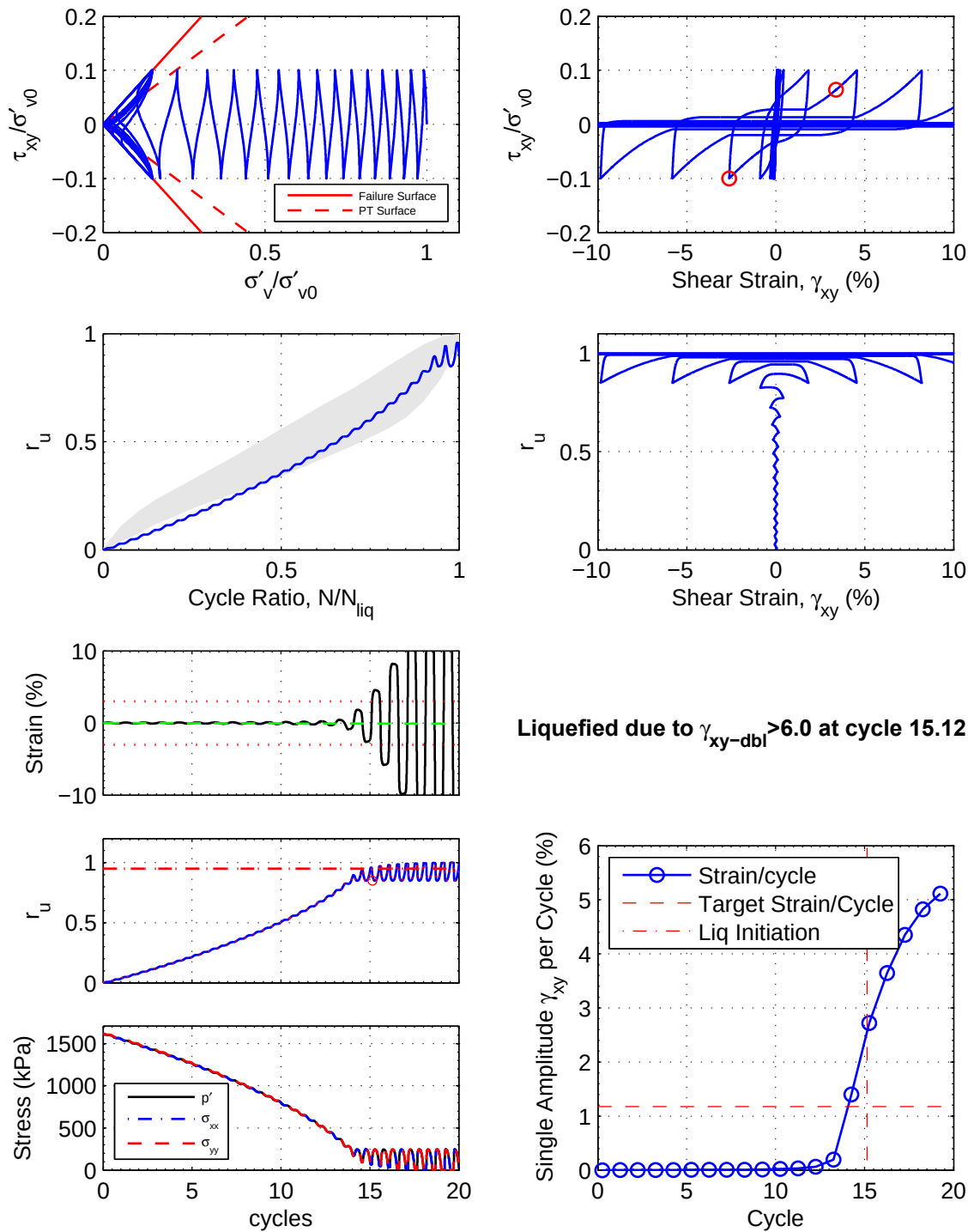


Figure C-35: Undrained CSS Simulation: PDMY2, $Dr=50$, $\sigma'_{v0}=16$ atm, $CSR=0.100$, $\phi=28.6$, $\phi_{PT}=20$

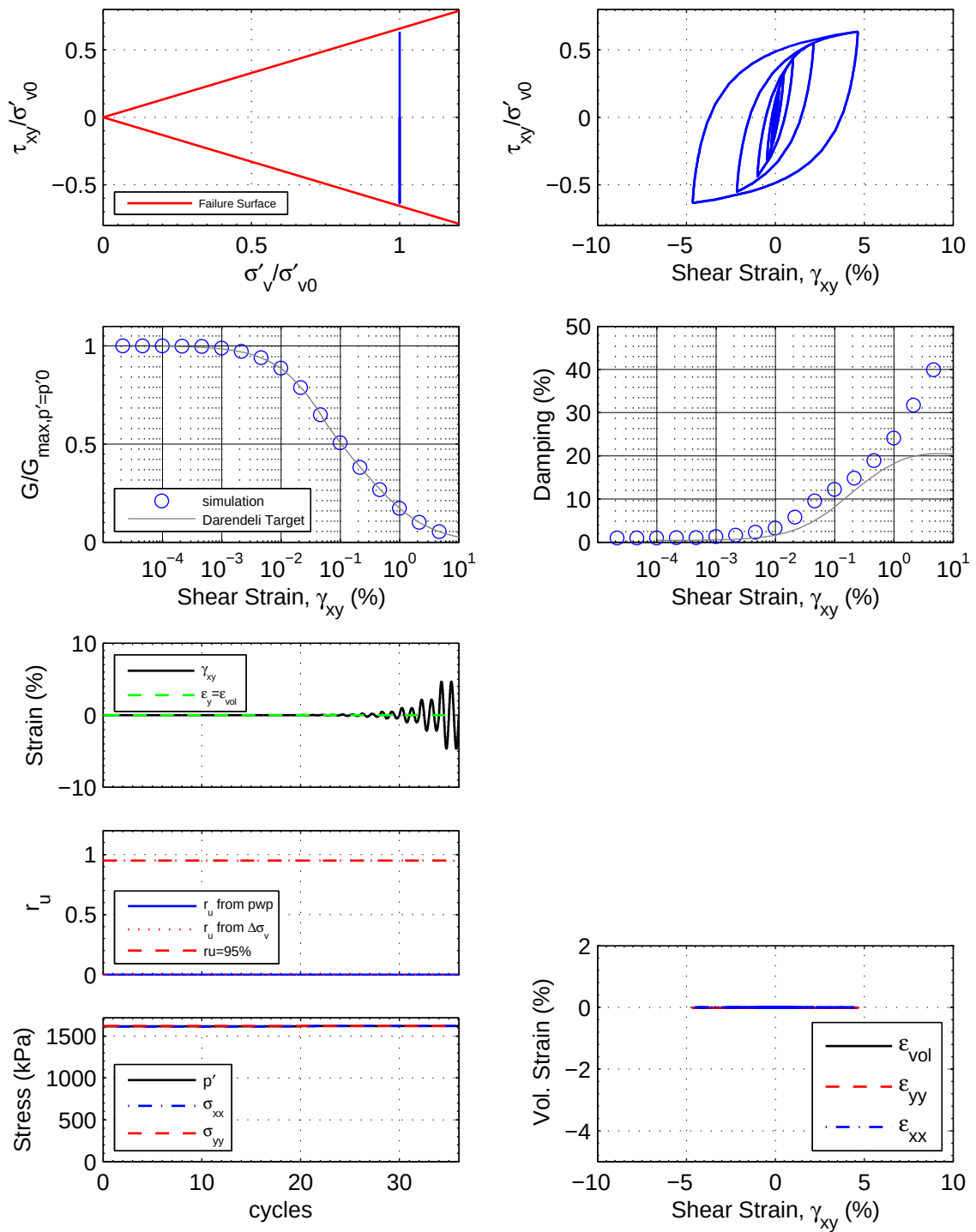


Figure C-36: Drained CSS Simulation: PIMY, $Dr=50$, $\sigma'_{v0}=16$ atm, $\phi=28.6$, $\phi_{PT}=na$

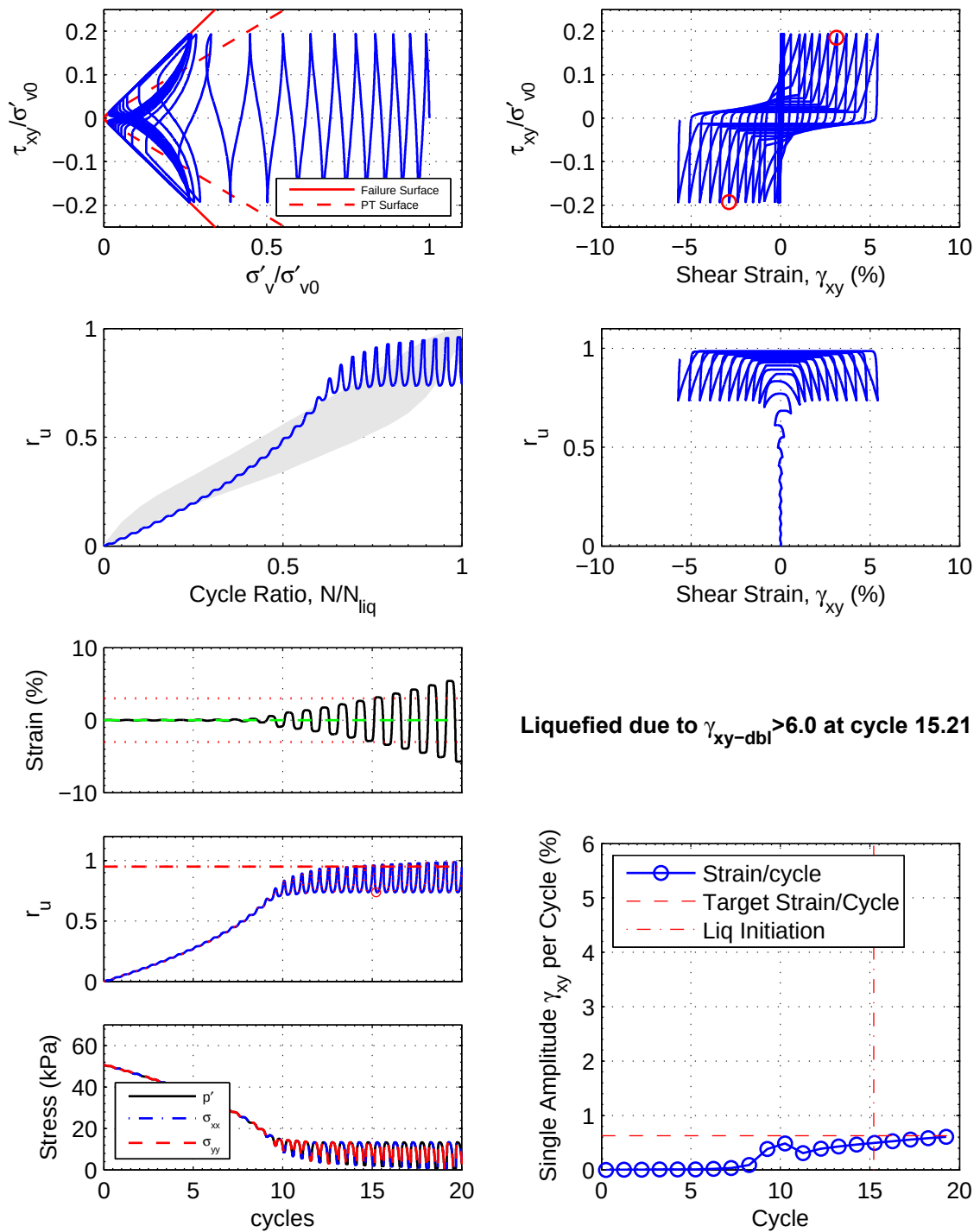


Figure C-37: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=0.5$ atm, $CSR=0.193$, $\phi=31.3$, $\phi_{PT}=20$

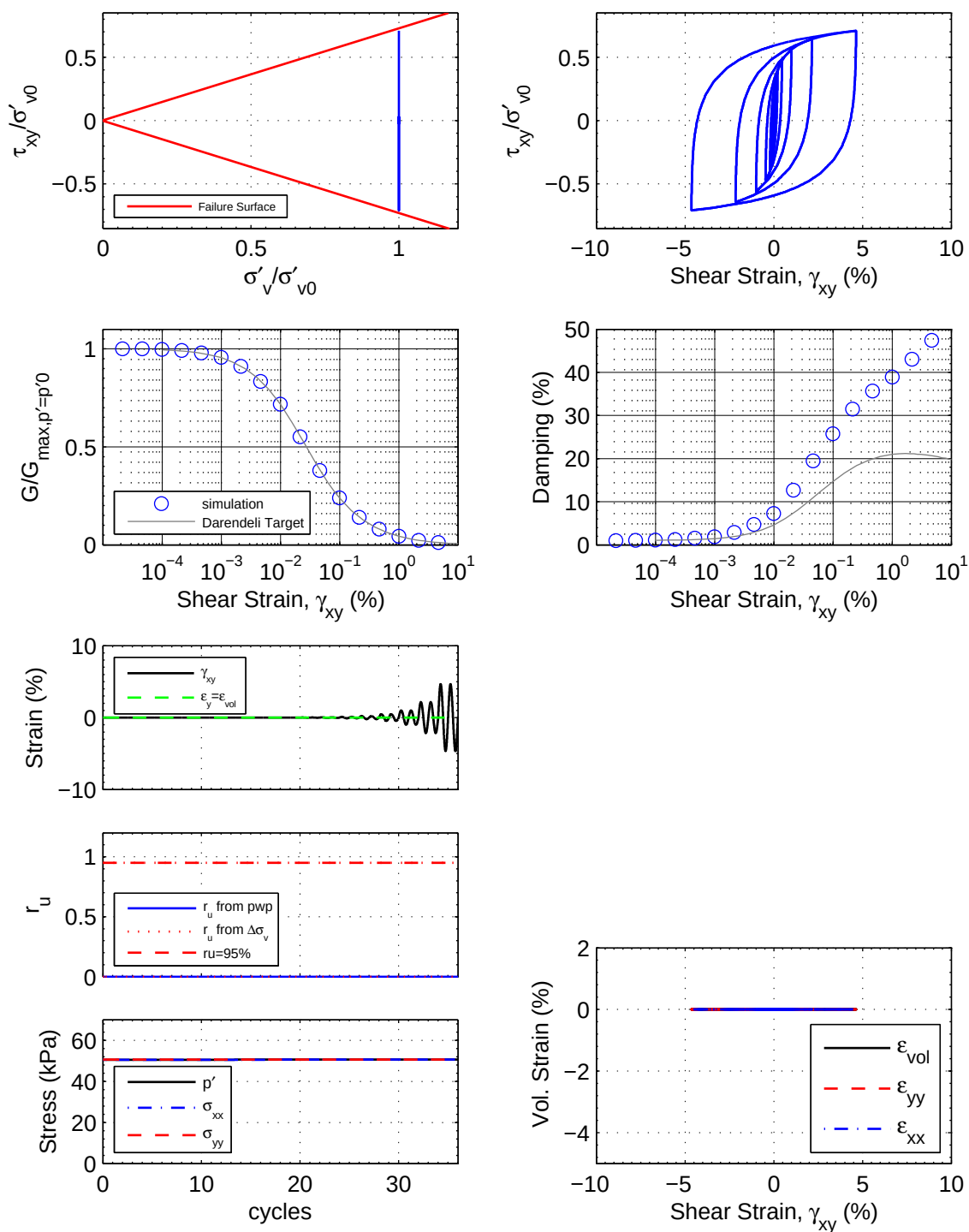


Figure C-38: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$

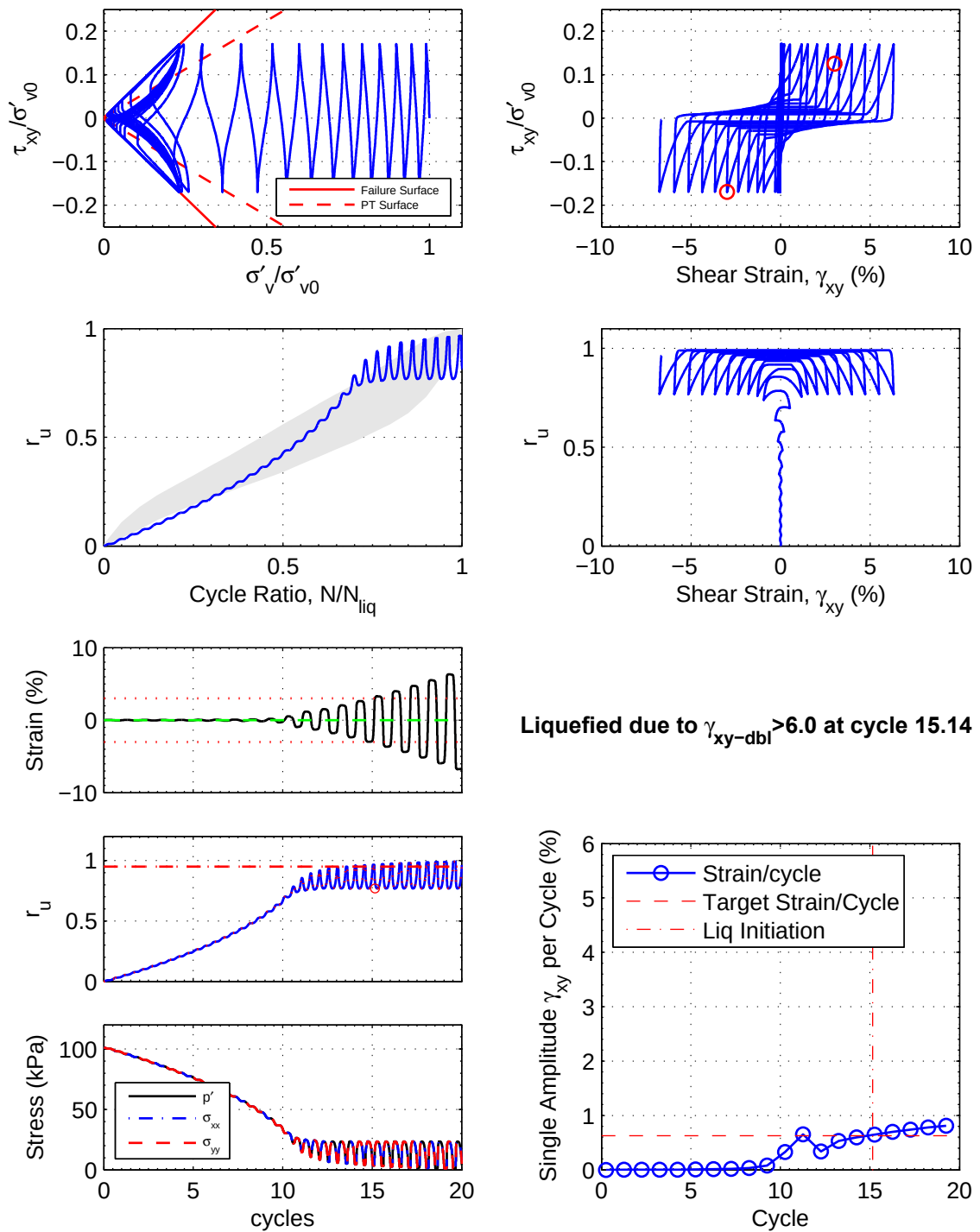


Figure C-39: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=1$ atm, $CSR=0.170$, $\phi=31.3$, $\phi_{PT}=20$

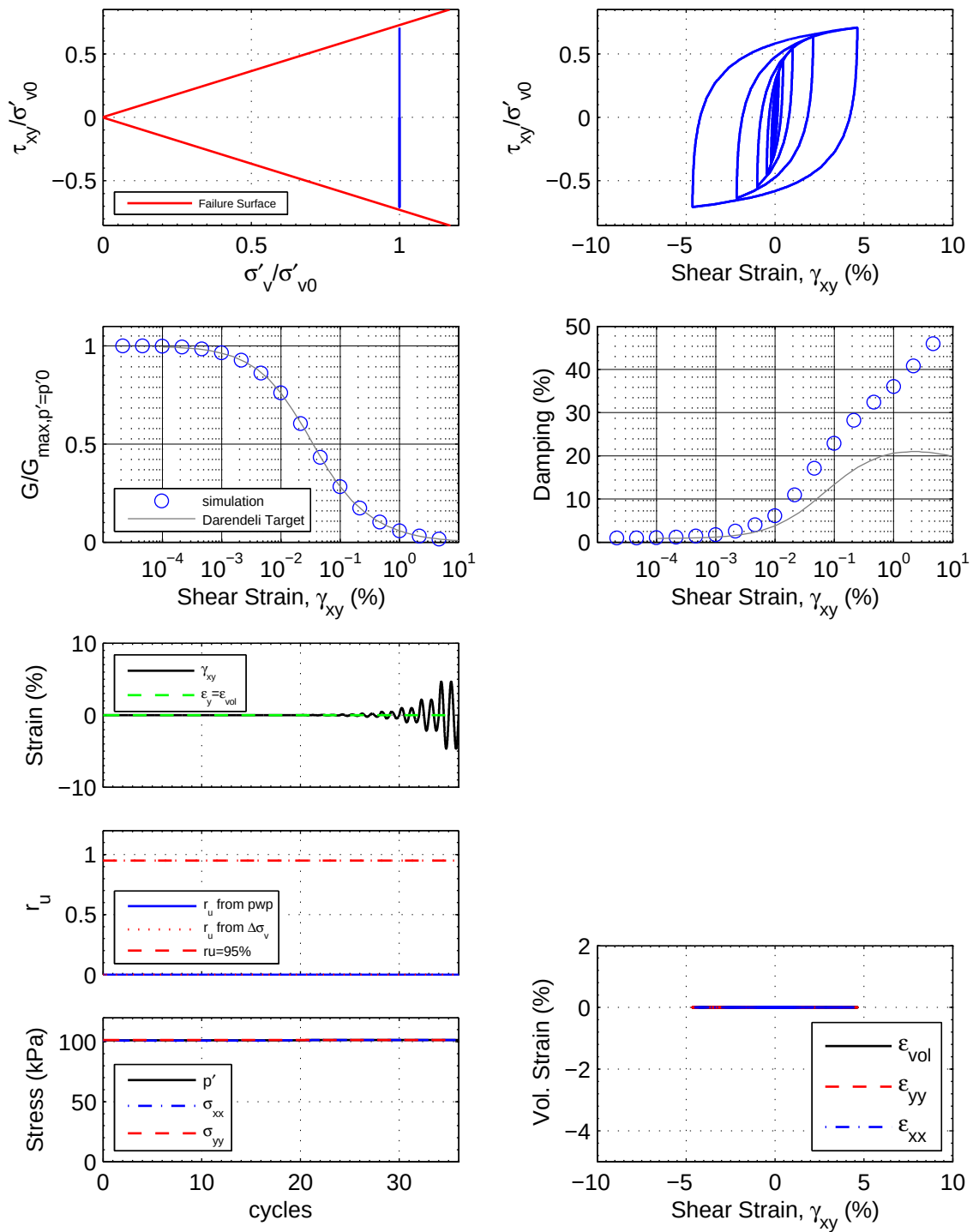


Figure C-40: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$

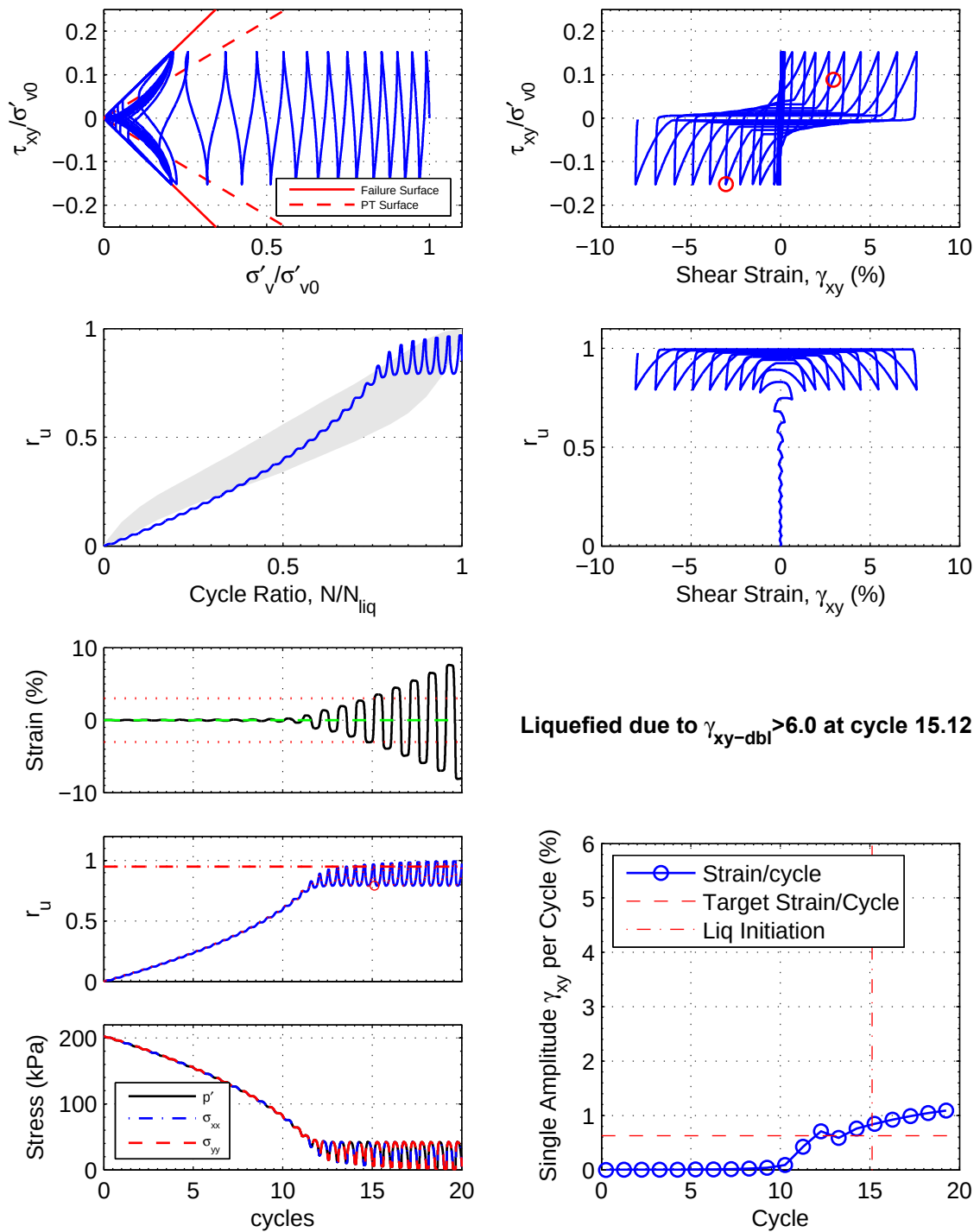


Figure C-41: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=2$ atm, $CSR=0.152$, $\phi=31.3$, $\phi_{PT}=20$

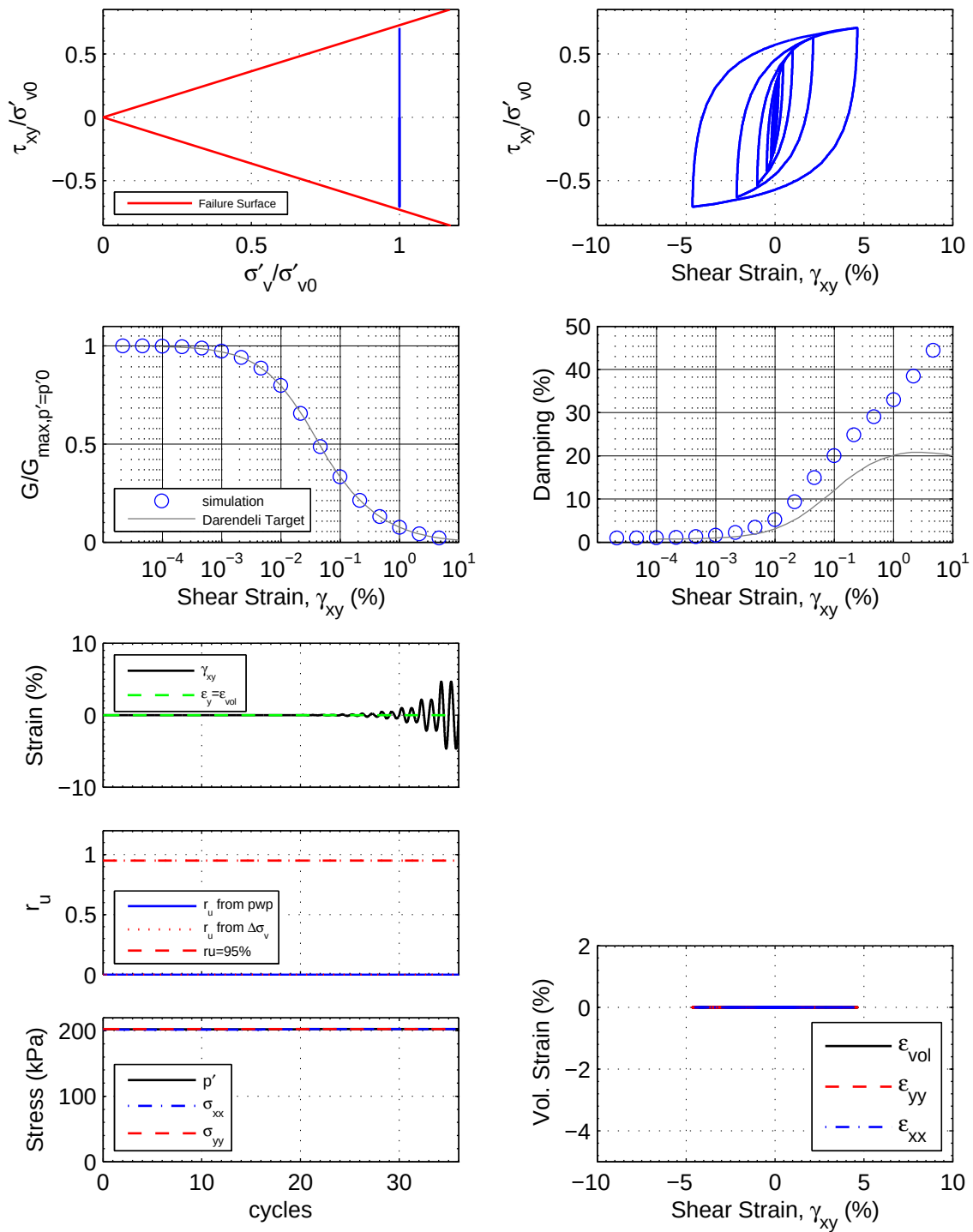


Figure C-42: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$

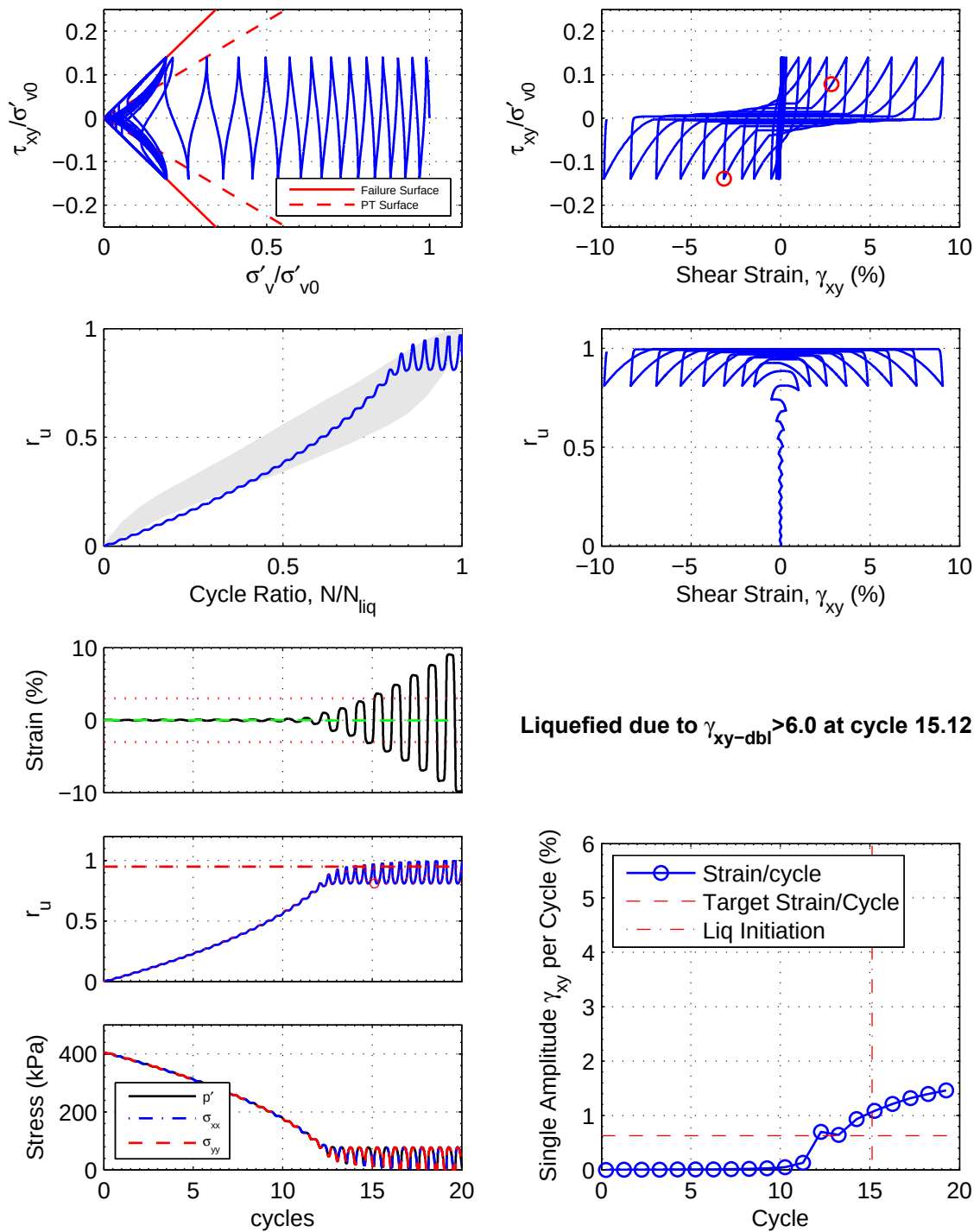


Figure C-43: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=4$ atm, $CSR=0.139$, $\phi=31.3$, $\phi_{PT}=20$

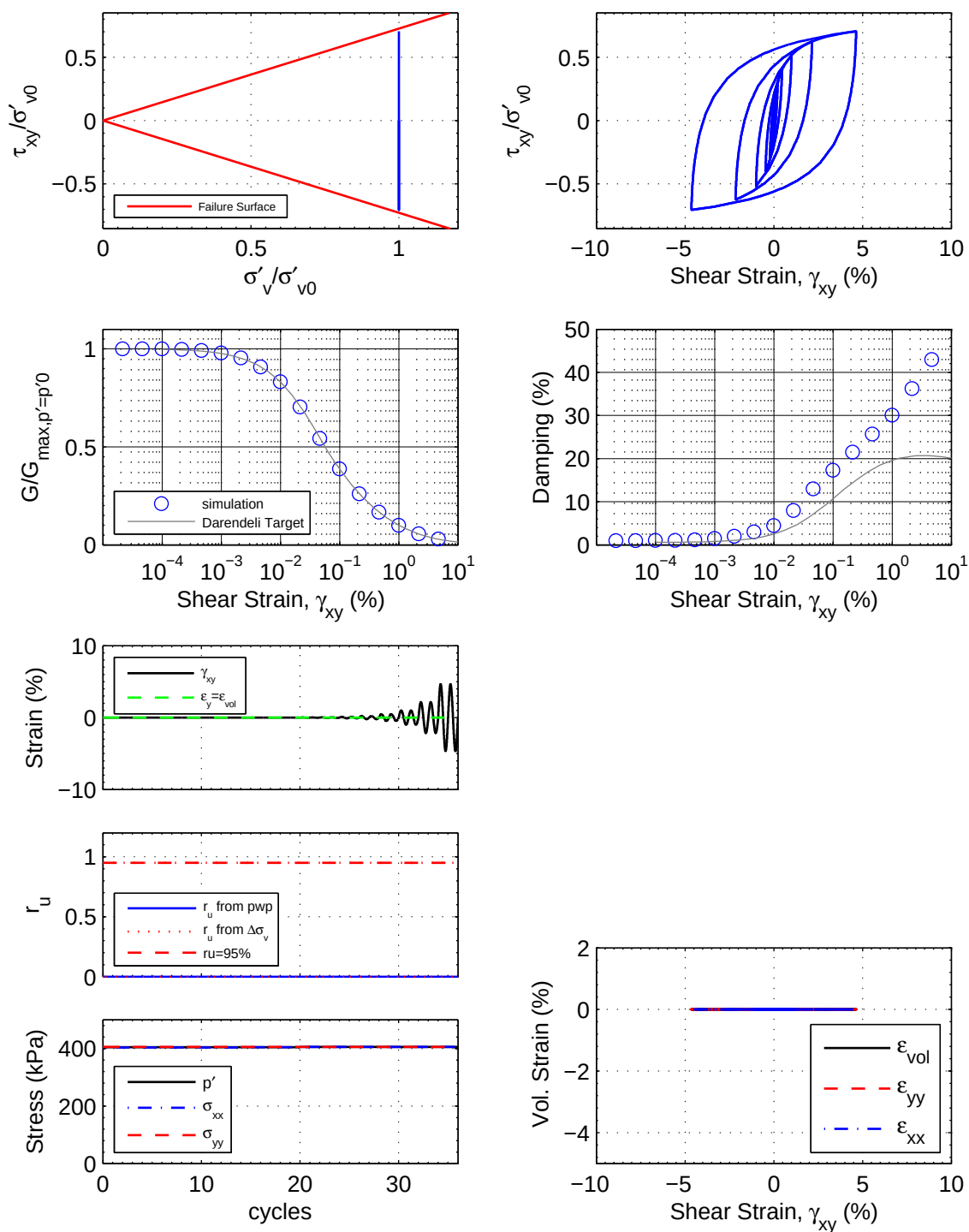


Figure C-44: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$

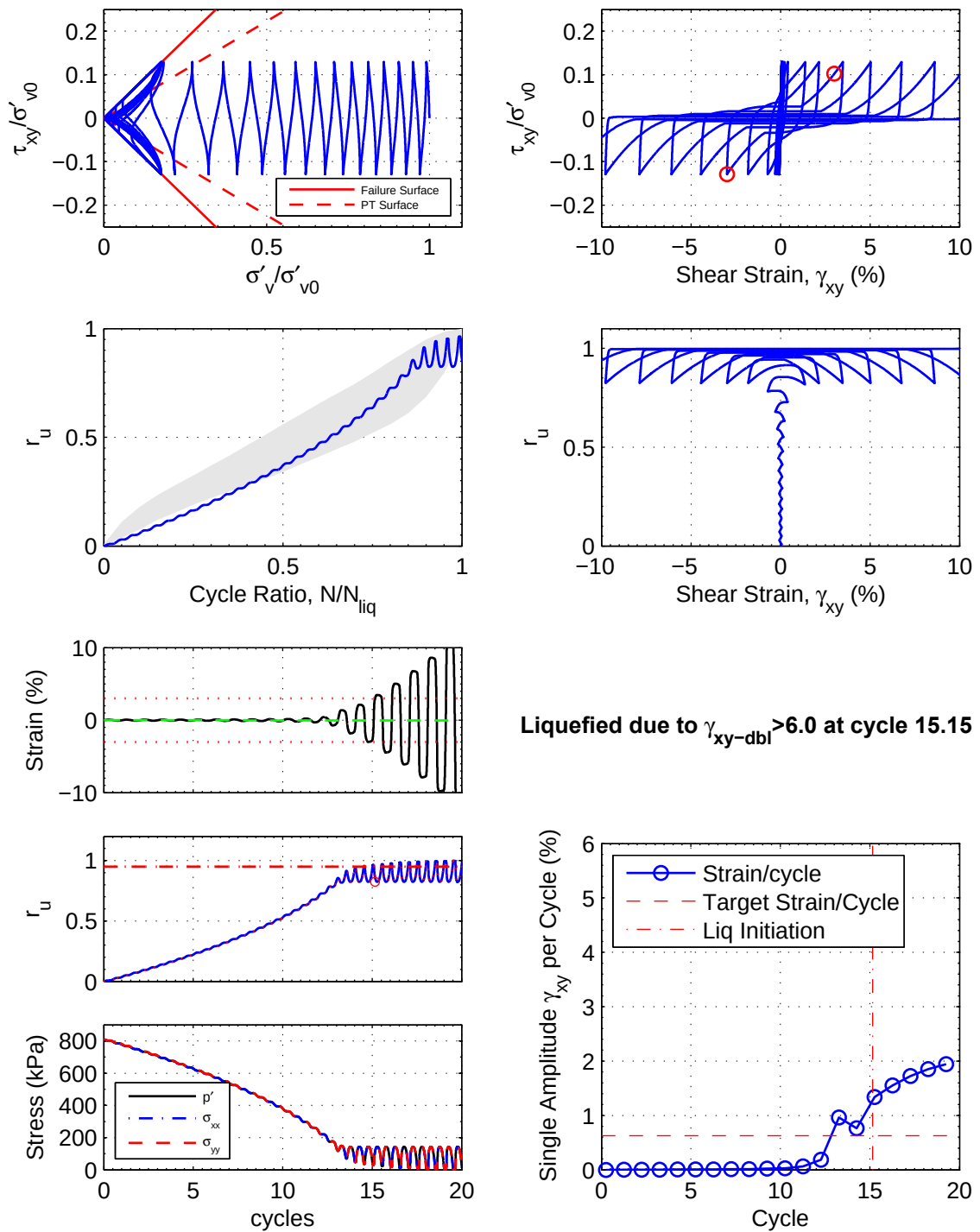


Figure C-45: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=8$ atm, $CSR=0.129$, $\phi=31.3$, $\phi_{PT}=20$

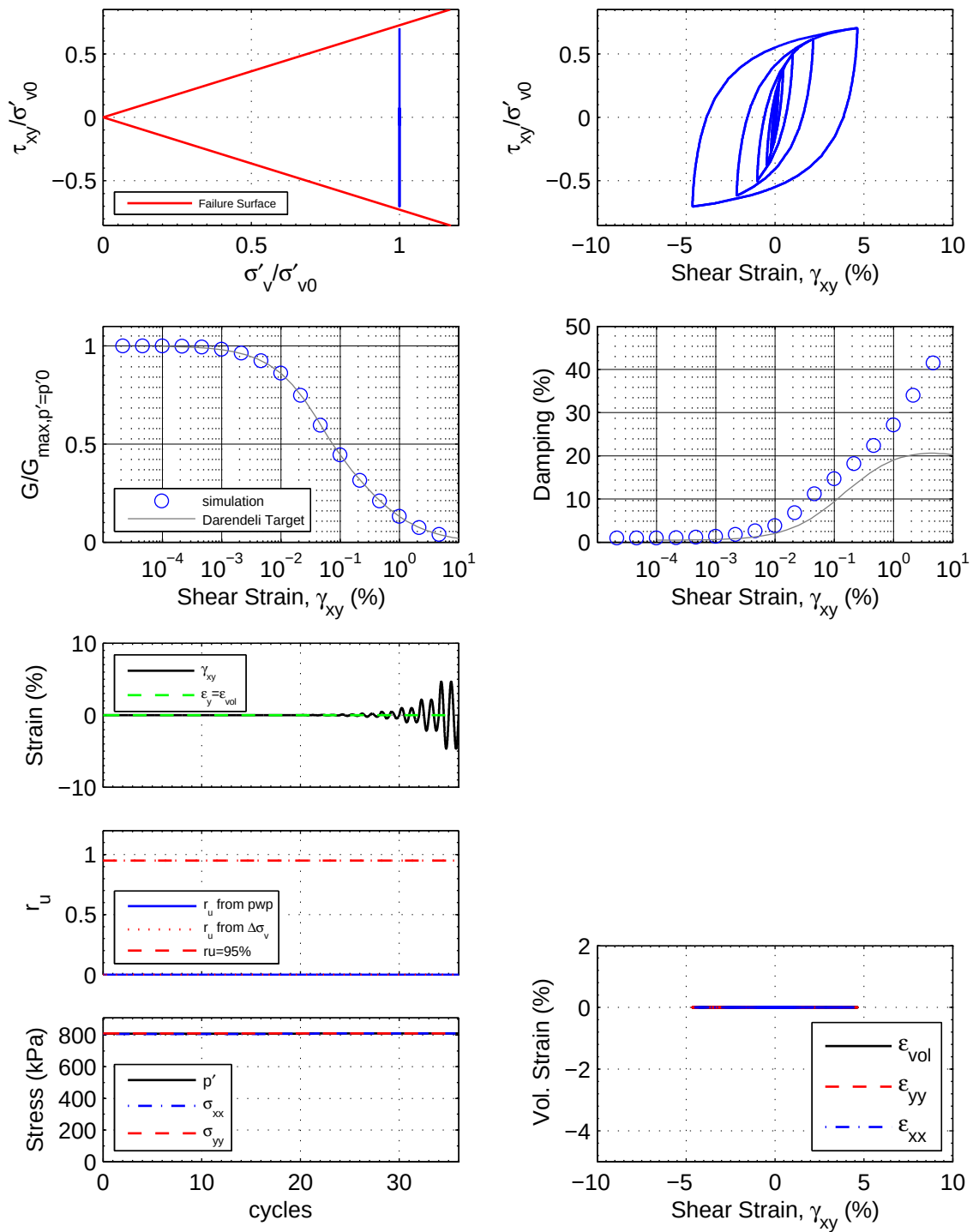


Figure C-46: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=31.3$, $\phi_{PT}=na$

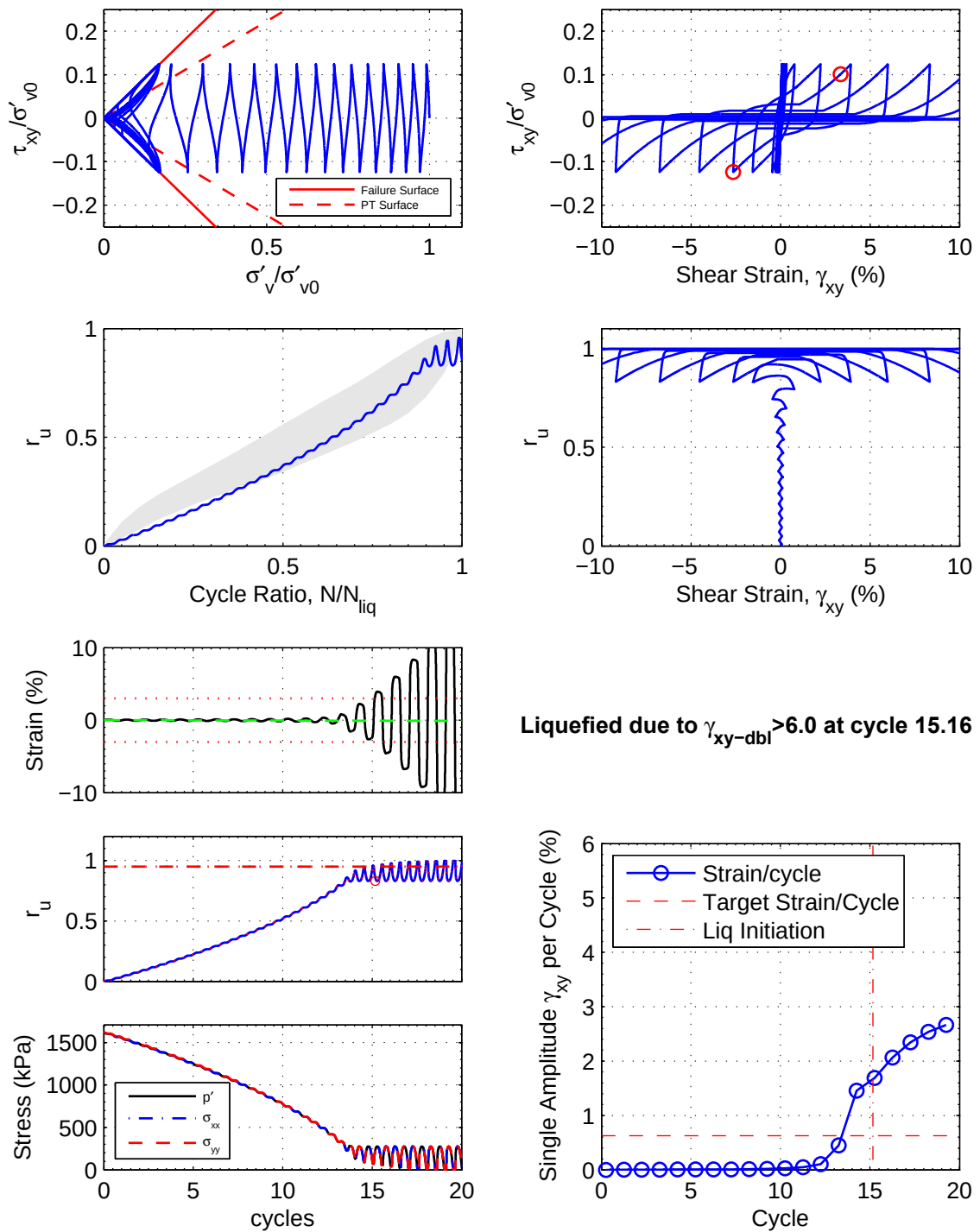


Figure C-47: Undrained CSS Simulation: PDMY2, $Dr=60$, $\sigma'_{v0}=16$ atm, $CSR=0.124$, $\phi=31.3$, $\phi_{PT}=20$

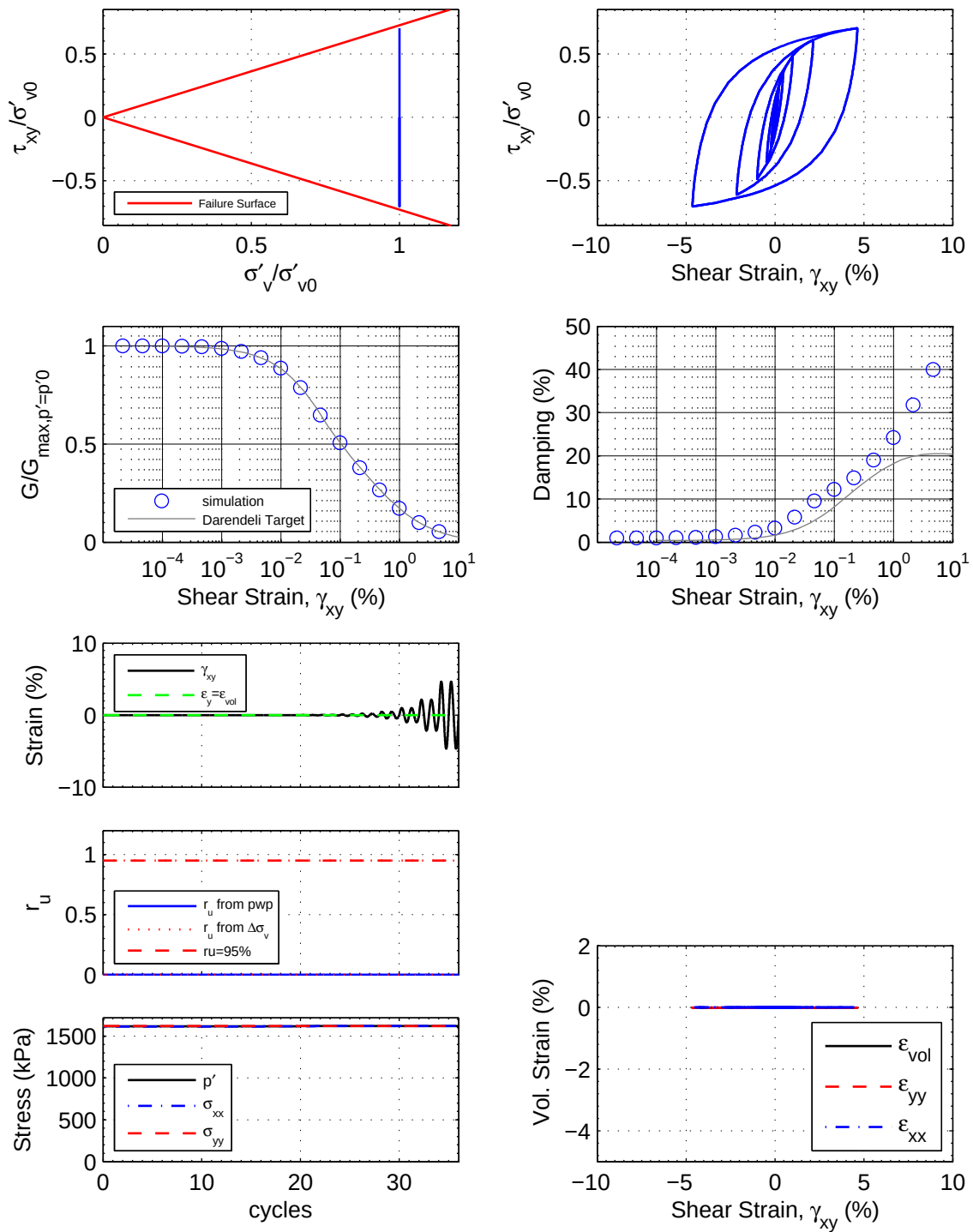


Figure C-48: Drained CSS Simulation: PIMY, $Dr=60$, $\sigma'_{v0}=16$ atm, $\phi=31.3$, $\phi_{PT}=na$

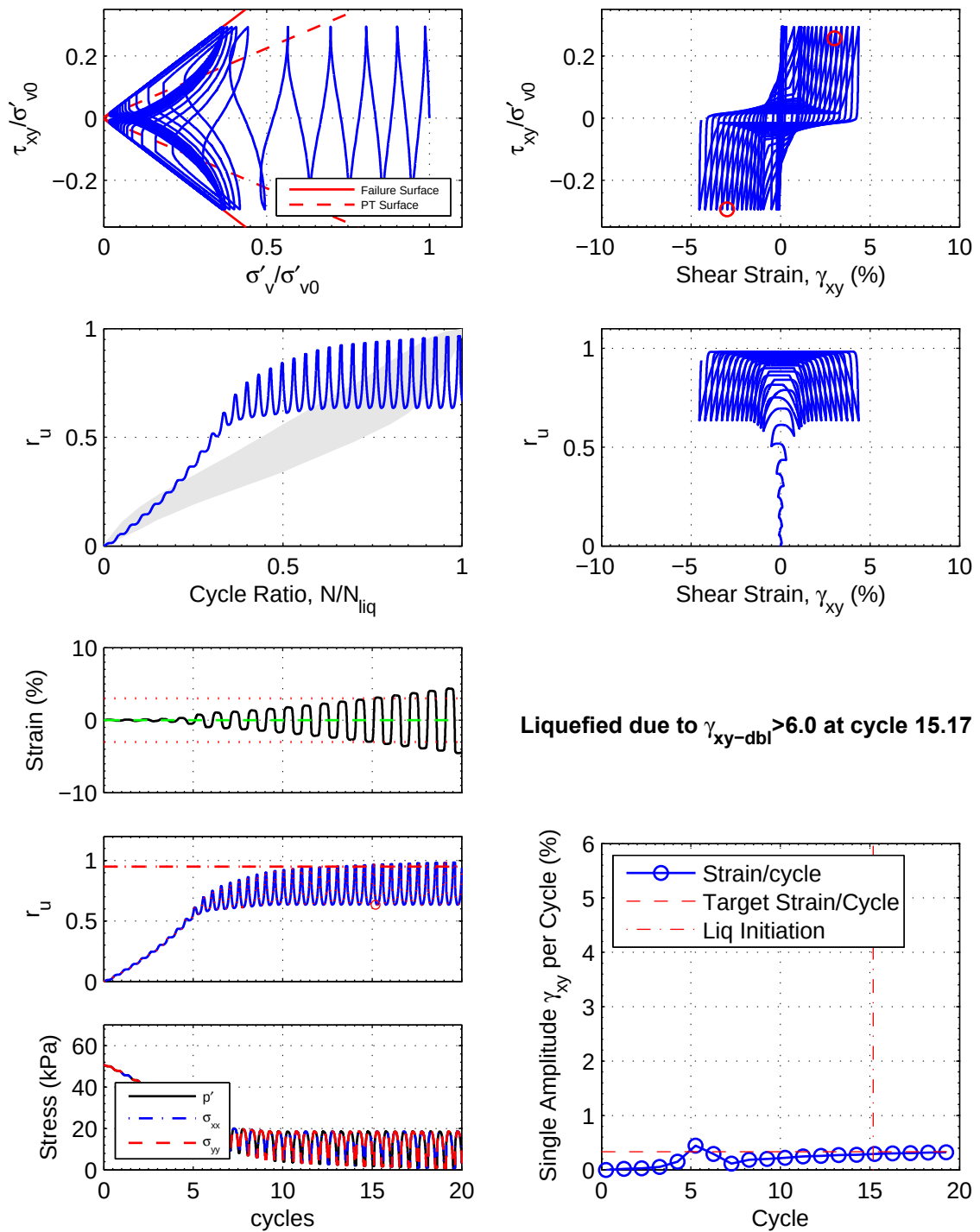


Figure C-49: Undrained CSS Simulation: PDMY2, $D_r=70$, $\sigma'_{v0}=0.5$ atm, $CSR=0.294$, $\phi=34.2$, $\phi_{PT}=20$

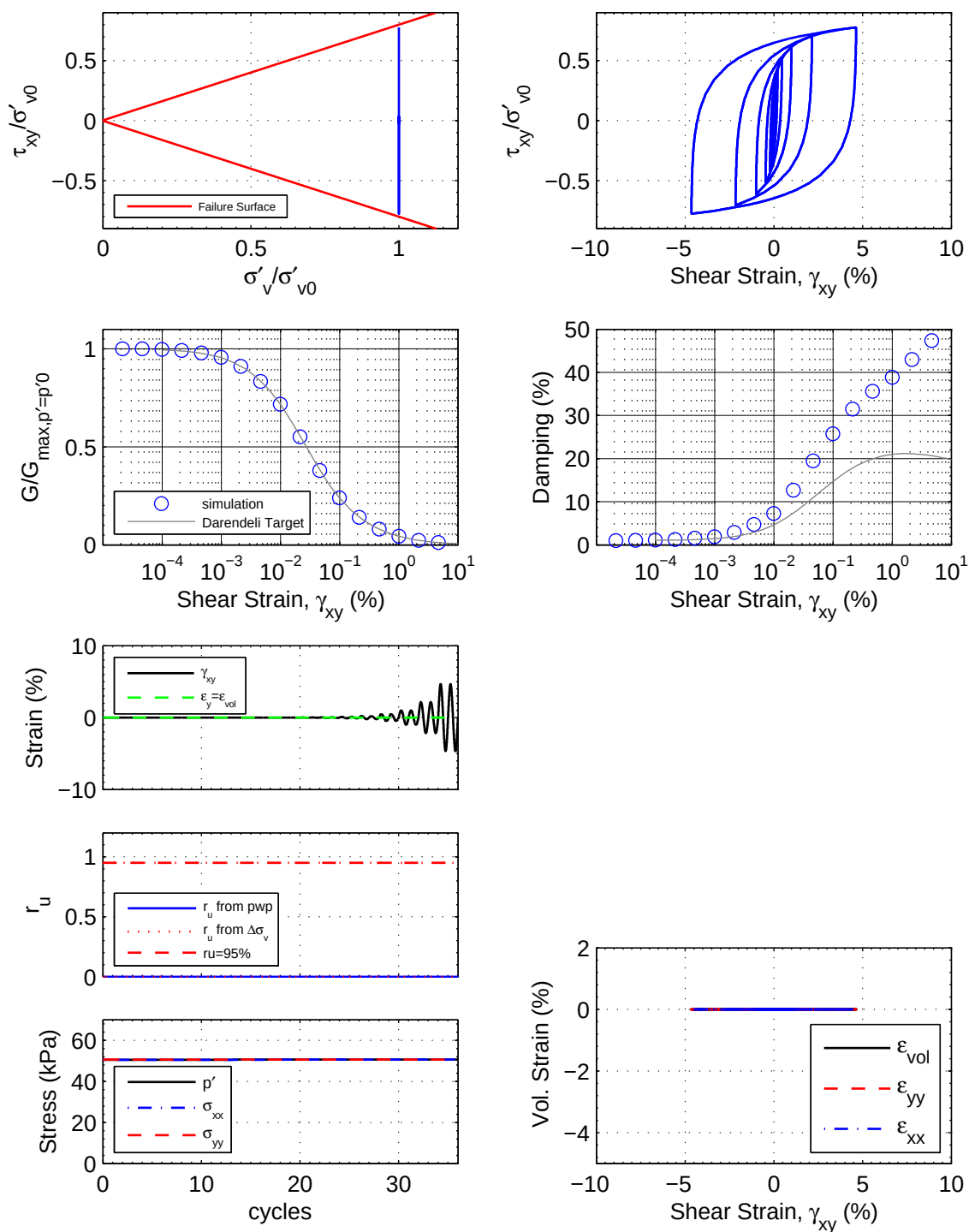


Figure C-50: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$

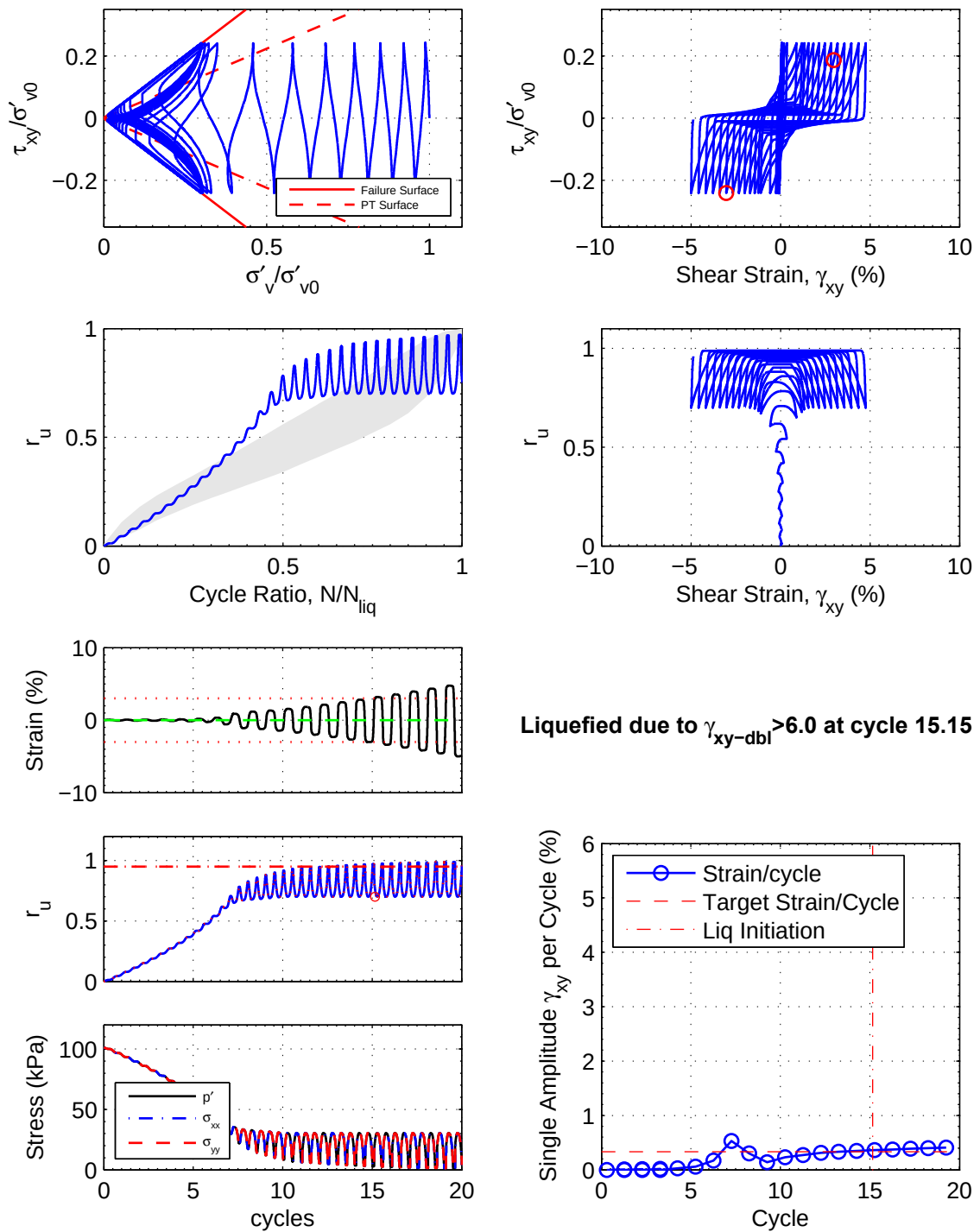


Figure C-51: Undrained CSS Simulation: PDMY2, $Dr=70$, $\sigma'_{v0}=1$ atm, $CSR=0.241$, $\phi=34.2$, $\phi_{PT}=20$

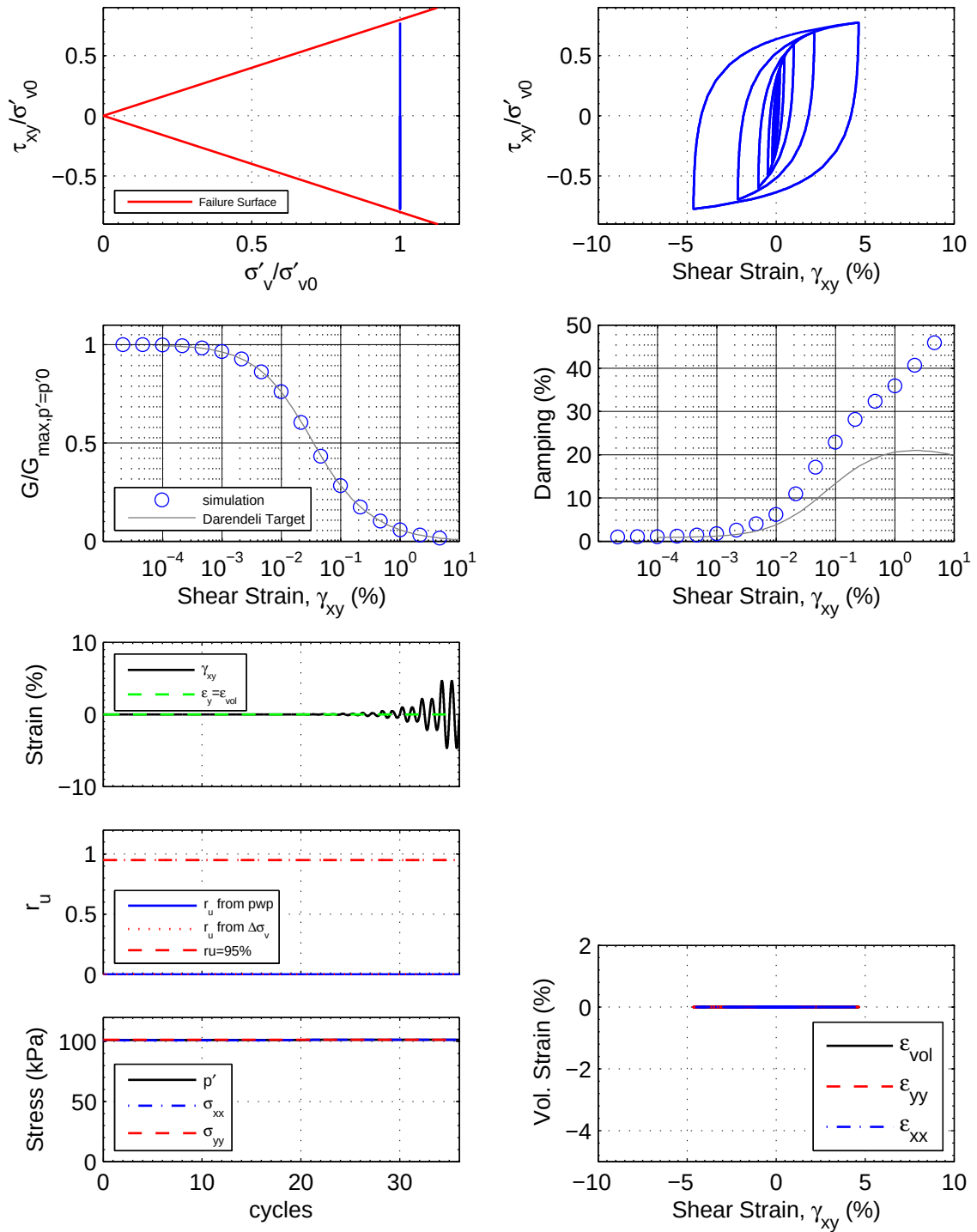


Figure C-52: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$

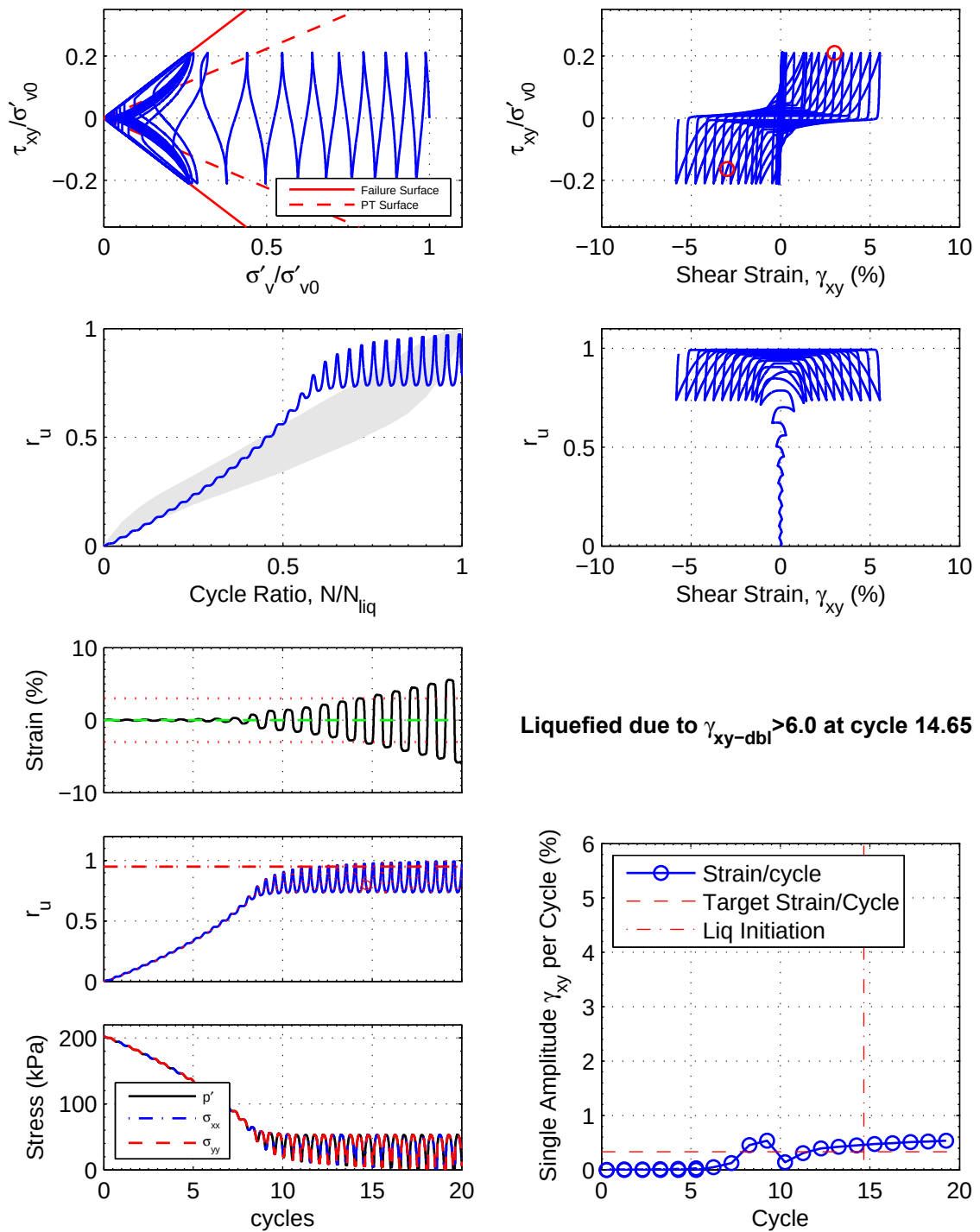


Figure C-53: Undrained CSS Simulation: PDMY2, $Dr=70$, $\sigma'_{v0}=2$ atm, $CSR=0.210$, $\phi=34.2$, $\phi_{PT}=20$

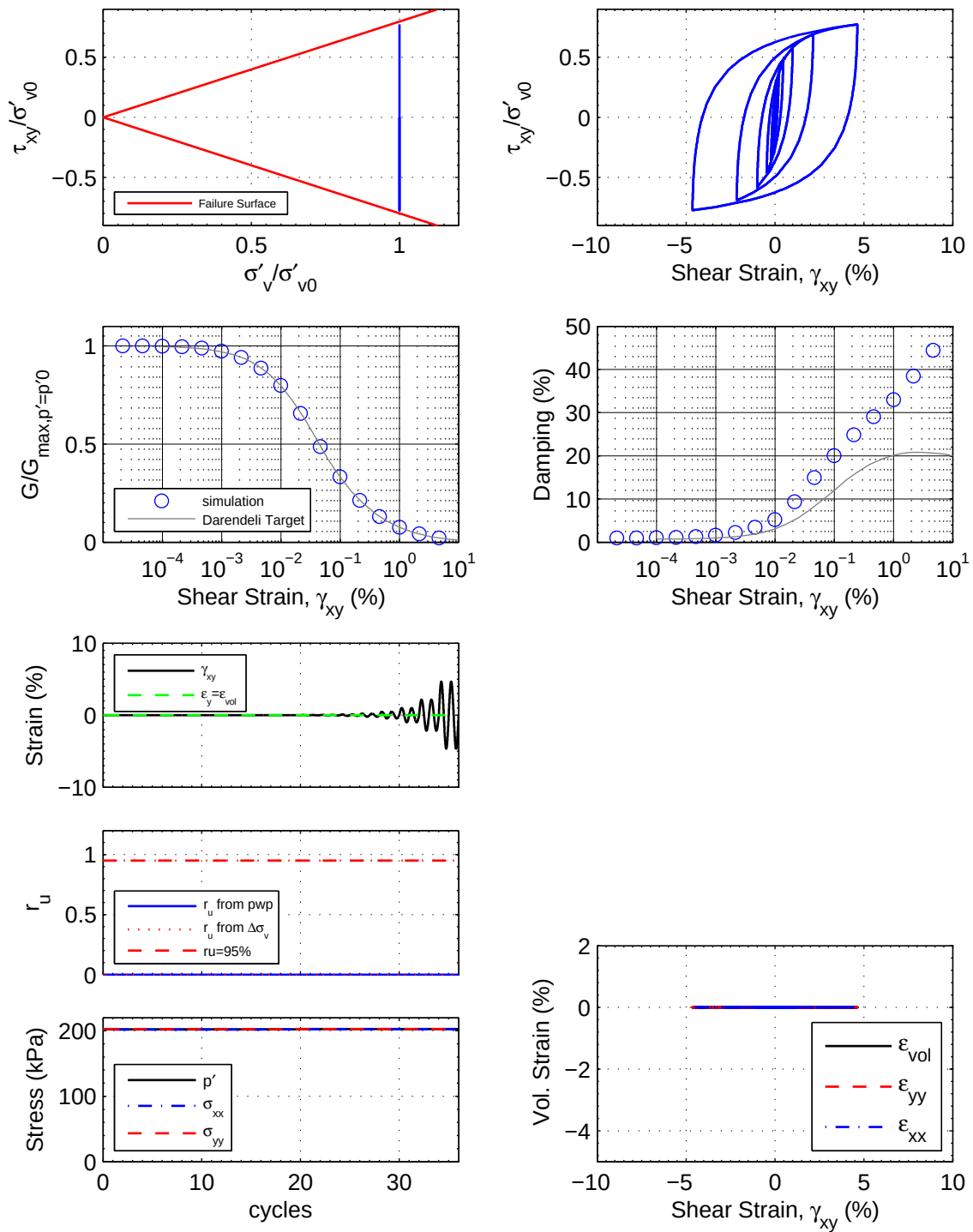


Figure C-54: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$

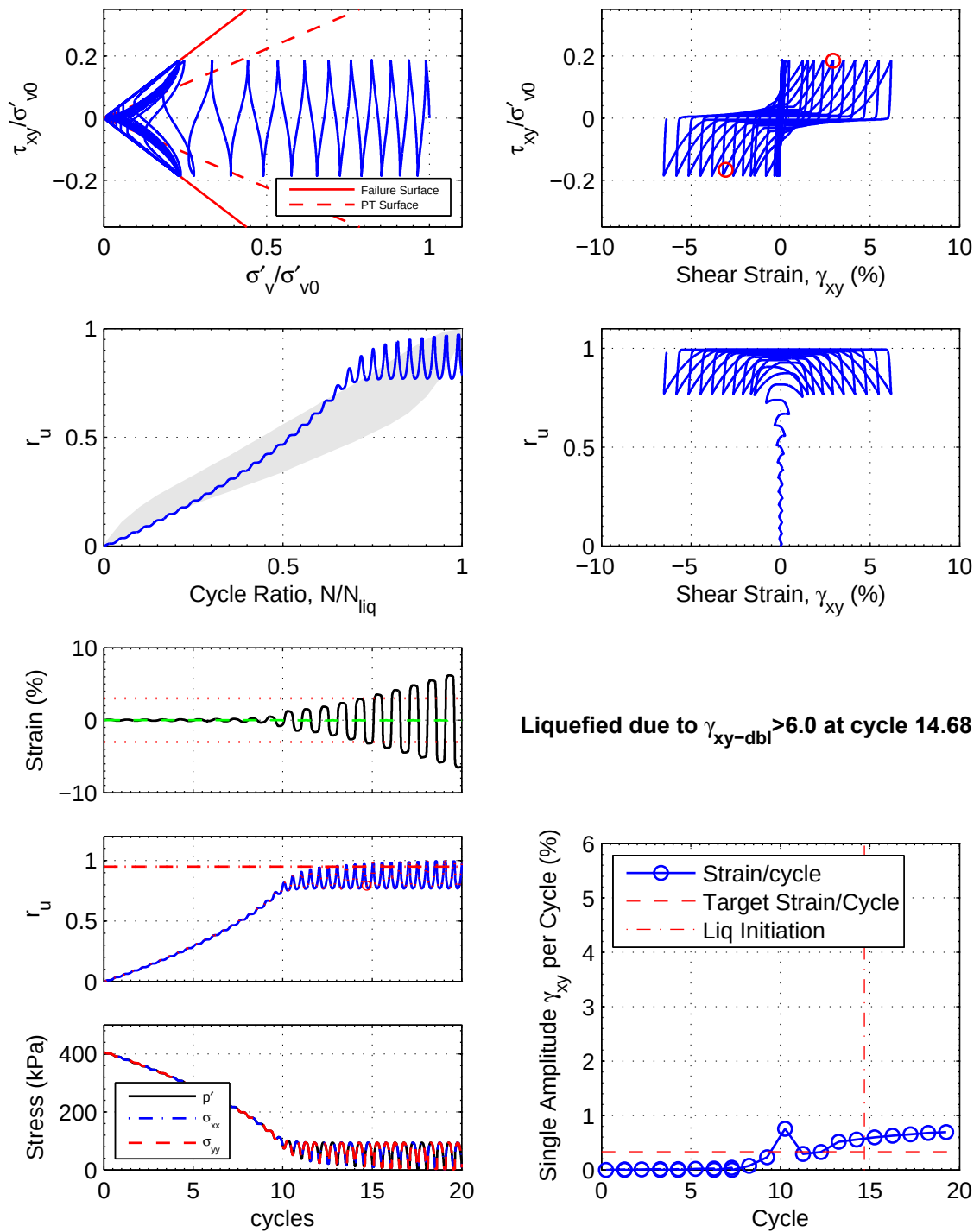


Figure C-55: Undrained CSS Simulation: PDMY2, $Dr=70$, $\sigma'_{v0}=4$ atm, $CSR=0.185$, $\phi=34.2$, $\phi_{PT}=20$

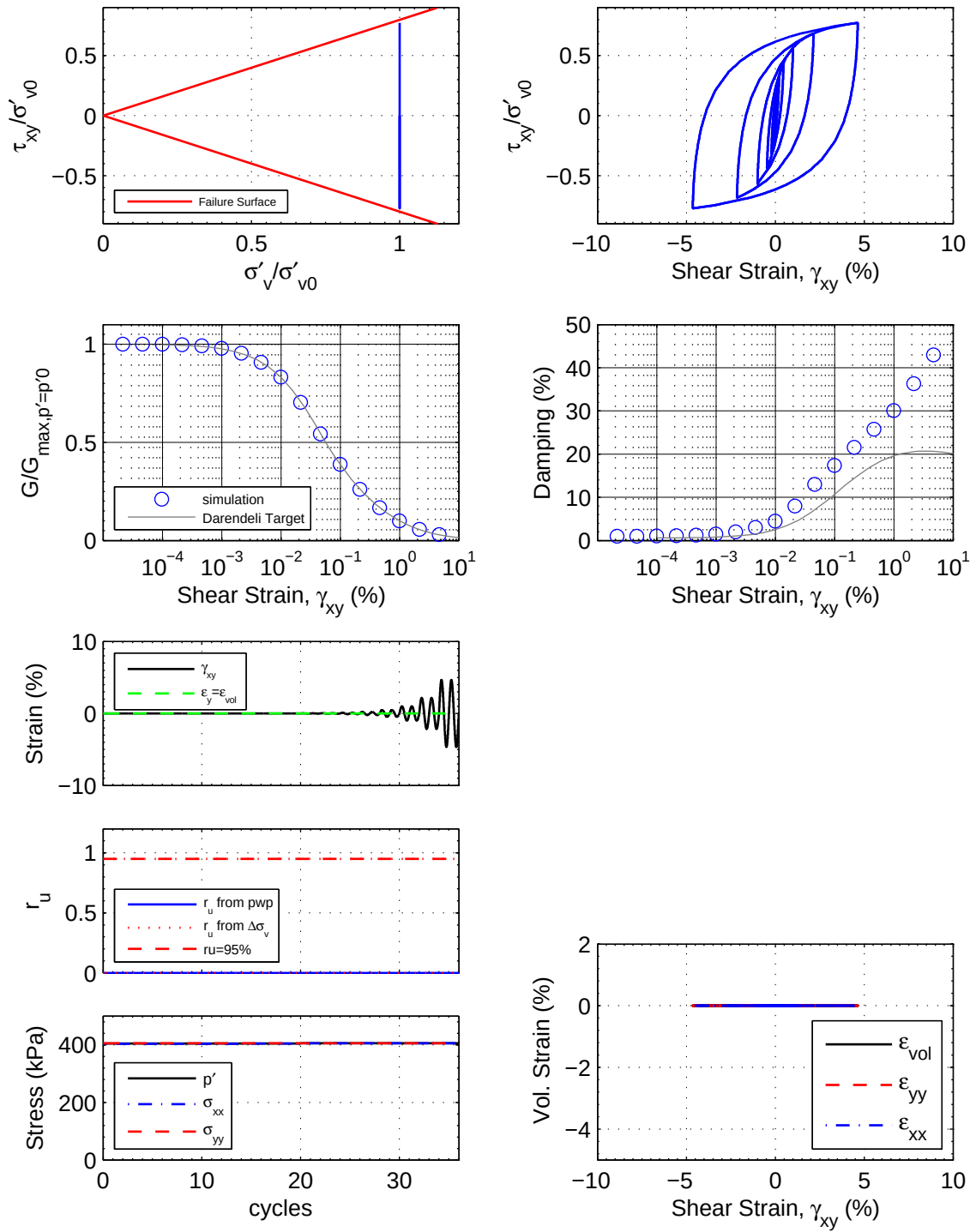


Figure C-56: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$

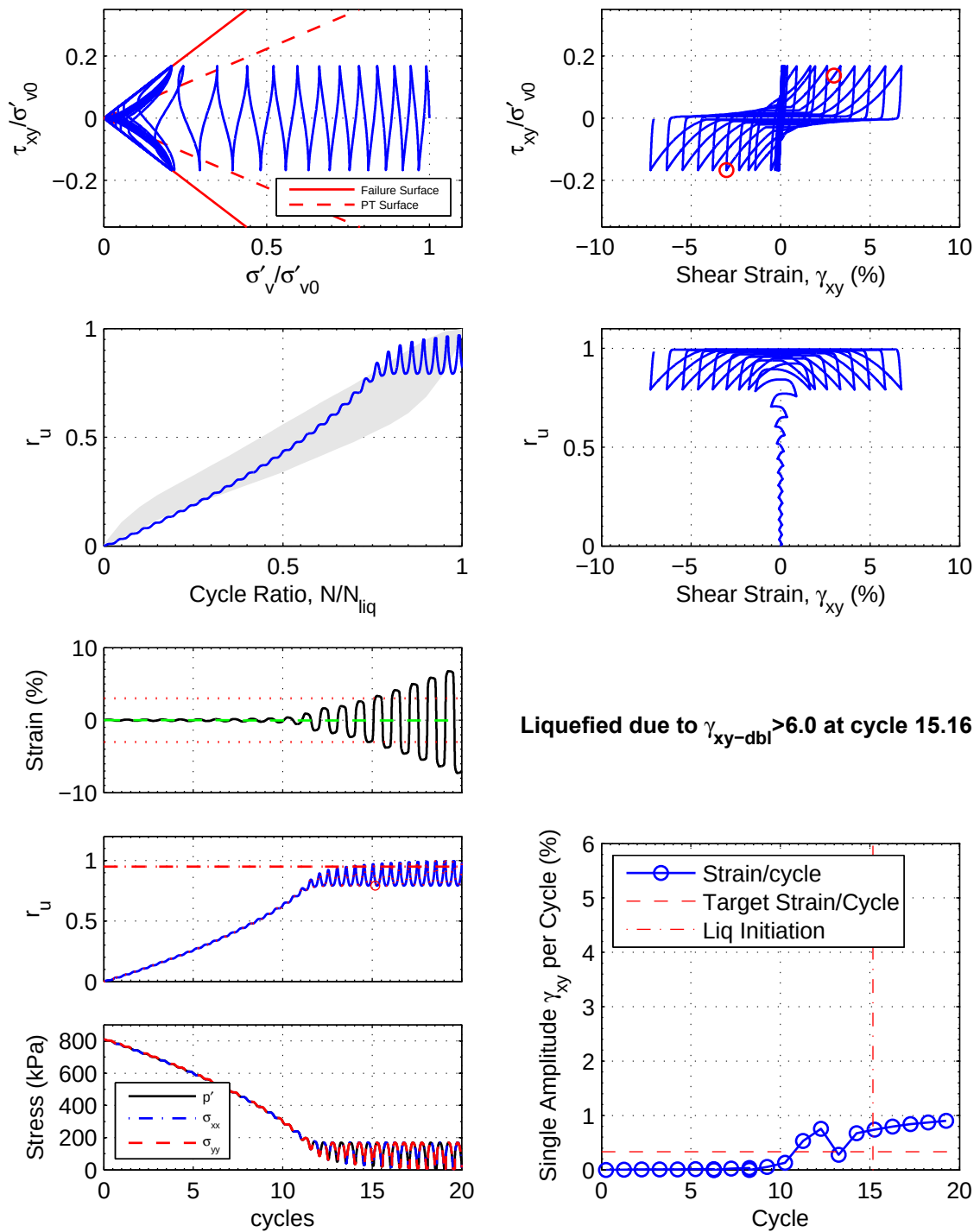


Figure C-57: Undrained CSS Simulation: PDMY2, $Dr=70$, $\sigma'_{v0}=8$ atm, $CSR=0.167$, $\phi=34.2$, $\phi_{PT}=20$

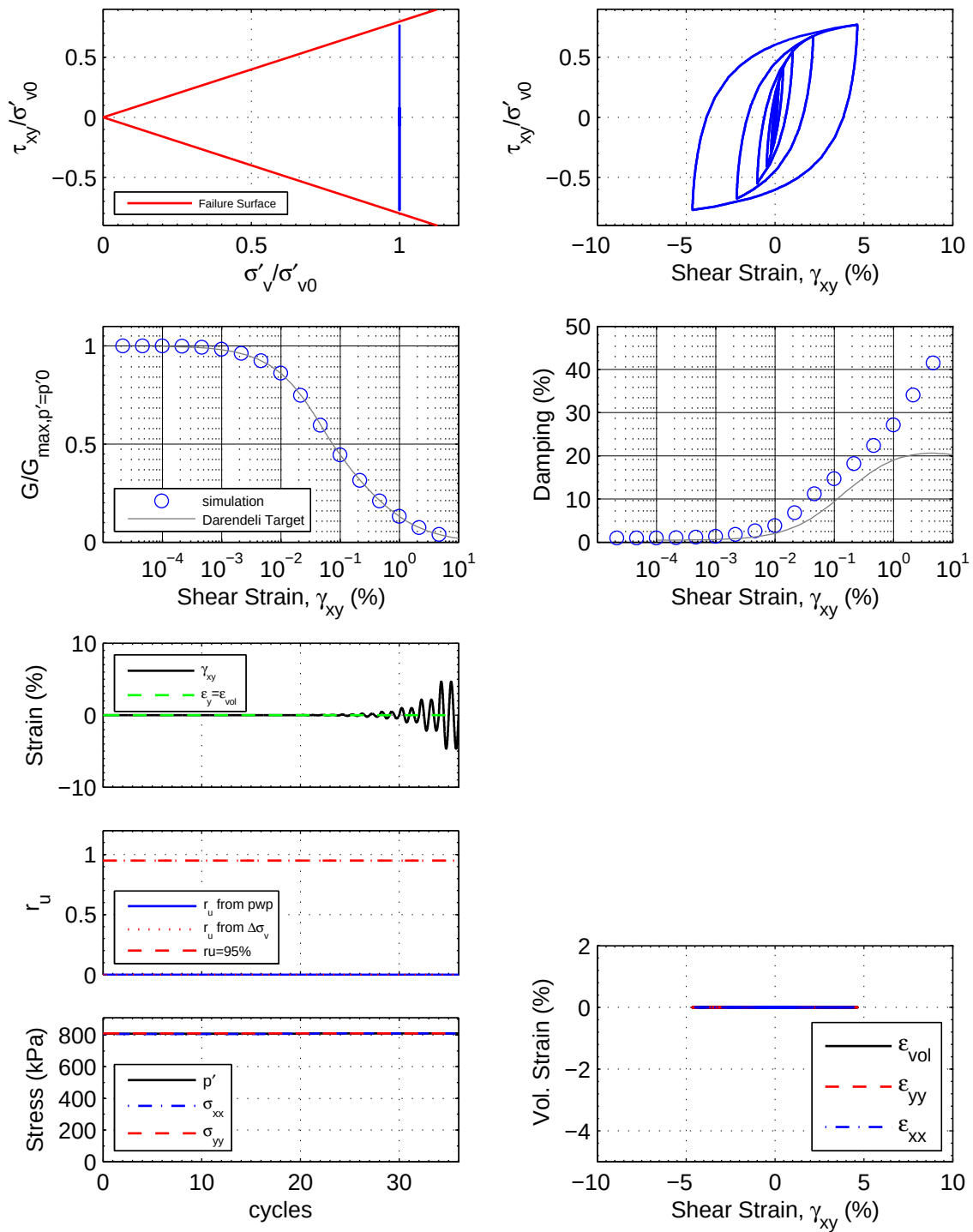


Figure C-58: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=34.2$, $\phi_{PT}=na$

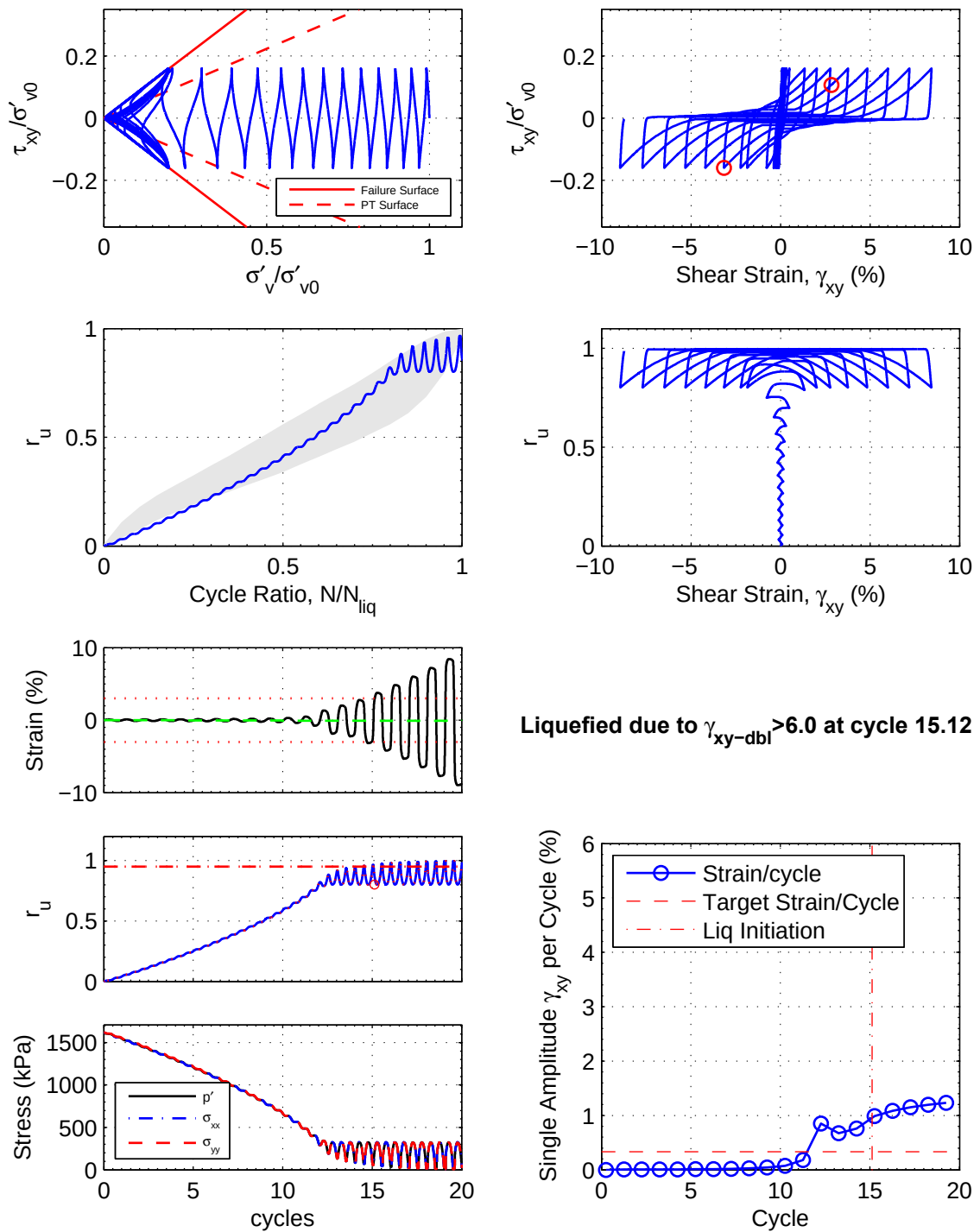


Figure C-59: Undrained CSS Simulation: PDMY2, $Dr=70$, $\sigma'_{v0}=16$ atm, $CSR=0.160$, $\phi=34.2$, $\phi_{PT}=20$

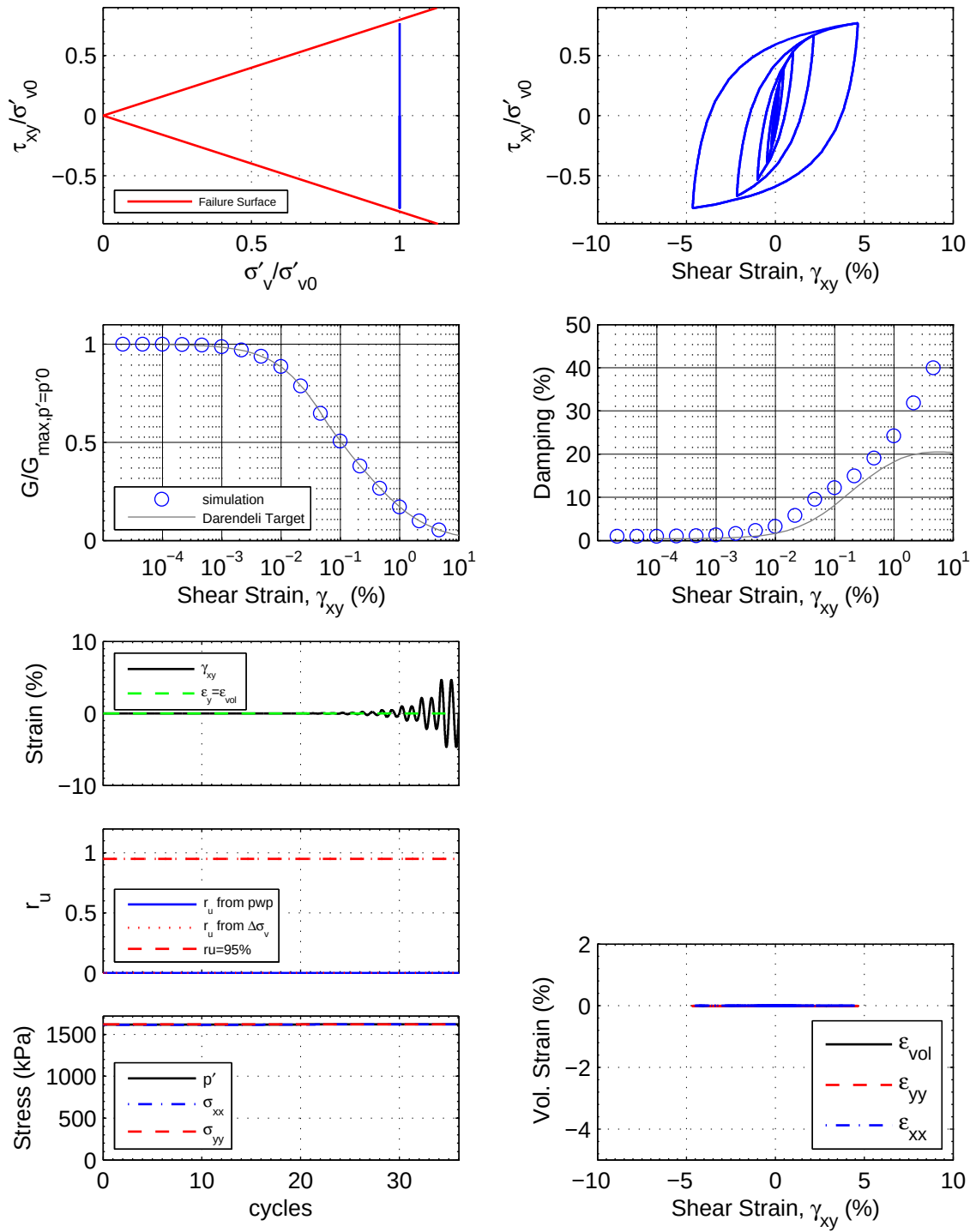


Figure C-60: Drained CSS Simulation: PIMY, $Dr=70$, $\sigma'_{v0}=16$ atm, $\phi=34.2$, $\phi_{PT}=na$

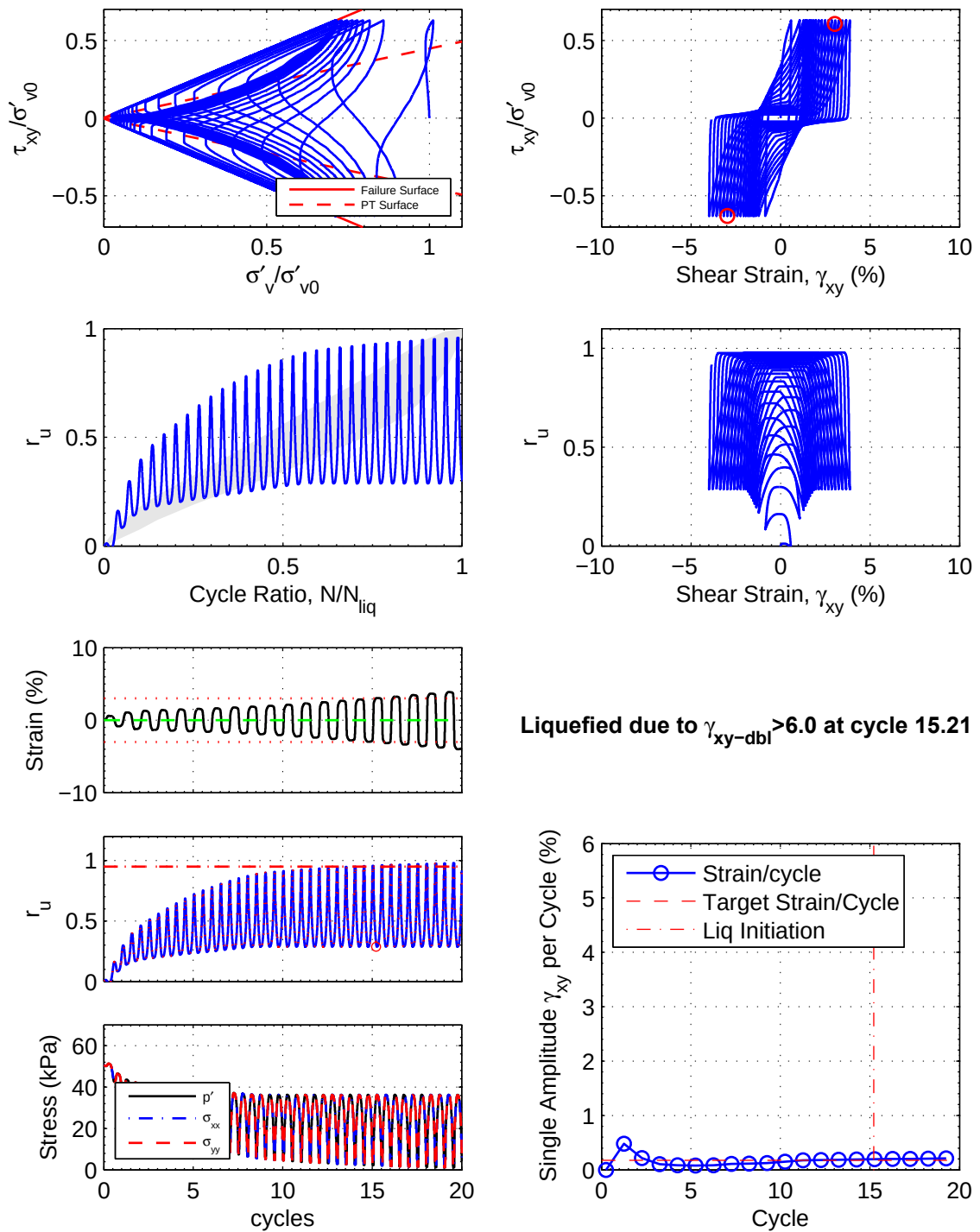


Figure C-61: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=0.5$ atm, $CSR=0.630$, $\phi=37.4$, $\phi_{PT}=20$

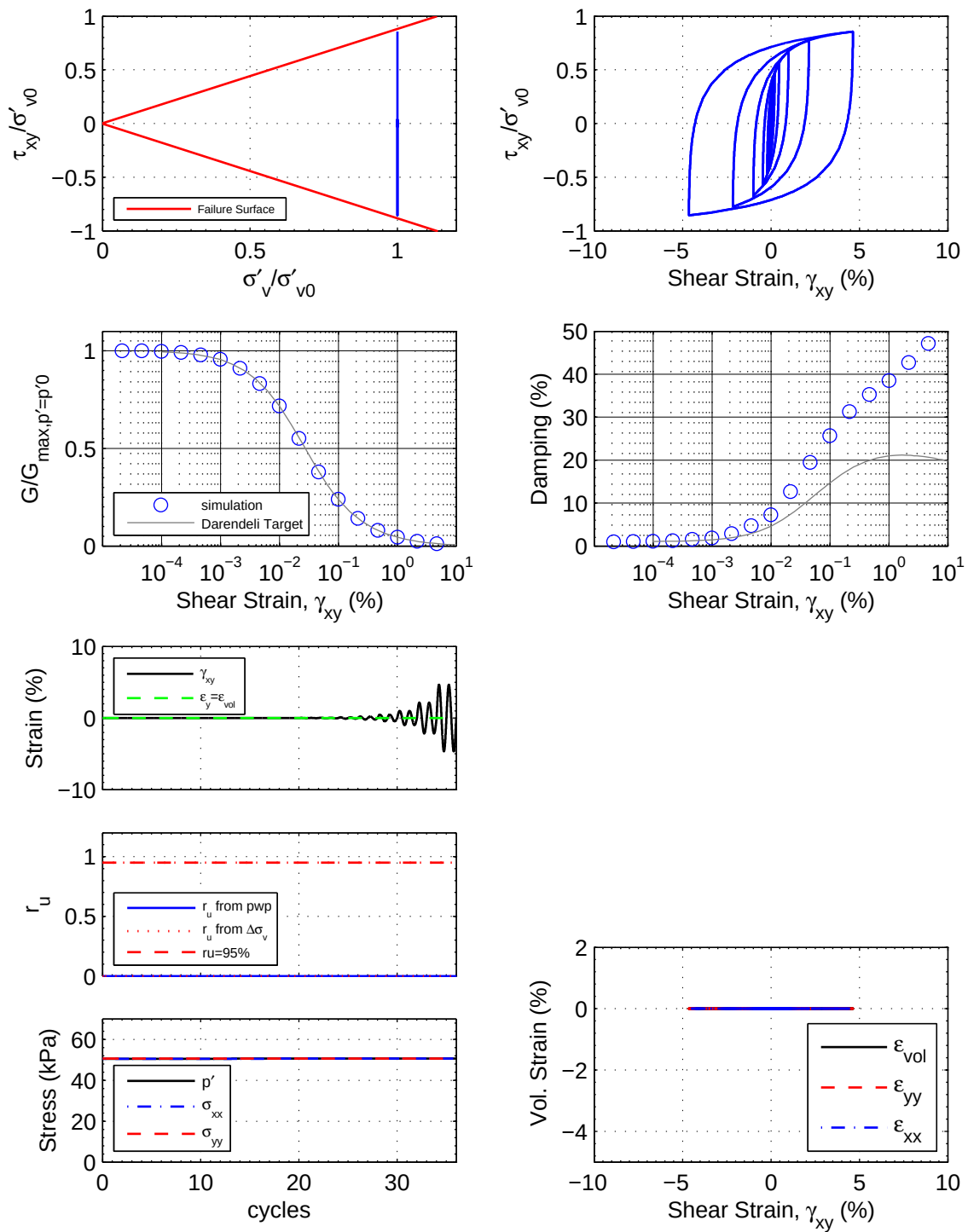


Figure C-62: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

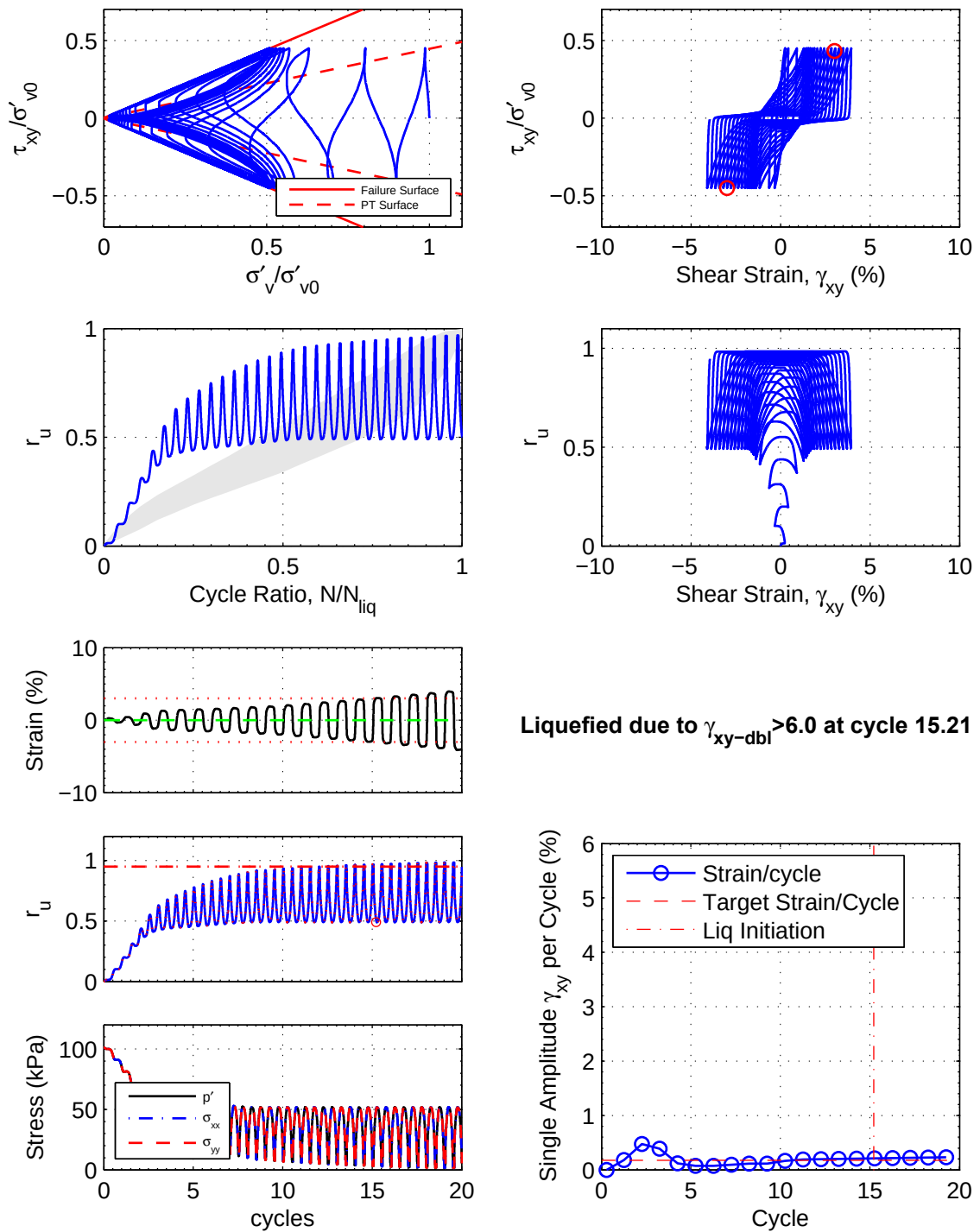


Figure C-63: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=1$ atm, $CSR=0.450$, $\phi=37.4$, $\phi_{PT}=20$

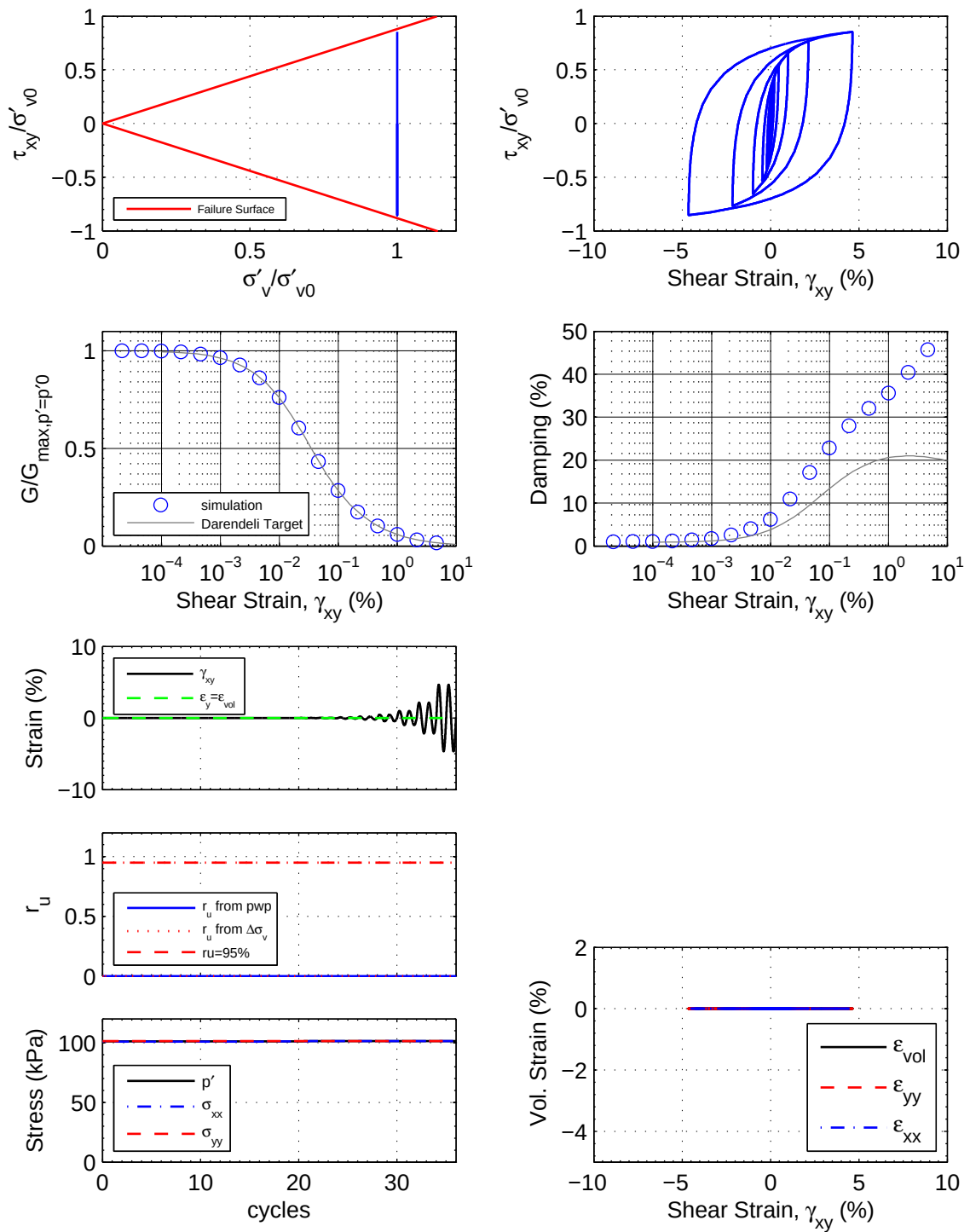


Figure C-64: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

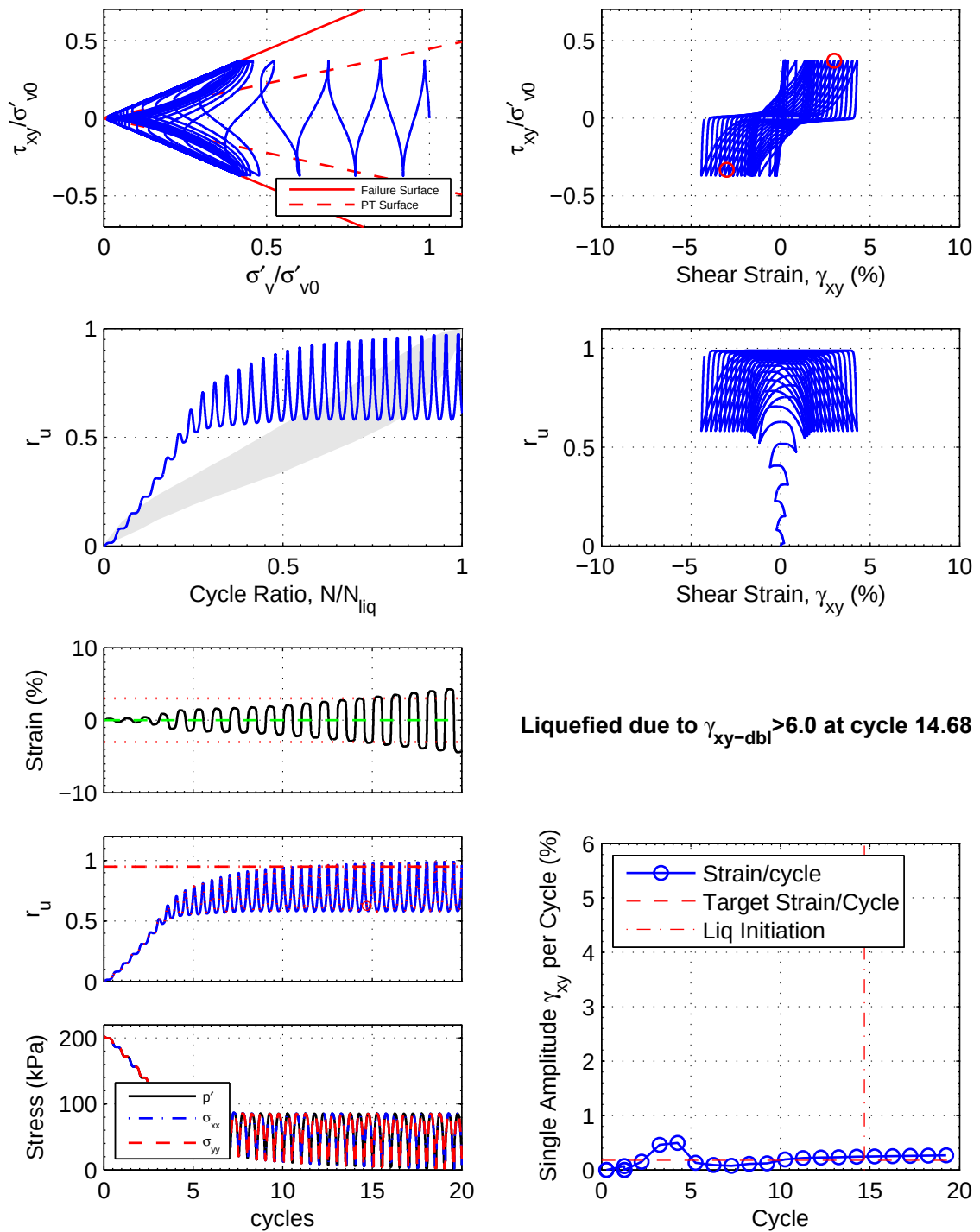


Figure C-65: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=2$ atm, $CSR=0.370$, $\phi=37.4$, $\phi_{PT}=20$

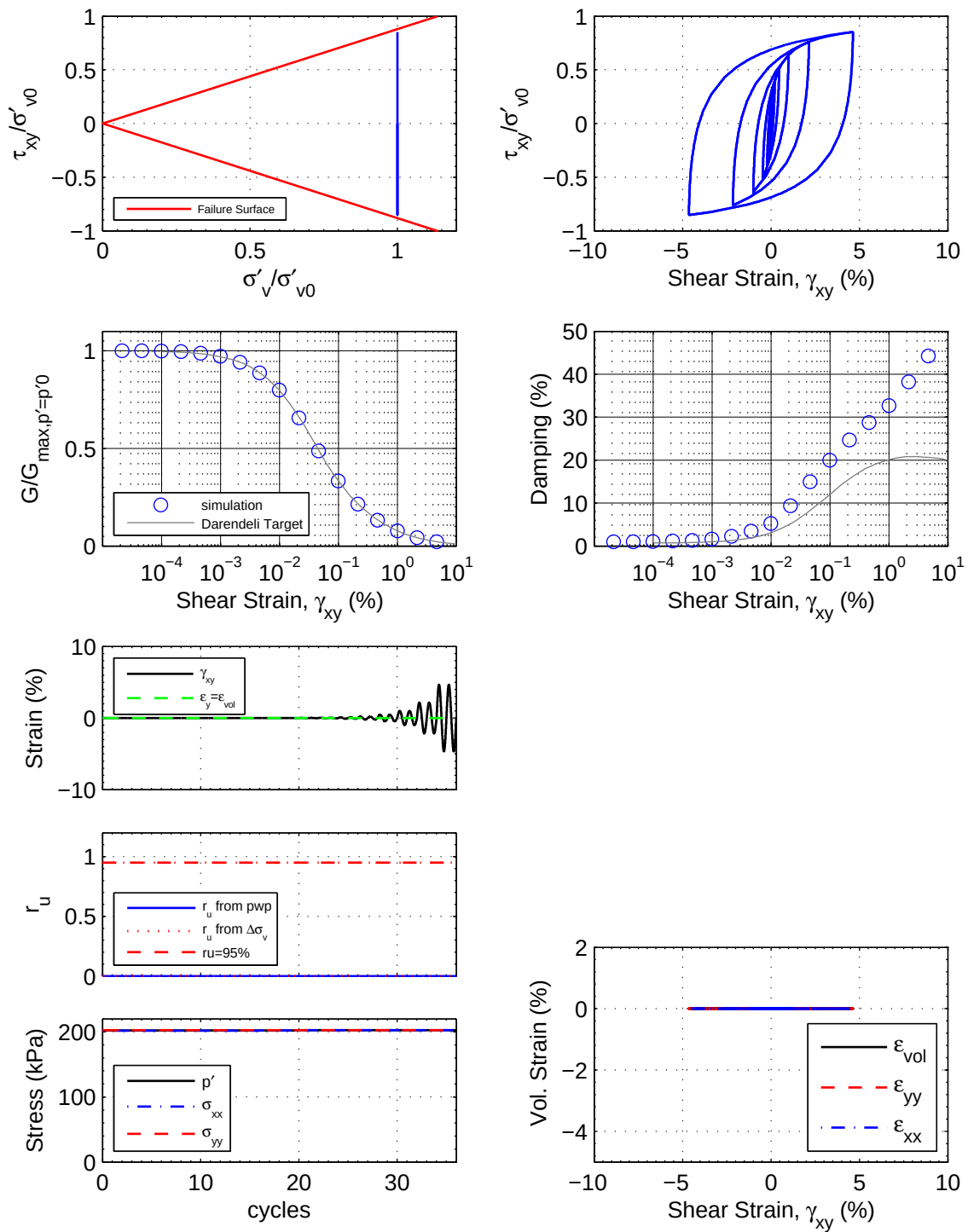


Figure C-66: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

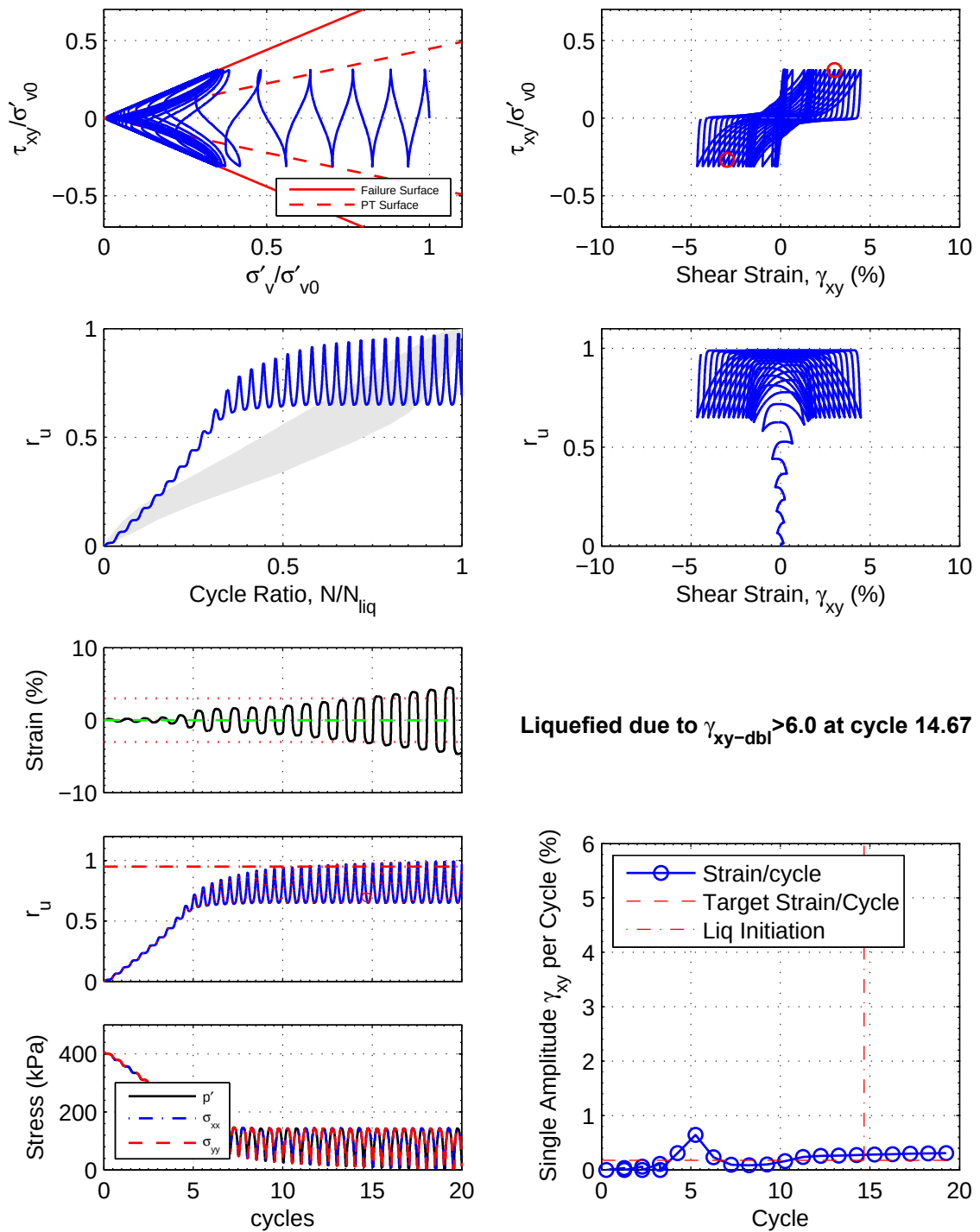


Figure C-67: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=4$ atm, $CSR=0.310$, $\phi=37.4$, $\phi_{PT}=20$

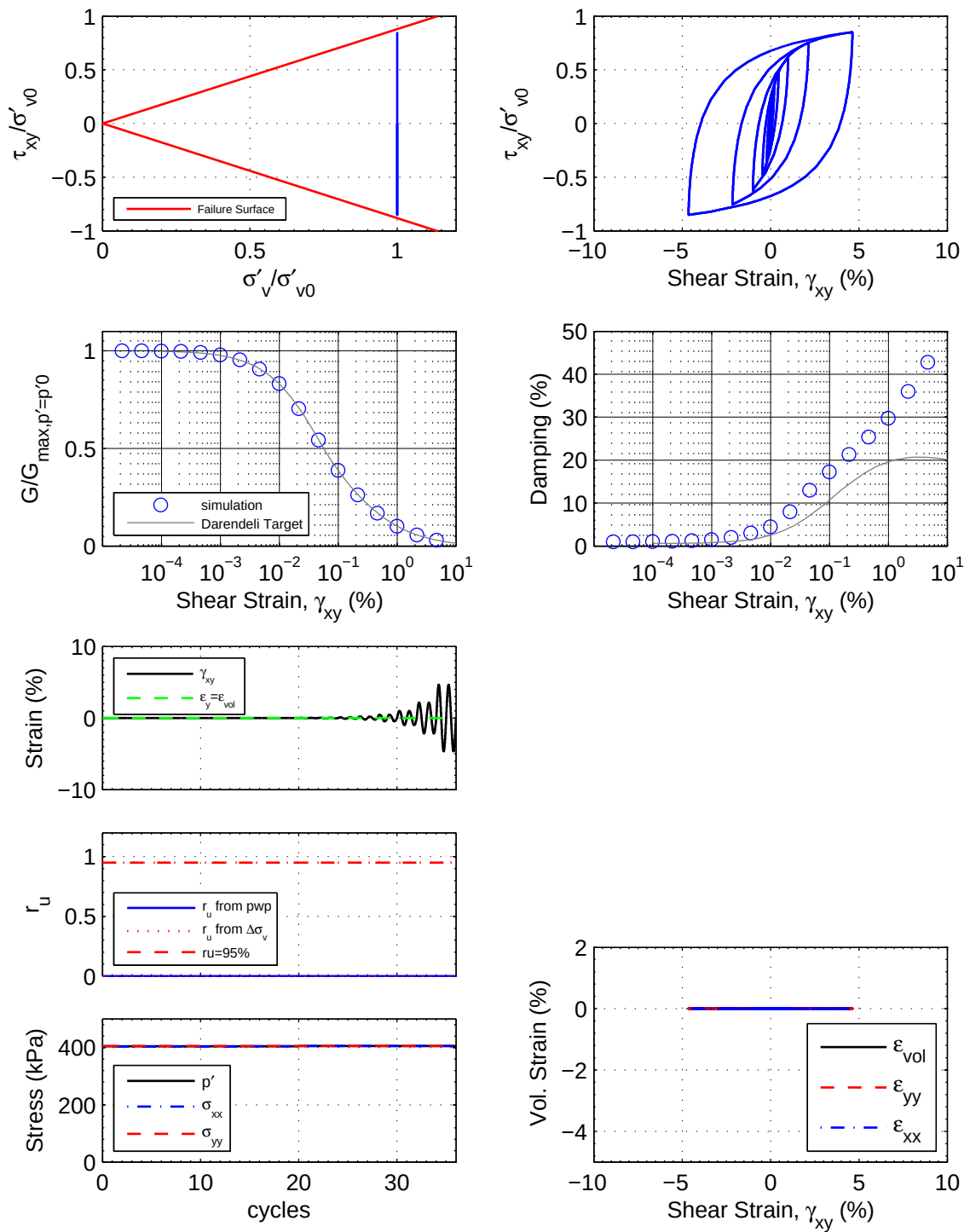


Figure C-68: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

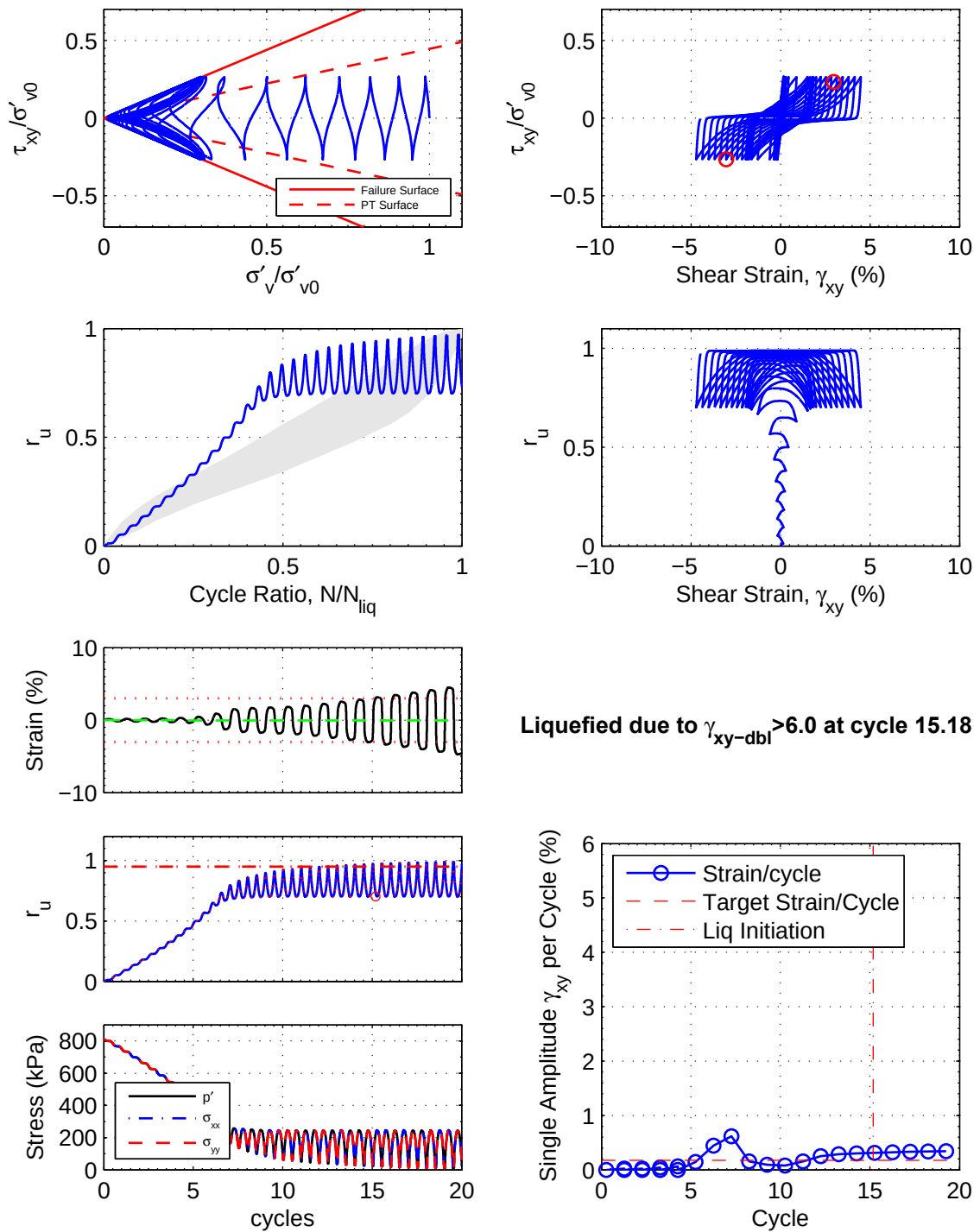


Figure C-69: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=8$ atm, $CSR=0.265$, $\phi=37.4$, $\phi_{PT}=20$

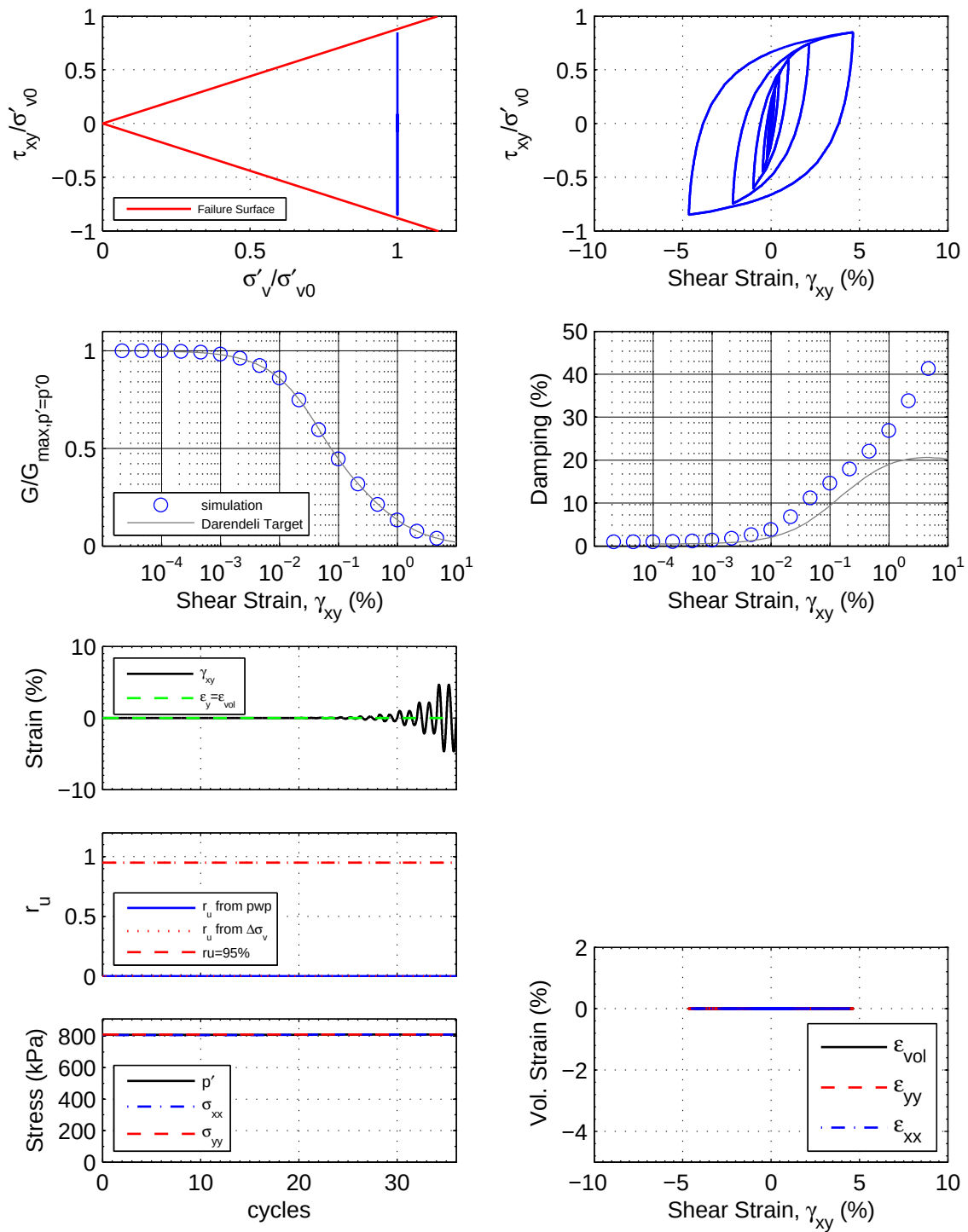


Figure C-70: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=8$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

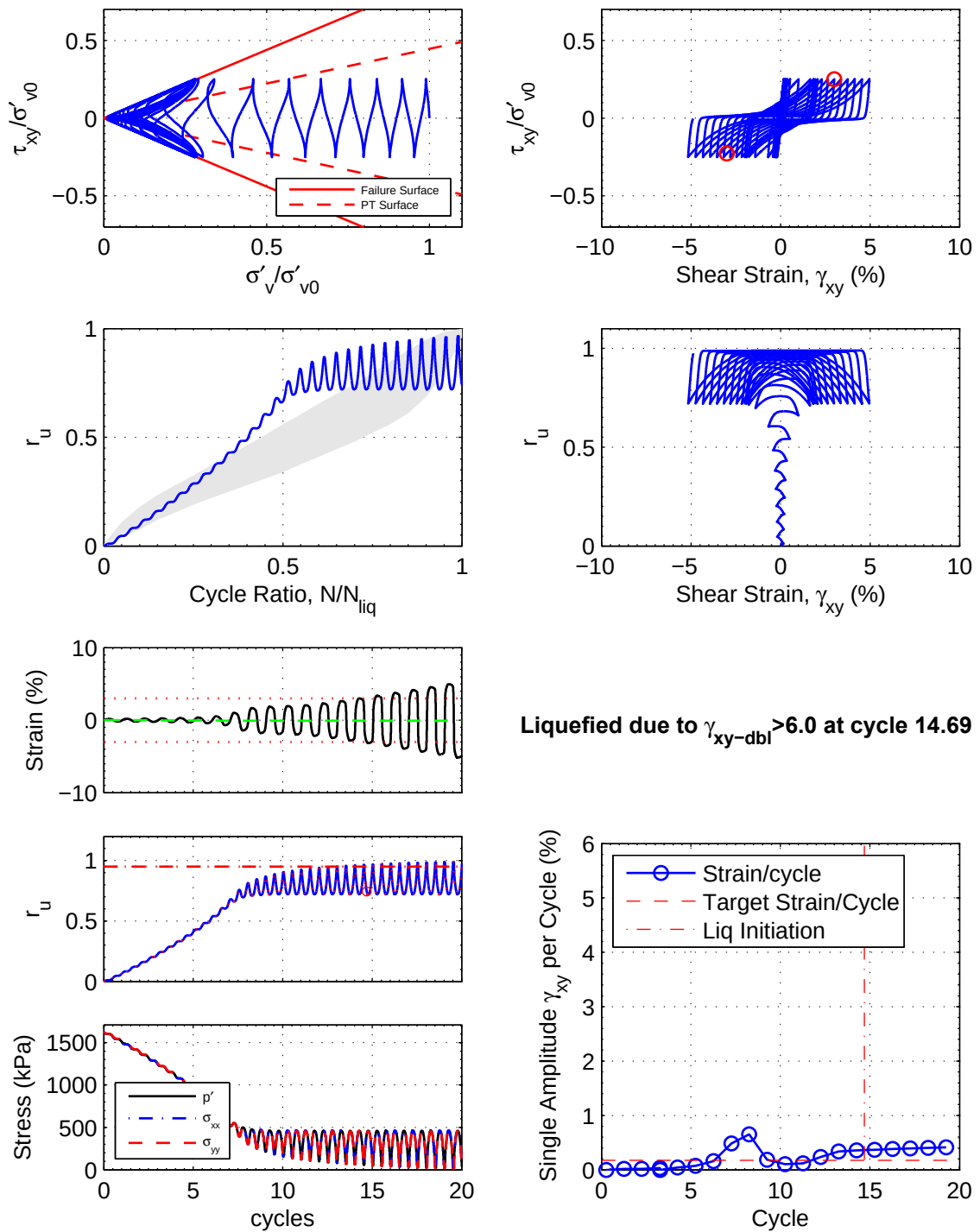


Figure C-71: Undrained CSS Simulation: PDMY2, $Dr=80$, $\sigma'_{v0}=16$ atm, $CSR=0.250$, $\phi=37.4$, $\phi_{PT}=20$

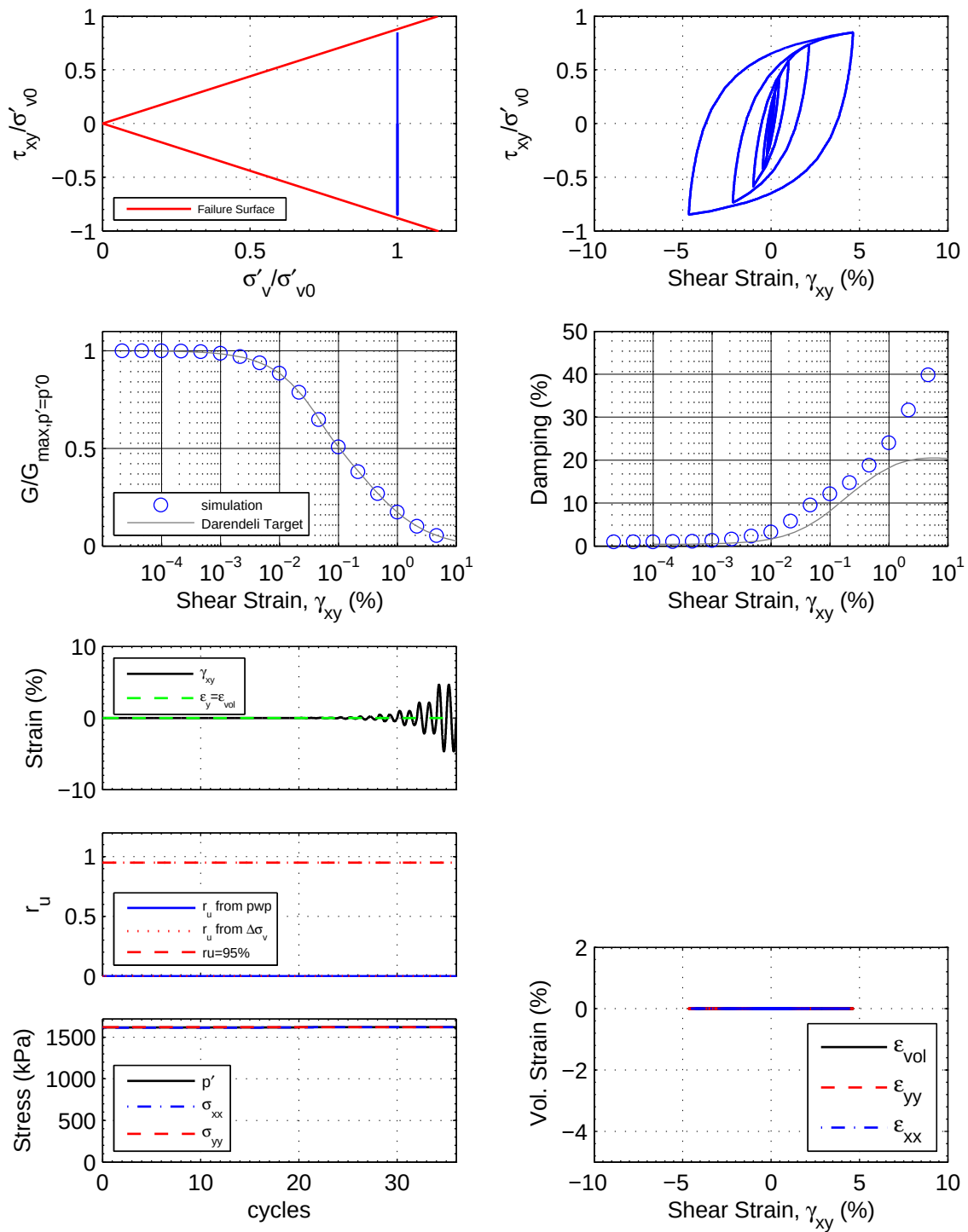


Figure C-72: Drained CSS Simulation: PIMY, $Dr=80$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=37.4$, $\phi_{PT}=na$

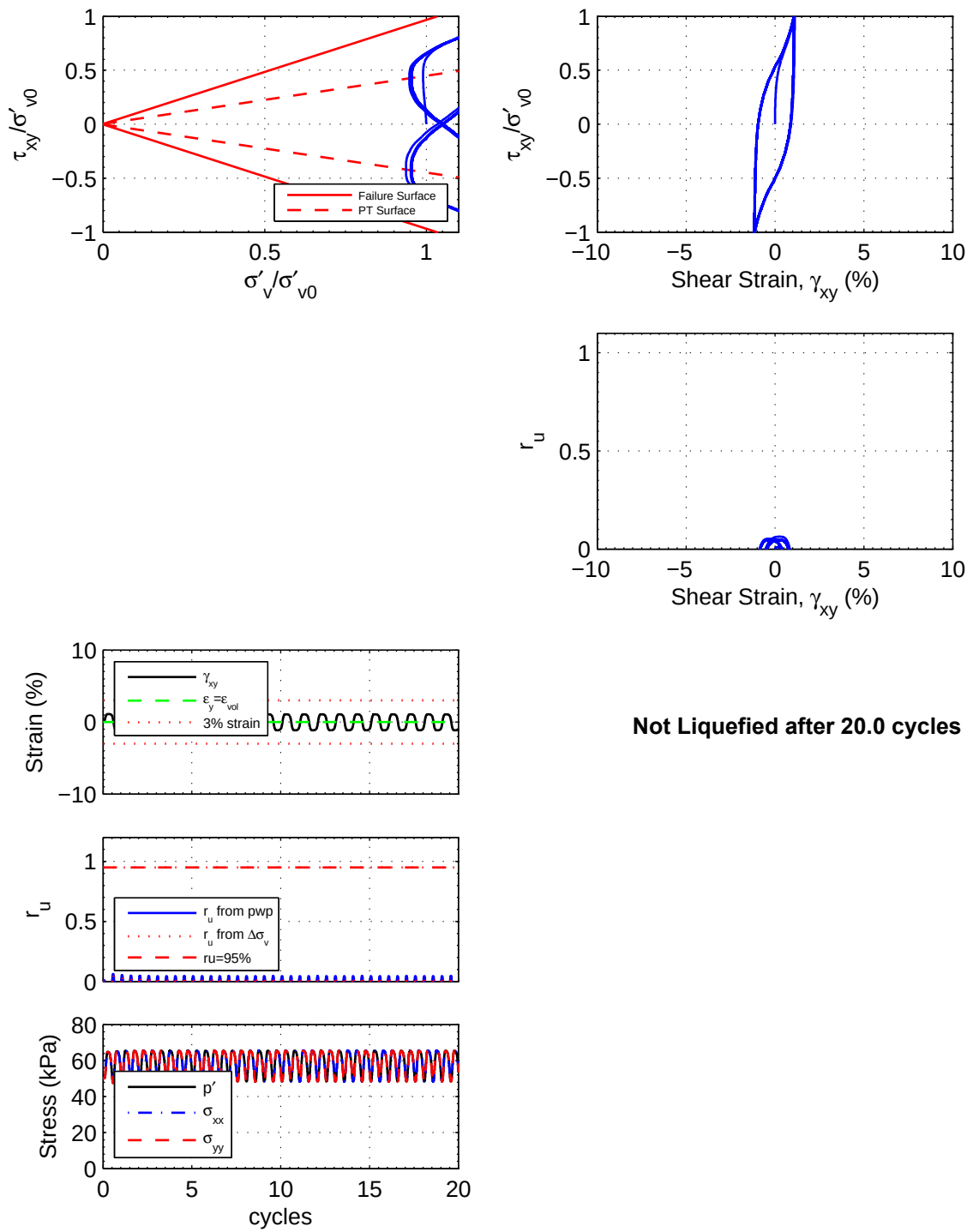


Figure C-73: Undrained CSS Simulation: PDMY2, $Dr=90$, $\sigma'_{v0}=0.5$ atm, $CSR=1p_008$, $\phi=40.8$, $\phi_{PT}=20$

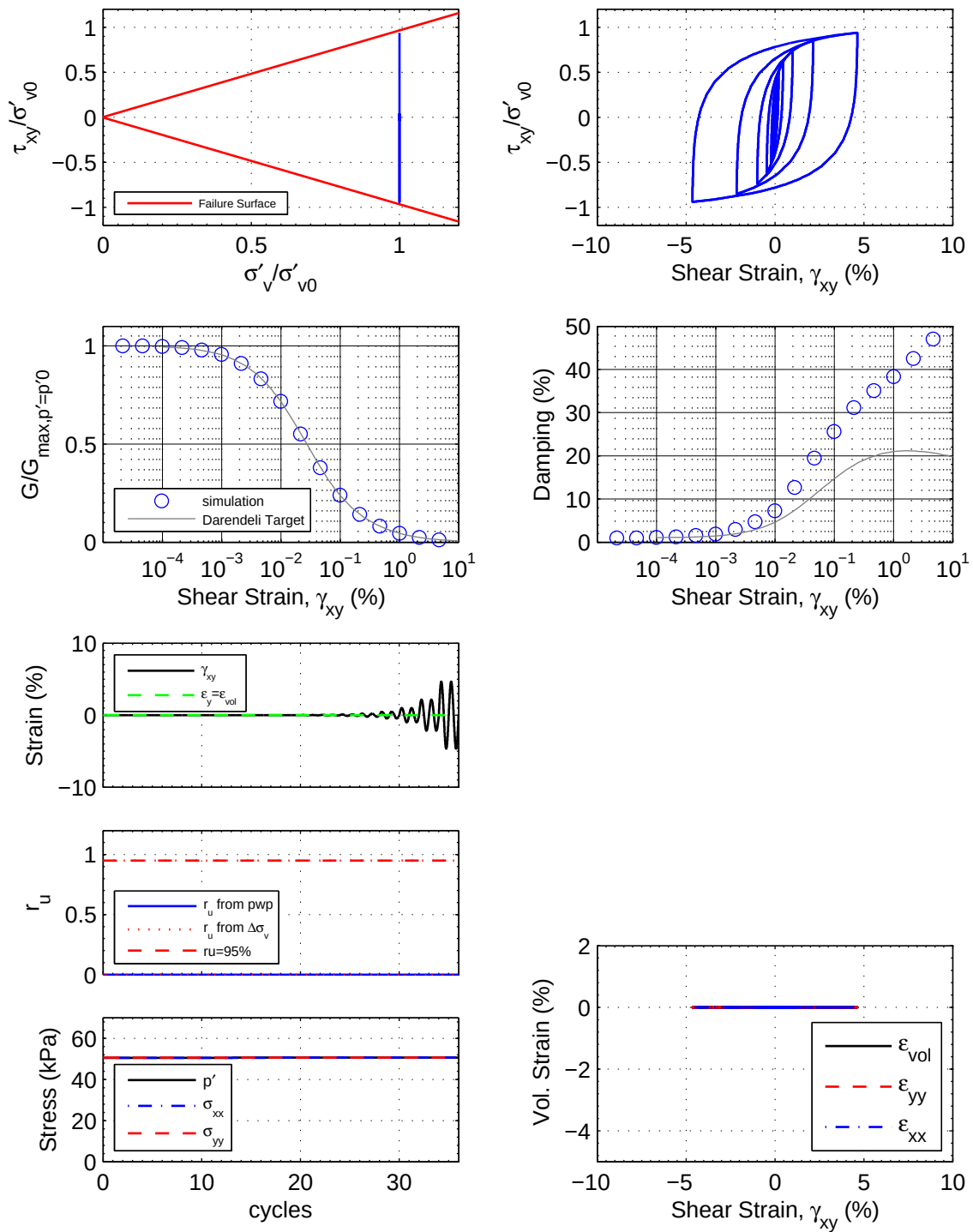


Figure C-74: Drained CSS Simulation: PIMY, $Dr=90$, $\sigma'_{v0}=0.5$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$

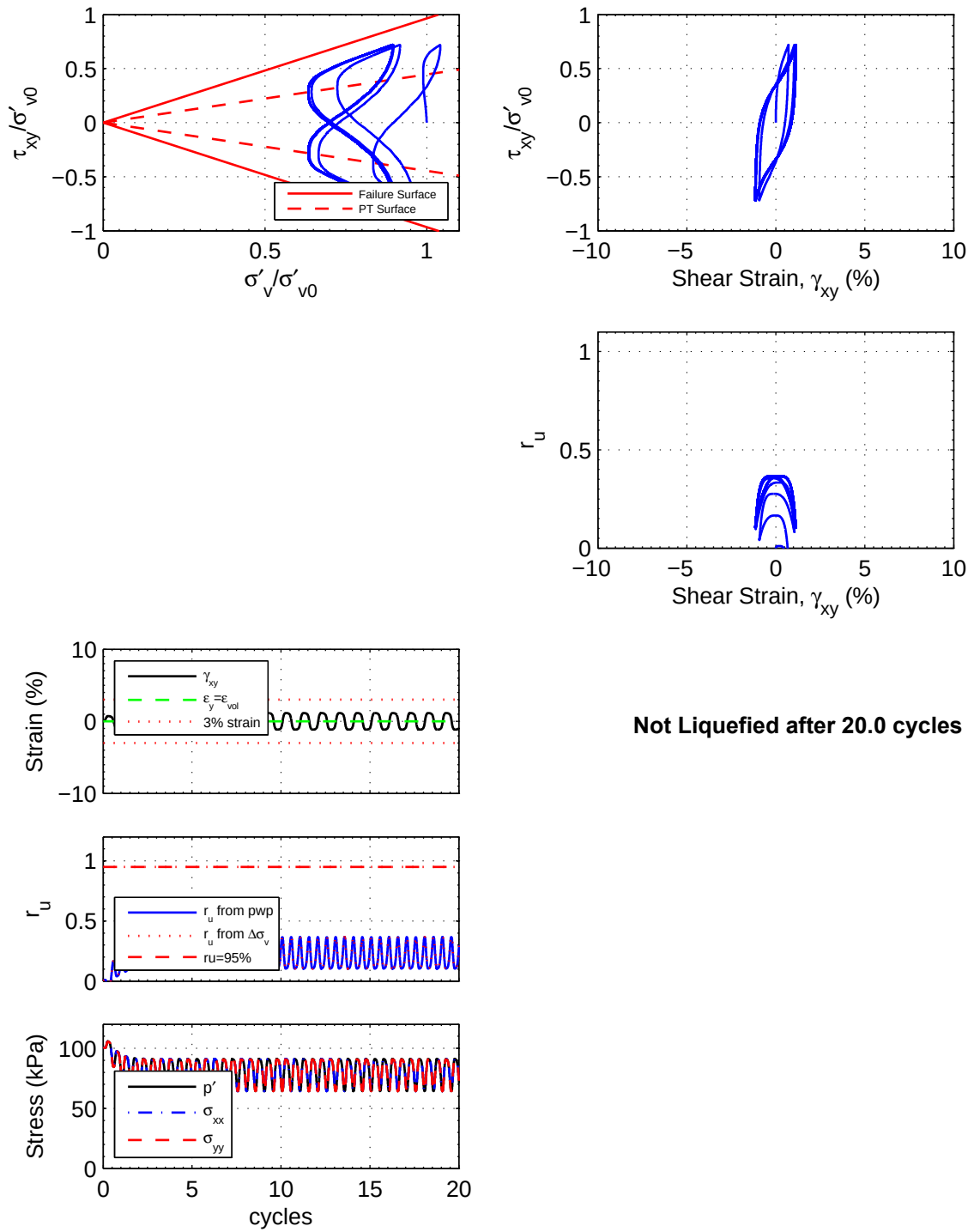


Figure C-75: Undrained CSS Simulation: PDMY2, $D_r=90$, $\sigma'_{v0}=1$ atm, $CSR=0.720$, $\phi=40.8$, $\phi_{PT}=20$

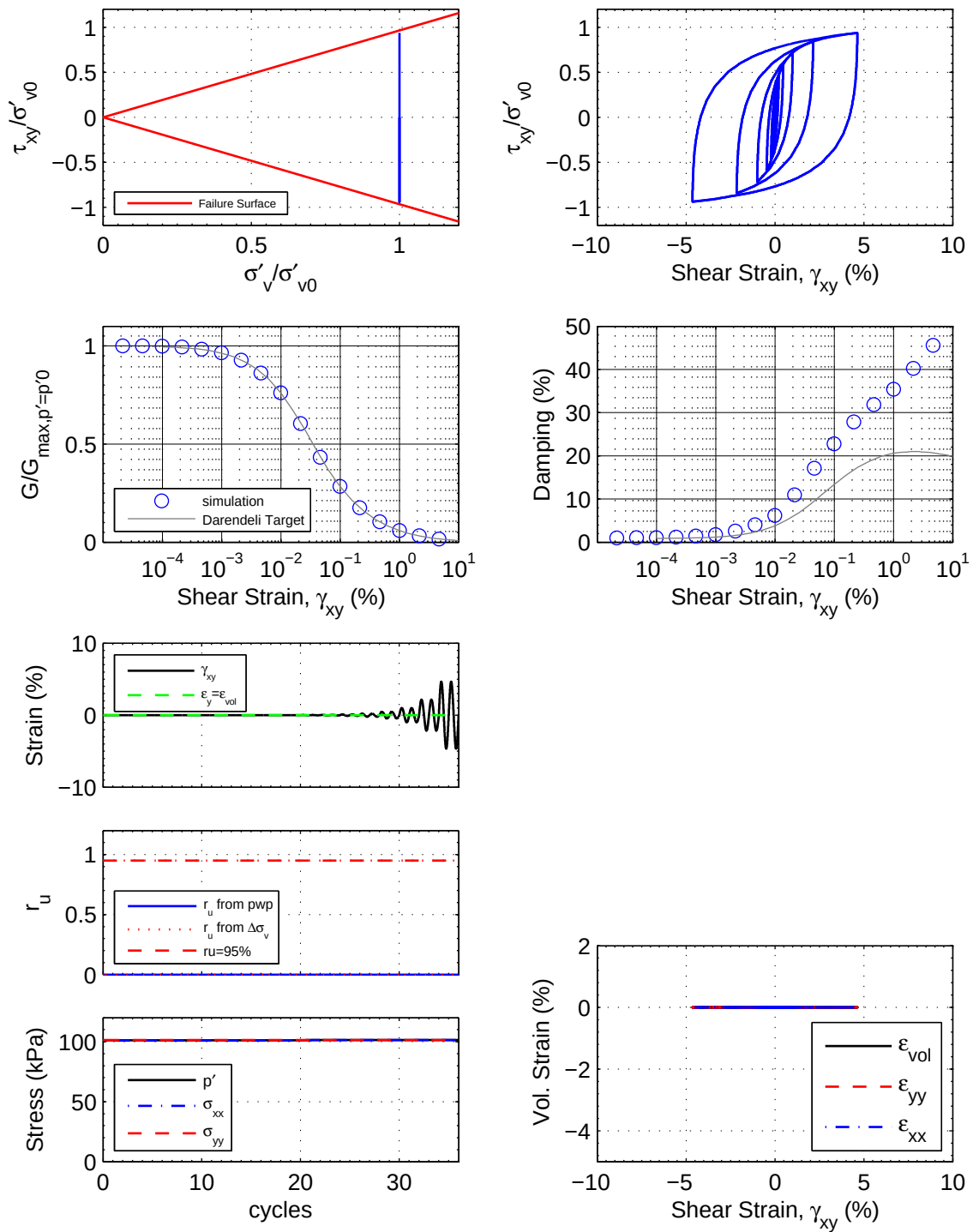


Figure C-76: Drained CSS Simulation: PIMY, $Dr=90$, $\sigma'_{v0}=1$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$

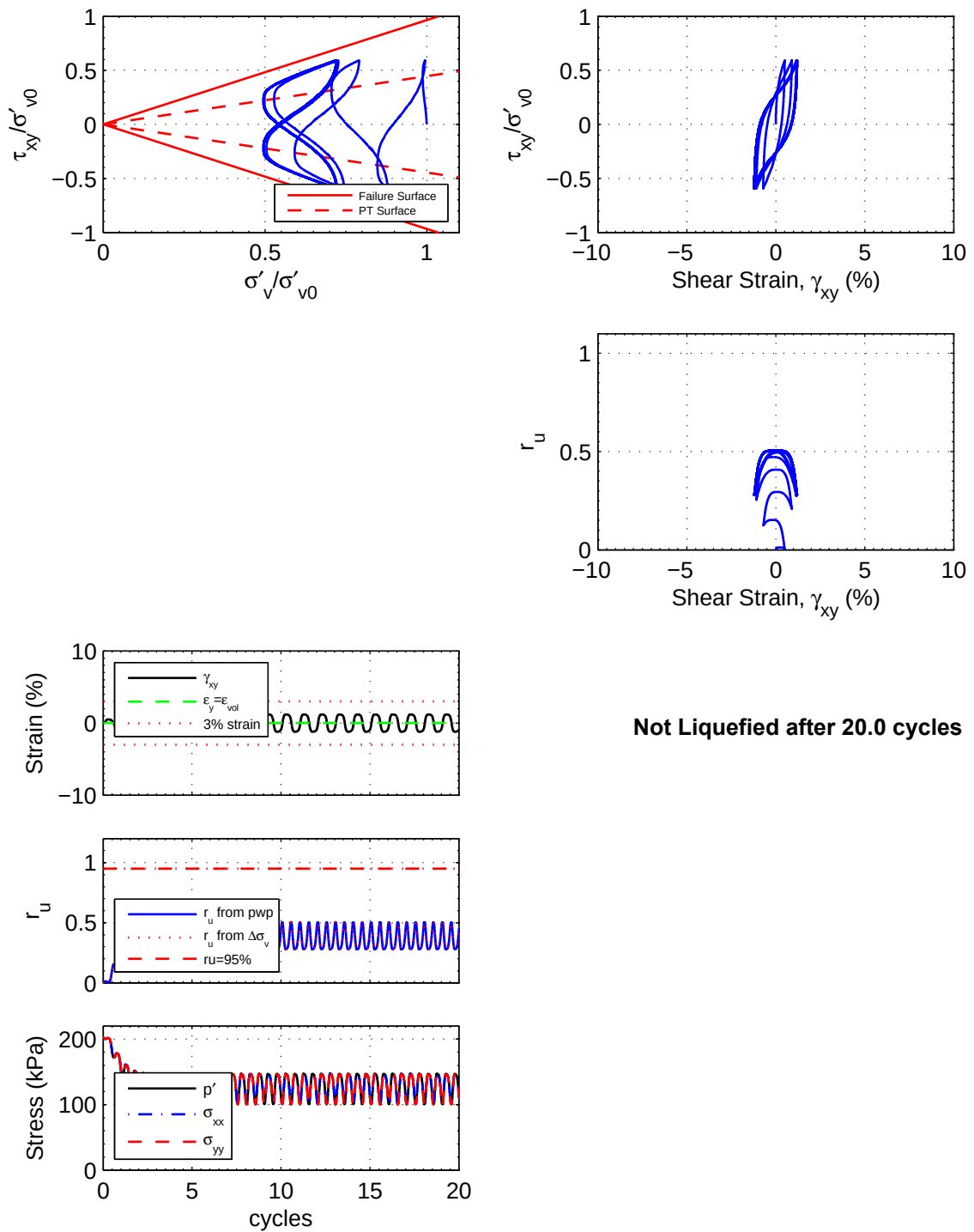


Figure C-77: Undrained CSS Simulation: PDMY2, $D_r=90$, $\sigma'_{v0}=2$ atm, $CSR=0.592$, $\phi=40.8$, $\phi_{PT}=20$

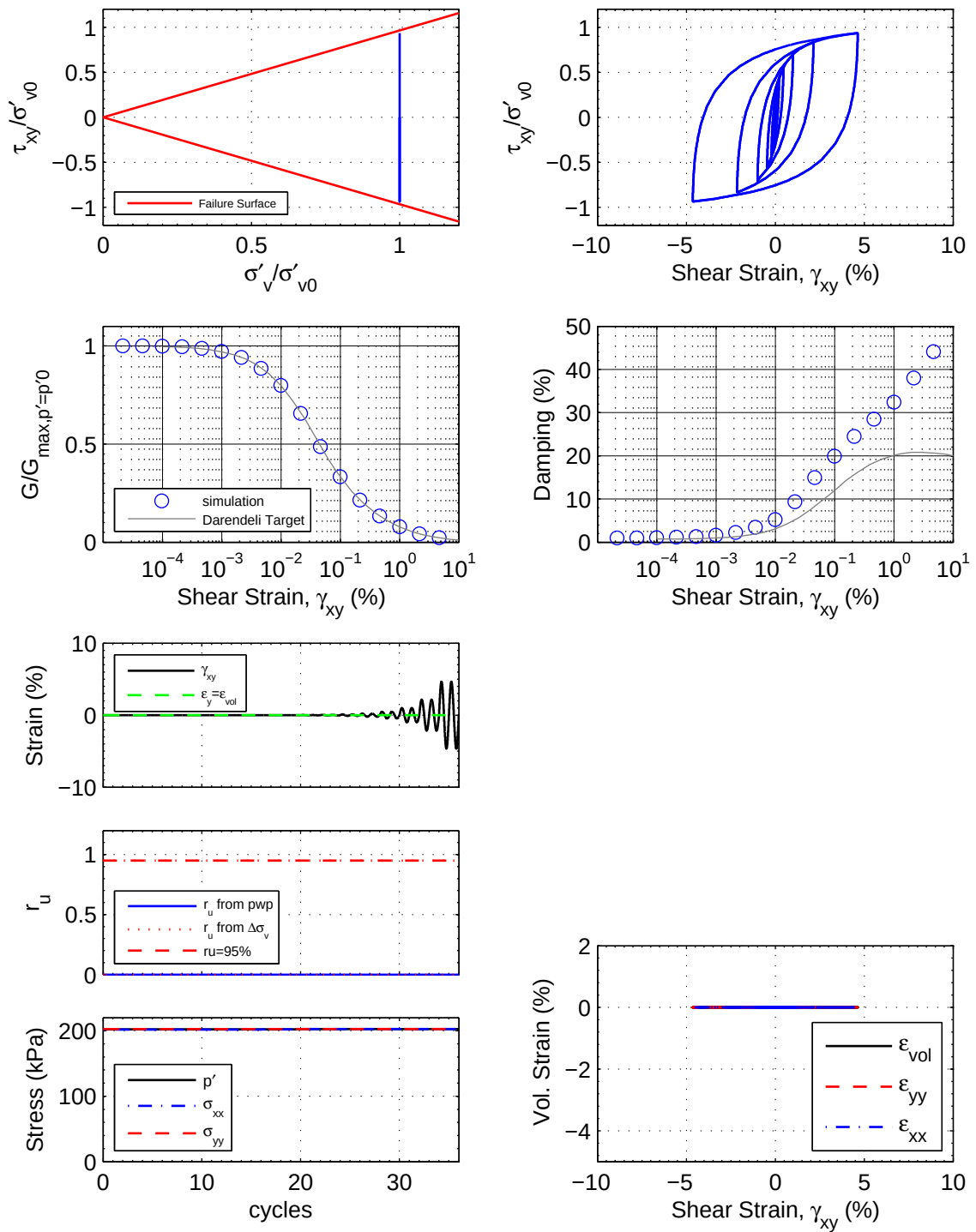


Figure C-78: Drained CSS Simulation: PIMY, $\text{Dr}=90$, $\sigma'_{v0}=2$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$

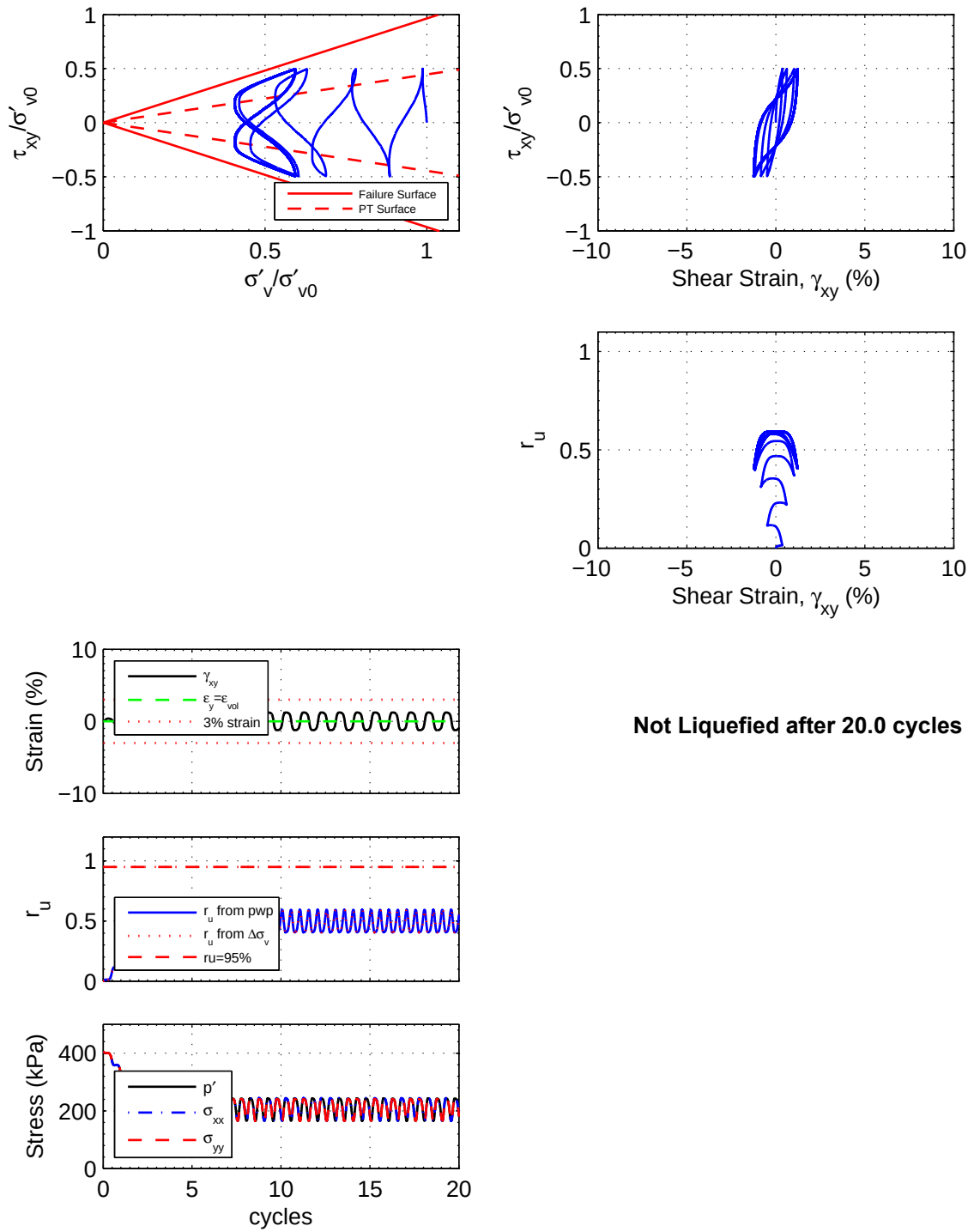


Figure C-79: Undrained CSS Simulation: PDMY2, $Dr=90$, $\sigma'_{v0}=4$ atm, $CSR=0.496$, $\phi=40.8$, $\phi_{PT}=20$

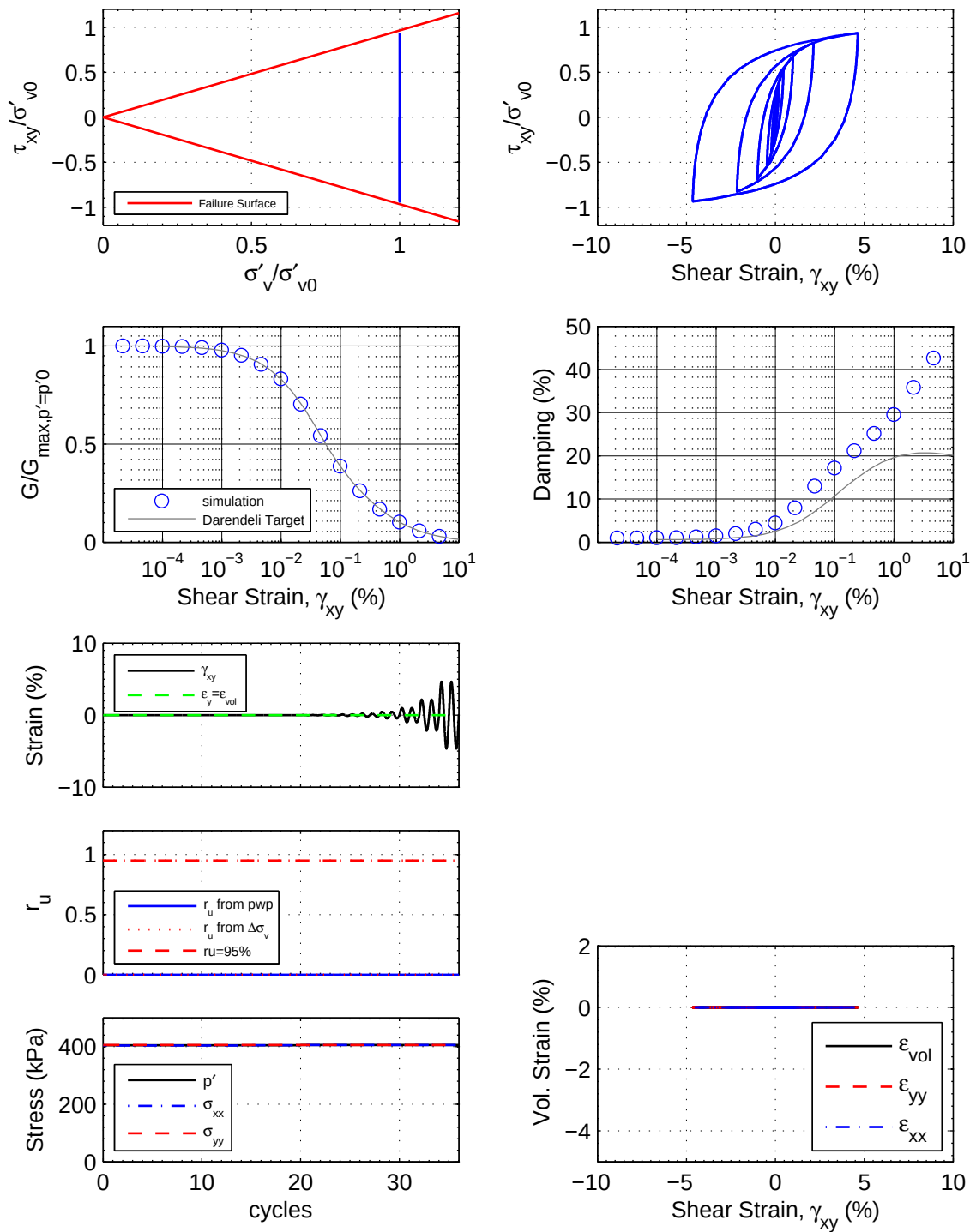


Figure C-80: Drained CSS Simulation: PIMY, $Dr=90$, $\sigma'_{v0}=4$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$

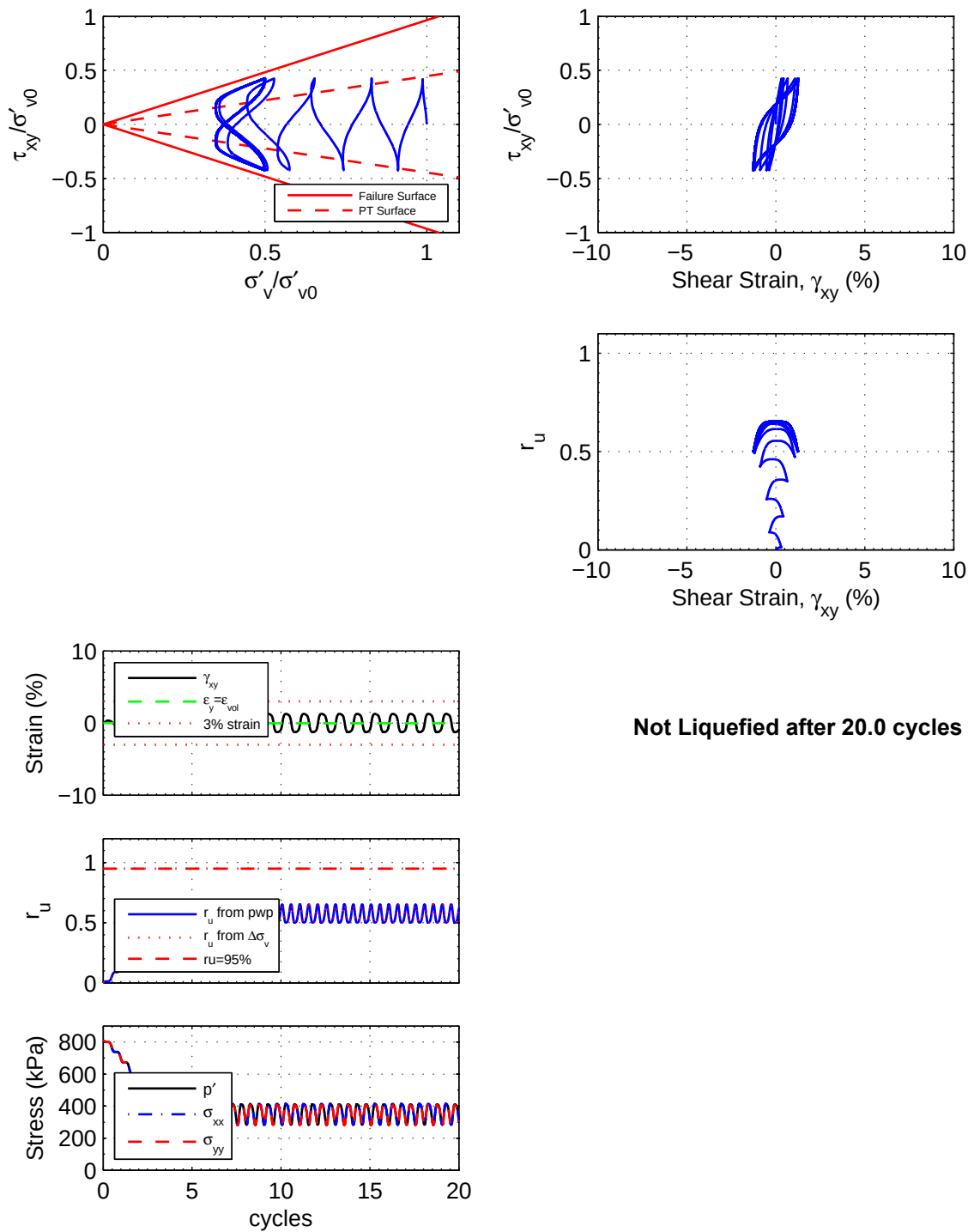


Figure C-81: Undrained CSS Simulation: PDMY2, $D_r=90$, $\sigma'_{v0}=8$ atm, $CSR=0.424$, $\phi=40.8$, $\phi_{PT}=20$

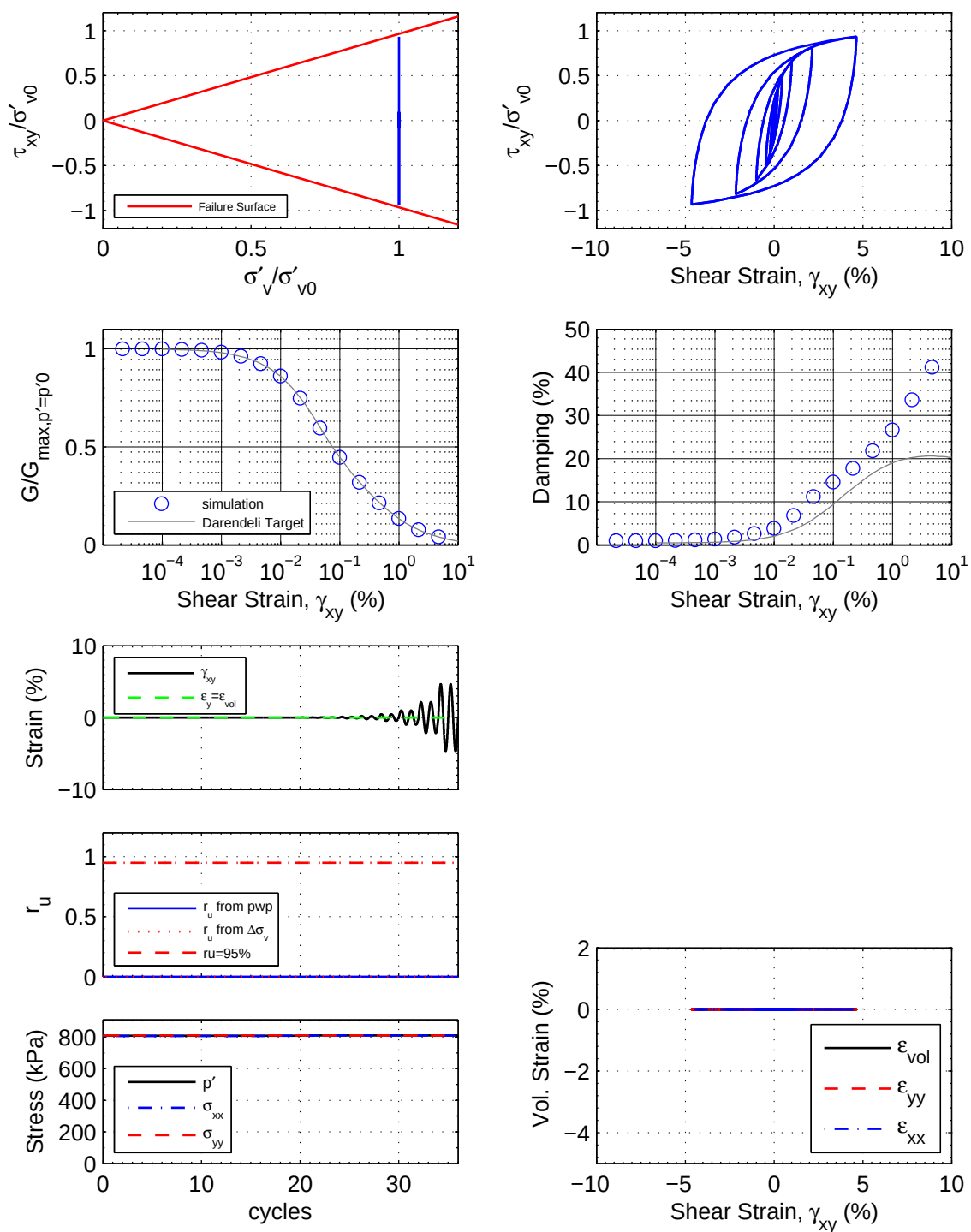


Figure C-82: Drained CSS Simulation: PIMY, $Dr=90$, $\sigma'_{v0}=8$ atm, $\phi=40.8$, $\phi_{PT}=na$

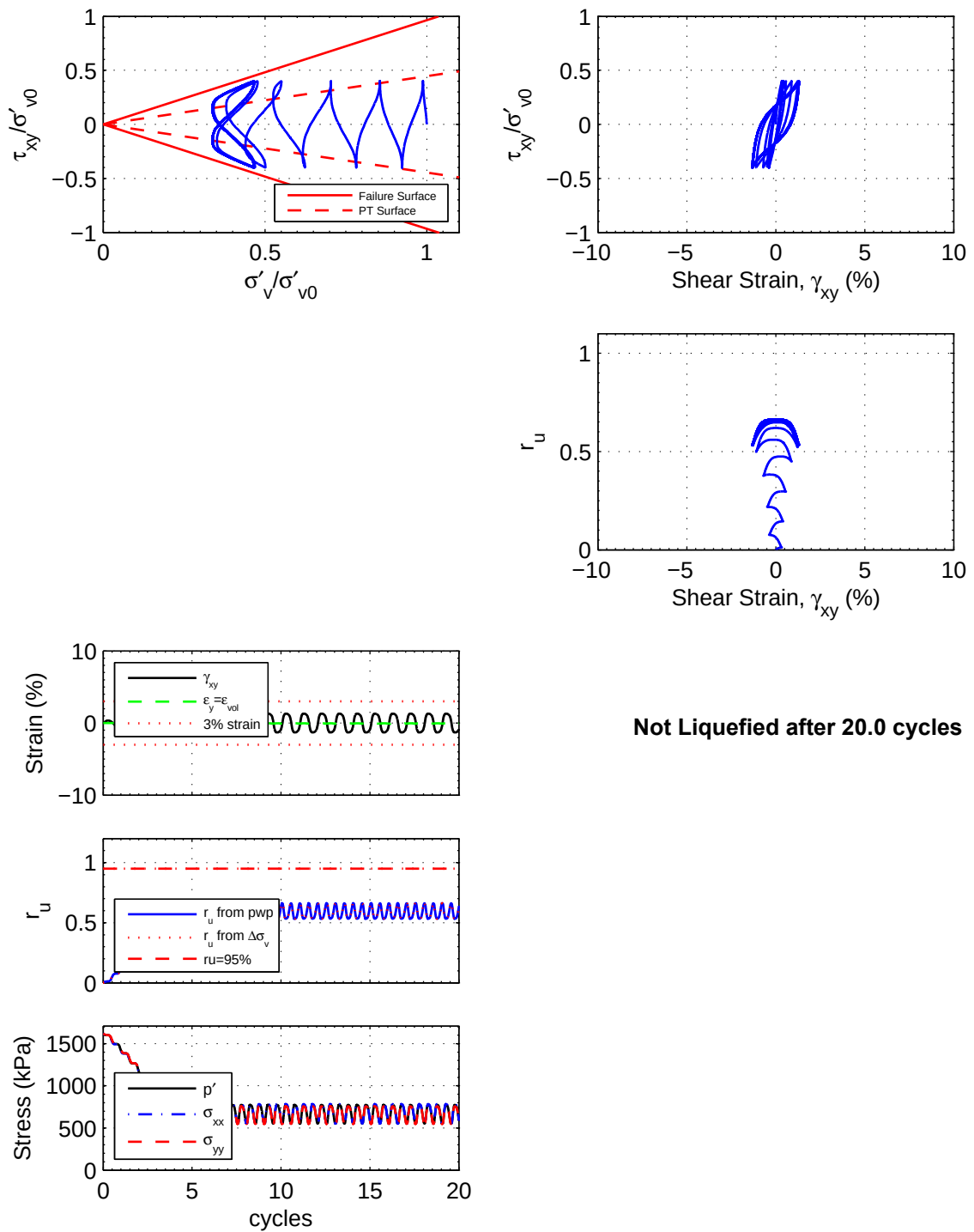


Figure C-83: Undrained CSS Simulation: PDMY2, $Dr=90$, $\sigma'_{v0}=16$ atm, $CSR=0.400$, $\phi=40.8$, $\phi_{PT}=20$

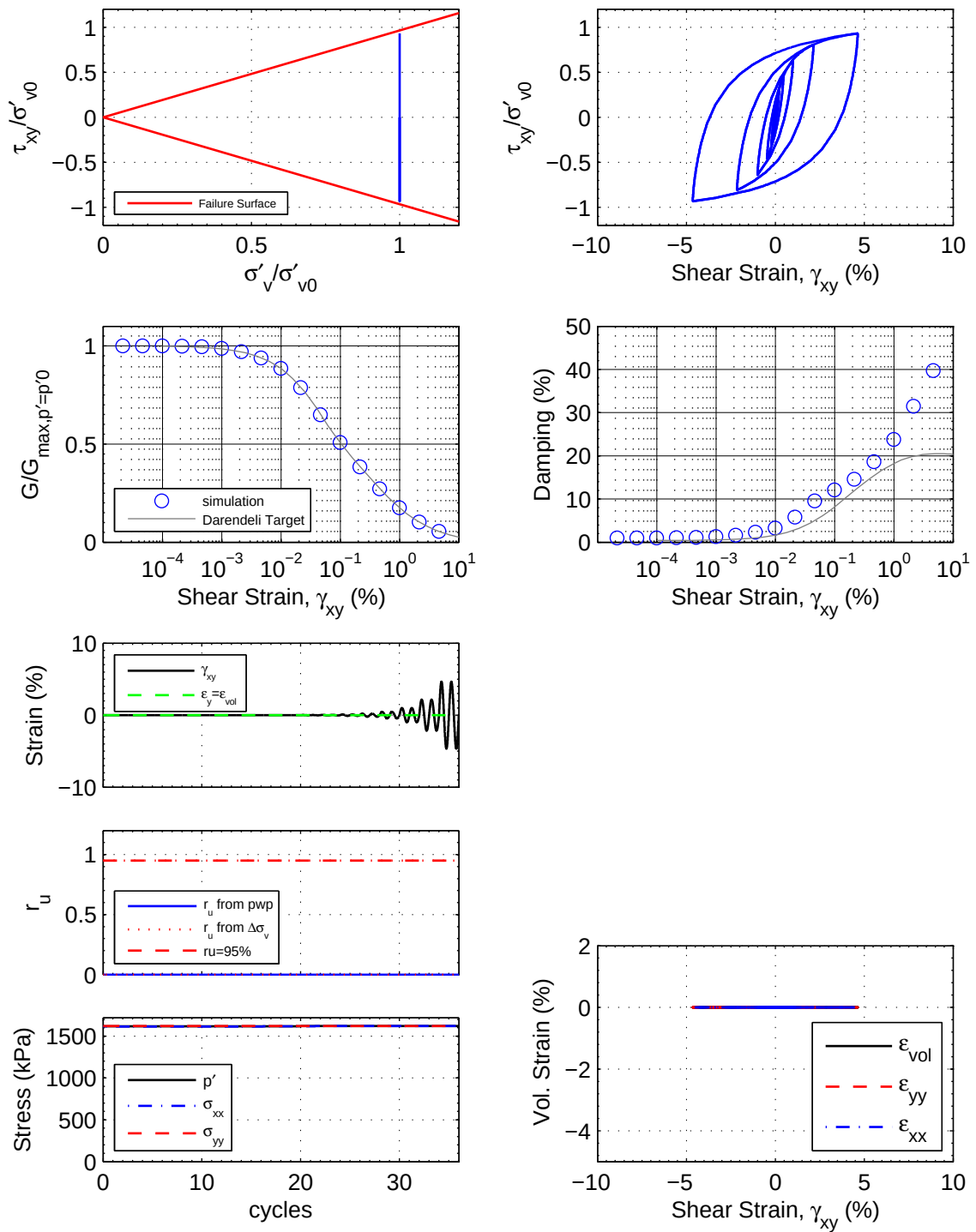


Figure C-84: Drained CSS Simulation: PIMY, $Dr=90$, $\sigma'_{v0}=16$ atm, CyclicStrain, $\phi=40.8$, $\phi_{PT}=na$

Appendix D

Input Ground Motions

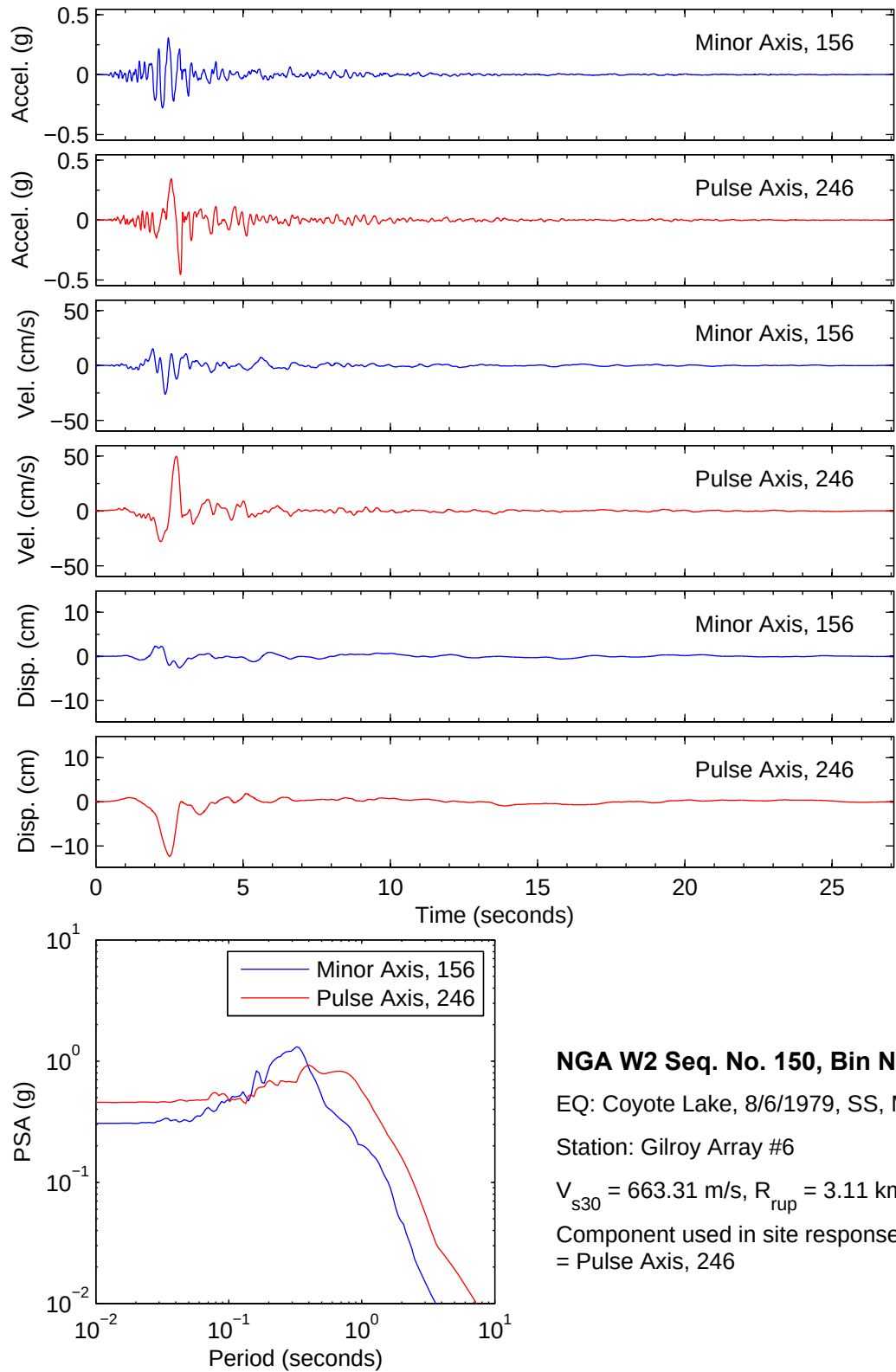


Figure D-1: Input motion NGA W2 Seq. No. 150, Bin No. 1

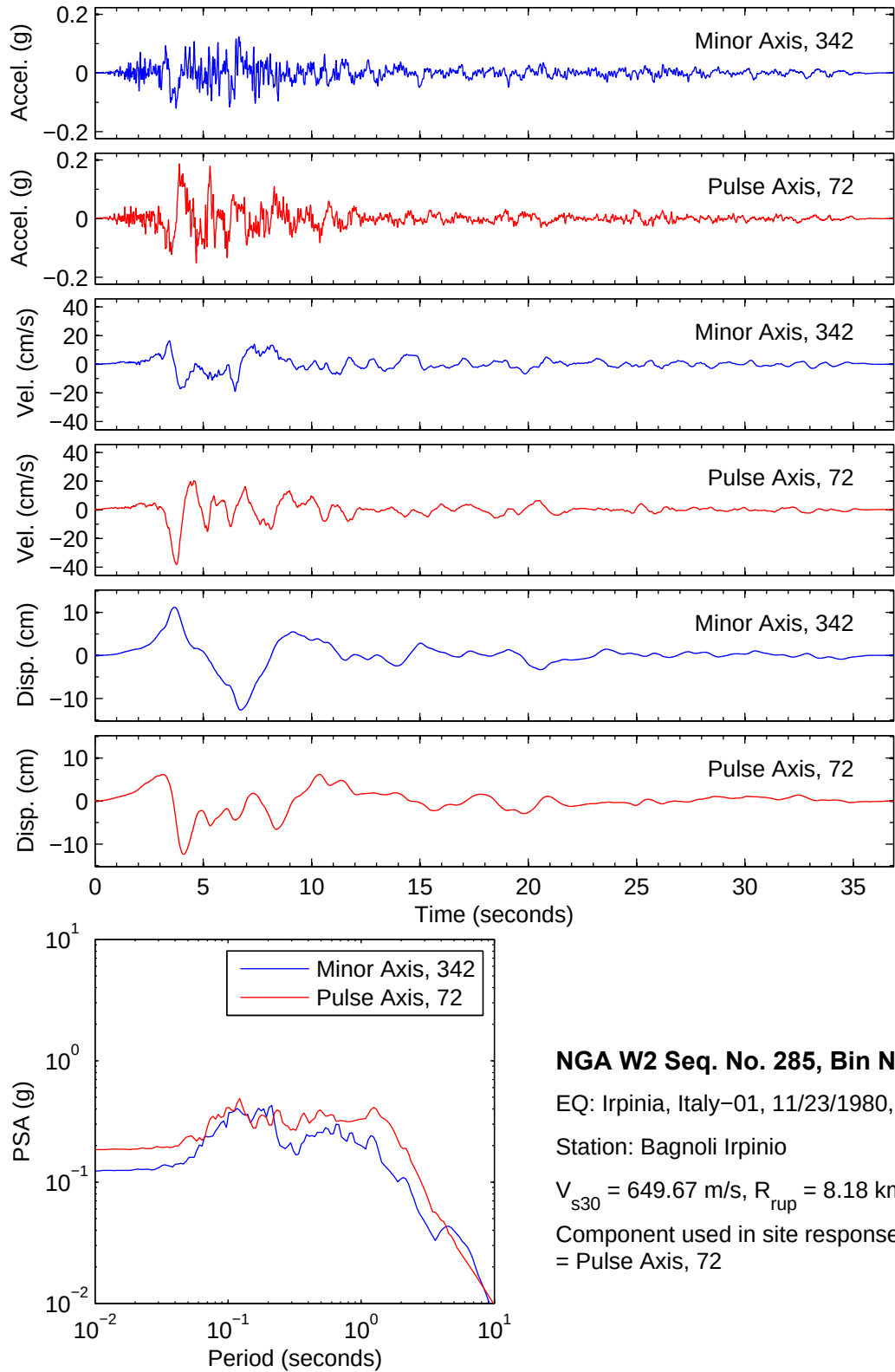


Figure D-2: Input motion NGA W2 Seq. No. 285, Bin No. 1

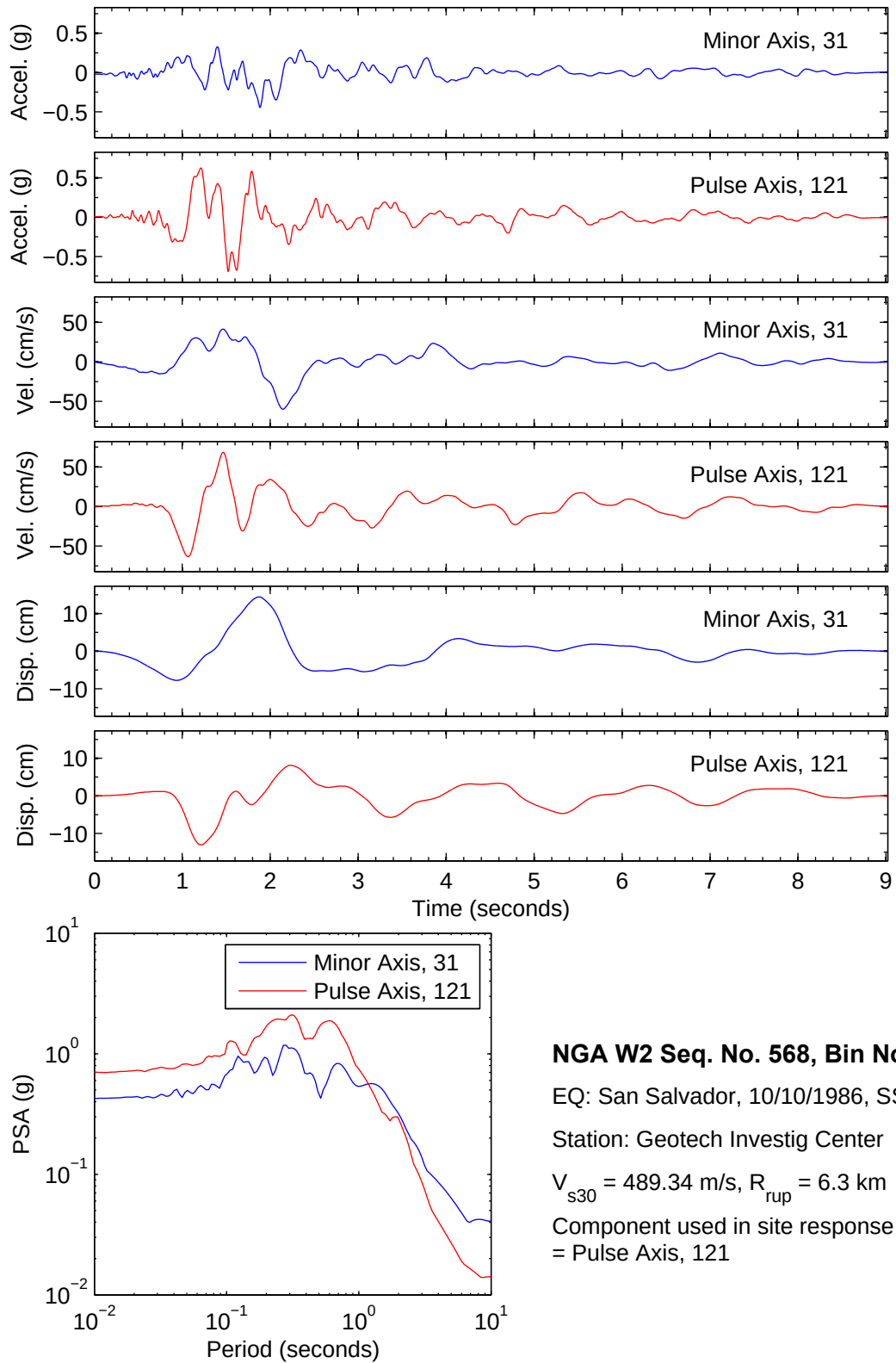


Figure D-3: Input motion NGA W2 Seq. No. 568, Bin No. 1

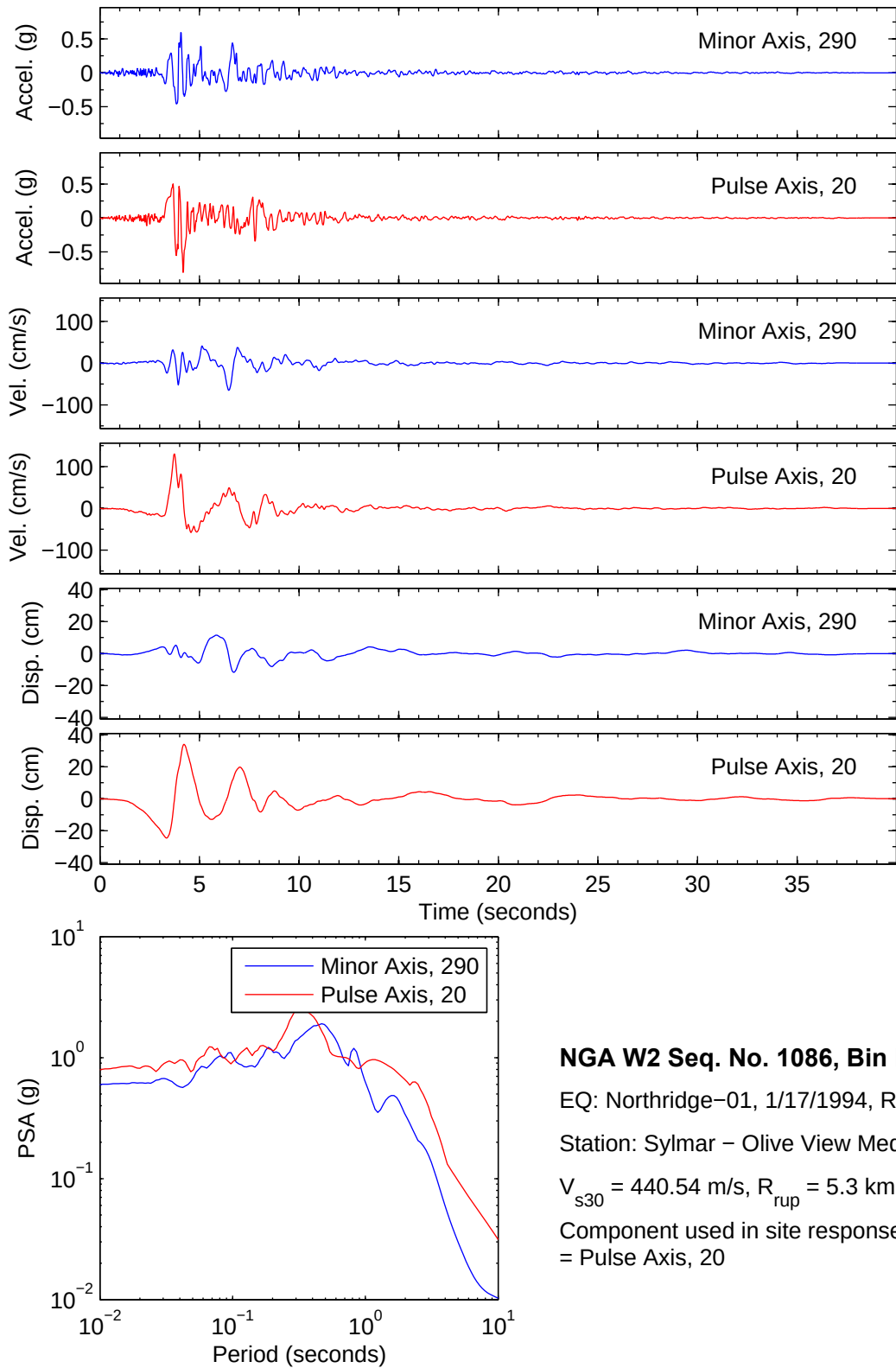


Figure D-4: Input motion NGA W2 Seq. No. 1086, Bin No. 1

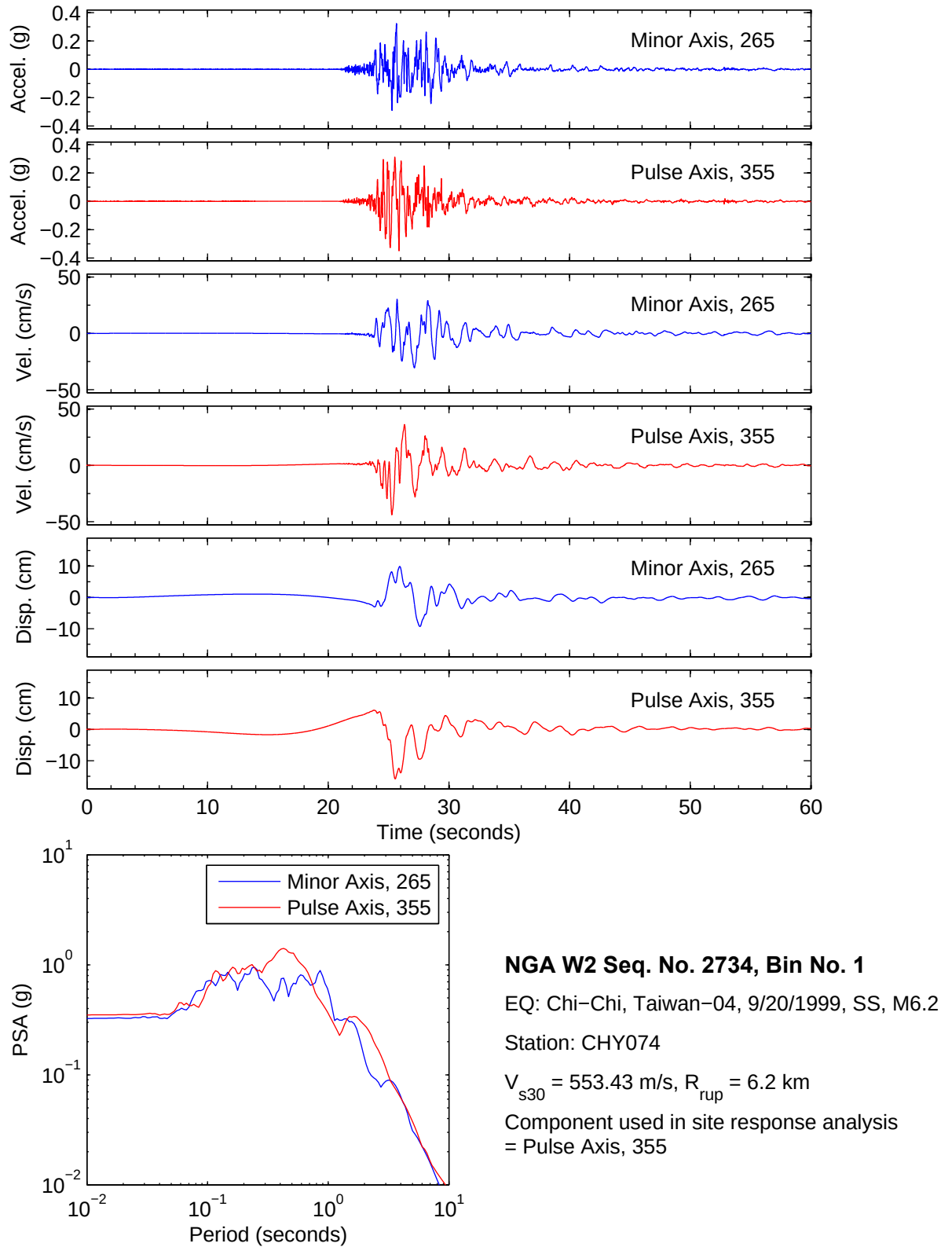


Figure D-5: Input motion NGA W2 Seq. No. 2734, Bin No. 1

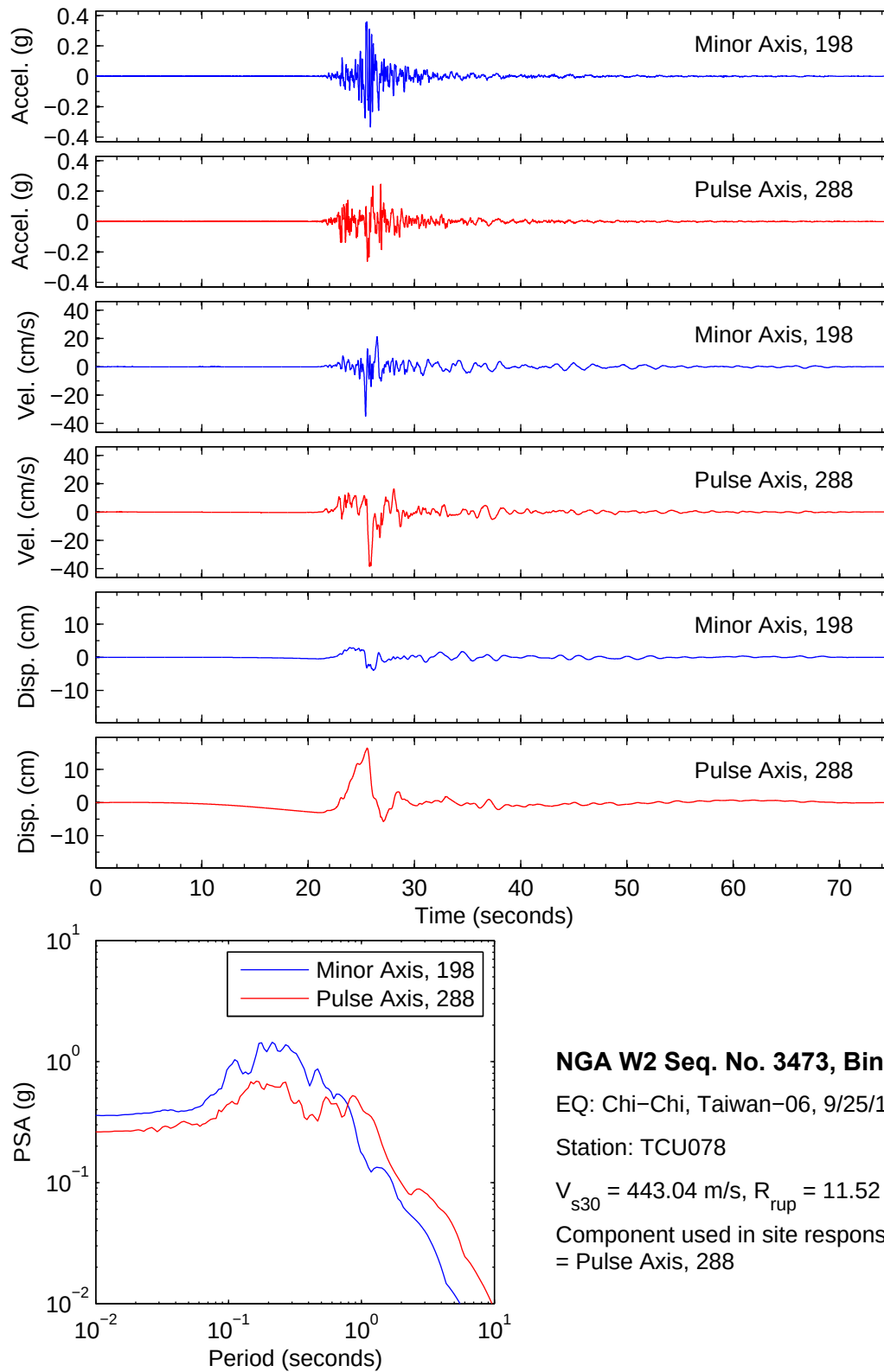


Figure D-6: Input motion NGA W2 Seq. No. 3473, Bin No. 1

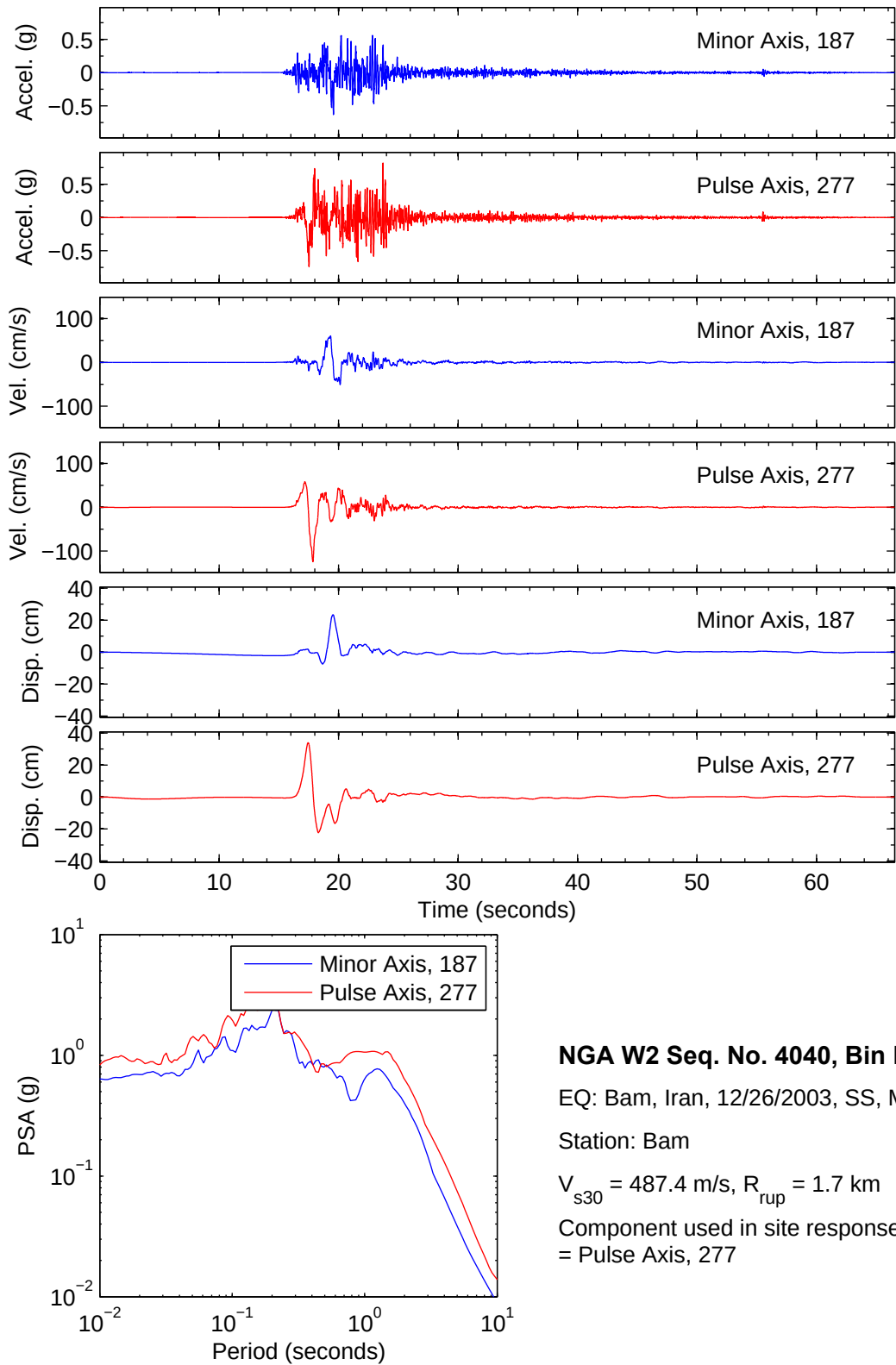


Figure D-7: Input motion NGA W2 Seq. No. 4040, Bin No. 1

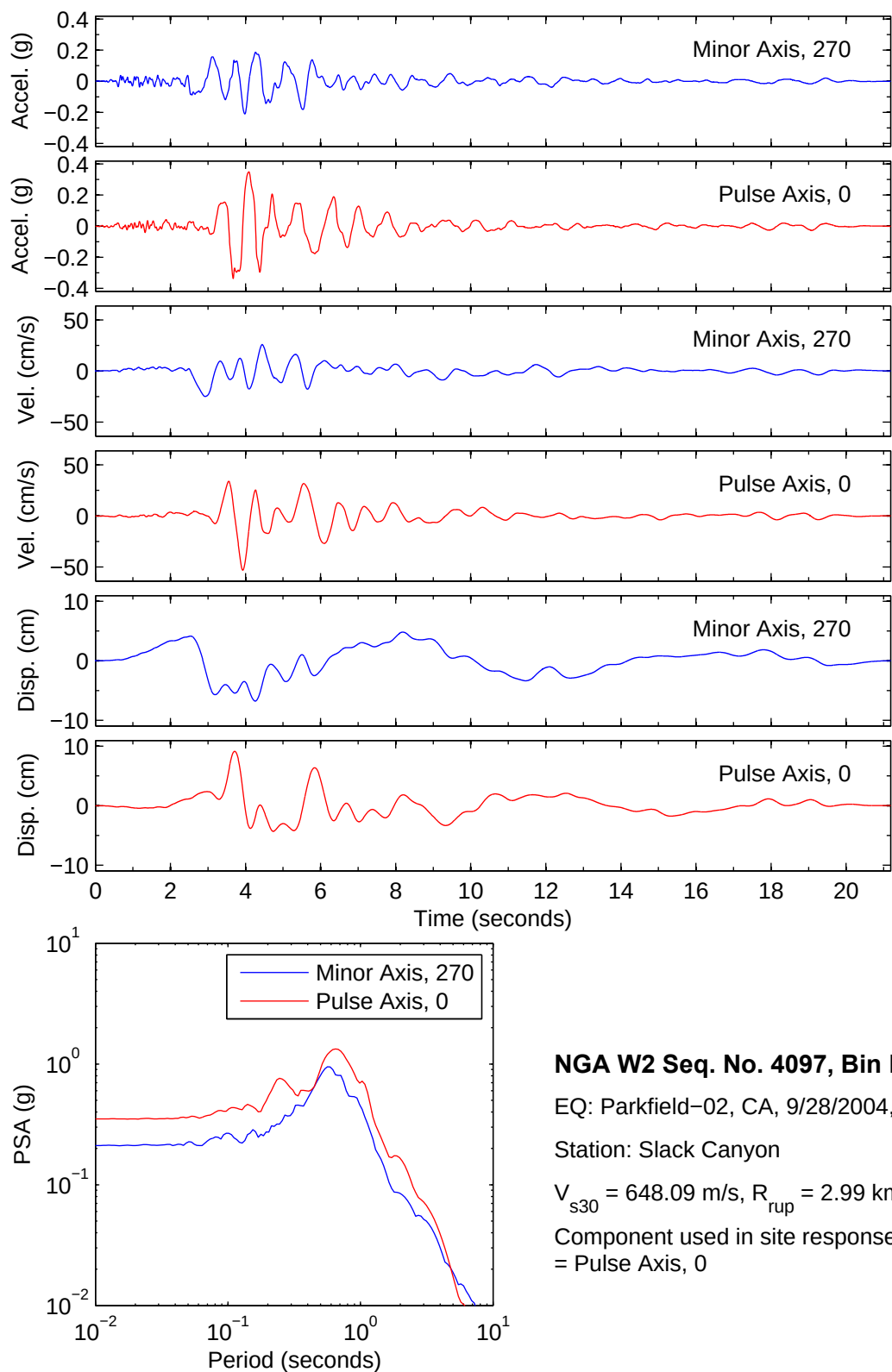


Figure D-8: Input motion NGA W2 Seq. No. 4097, Bin No. 1

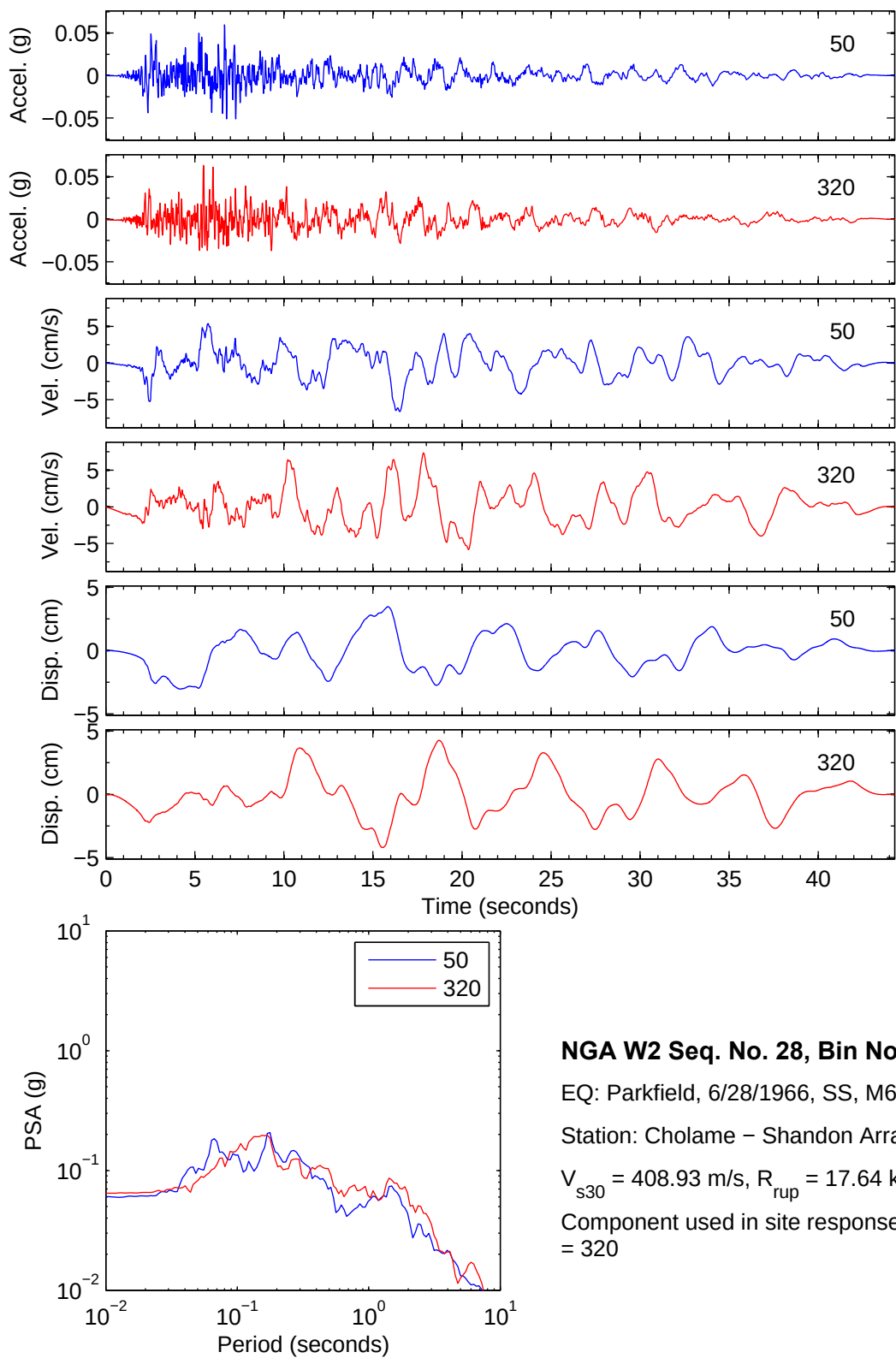


Figure D-9: Input motion NGA W2 Seq. No. 28, Bin No. 2

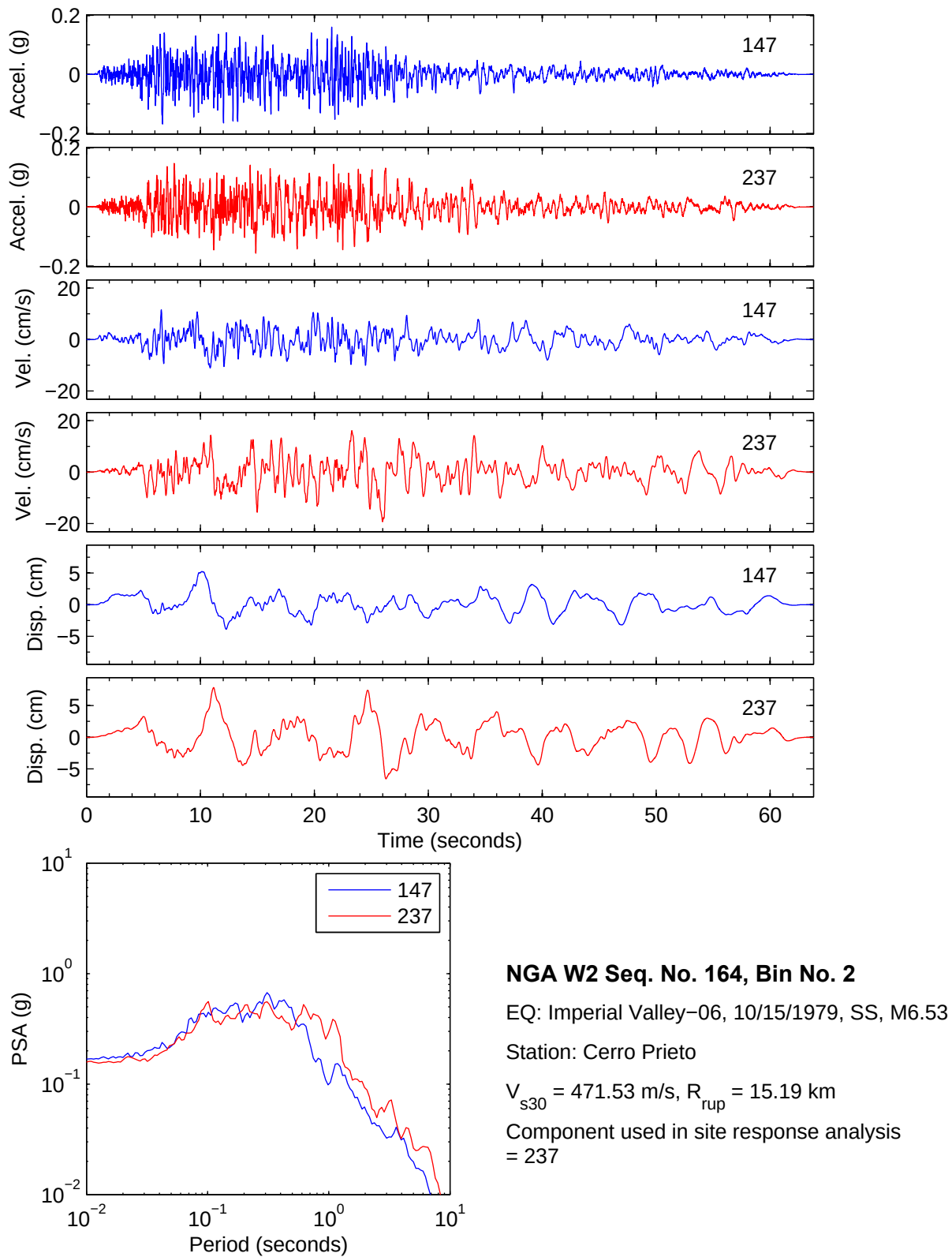


Figure D-10: Input motion NGA W2 Seq. No. 164, Bin No. 2

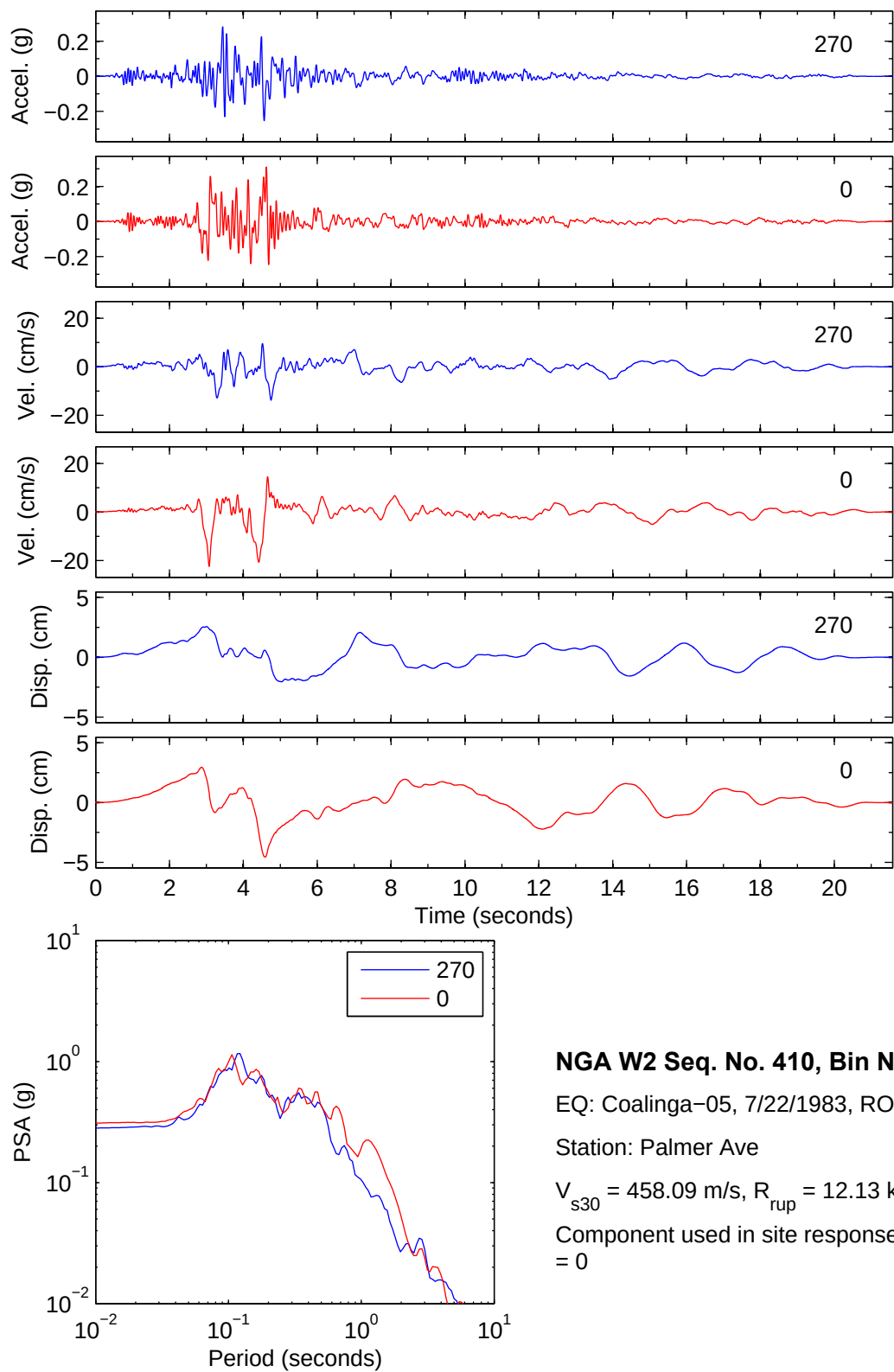


Figure D-11: Input motion NGA W2 Seq. No. 410, Bin No. 2

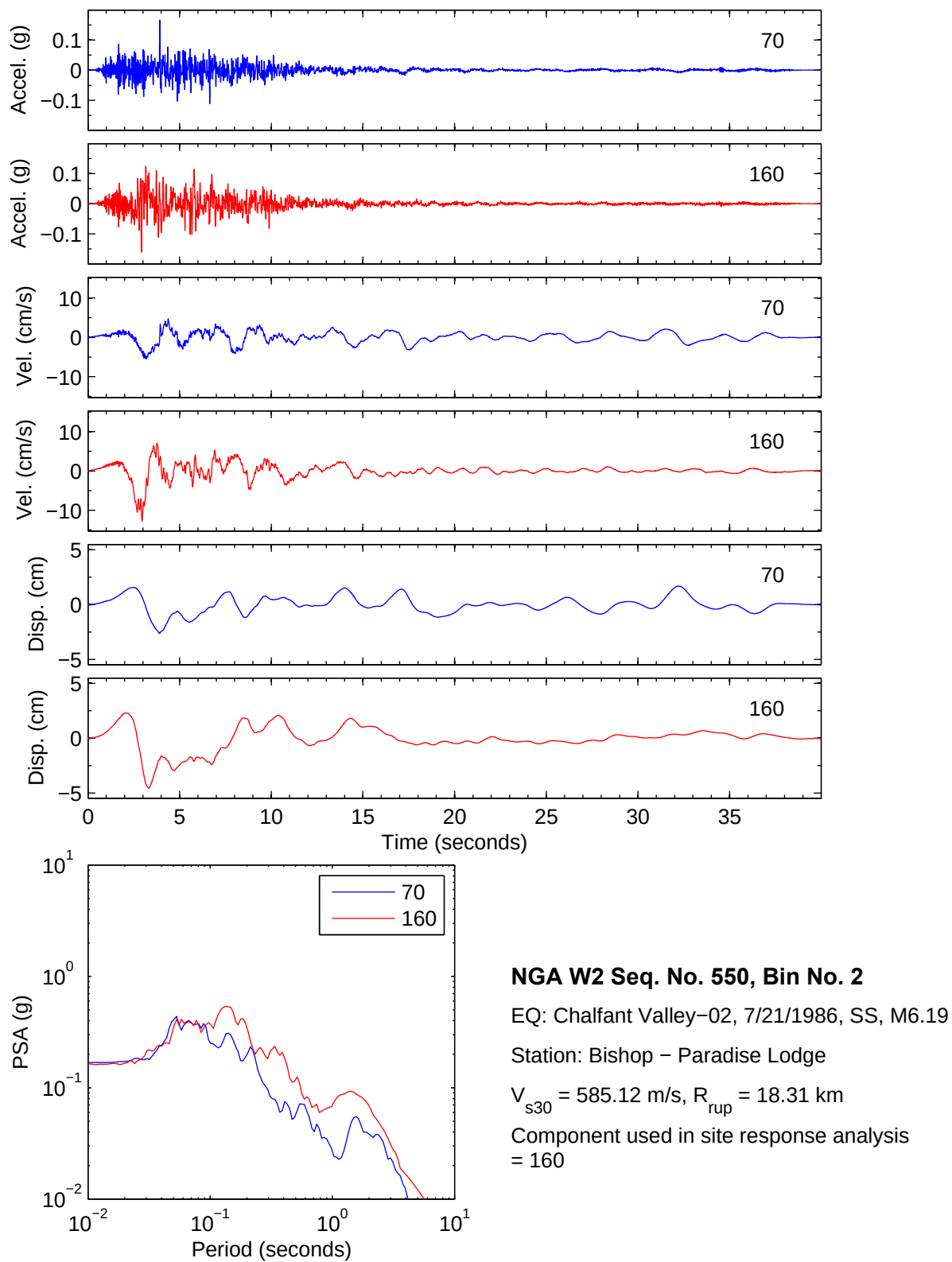


Figure D-12: Input motion NGA W2 Seq. No. 550, Bin No. 2

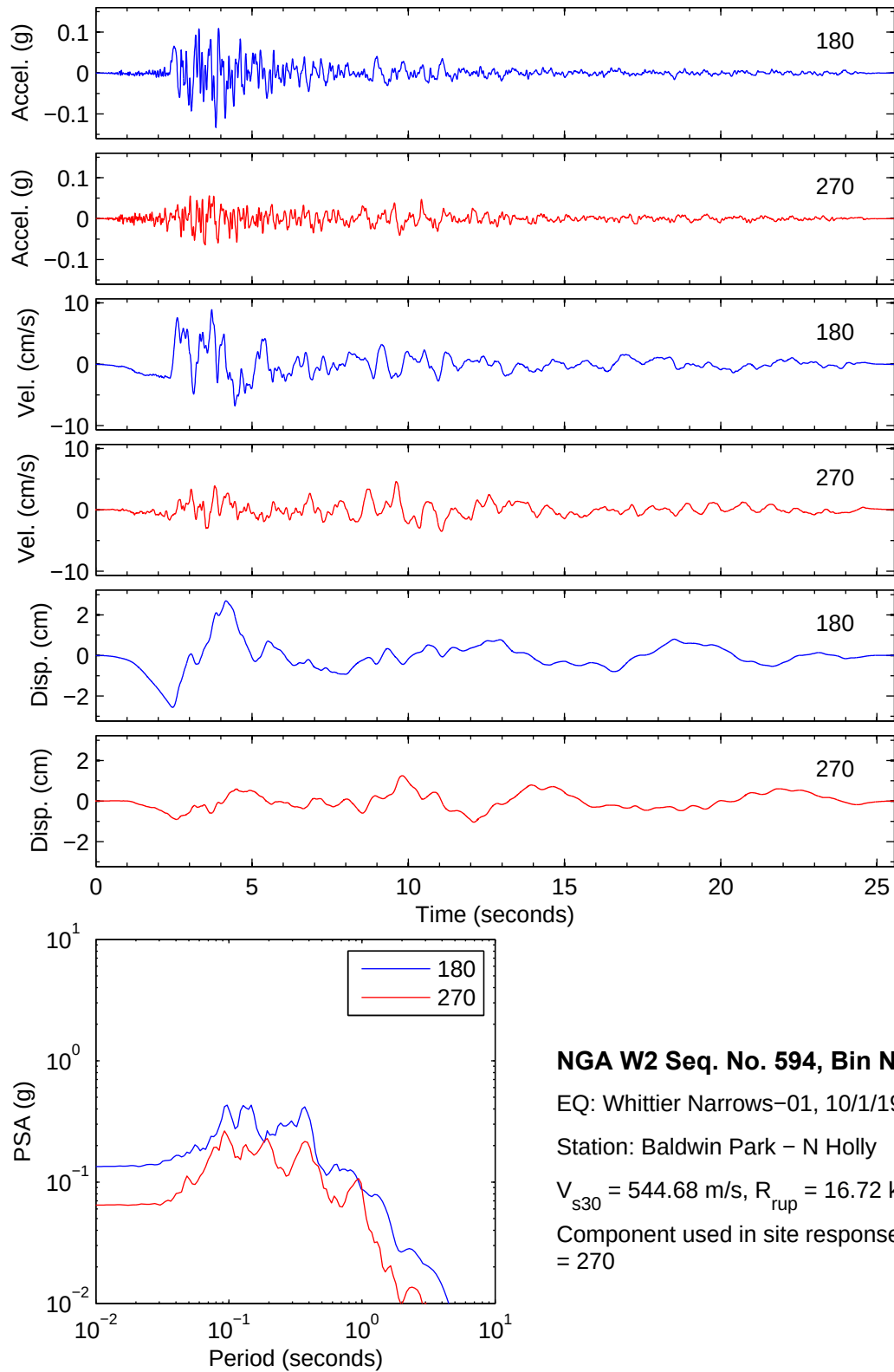


Figure D-13: Input motion NGA W2 Seq. No. 594, Bin No. 2

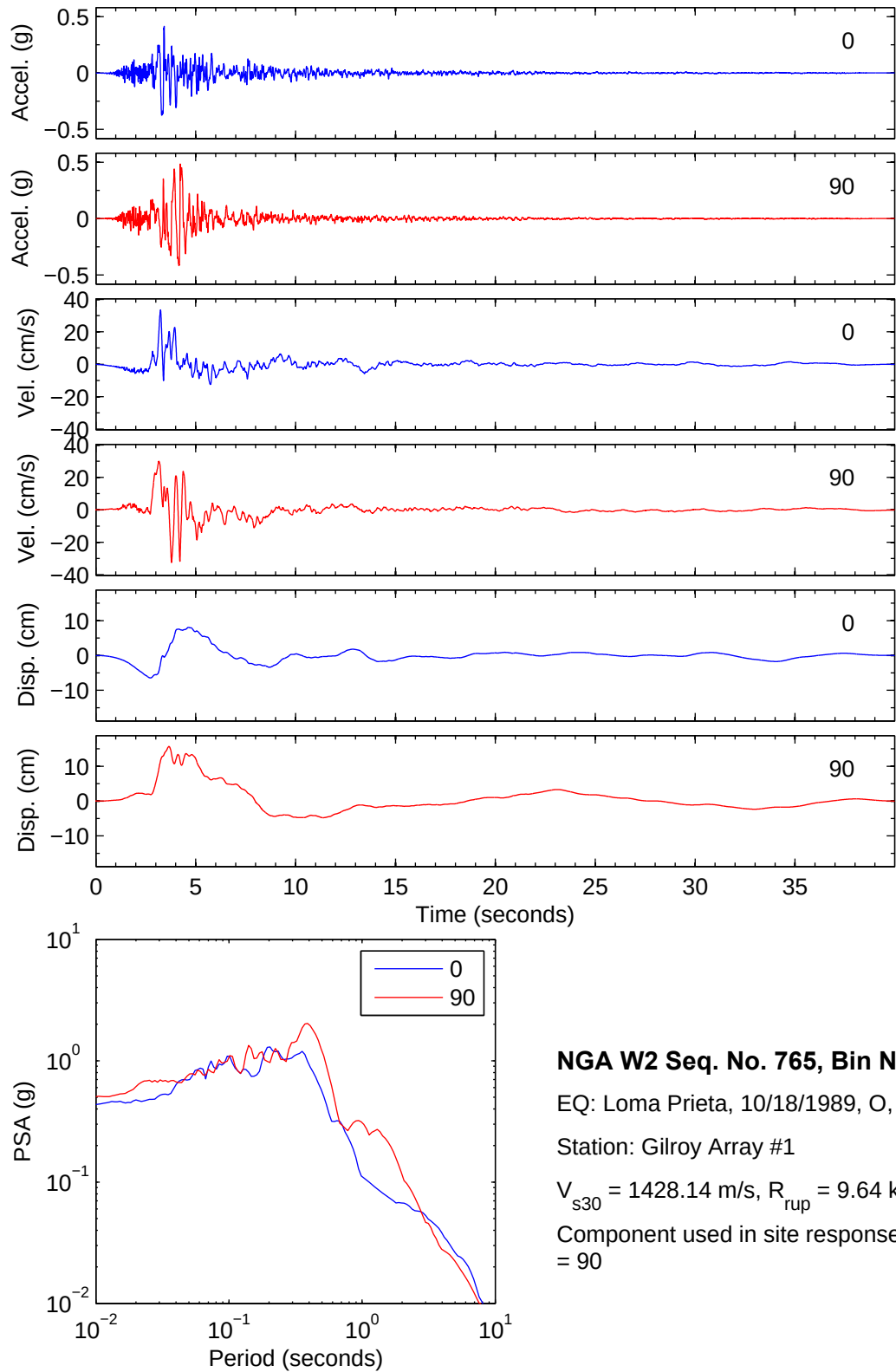


Figure D-14: Input motion NGA W2 Seq. No. 765, Bin No. 2

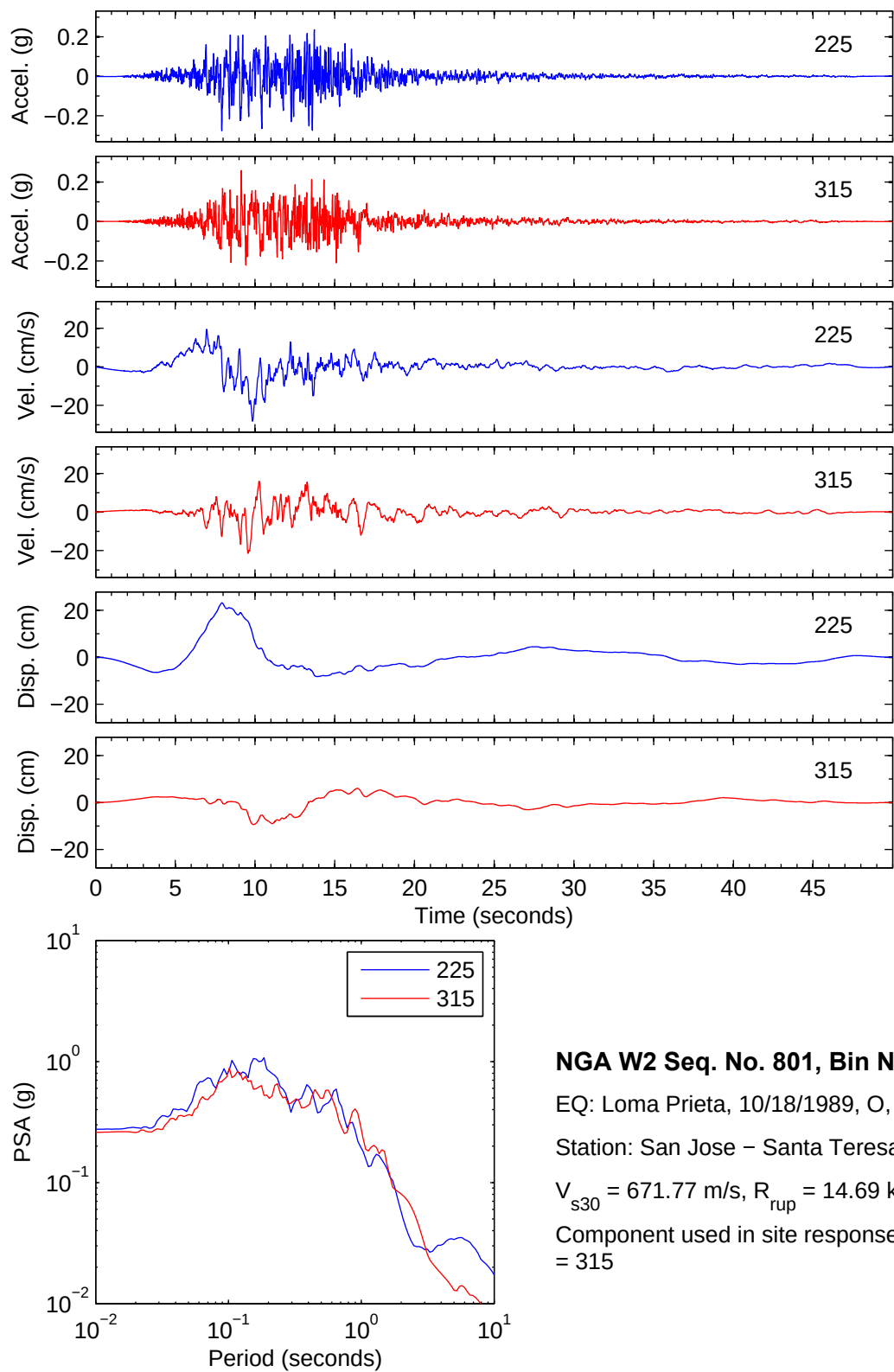


Figure D-15: Input motion NGA W2 Seq. No. 801, Bin No. 2

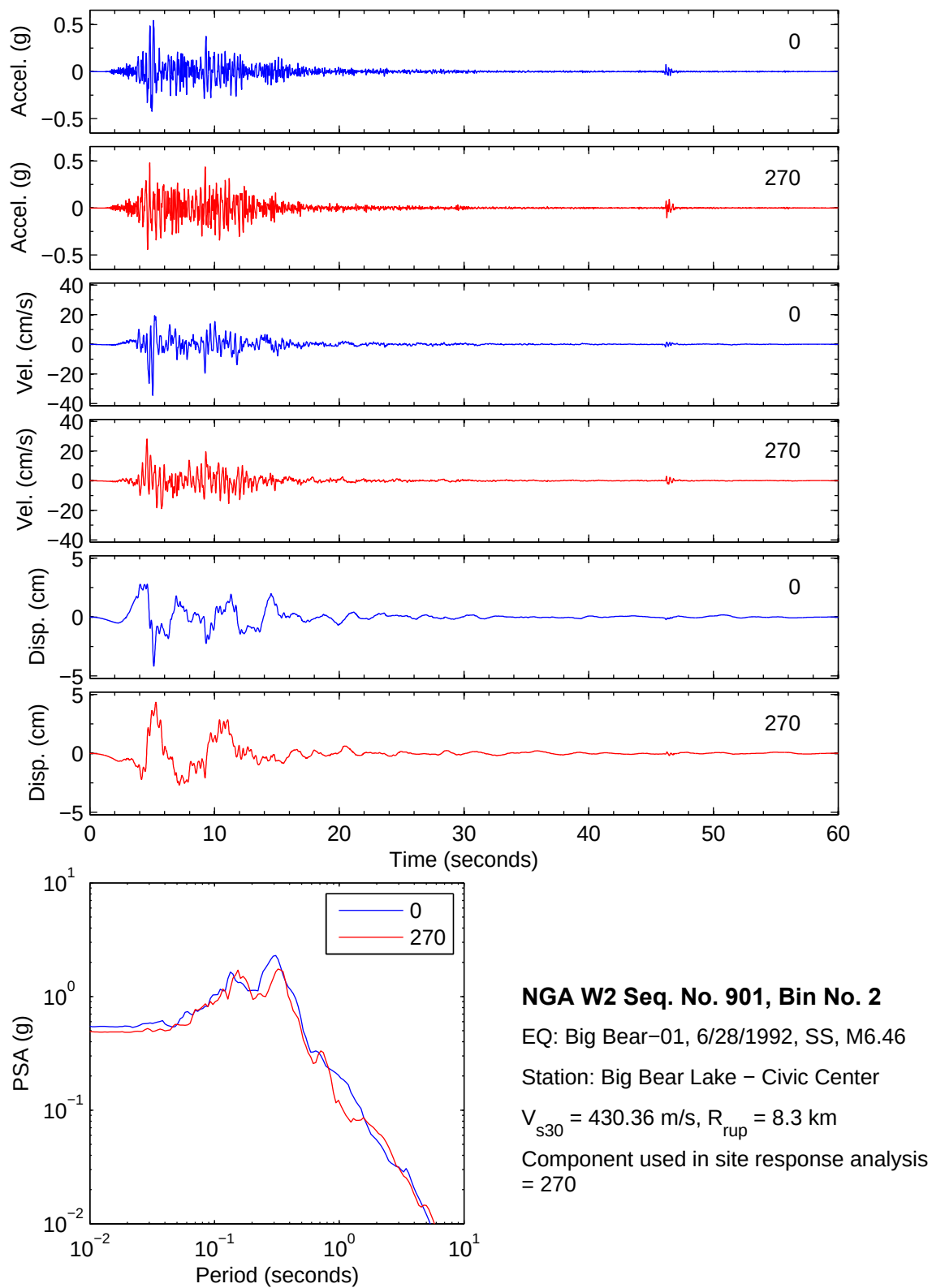


Figure D-16: Input motion NGA W2 Seq. No. 901, Bin No. 2

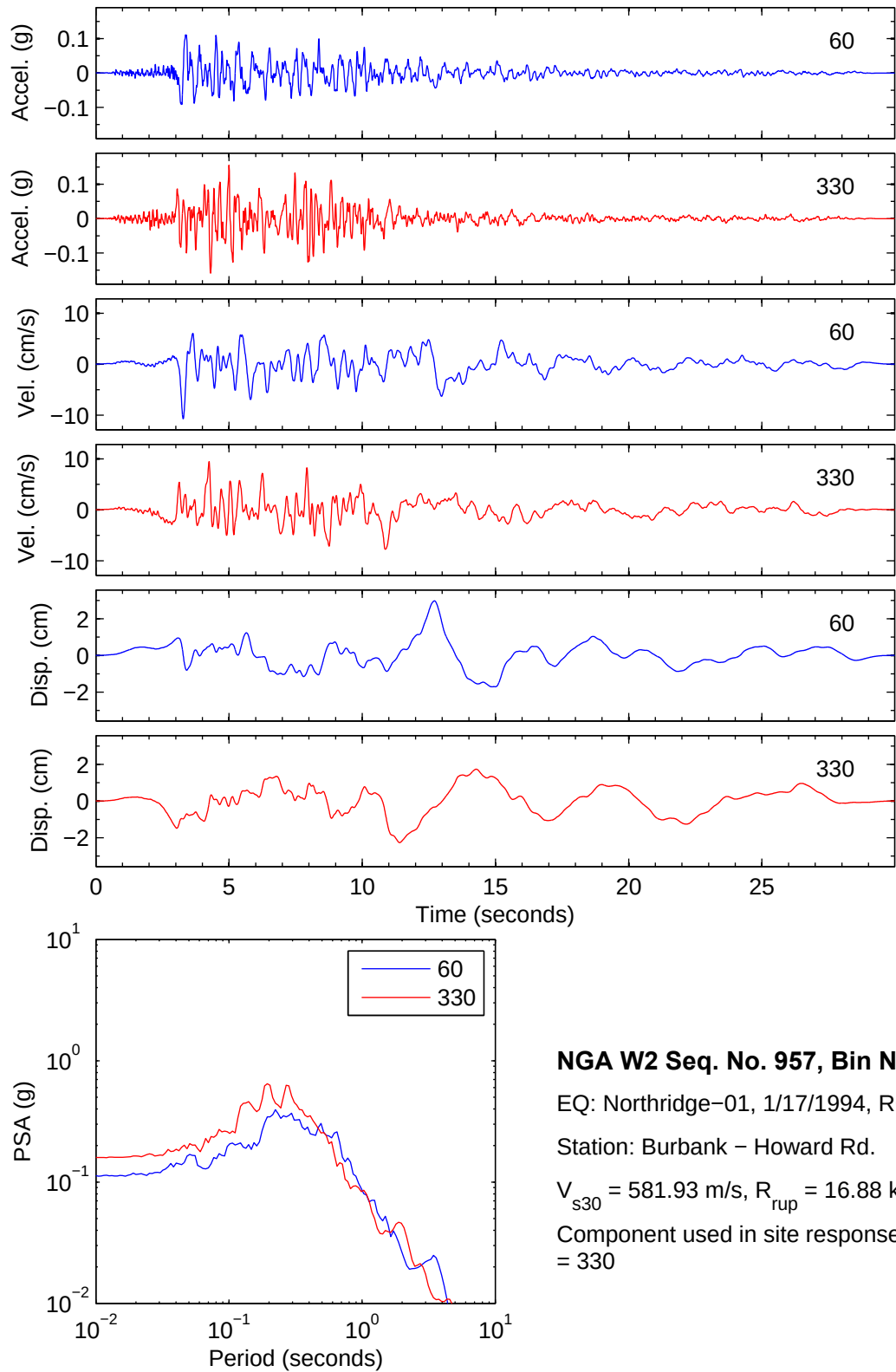


Figure D-17: Input motion NGA W2 Seq. No. 957, Bin No. 2

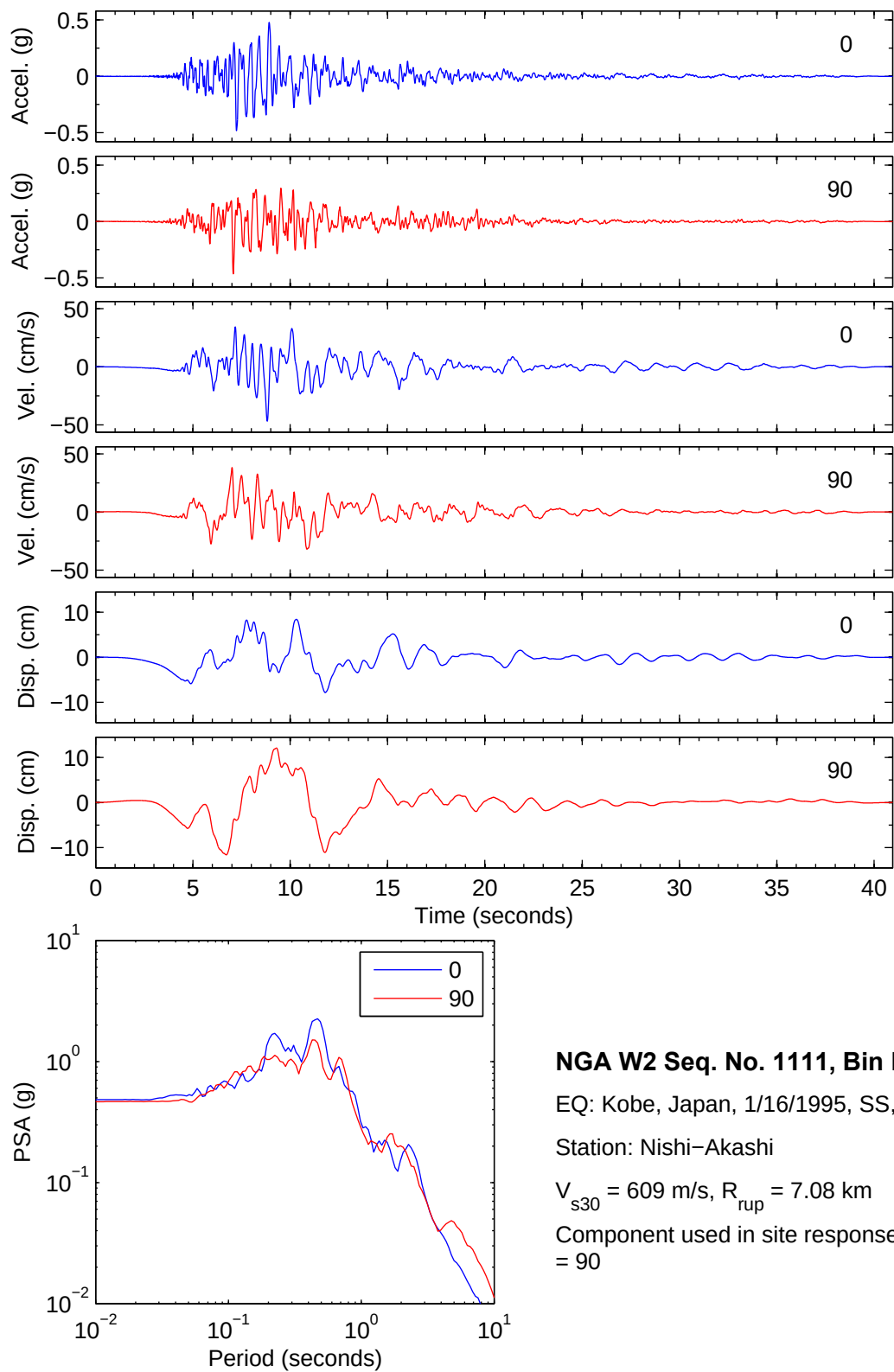


Figure D-18: Input motion NGA W2 Seq. No. 1111, Bin No. 2

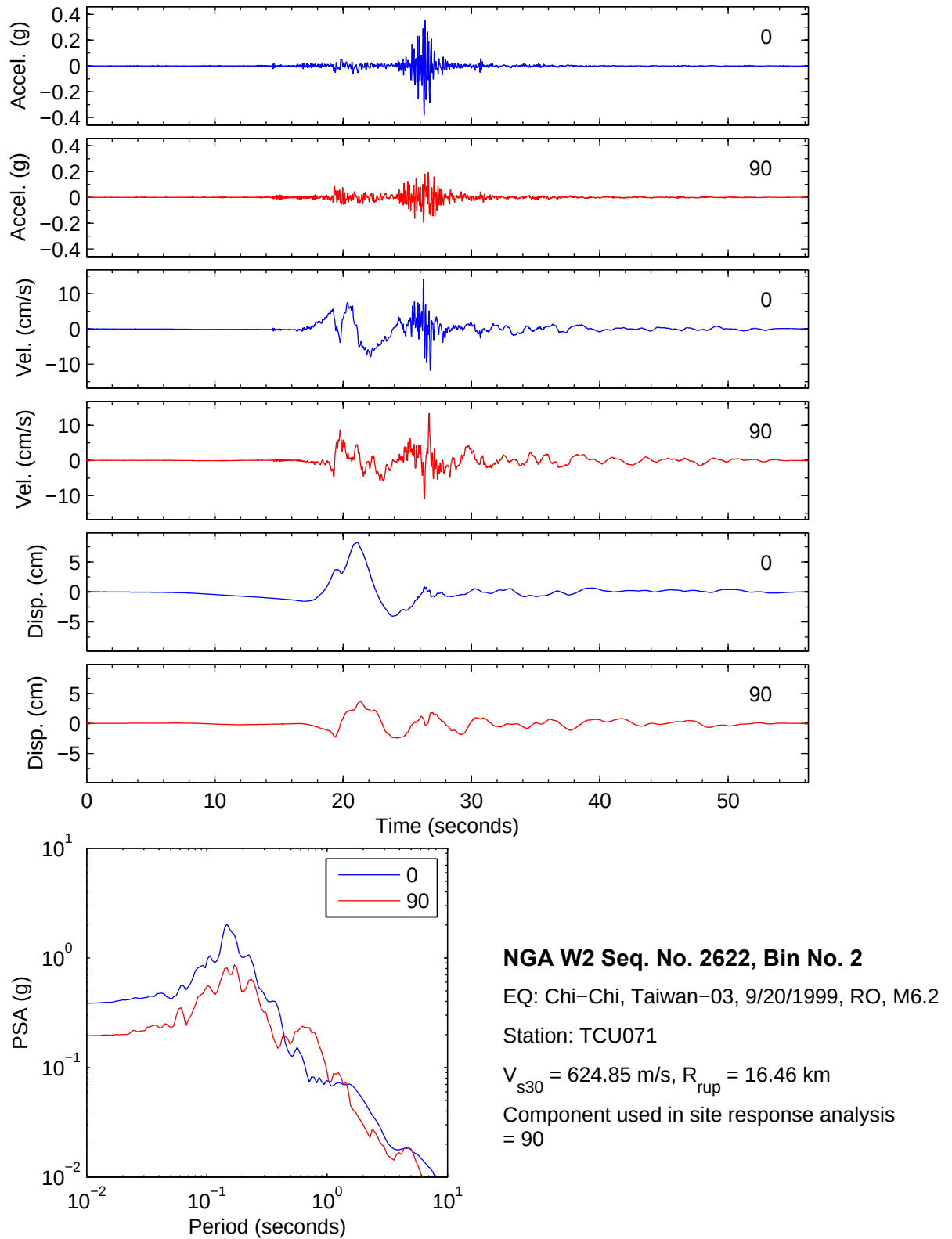


Figure D-19: Input motion NGA W2 Seq. No. 2622, Bin No. 2

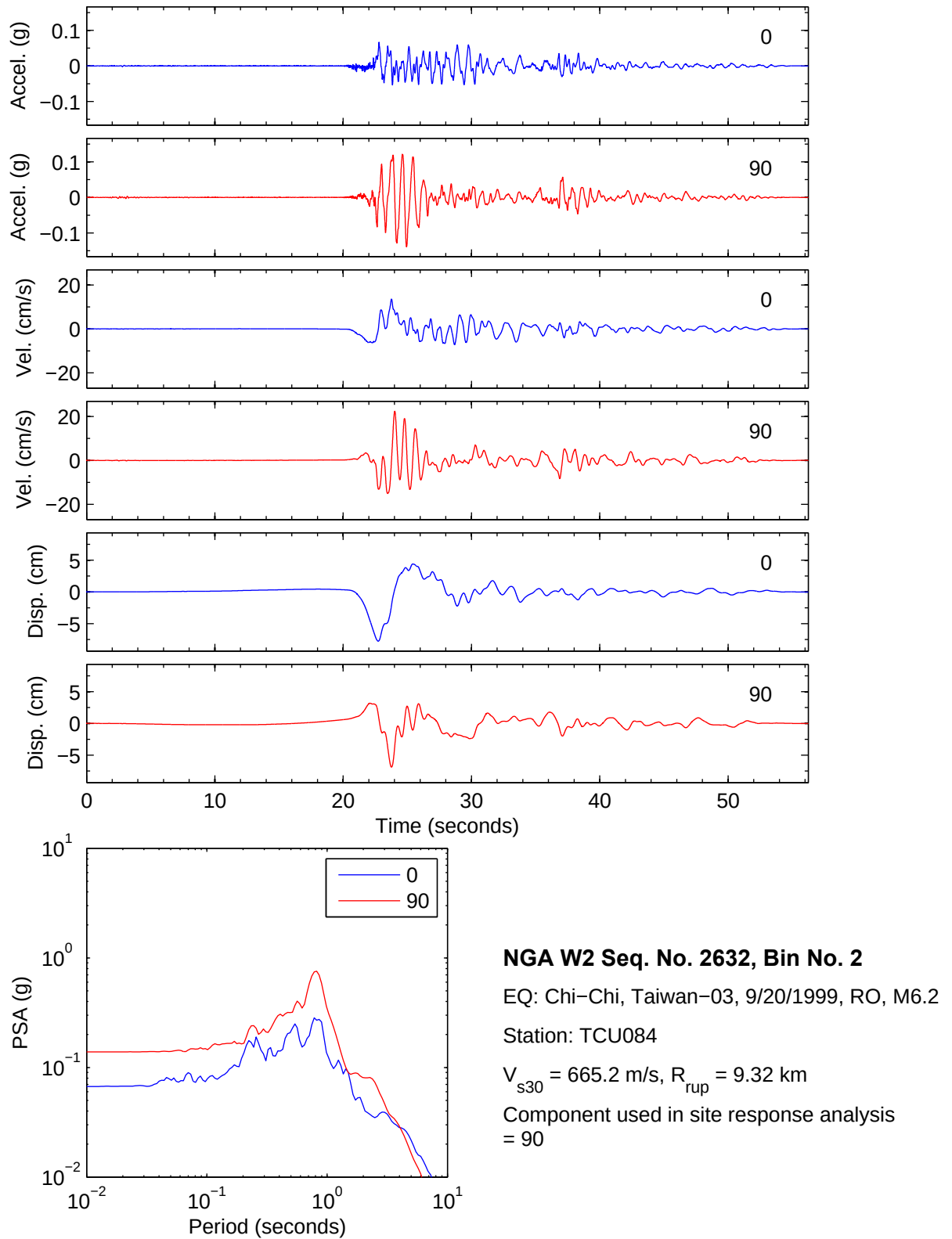


Figure D-20: Input motion NGA W2 Seq. No. 2632, Bin No. 2

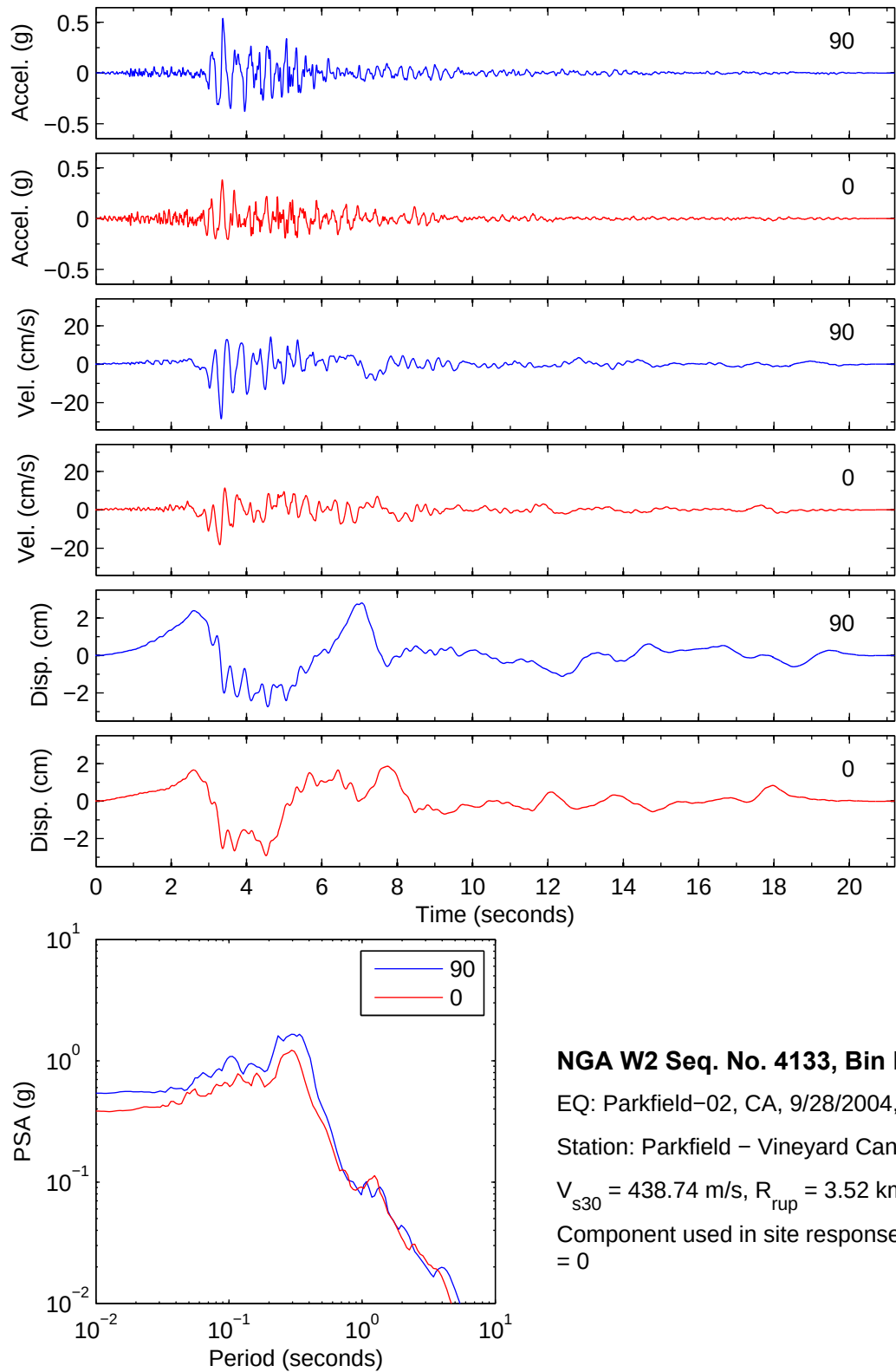


Figure D-21: Input motion NGA W2 Seq. No. 4133, Bin No. 2

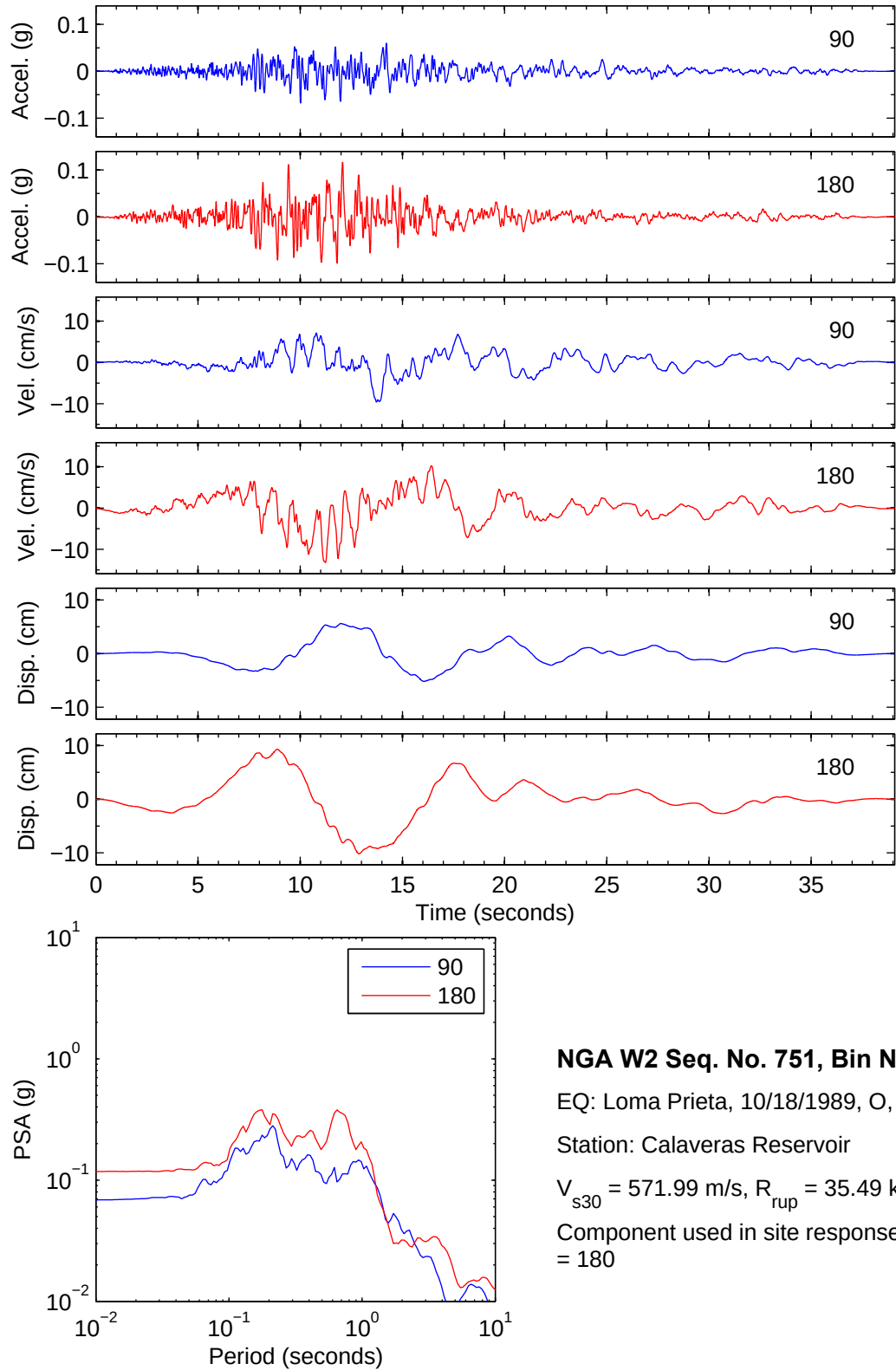


Figure D-22: Input motion NGA W2 Seq. No. 751, Bin No. 3

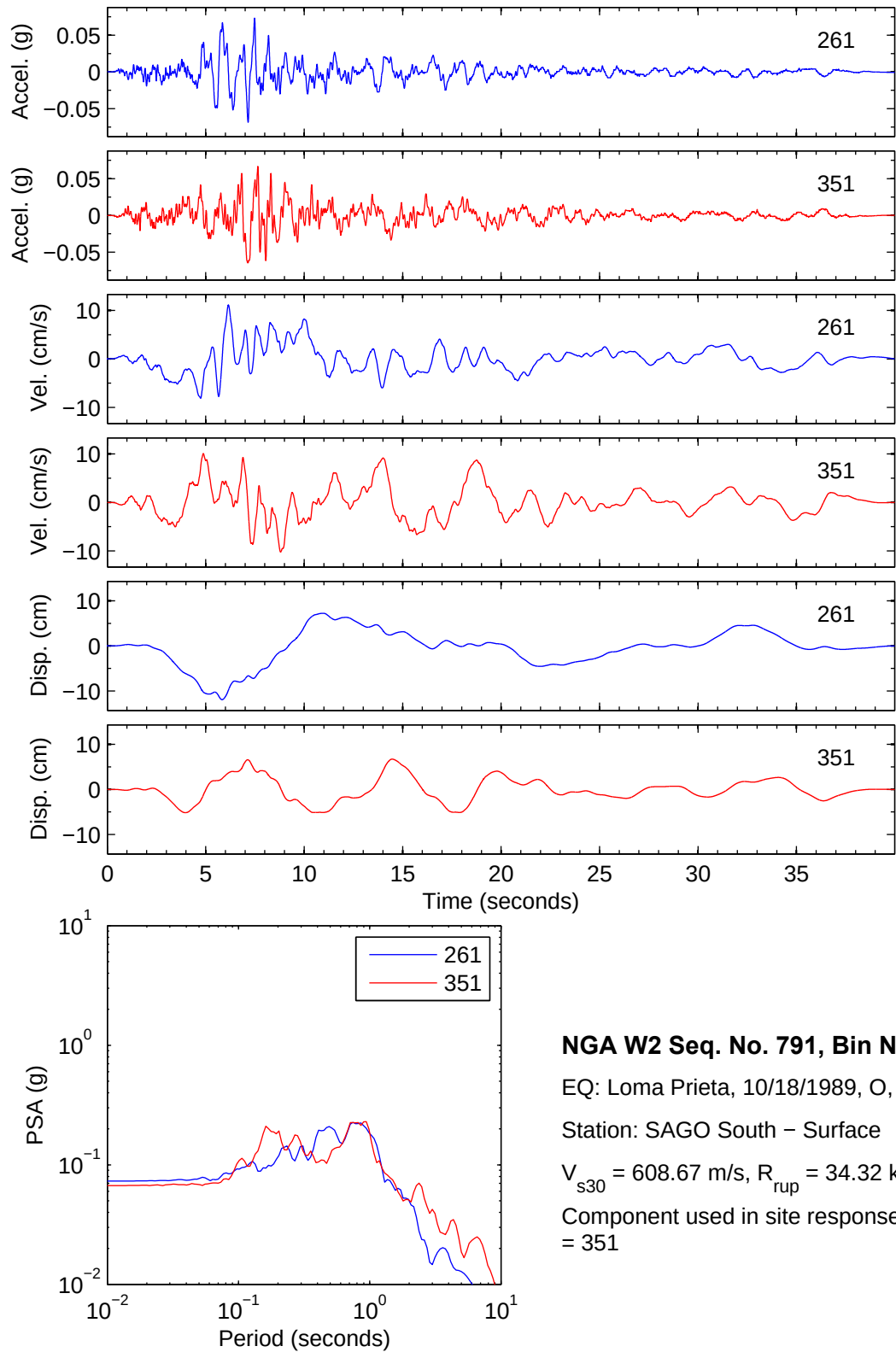


Figure D-23: Input motion NGA W2 Seq. No. 791, Bin No. 3

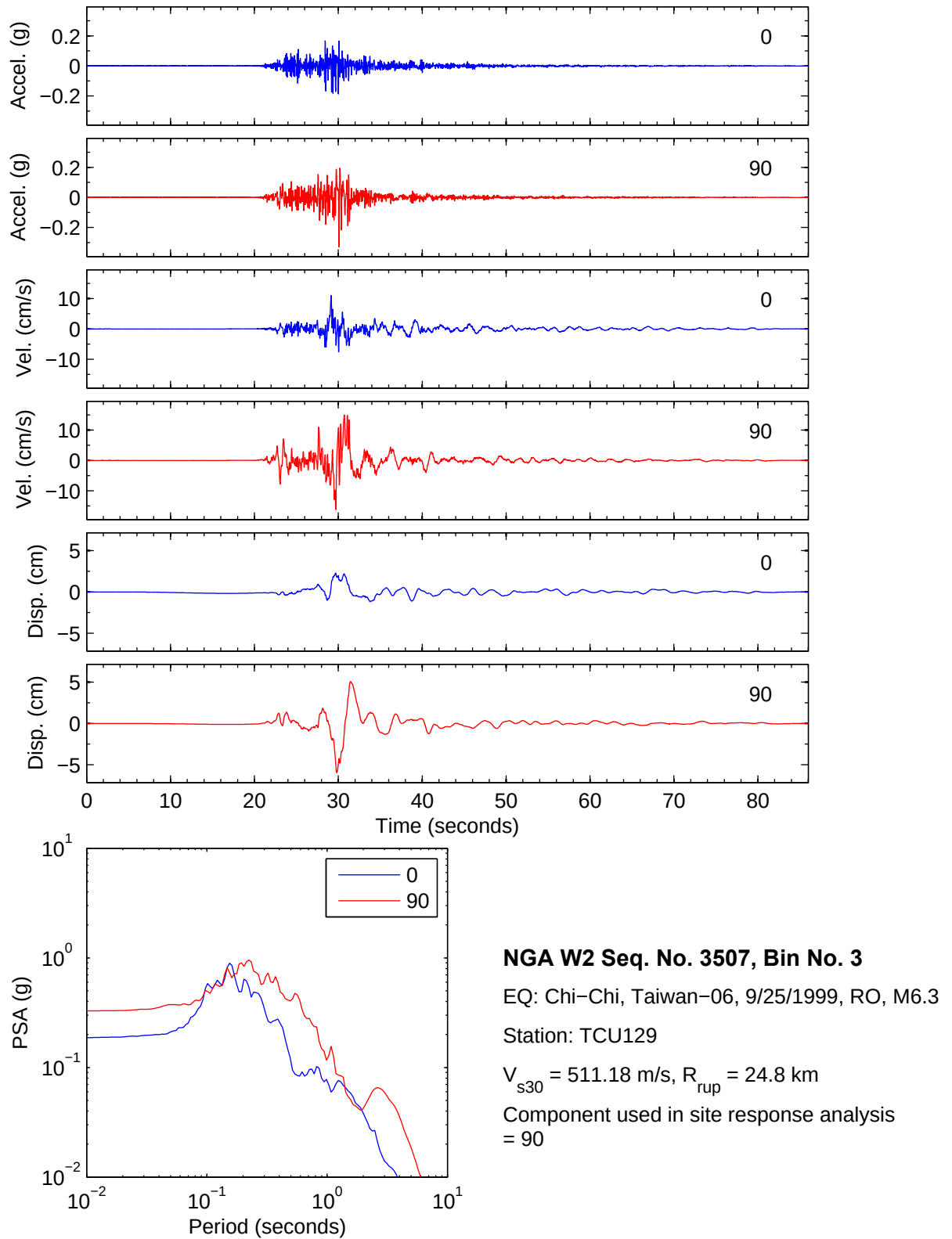


Figure D-24: Input motion NGA W2 Seq. No. 3507, Bin No. 3

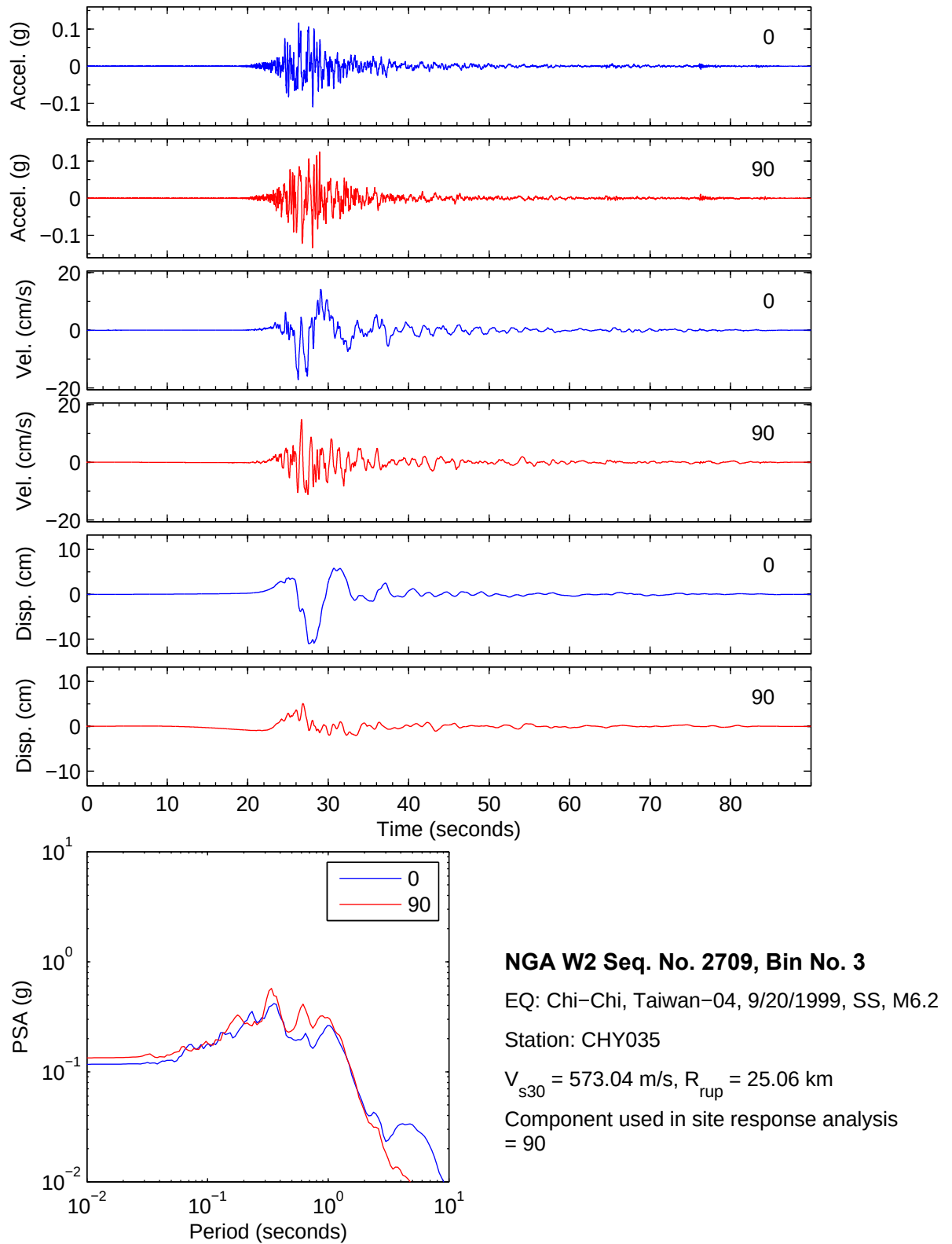


Figure D-25: Input motion NGA W2 Seq. No. 2709, Bin No. 3

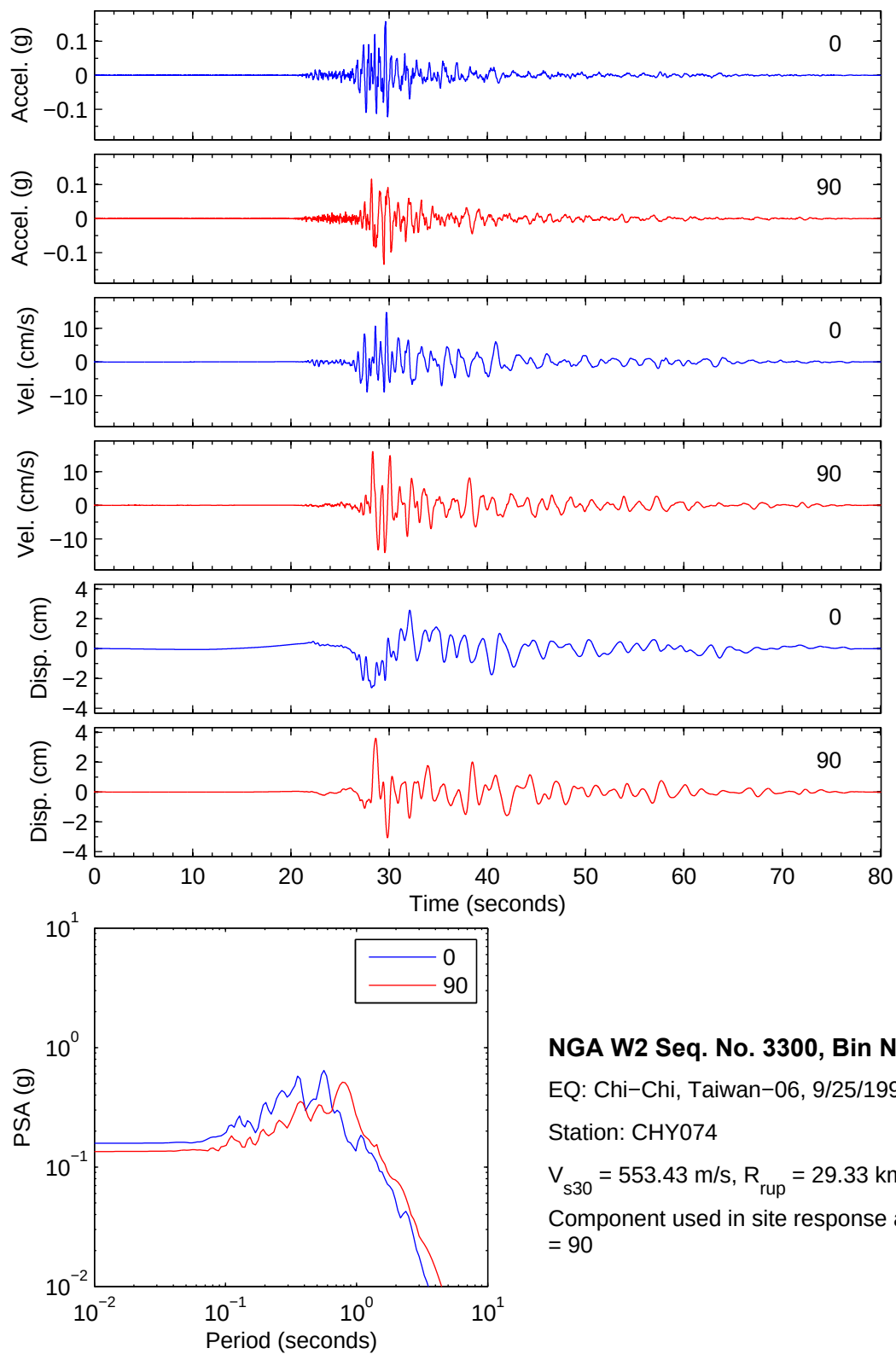


Figure D-26: Input motion NGA W2 Seq. No. 3300, Bin No. 3

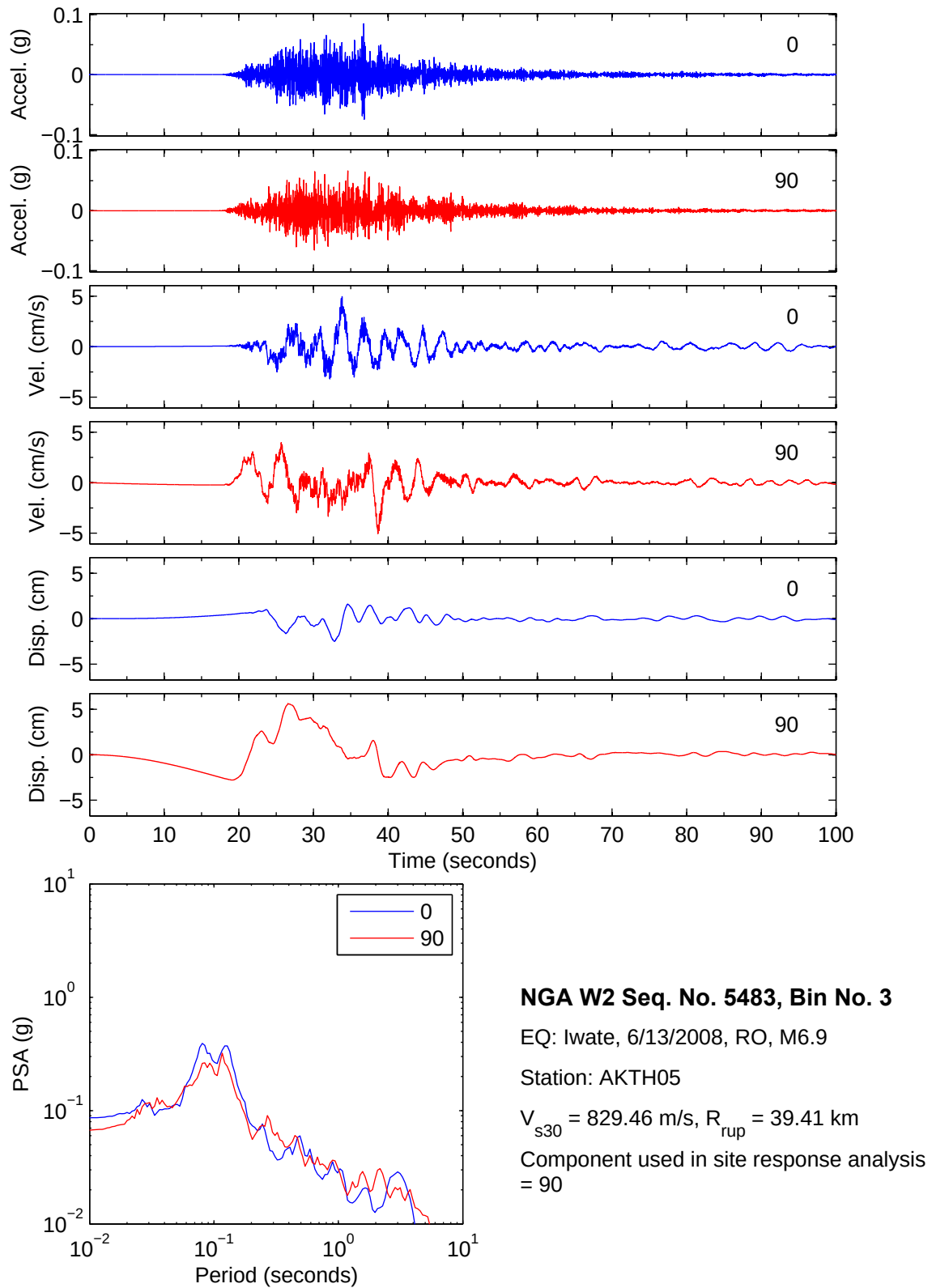


Figure D-27: Input motion NGA W2 Seq. No. 5483, Bin No. 3

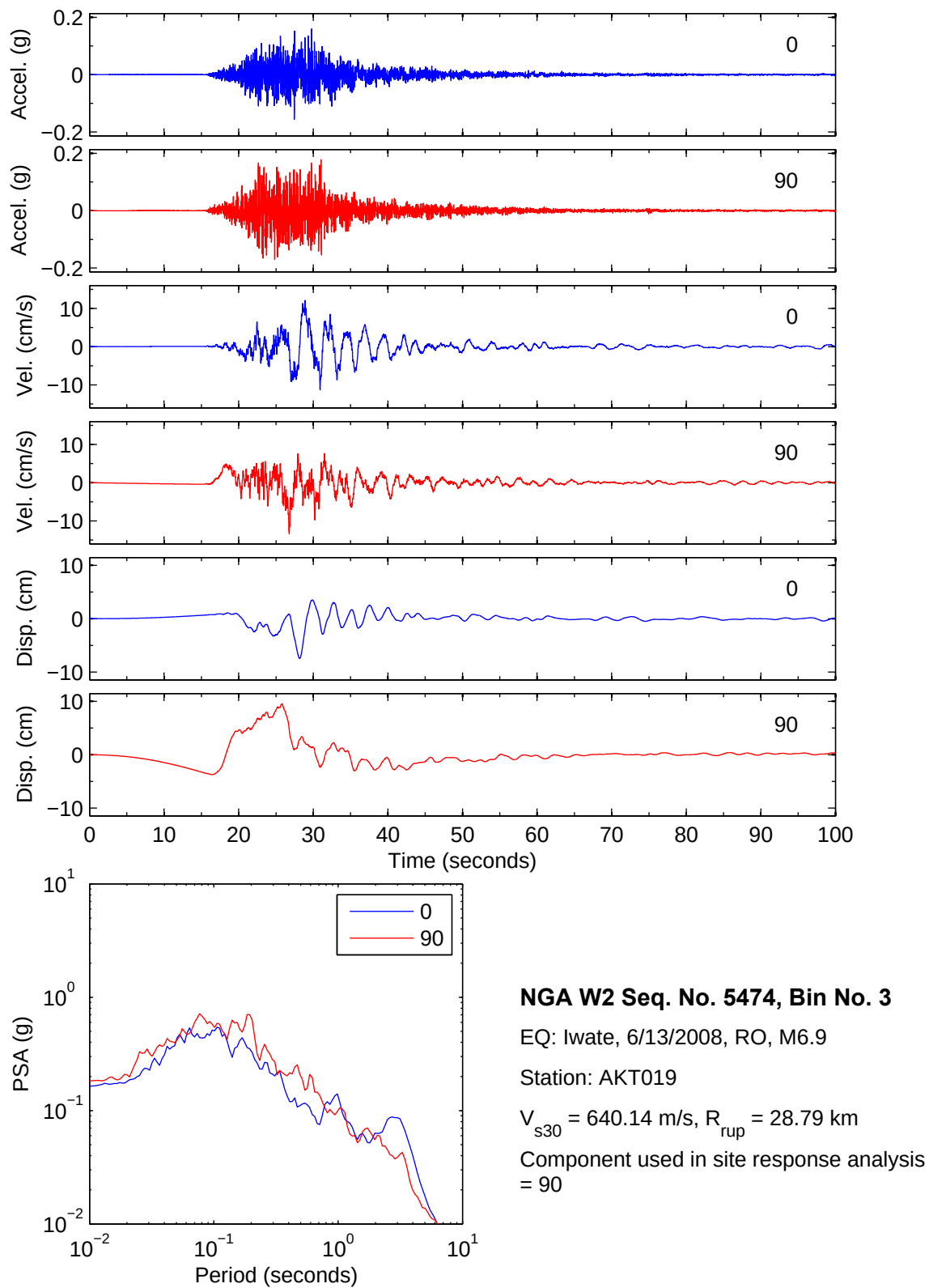


Figure D-28: Input motion NGA W2 Seq. No. 5474, Bin No. 3

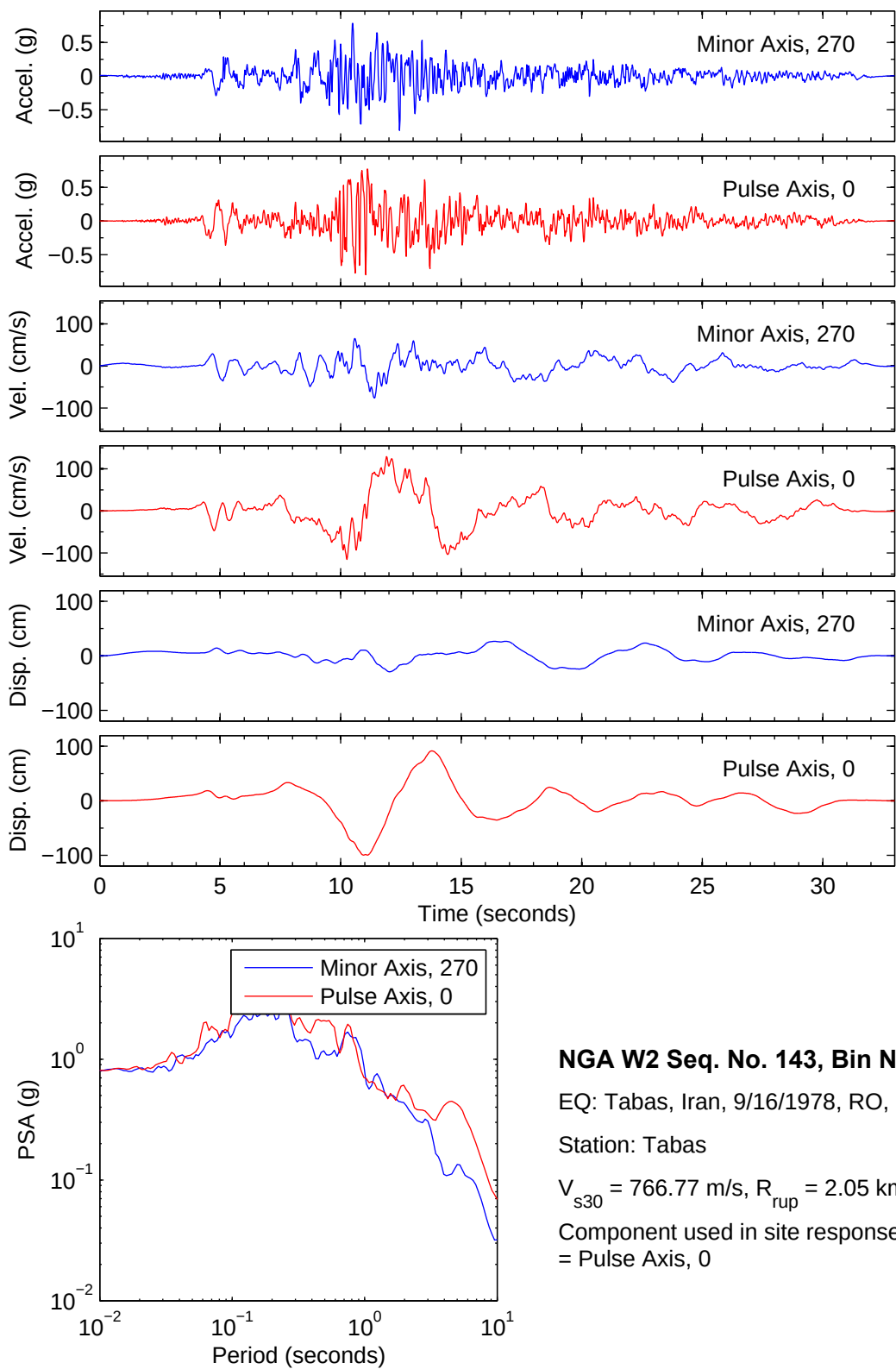


Figure D-29: Input motion NGA W2 Seq. No. 143, Bin No. 4

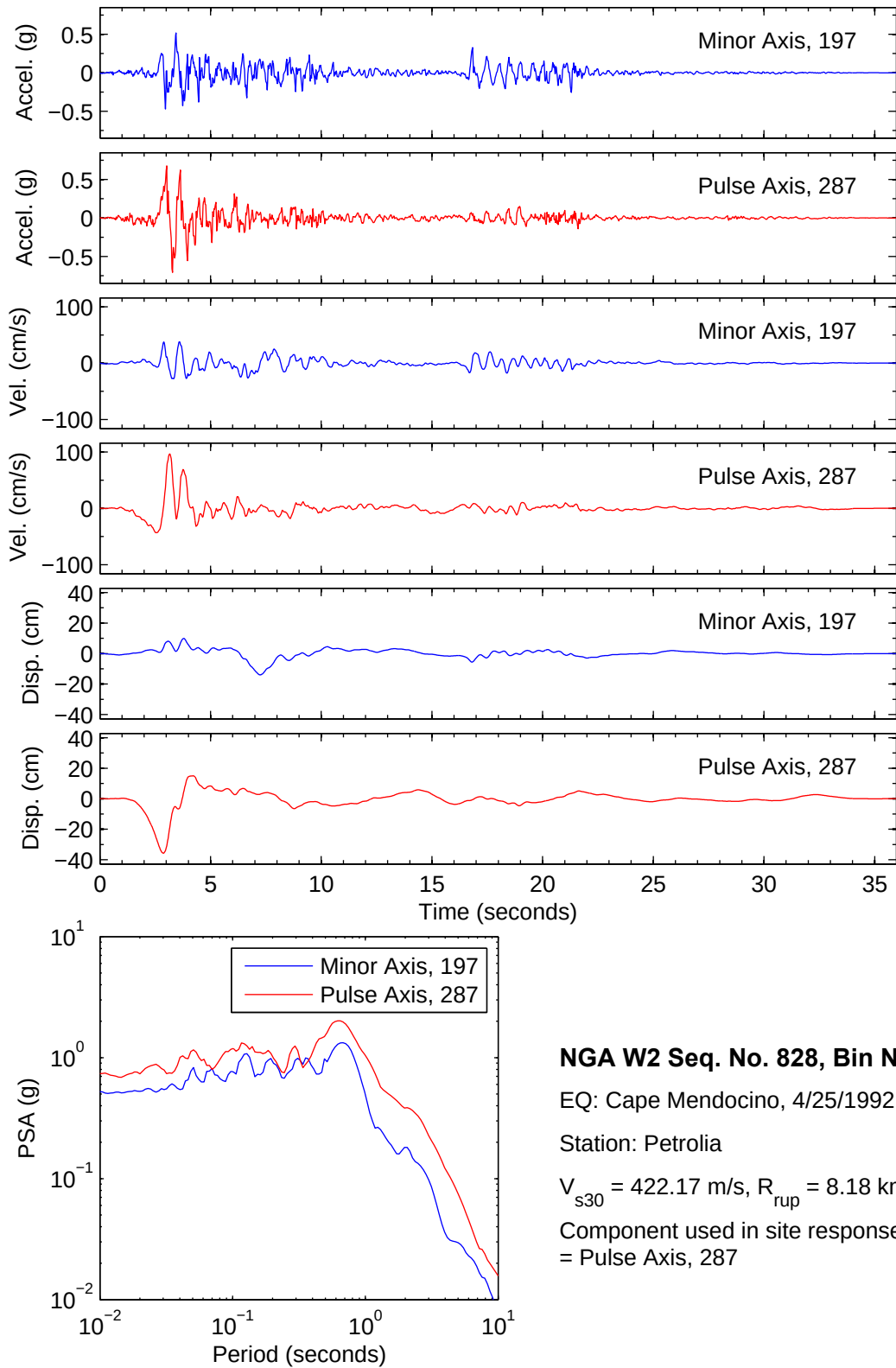


Figure D-30: Input motion NGA W2 Seq. No. 828, Bin No. 4

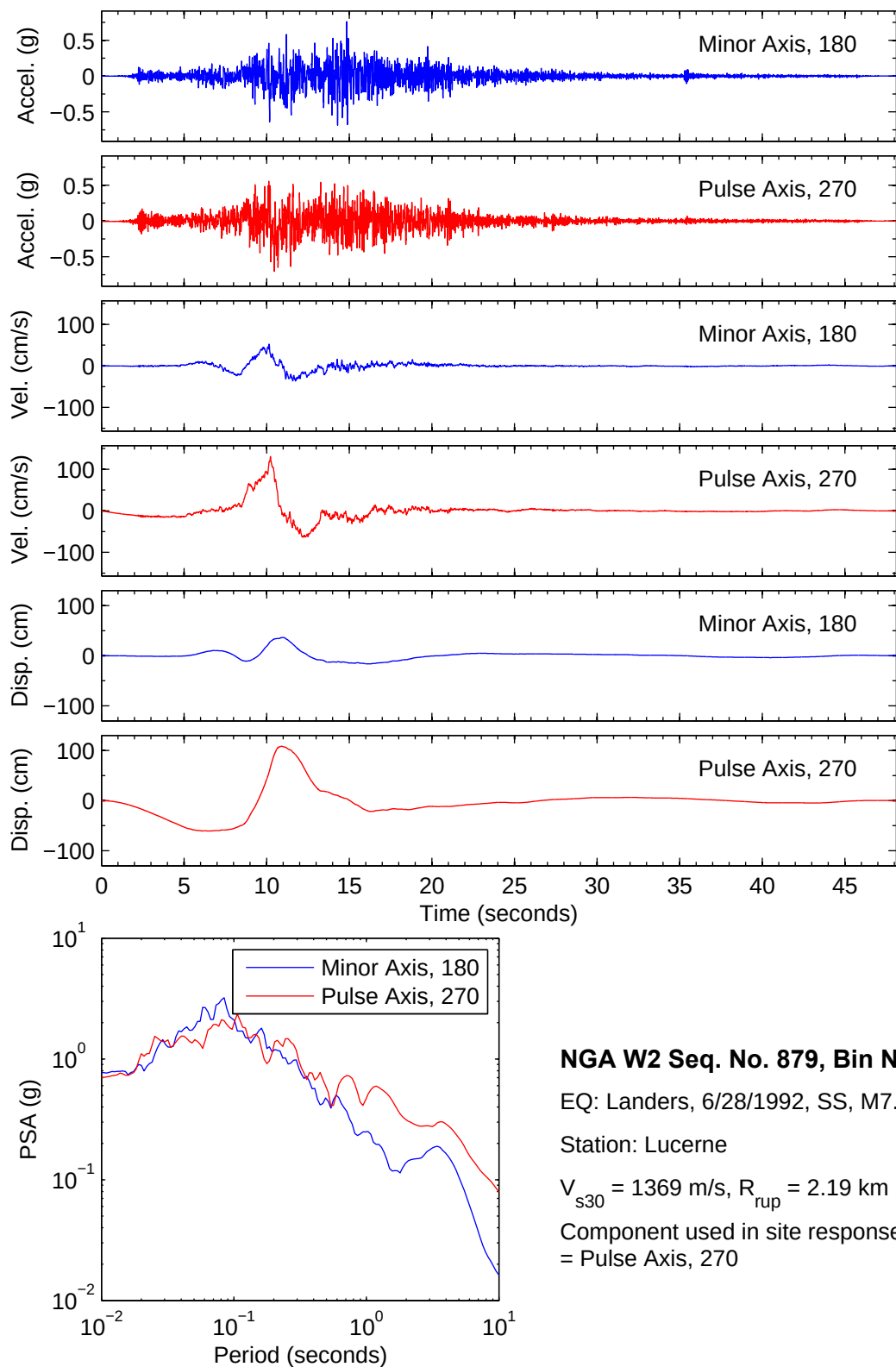


Figure D-31: Input motion NGA W2 Seq. No. 879, Bin No. 4

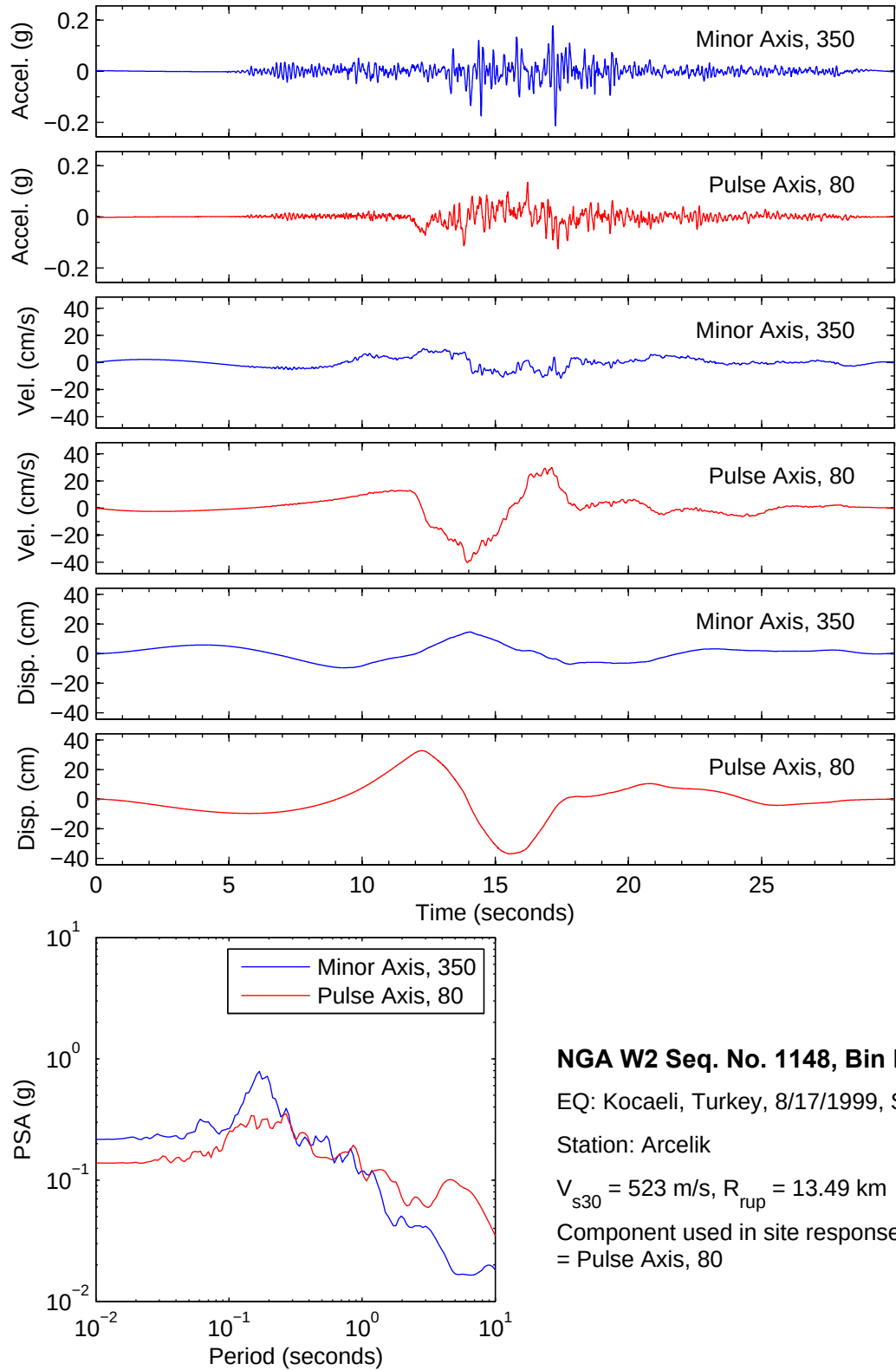


Figure D-32: Input motion NGA W2 Seq. No. 1148, Bin No. 4

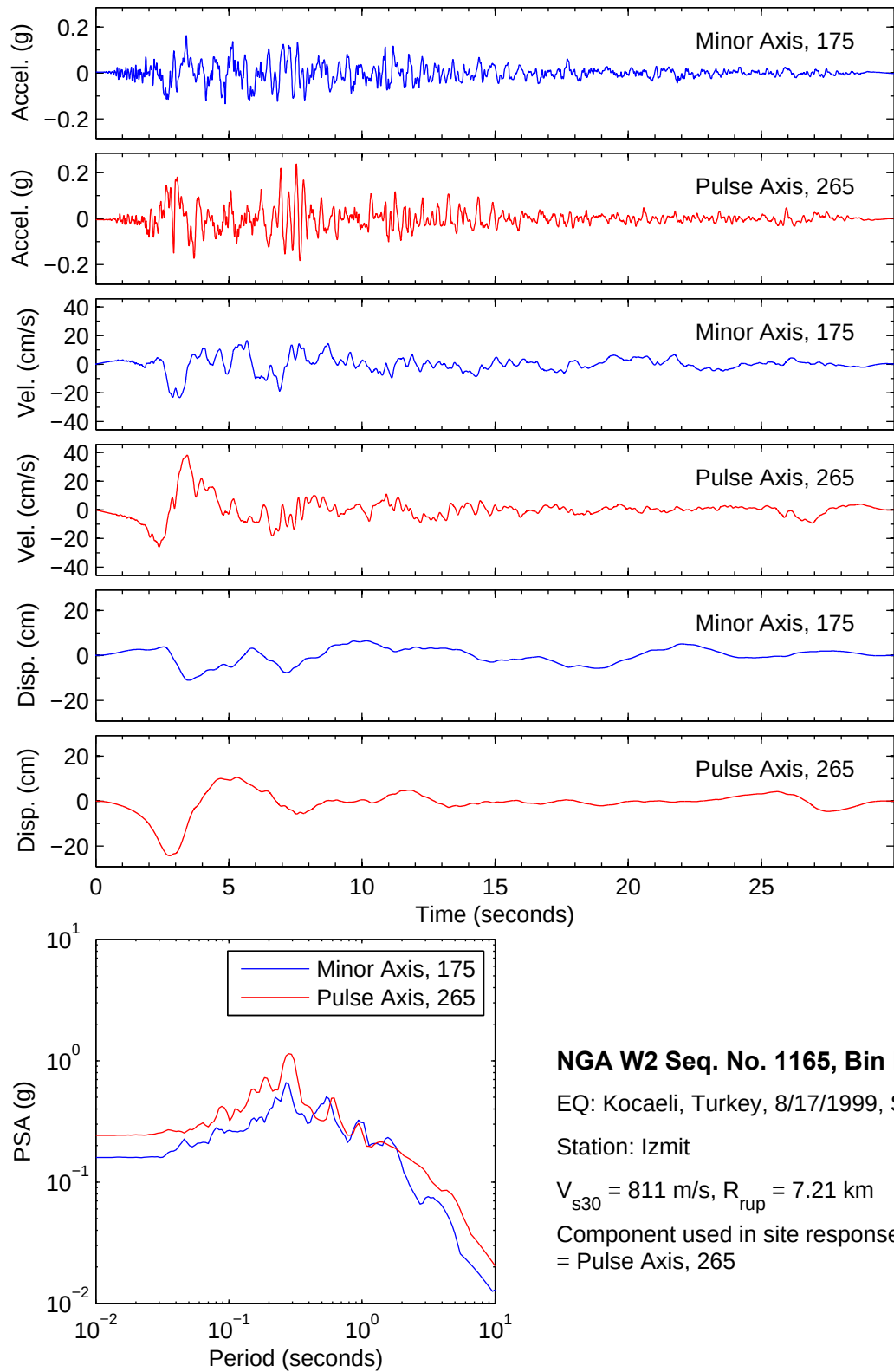


Figure D-33: Input motion NGA W2 Seq. No. 1165, Bin No. 4

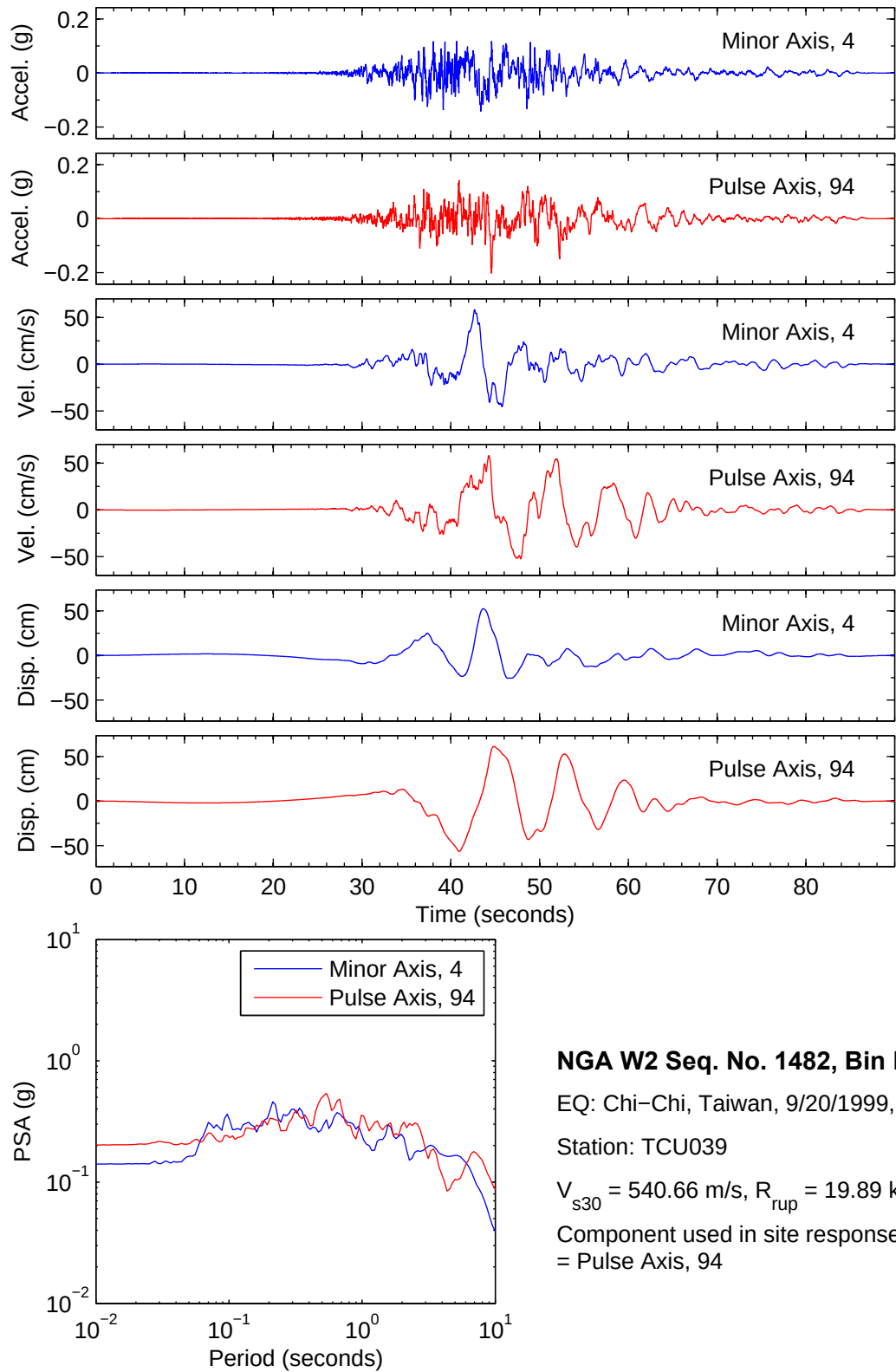


Figure D-34: Input motion NGA W2 Seq. No. 1482, Bin No. 4

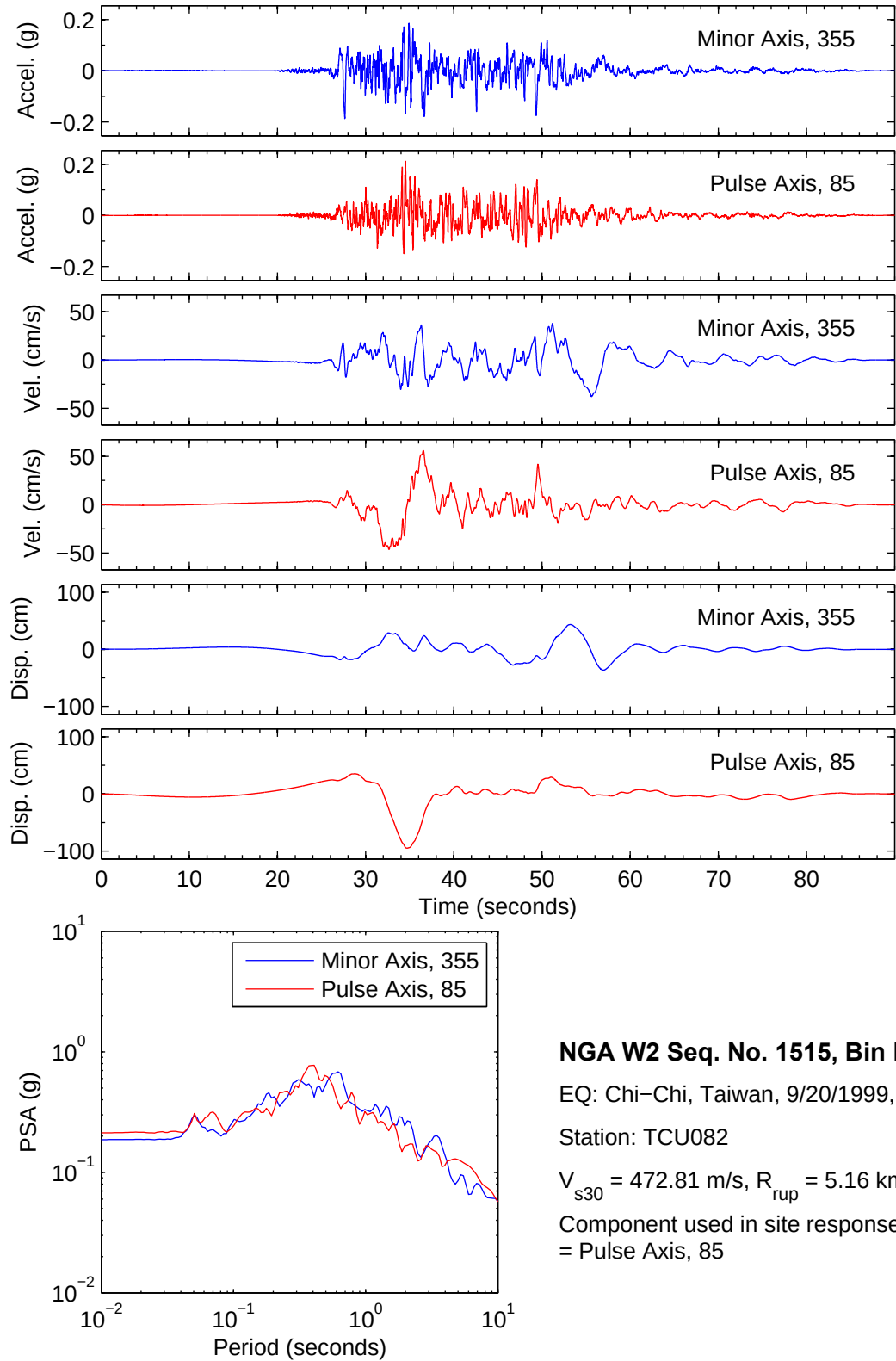


Figure D-35: Input motion NGA W2 Seq. No. 1515, Bin No. 4

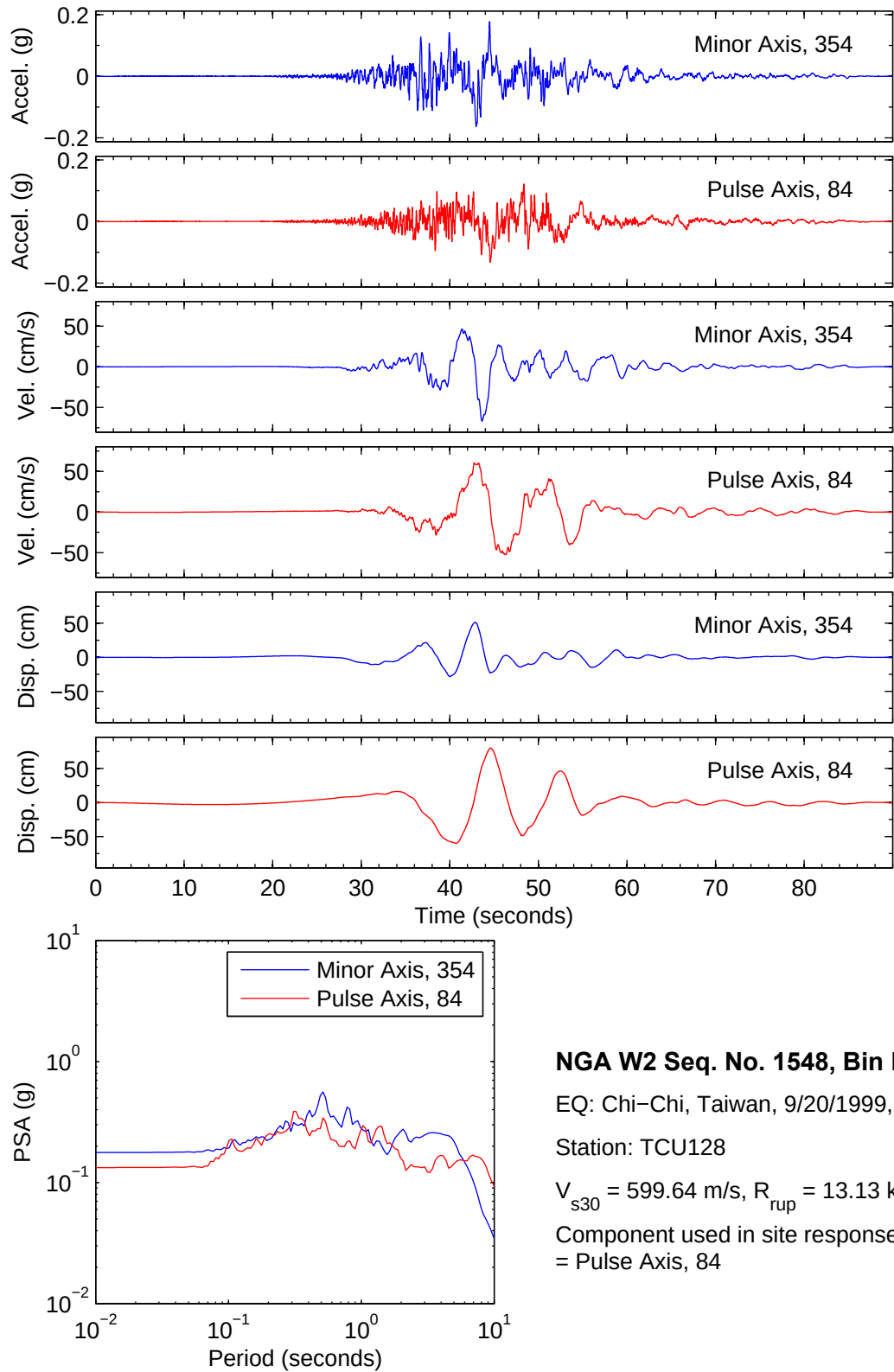
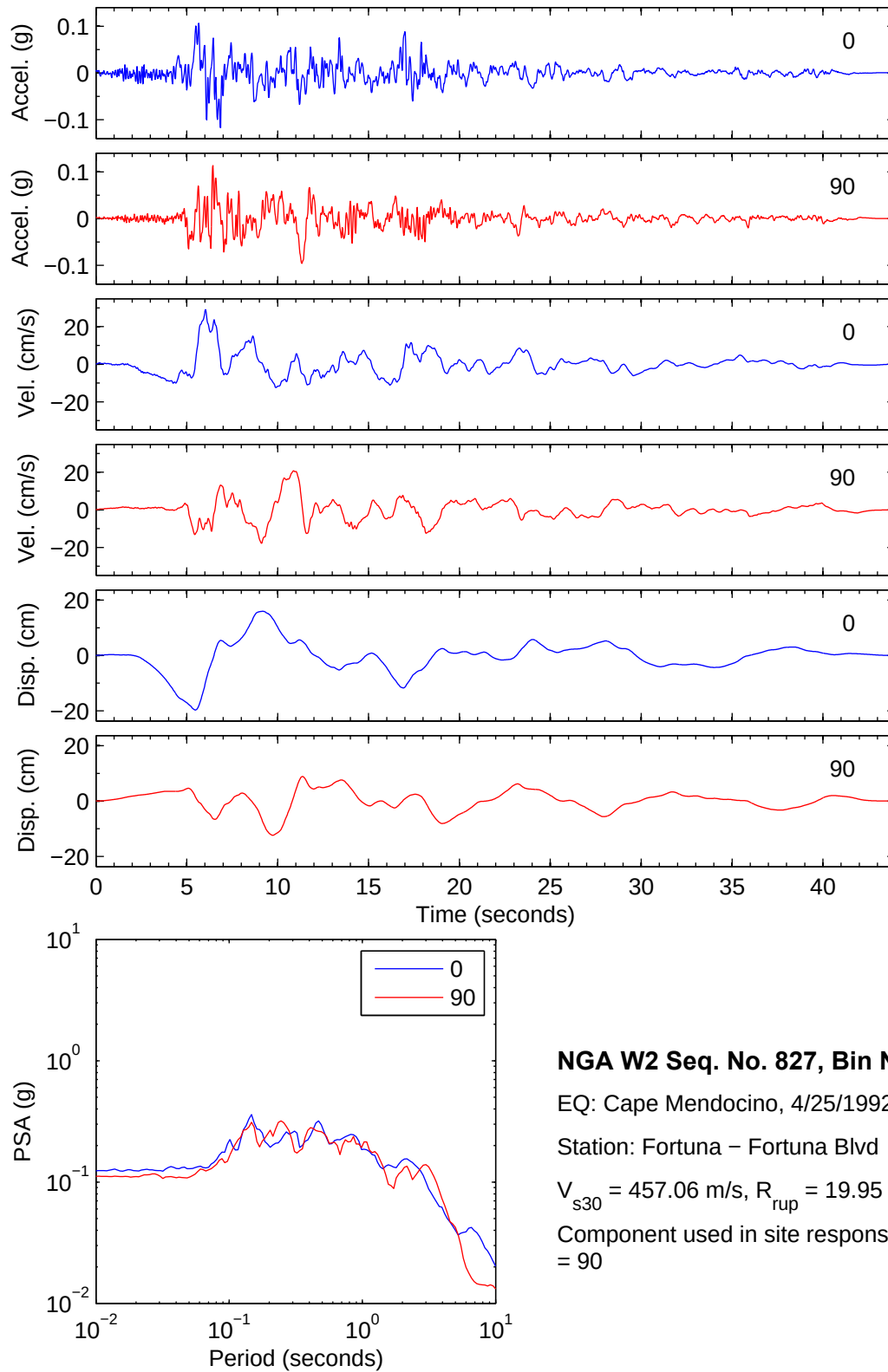


Figure D-36: Input motion NGA W2 Seq. No. 1548, Bin No. 4



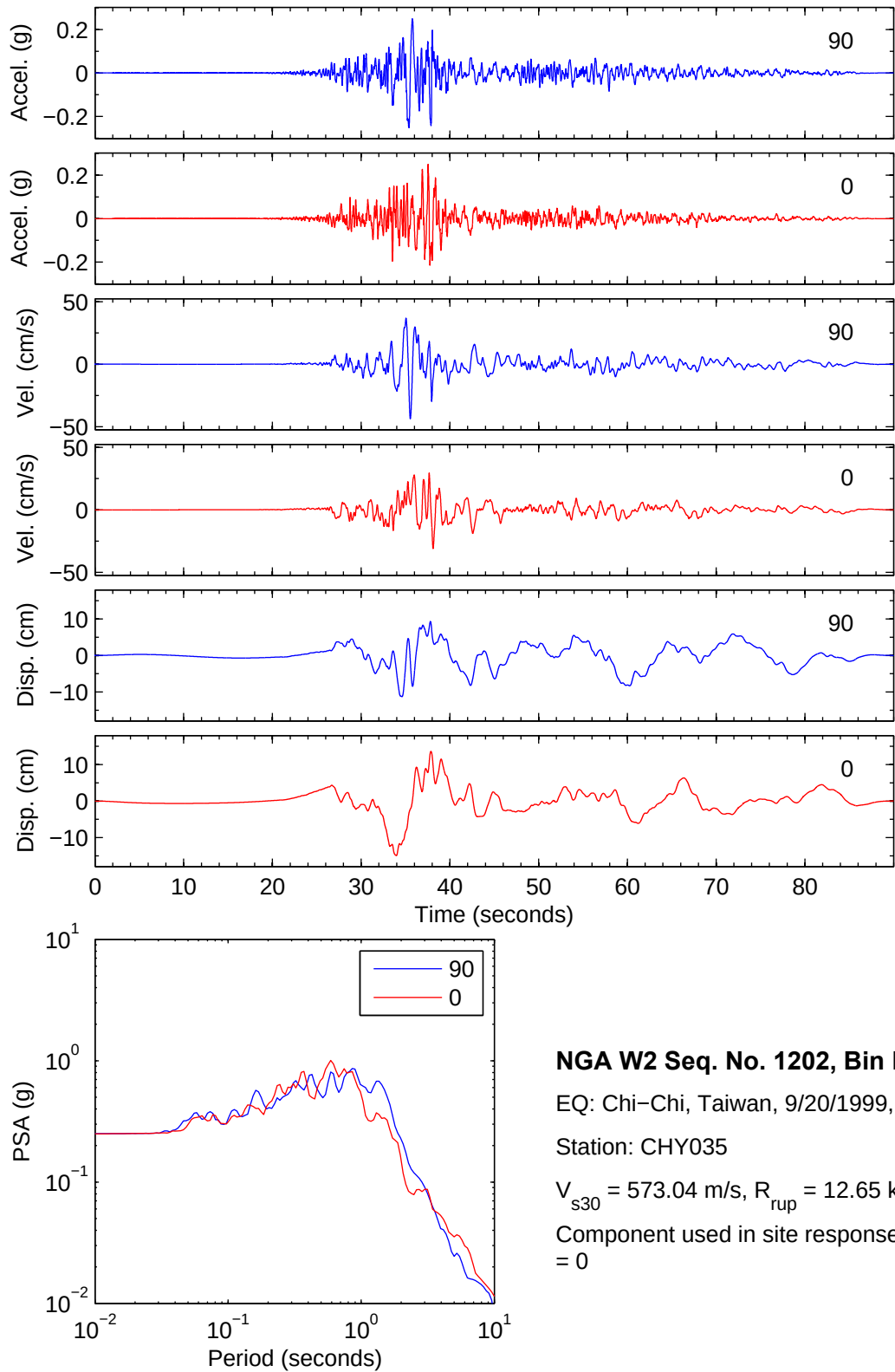


Figure D-38: Input motion NGA W2 Seq. No. 1202, Bin No. 5

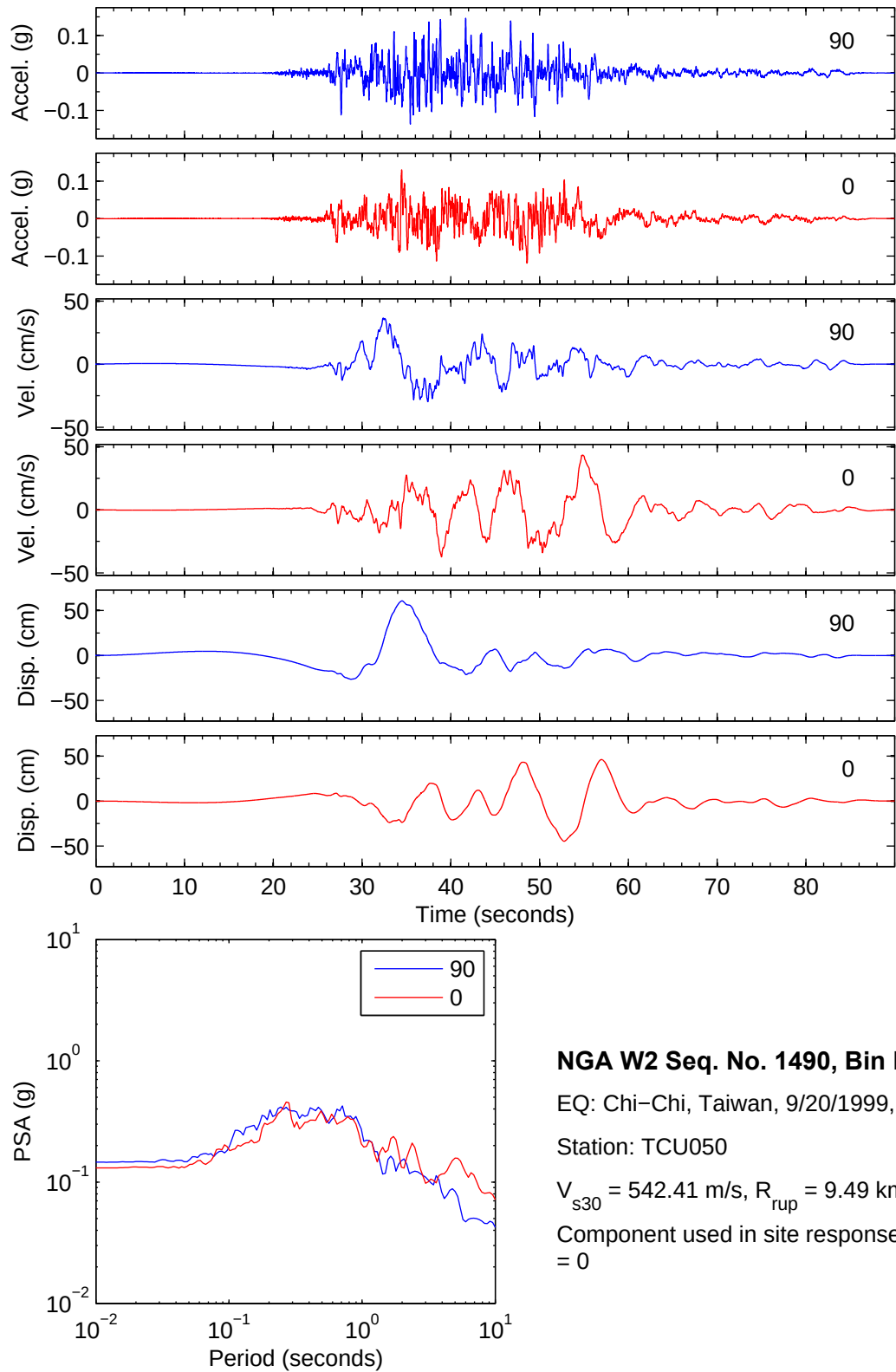


Figure D-39: Input motion NGA W2 Seq. No. 1490, Bin No. 5

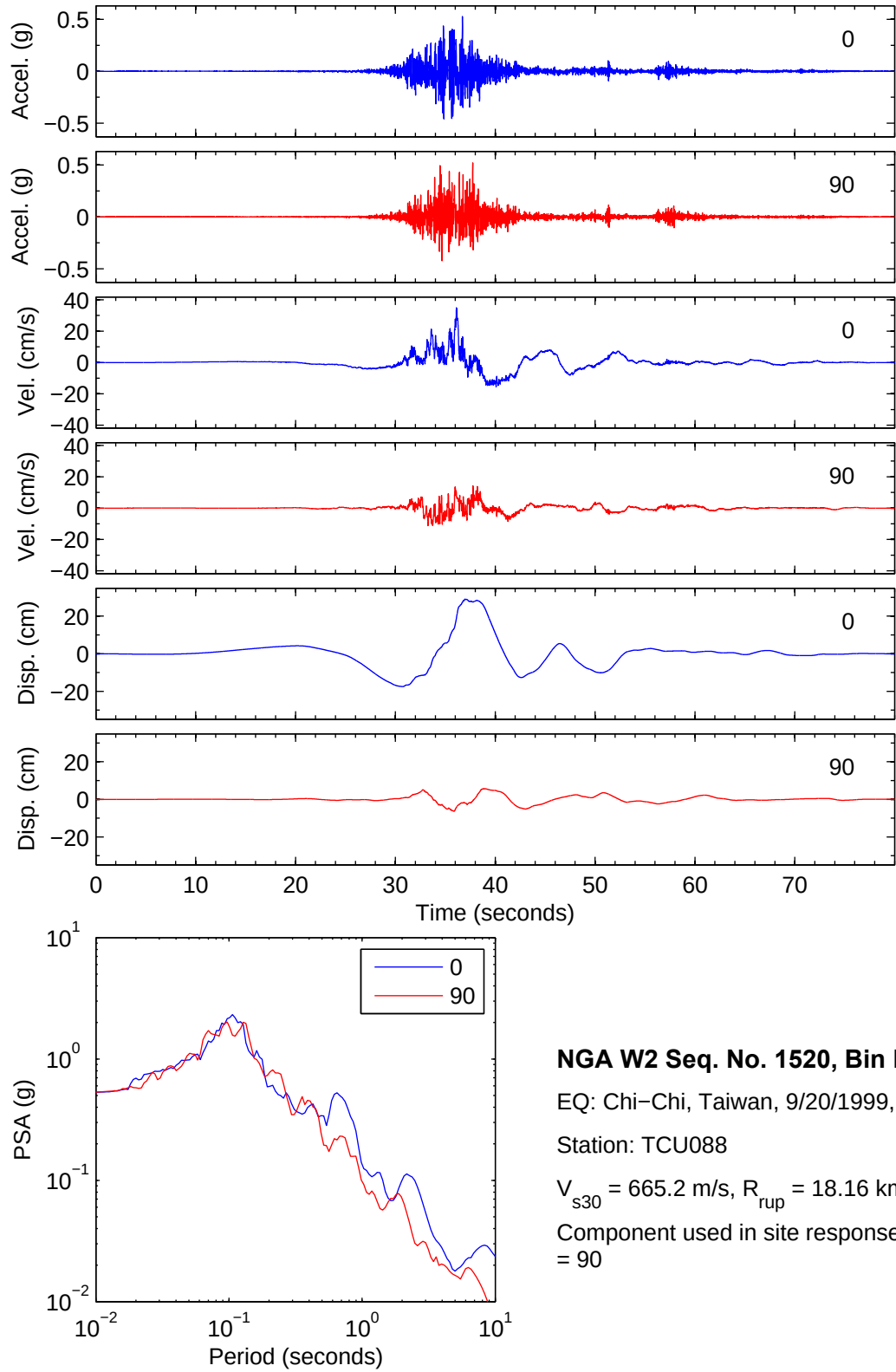


Figure D-40: Input motion NGA W2 Seq. No. 1520, Bin No. 5

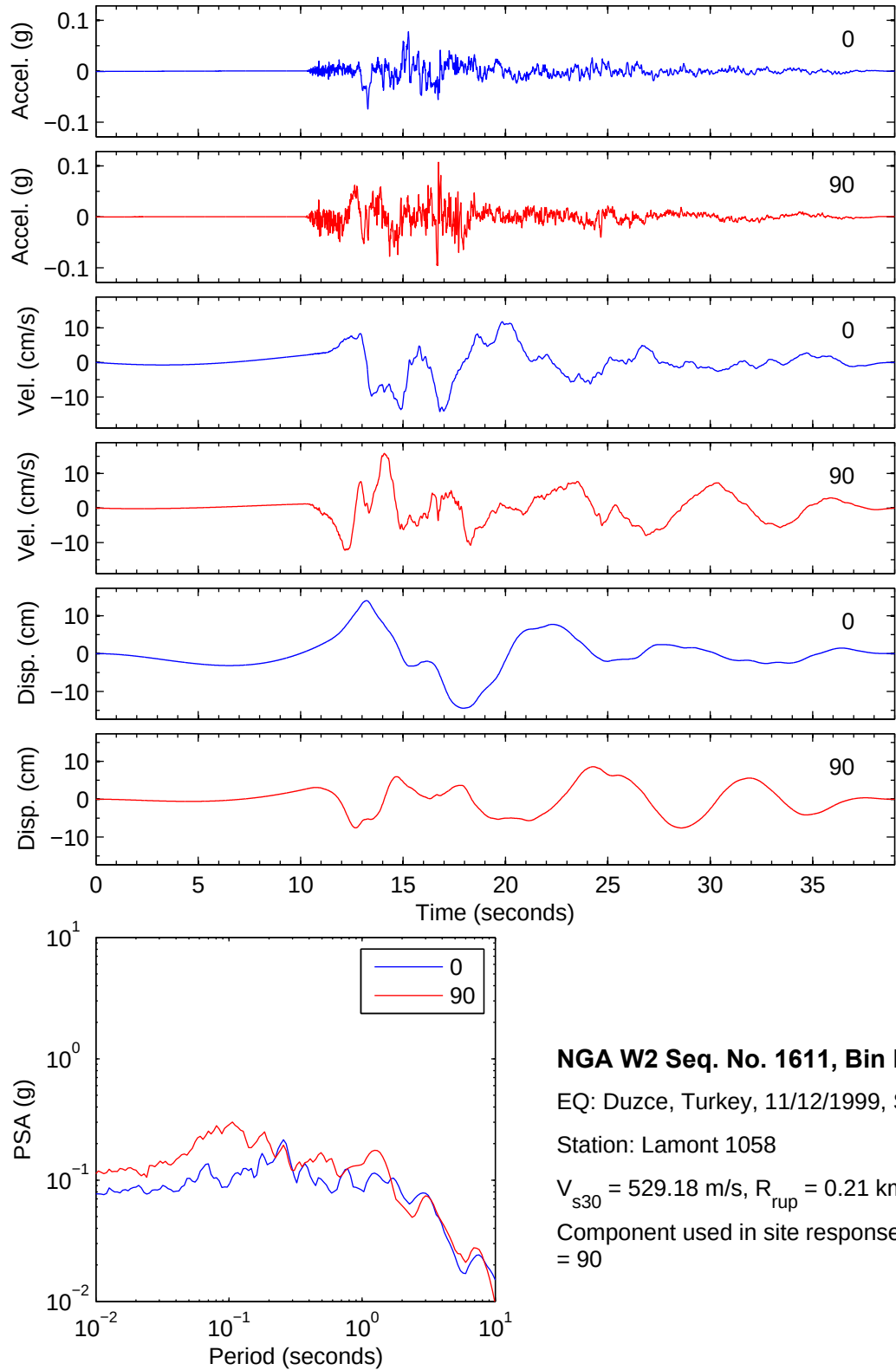


Figure D-41: Input motion NGA W2 Seq. No. 1611, Bin No. 5

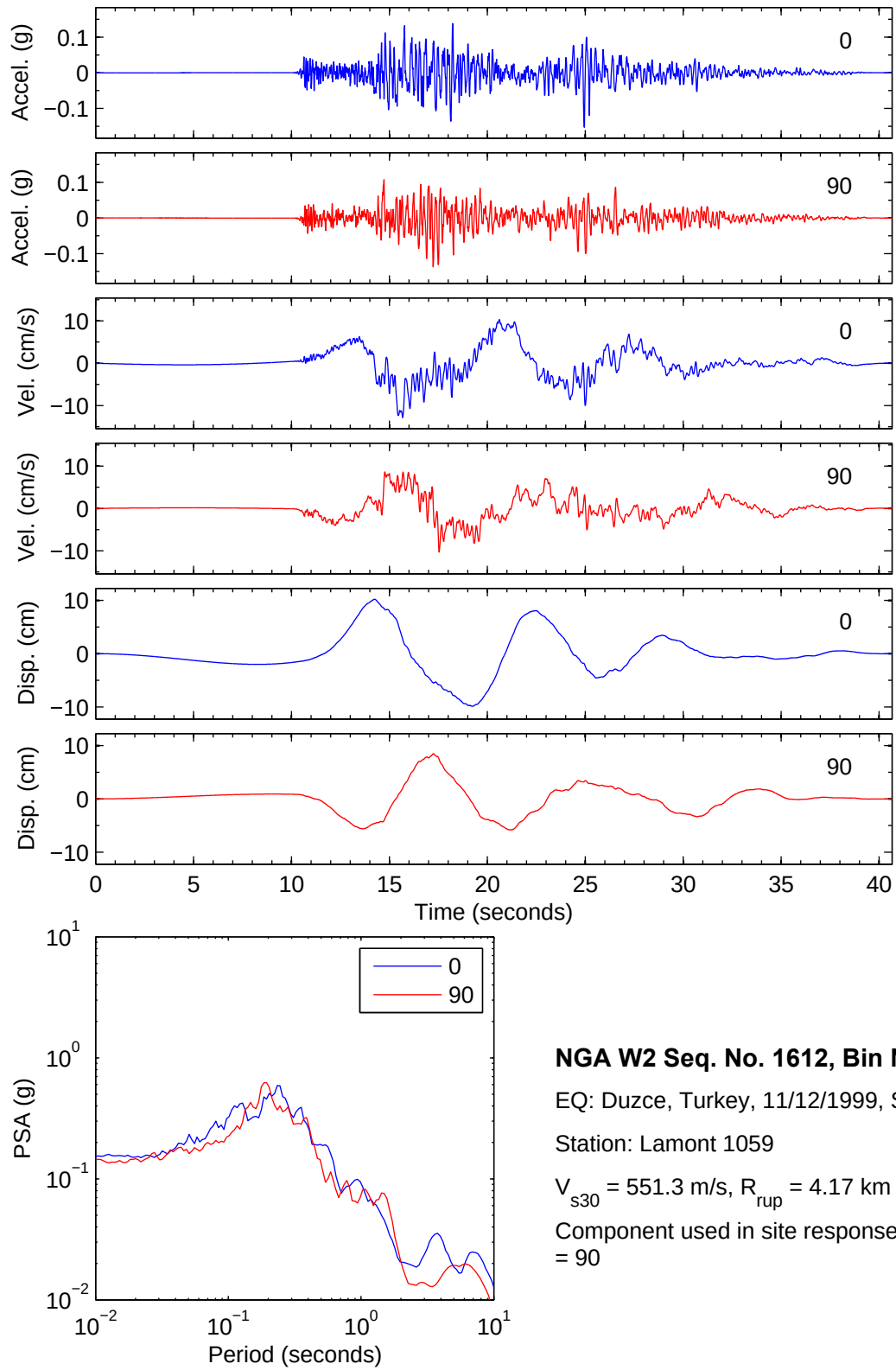


Figure D-42: Input motion NGA W2 Seq. No. 1612, Bin No. 5

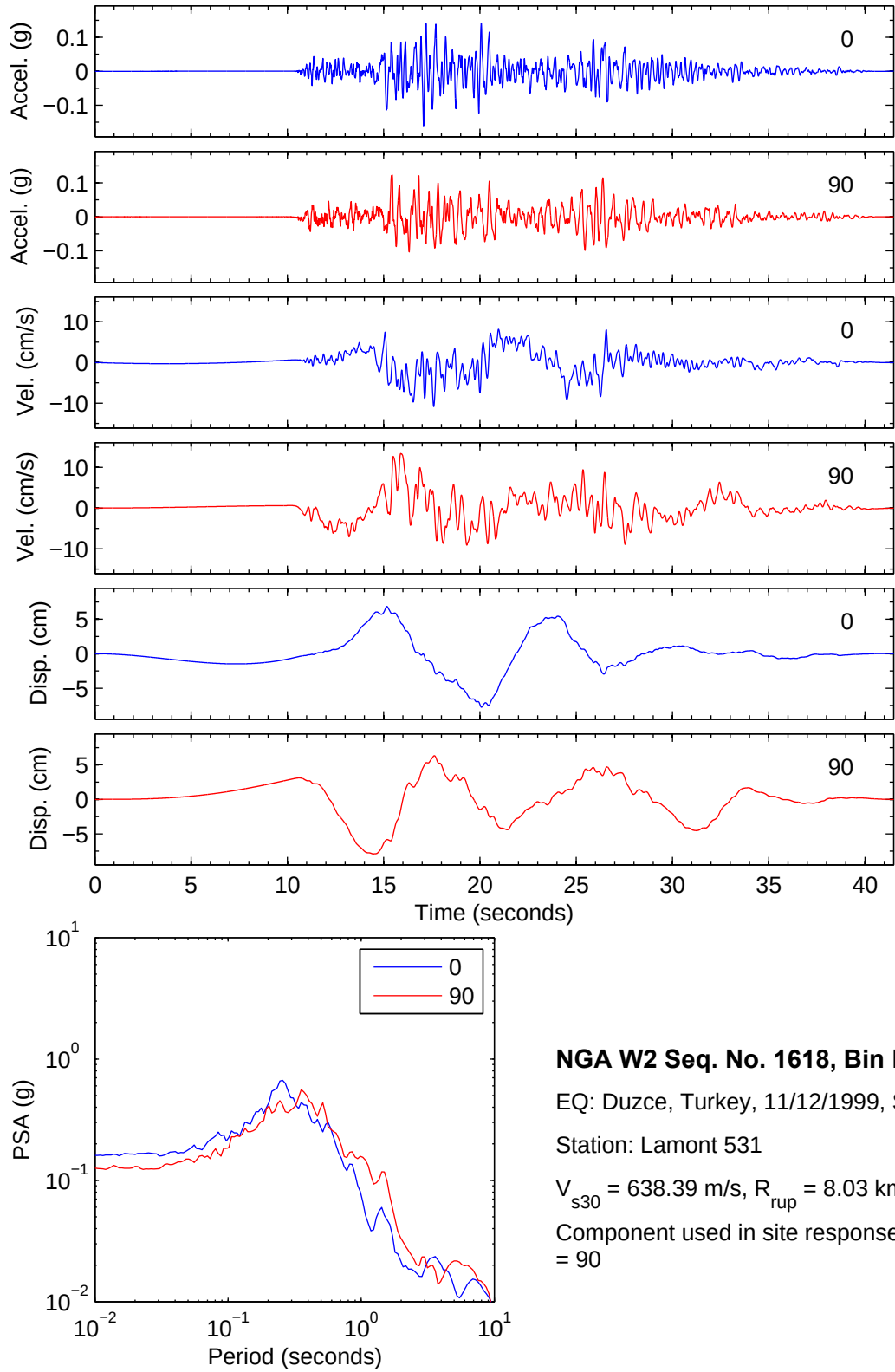


Figure D-43: Input motion NGA W2 Seq. No. 1618, Bin No. 5

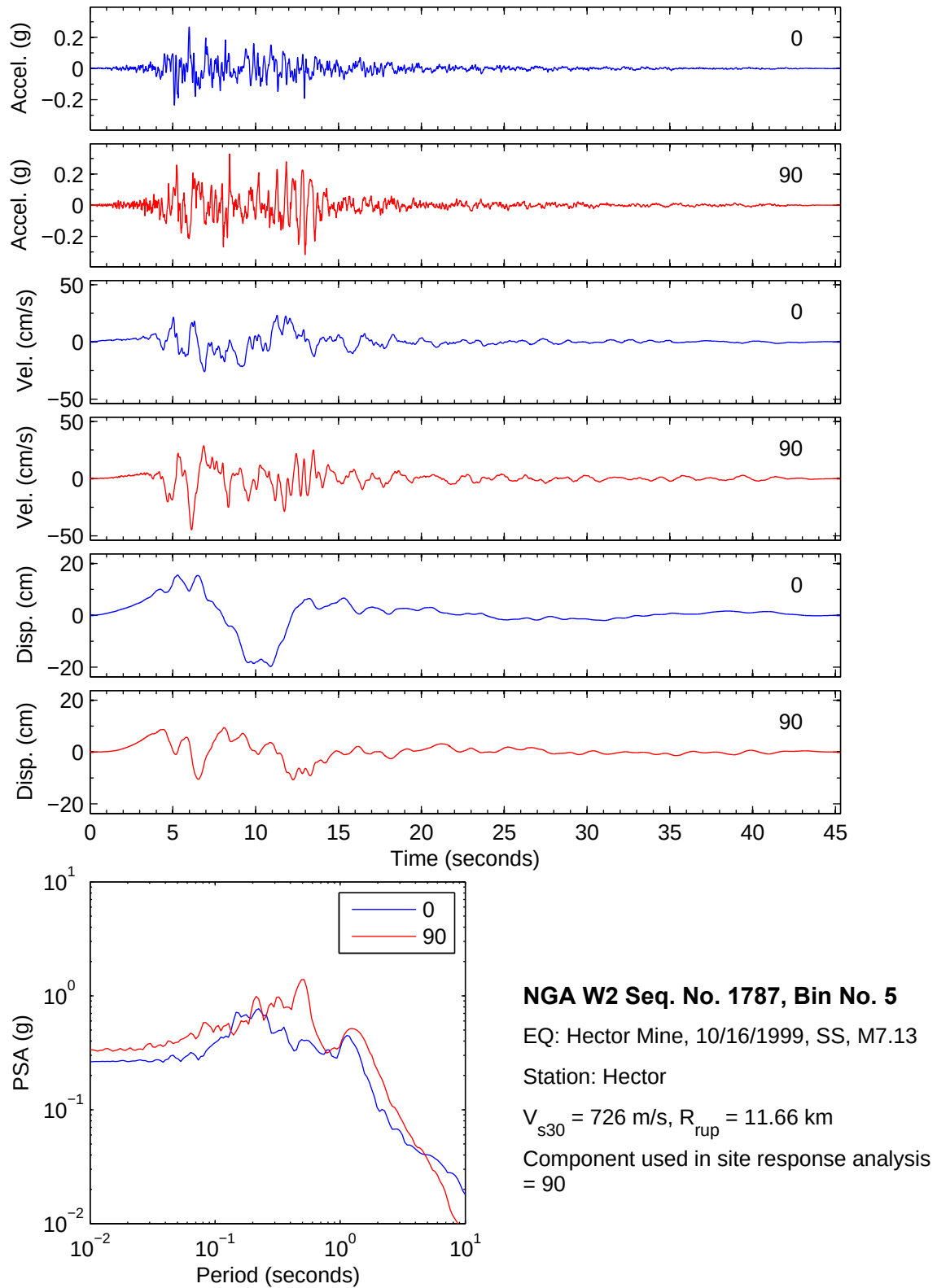


Figure D-44: Input motion NGA W2 Seq. No. 1787, Bin No. 5

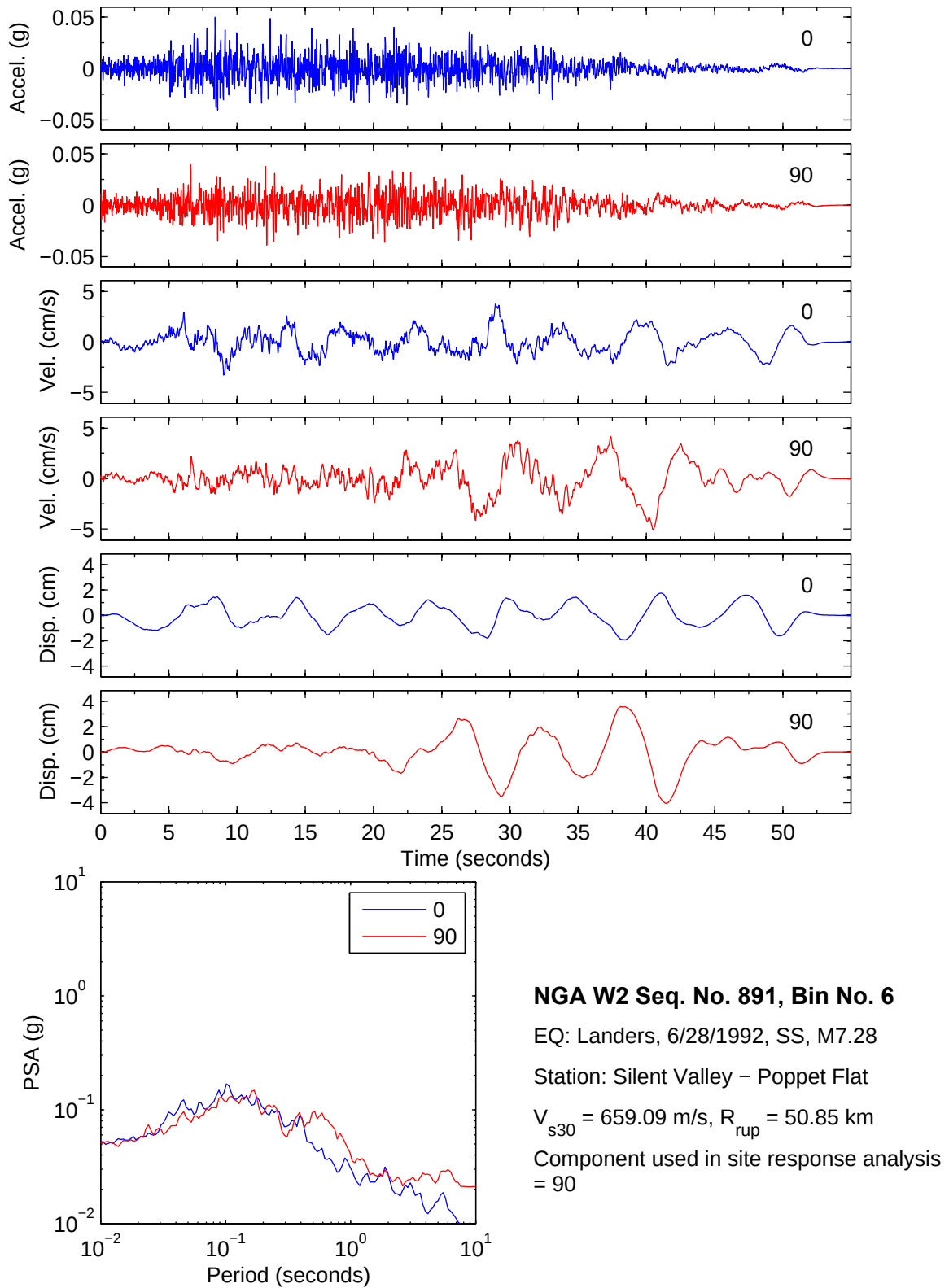


Figure D-45: Input motion NGA W2 Seq. No. 891, Bin No. 6

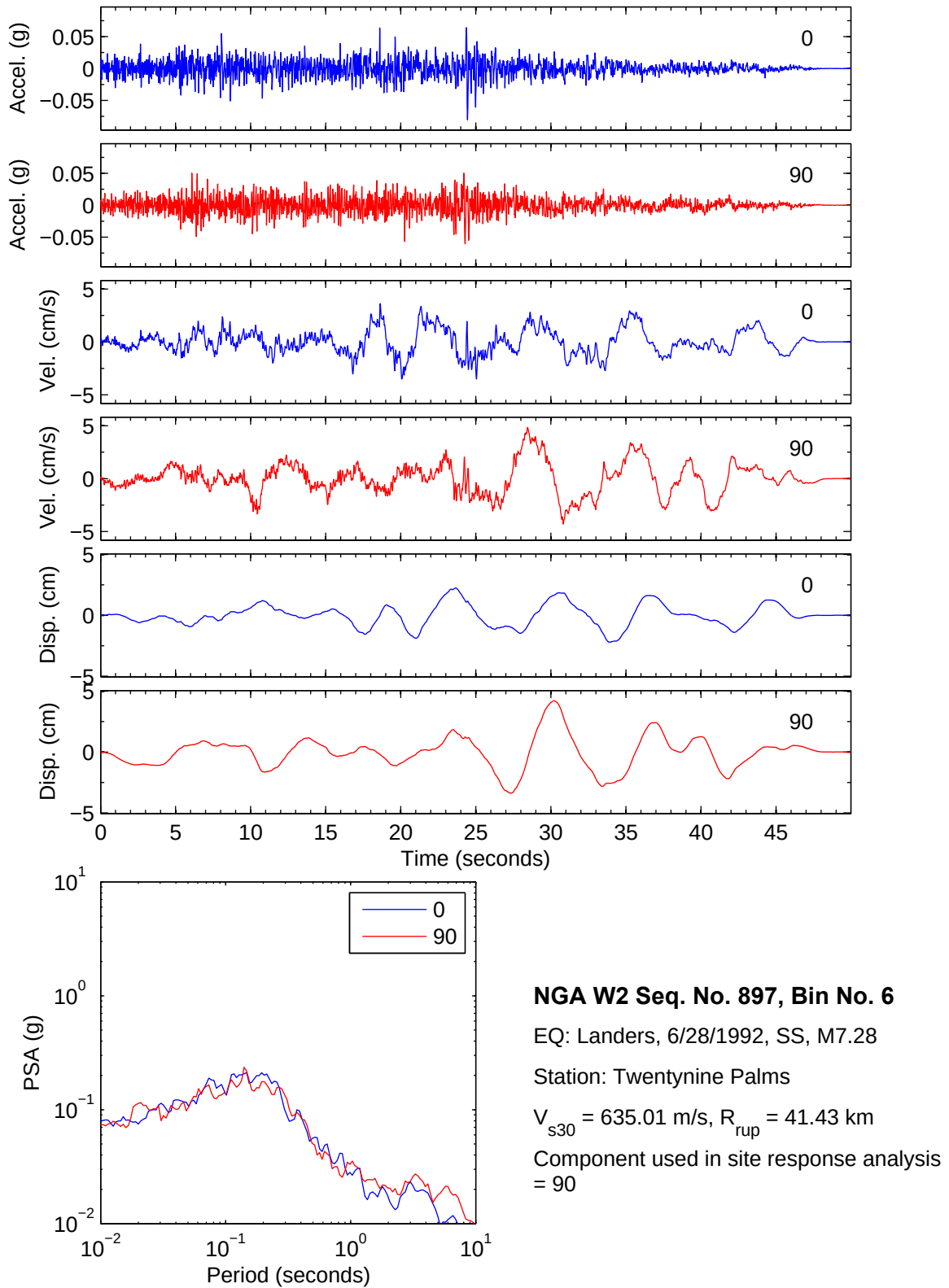


Figure D-46: Input motion NGA W2 Seq. No. 897, Bin No. 6

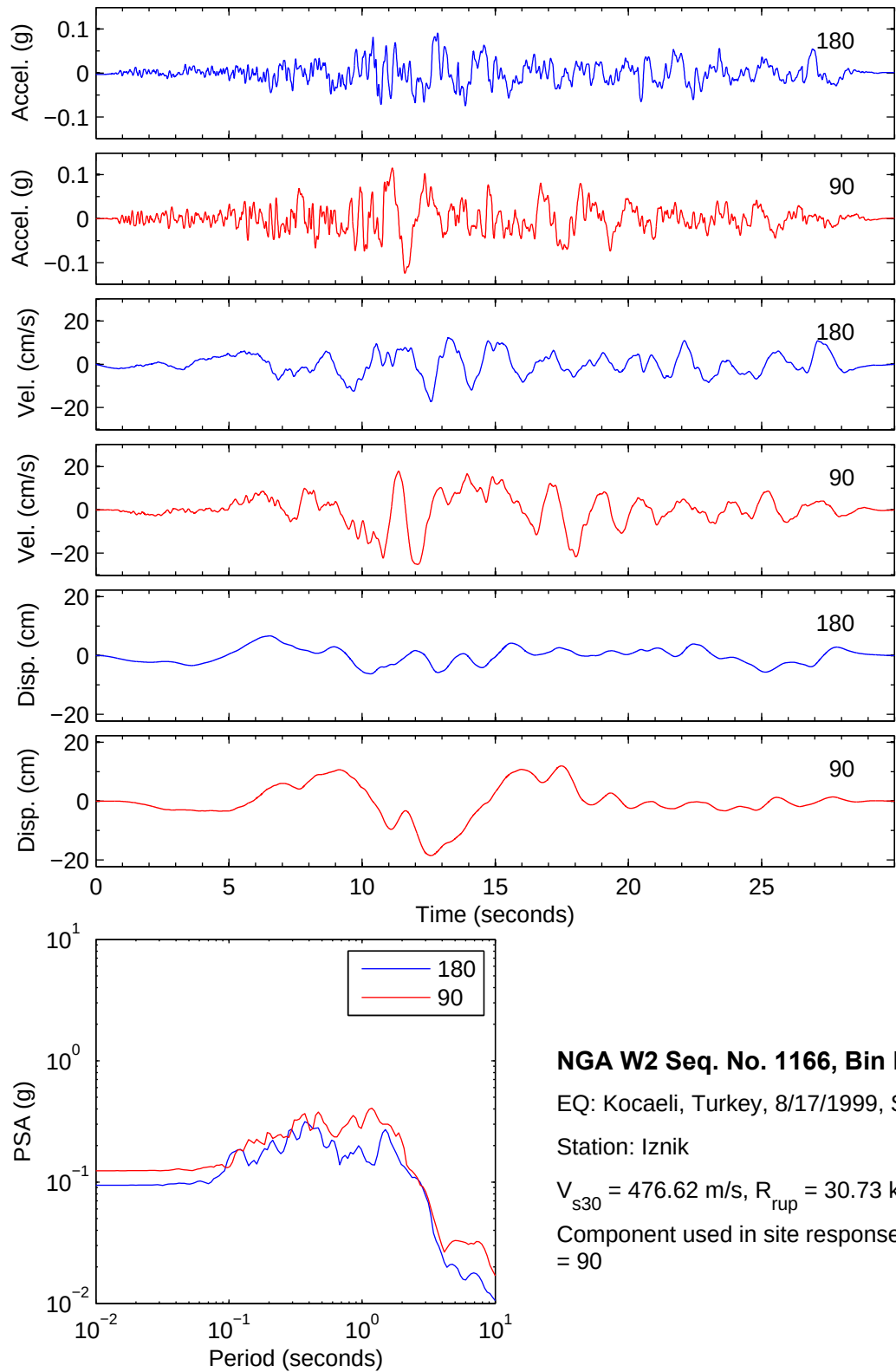


Figure D-47: Input motion NGA W2 Seq. No. 1166, Bin No. 6

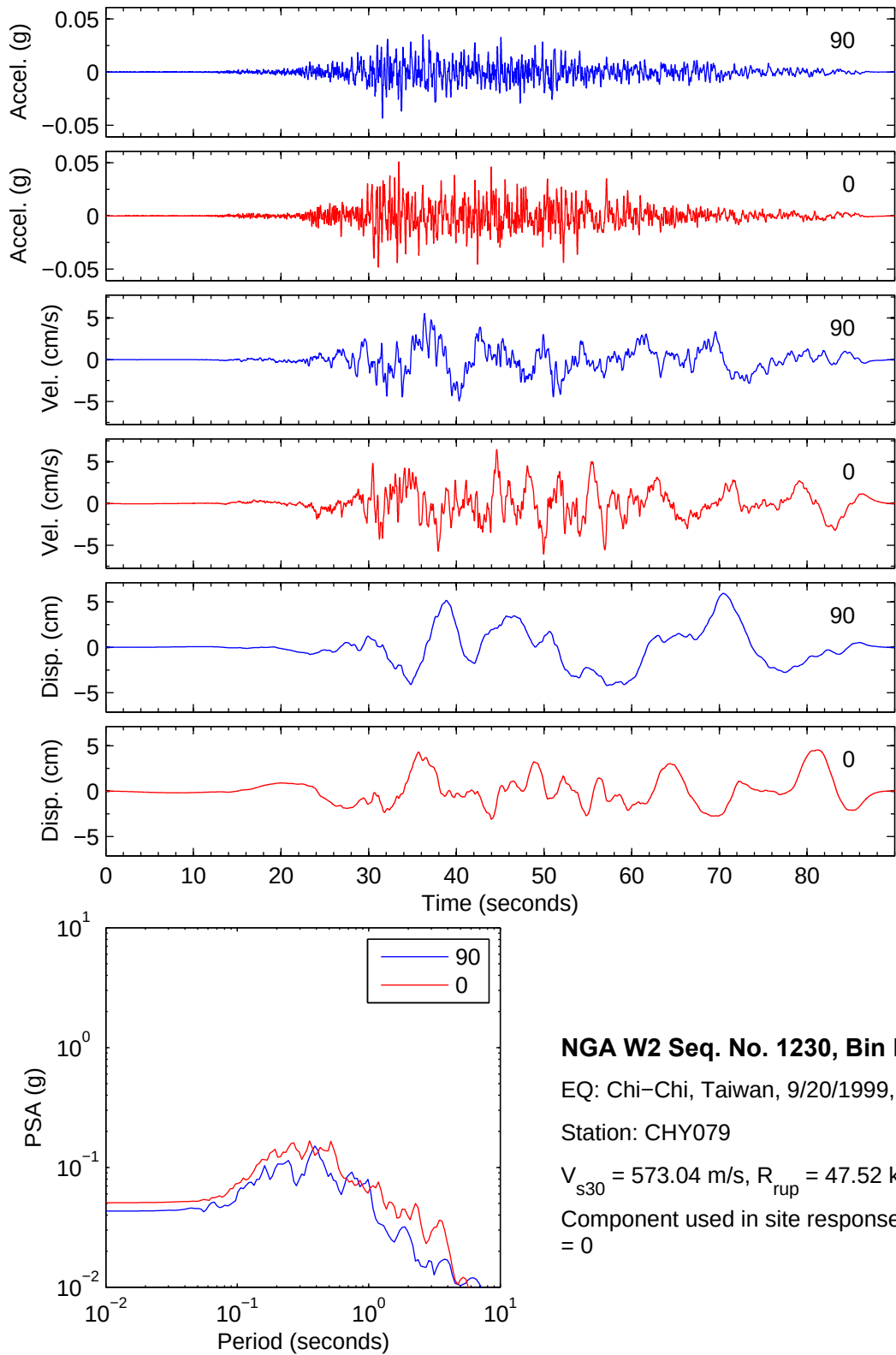


Figure D-48: Input motion NGA W2 Seq. No. 1230, Bin No. 6

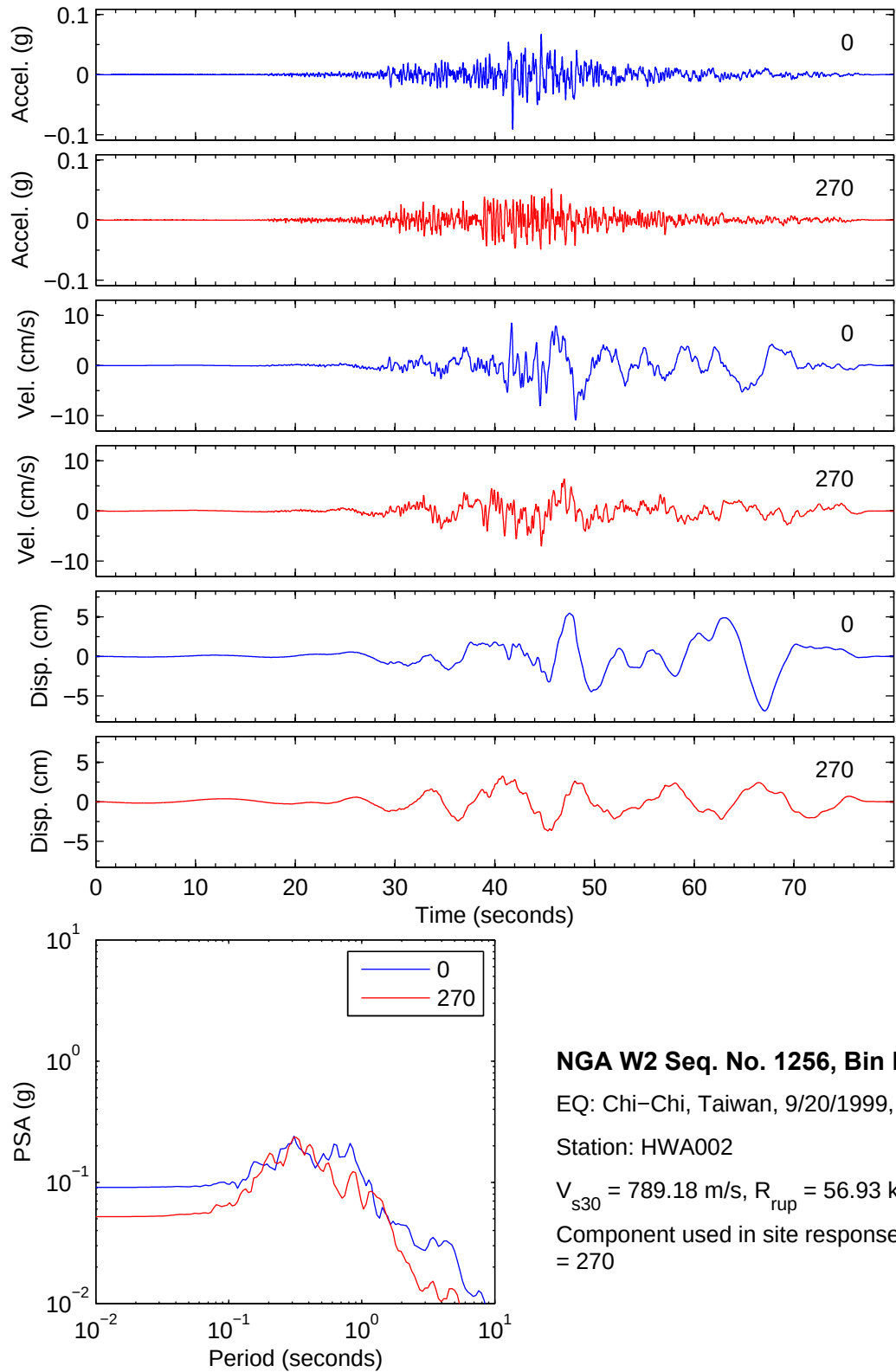


Figure D-49: Input motion NGA W2 Seq. No. 1256, Bin No. 6

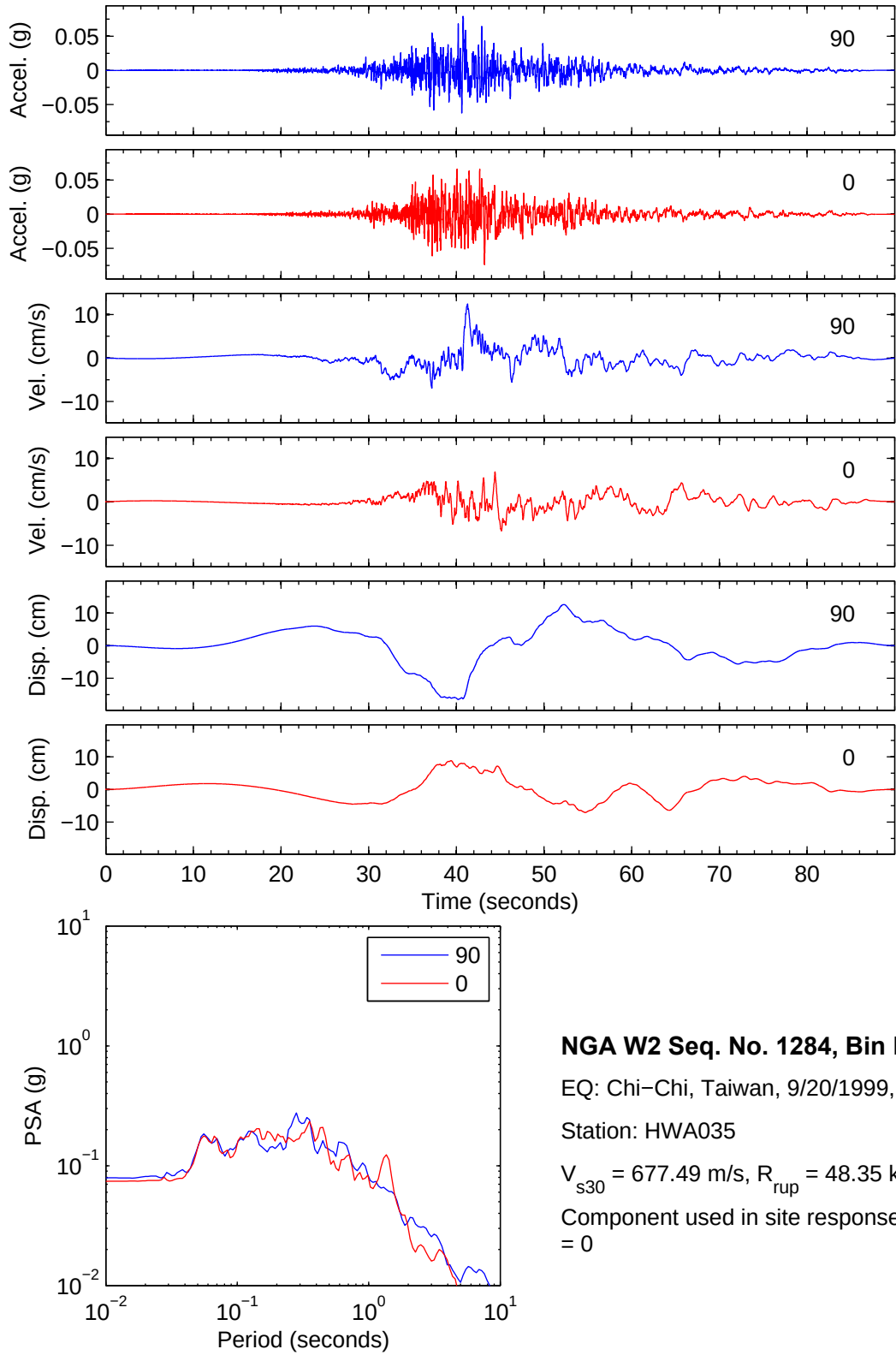


Figure D-50: Input motion NGA W2 Seq. No. 1284, Bin No. 6

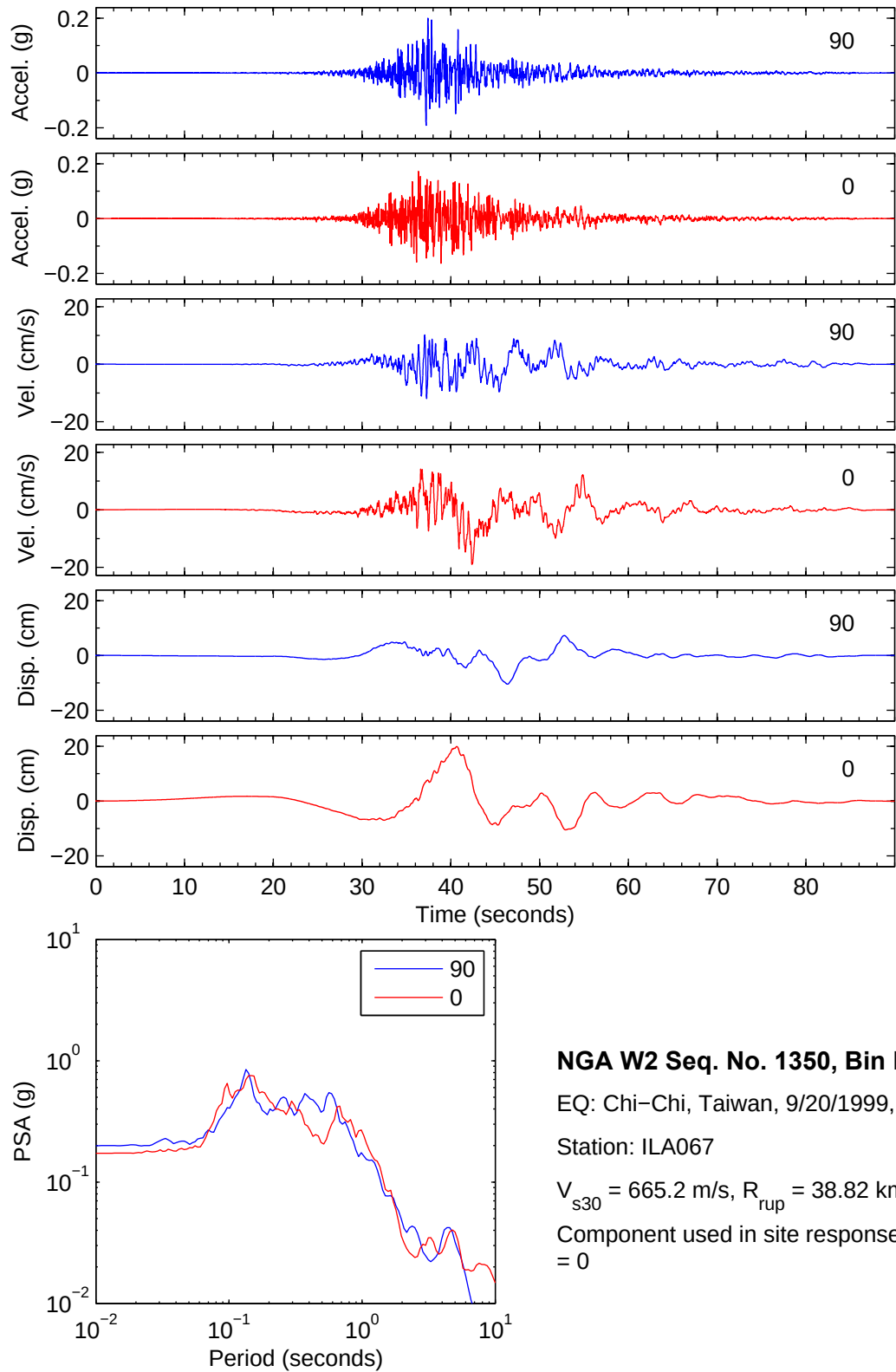


Figure D-51: Input motion NGA W2 Seq. No. 1350, Bin No. 6

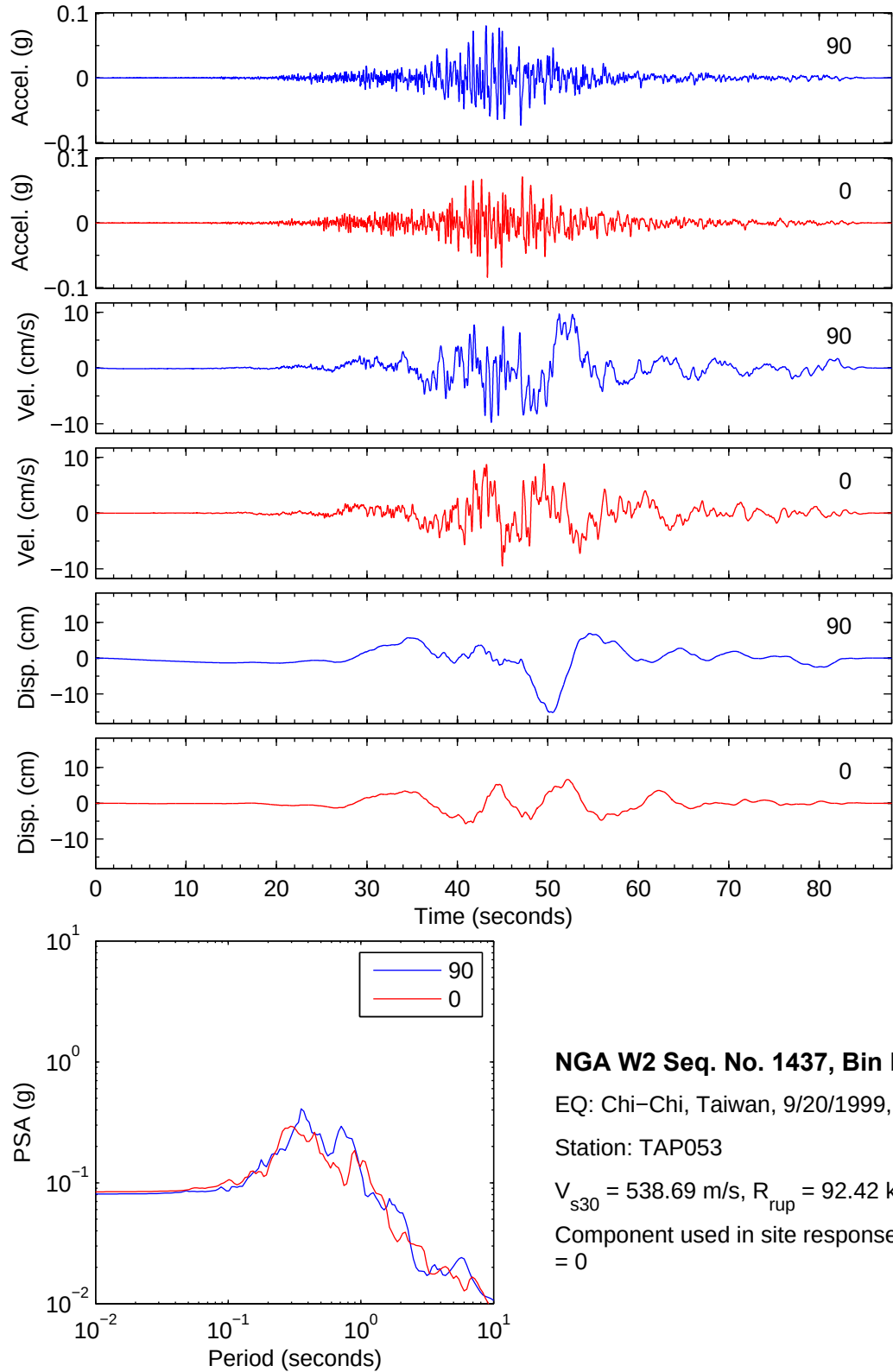
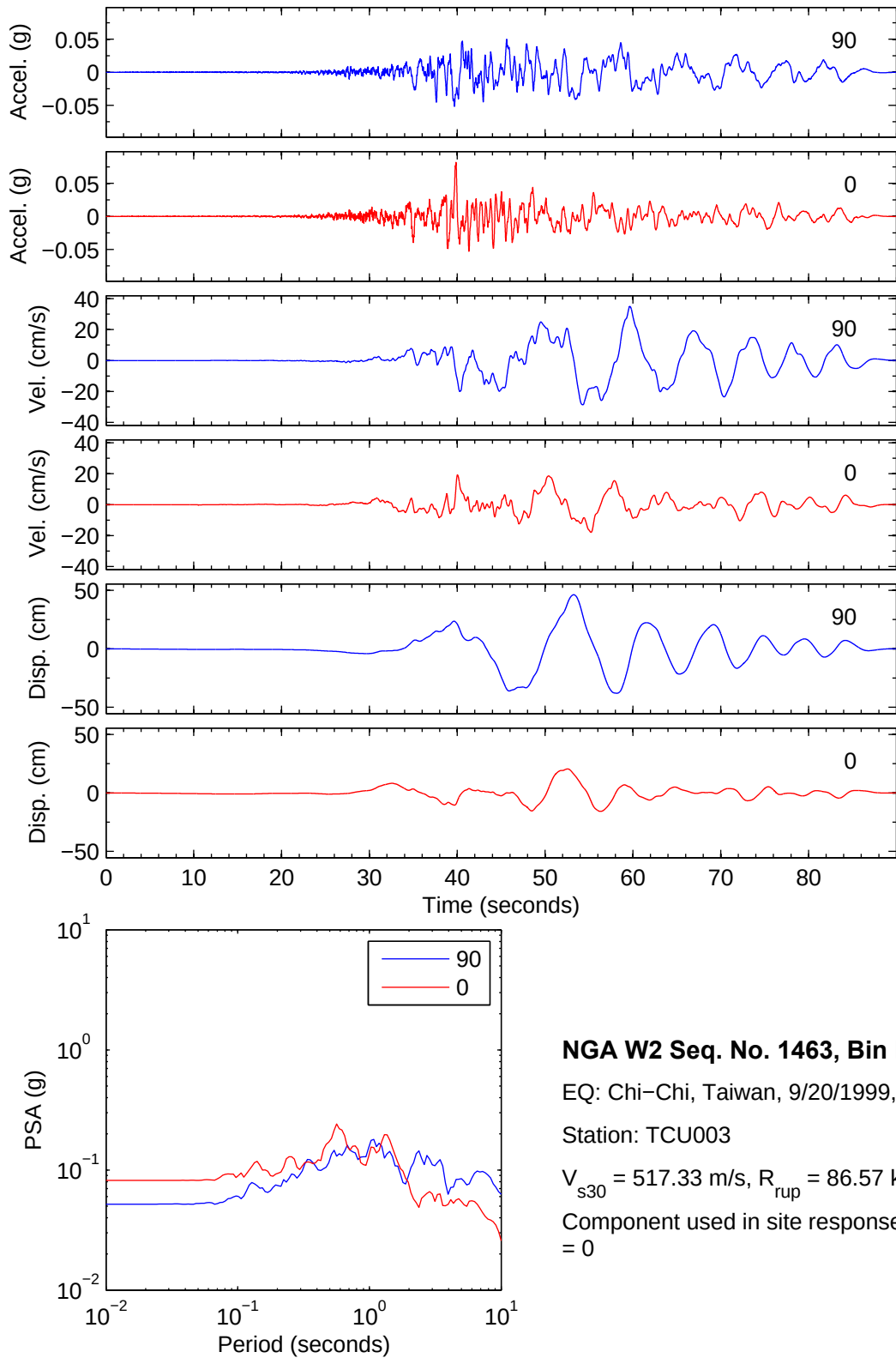


Figure D-52: Input motion NGA W2 Seq. No. 1437, Bin No. 6



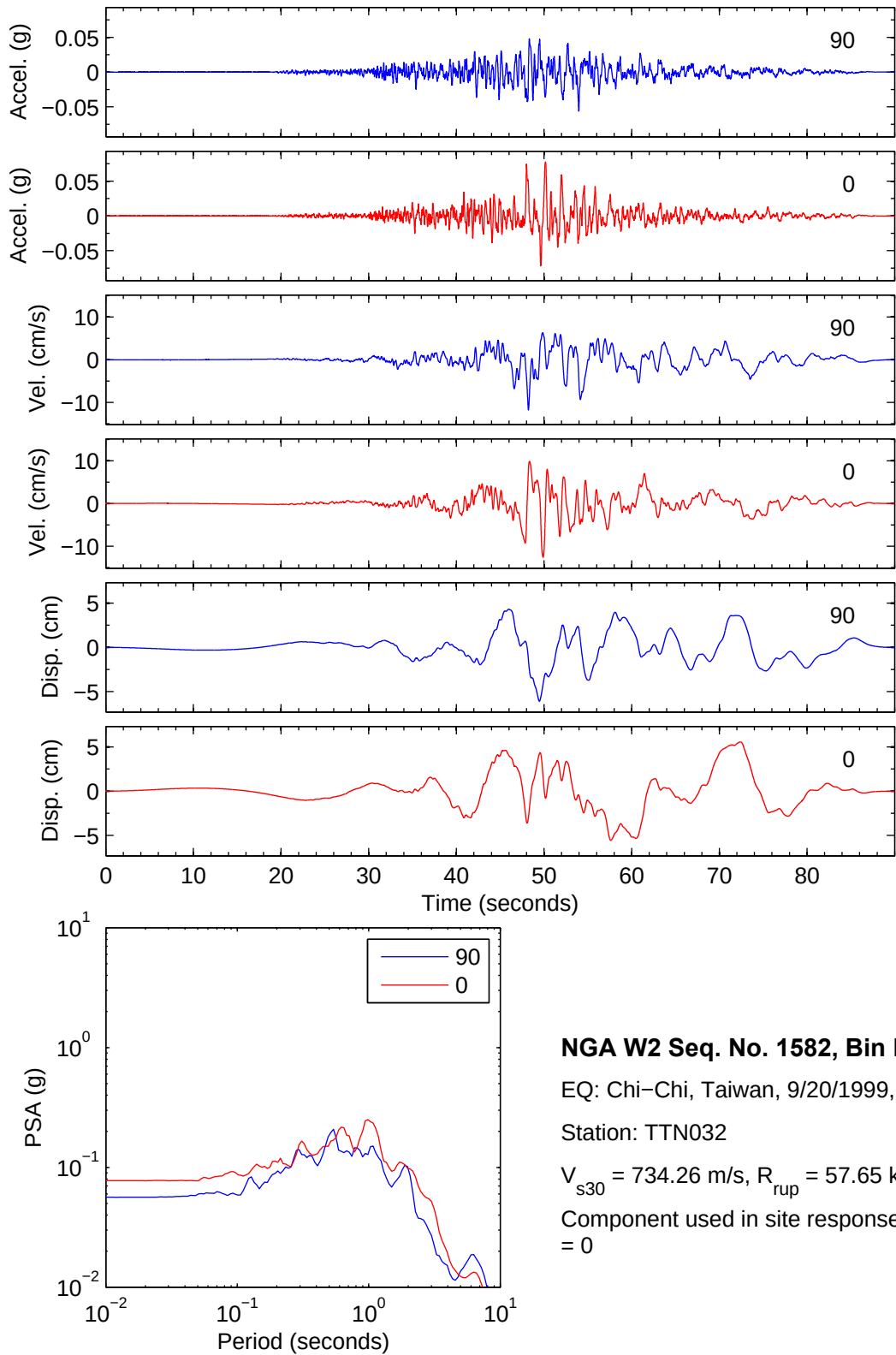


Figure D-54: Input motion NGA W2 Seq. No. 1582, Bin No. 6

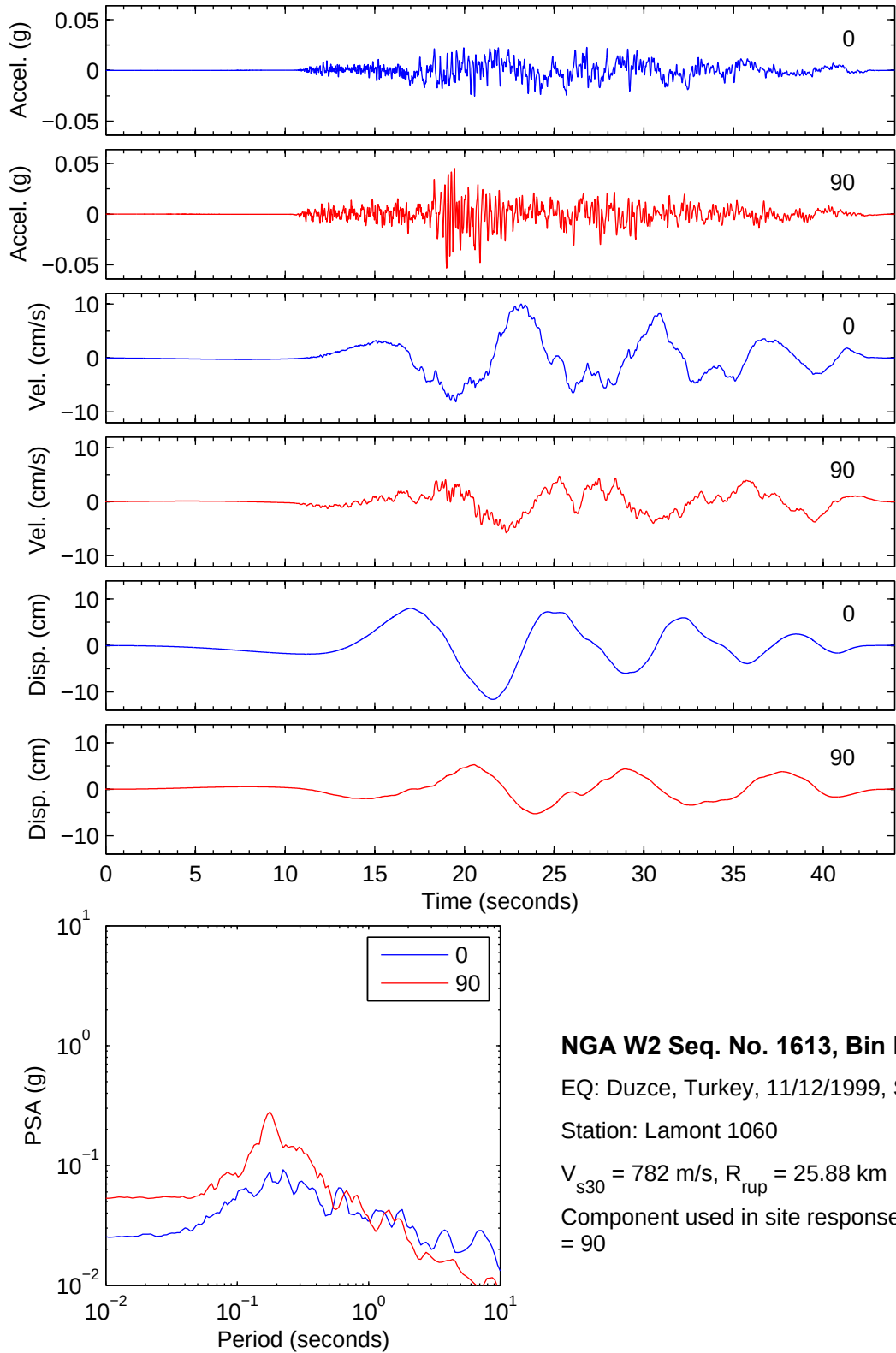


Figure D-55: Input motion NGA W2 Seq. No. 1613, Bin No. 6

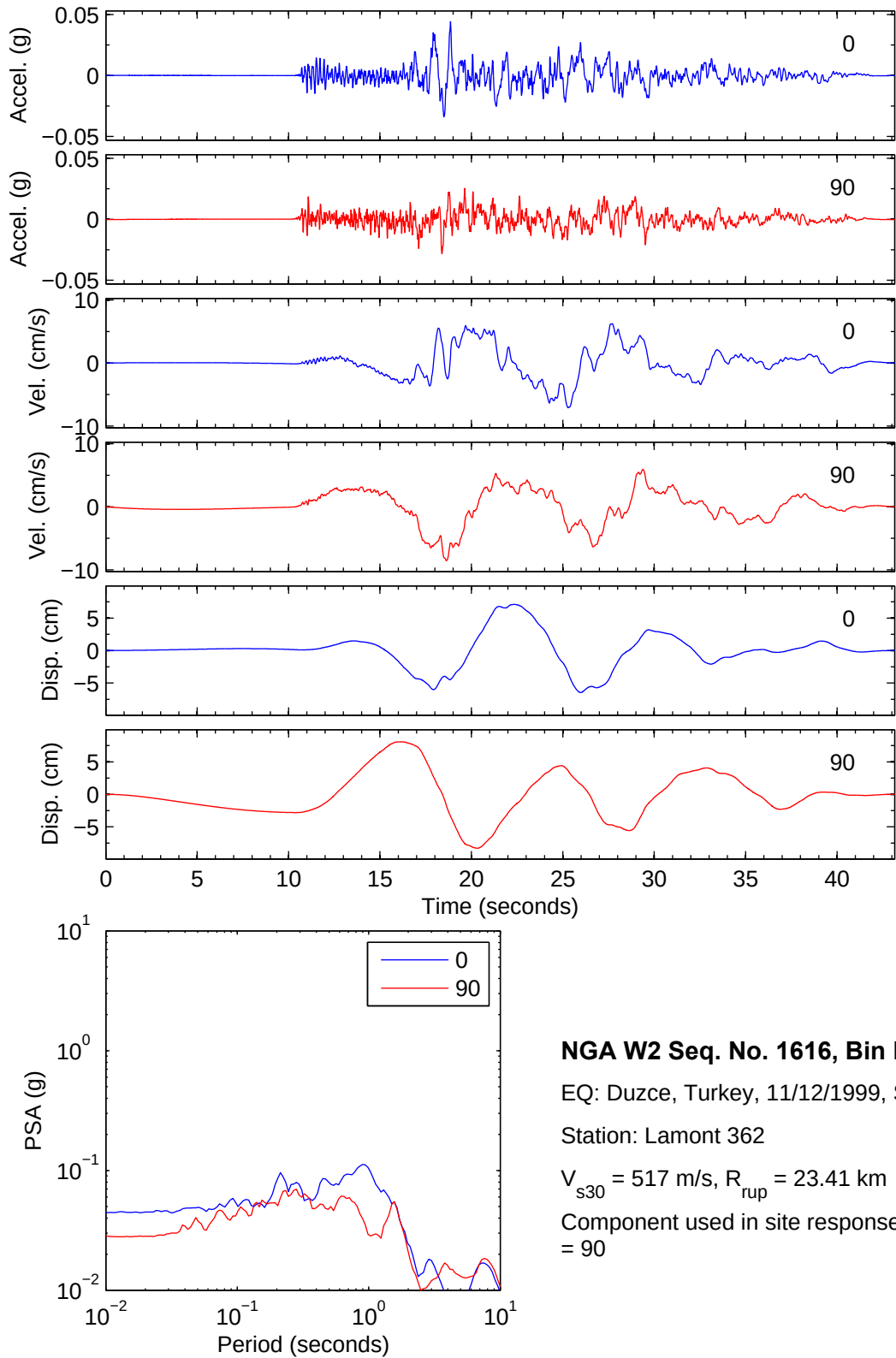


Figure D-56: Input motion NGA W2 Seq. No. 1616, Bin No. 6

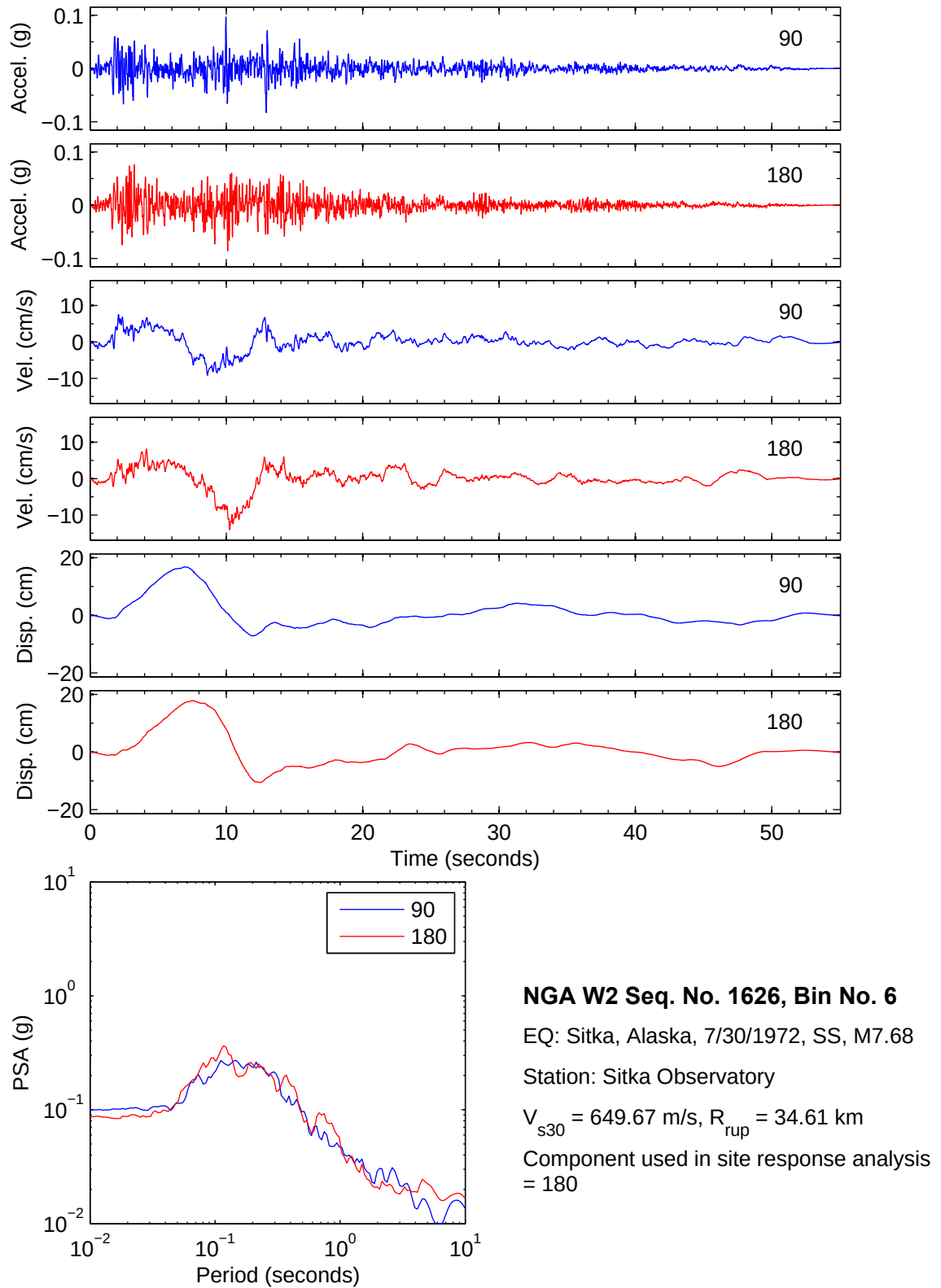


Figure D-57: Input motion NGA W2 Seq. No. 1626, Bin No. 6

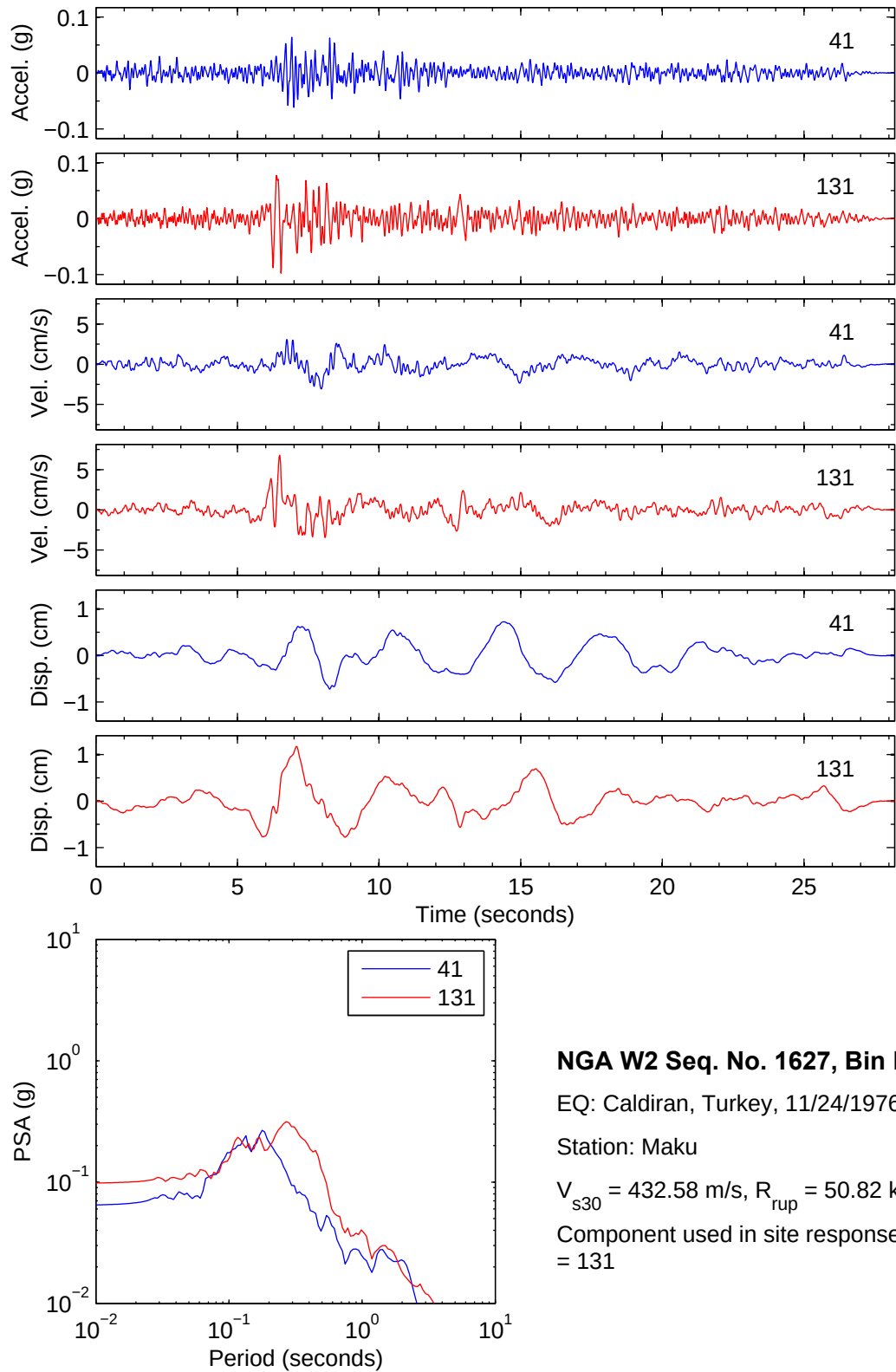


Figure D-58: Input motion NGA W2 Seq. No. 1627, Bin No. 6

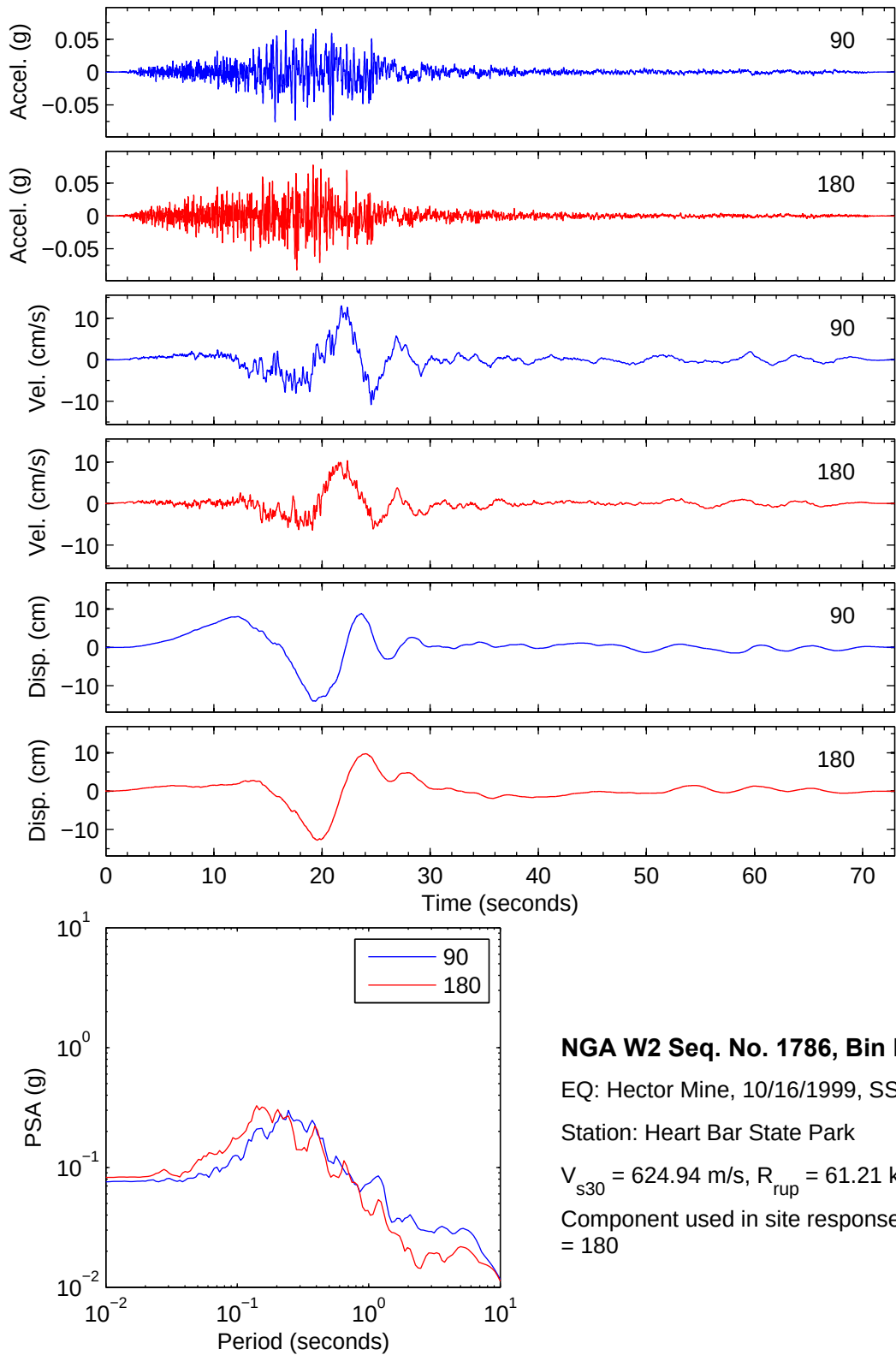


Figure D-59: Input motion NGA W2 Seq. No. 1786, Bin No. 6

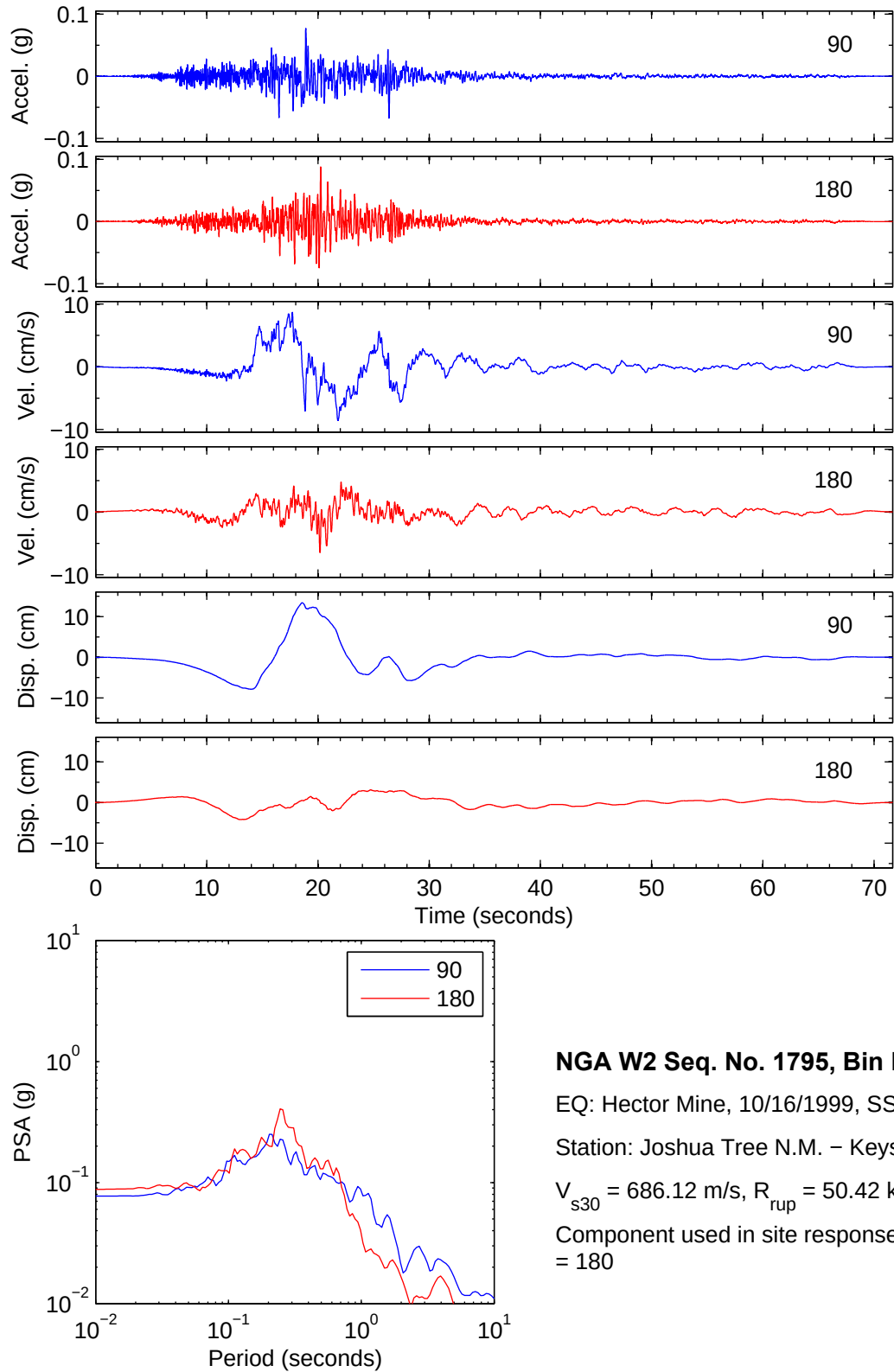


Figure D-60: Input motion NGA W2 Seq. No. 1795, Bin No. 6

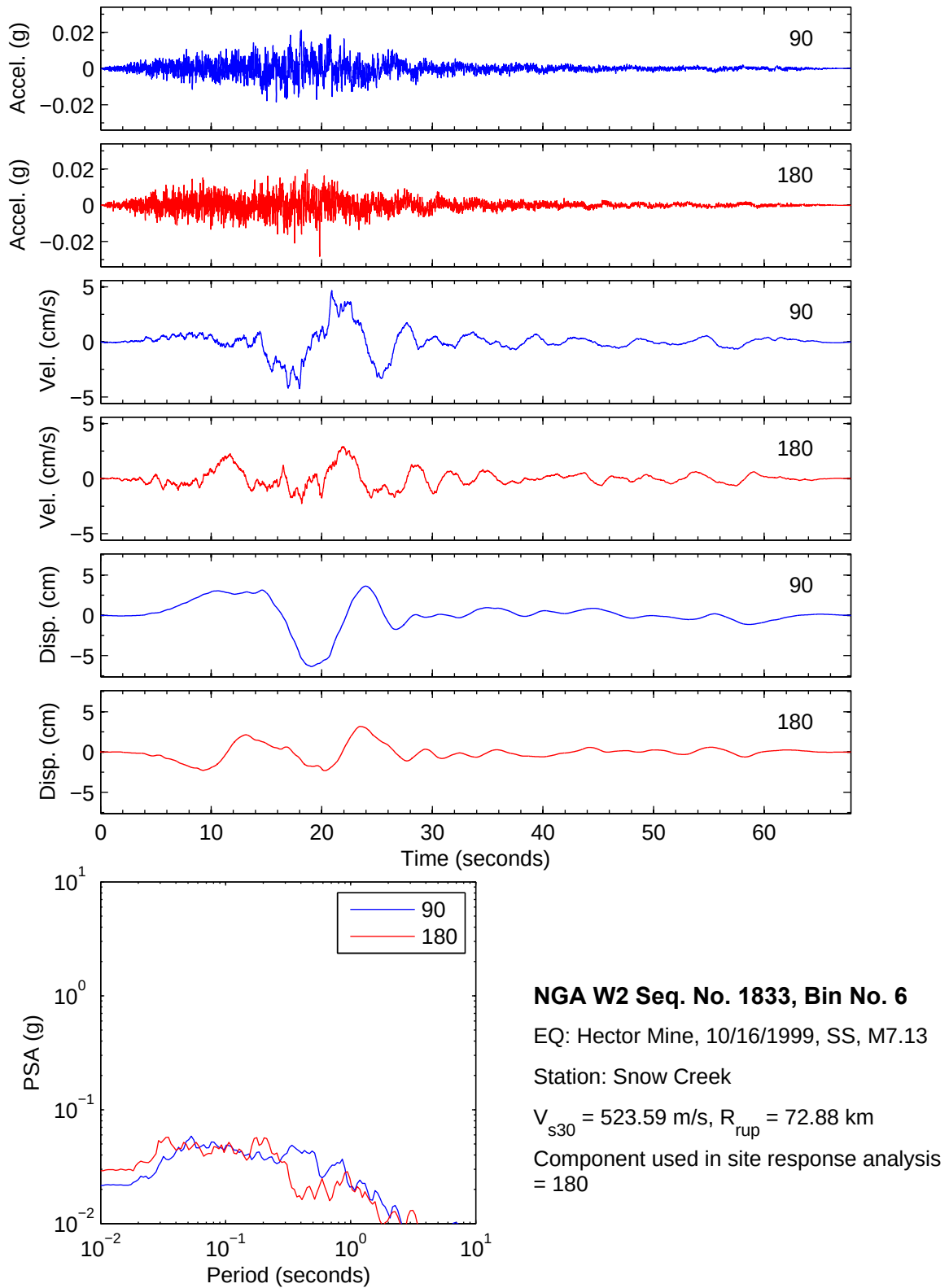


Figure D-61: Input motion NGA W2 Seq. No. 1833, Bin No. 6

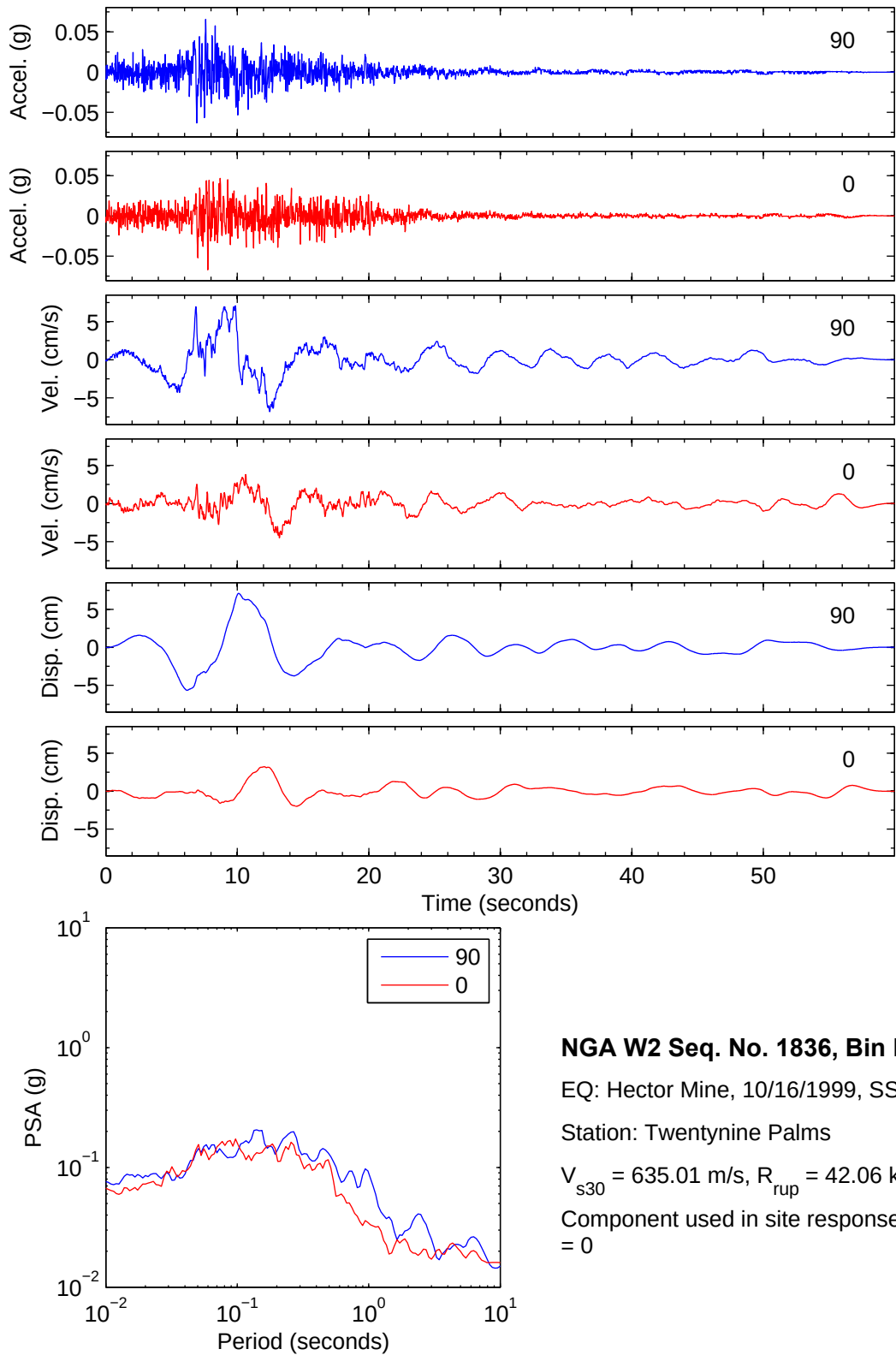


Figure D-62: Input motion NGA W2 Seq. No. 1836, Bin No. 6

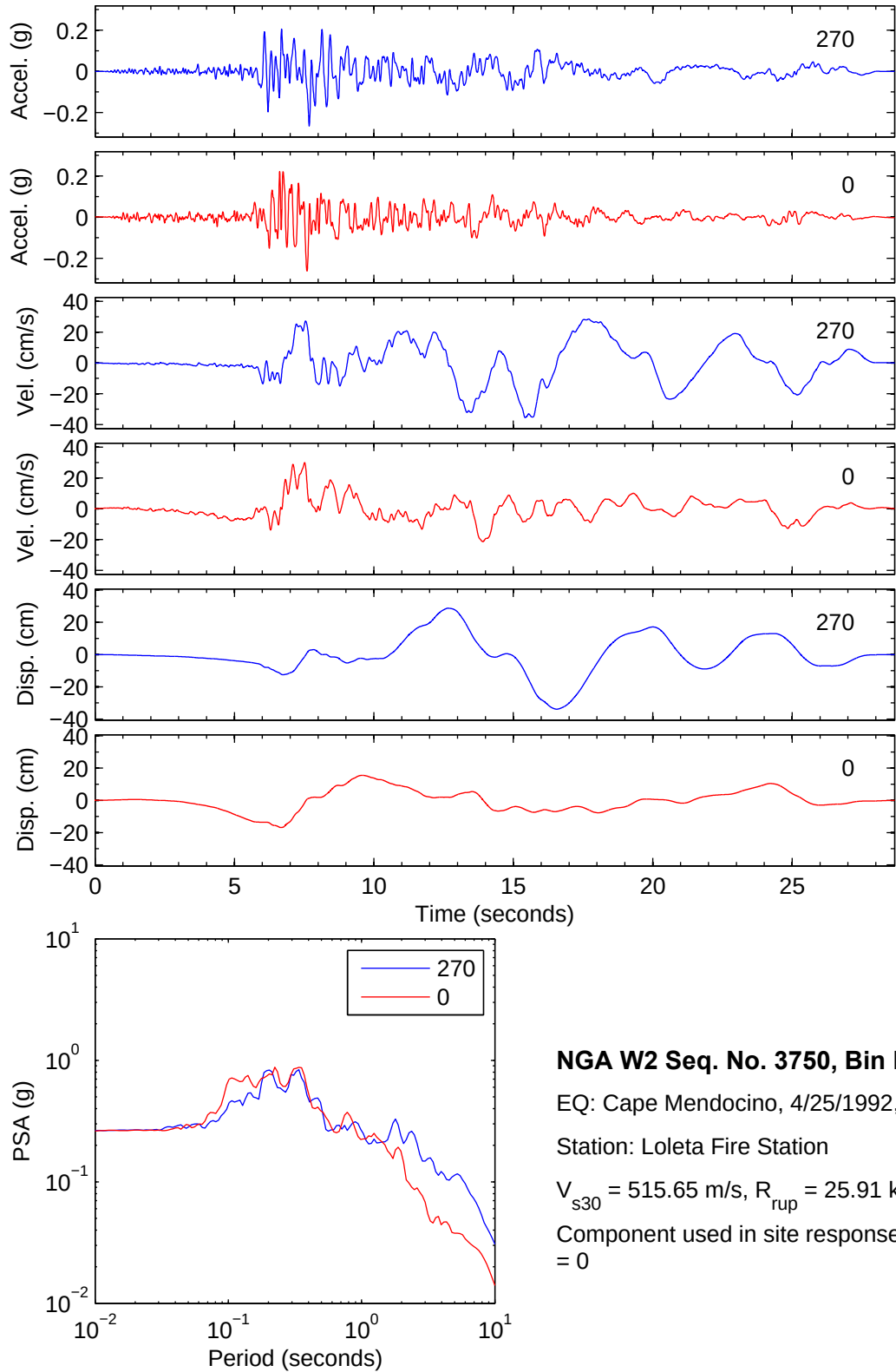


Figure D-63: Input motion NGA W2 Seq. No. 3750, Bin No. 6

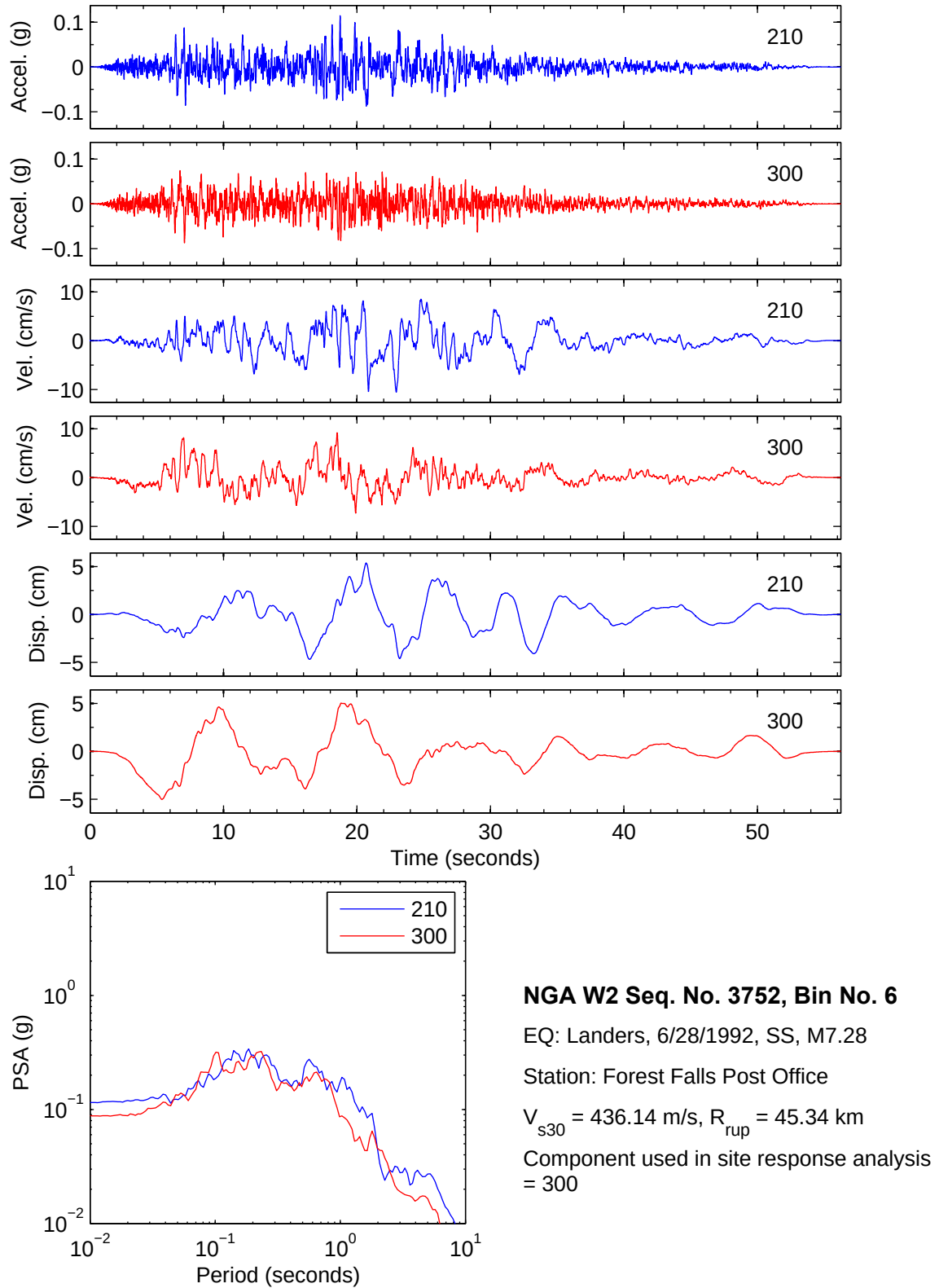


Figure D-64: Input motion NGA W2 Seq. No. 3752, Bin No. 6

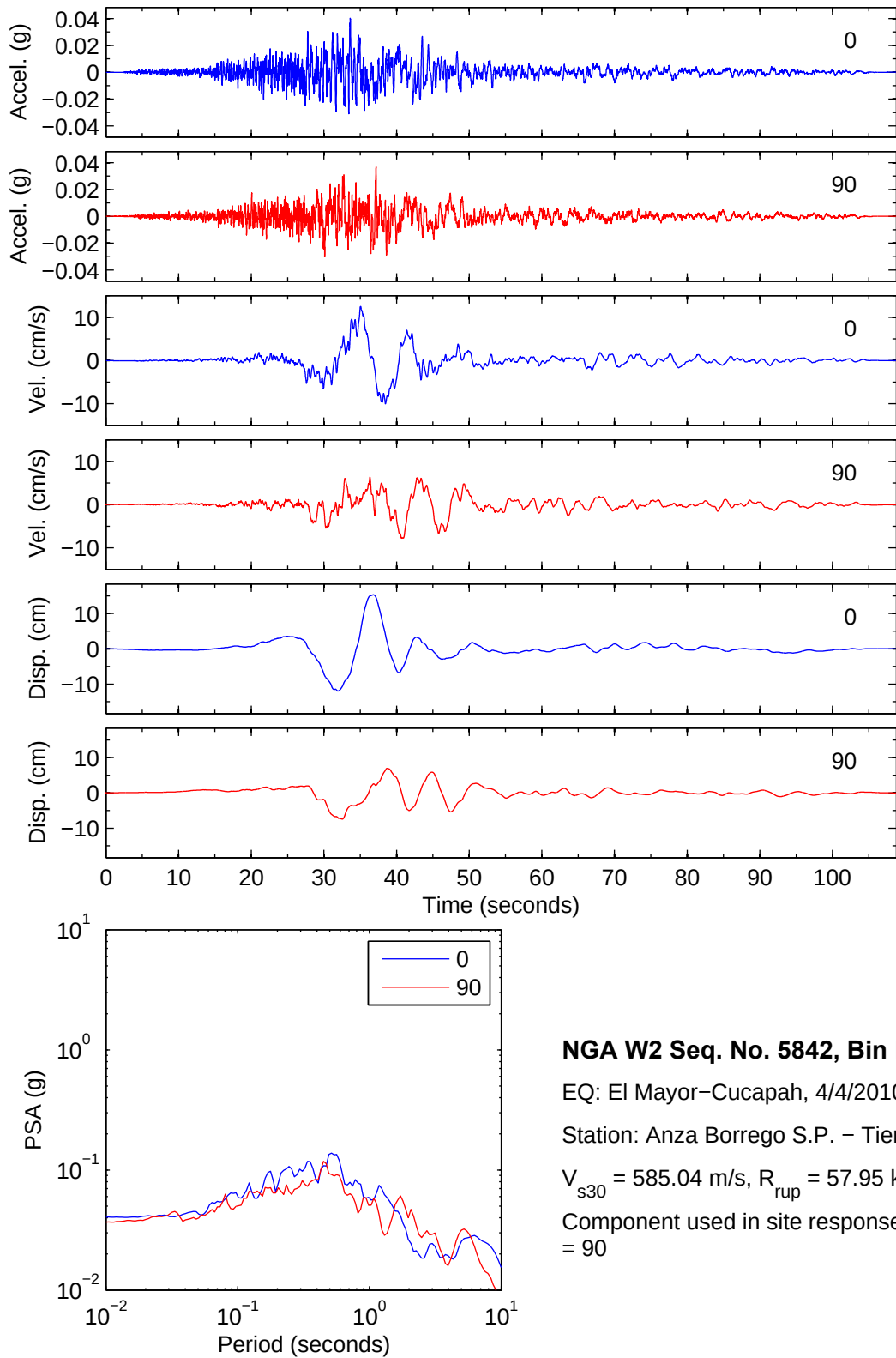


Figure D-65: Input motion NGA W2 Seq. No. 5842, Bin No. 6

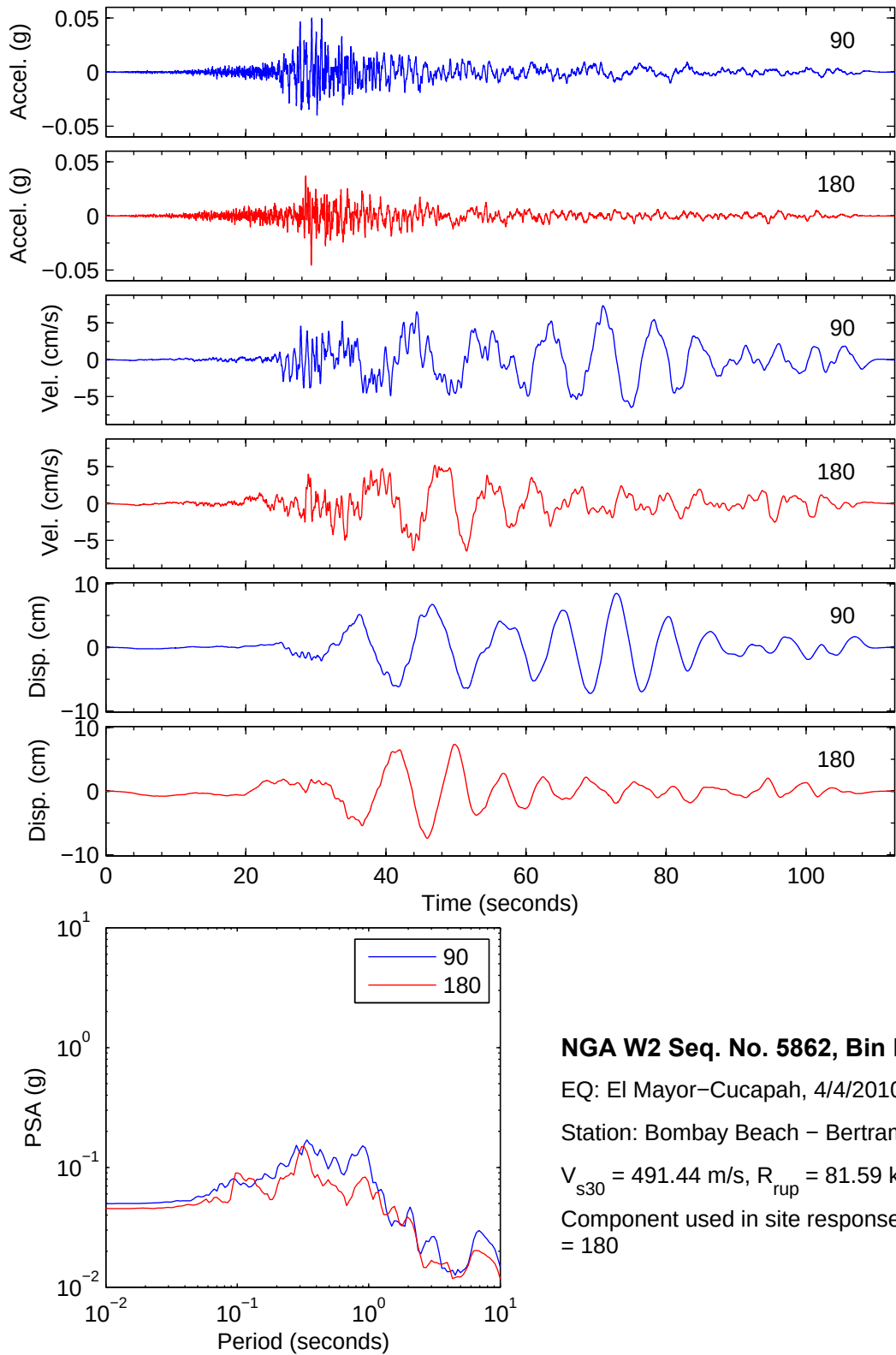


Figure D-66: Input motion NGA W2 Seq. No. 5862, Bin No. 6

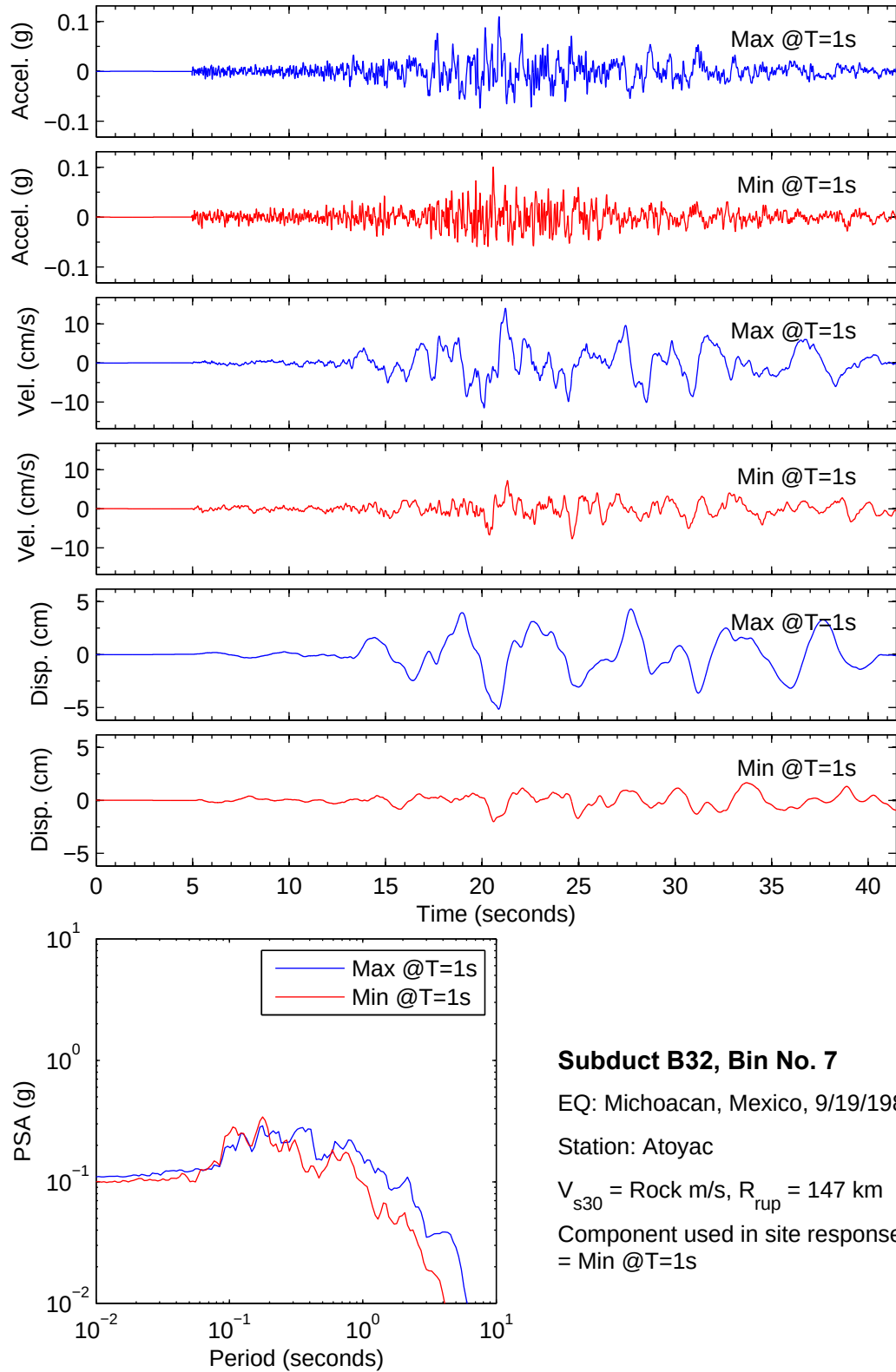


Figure D-67: Input motion Subduct B32, Bin No. 7

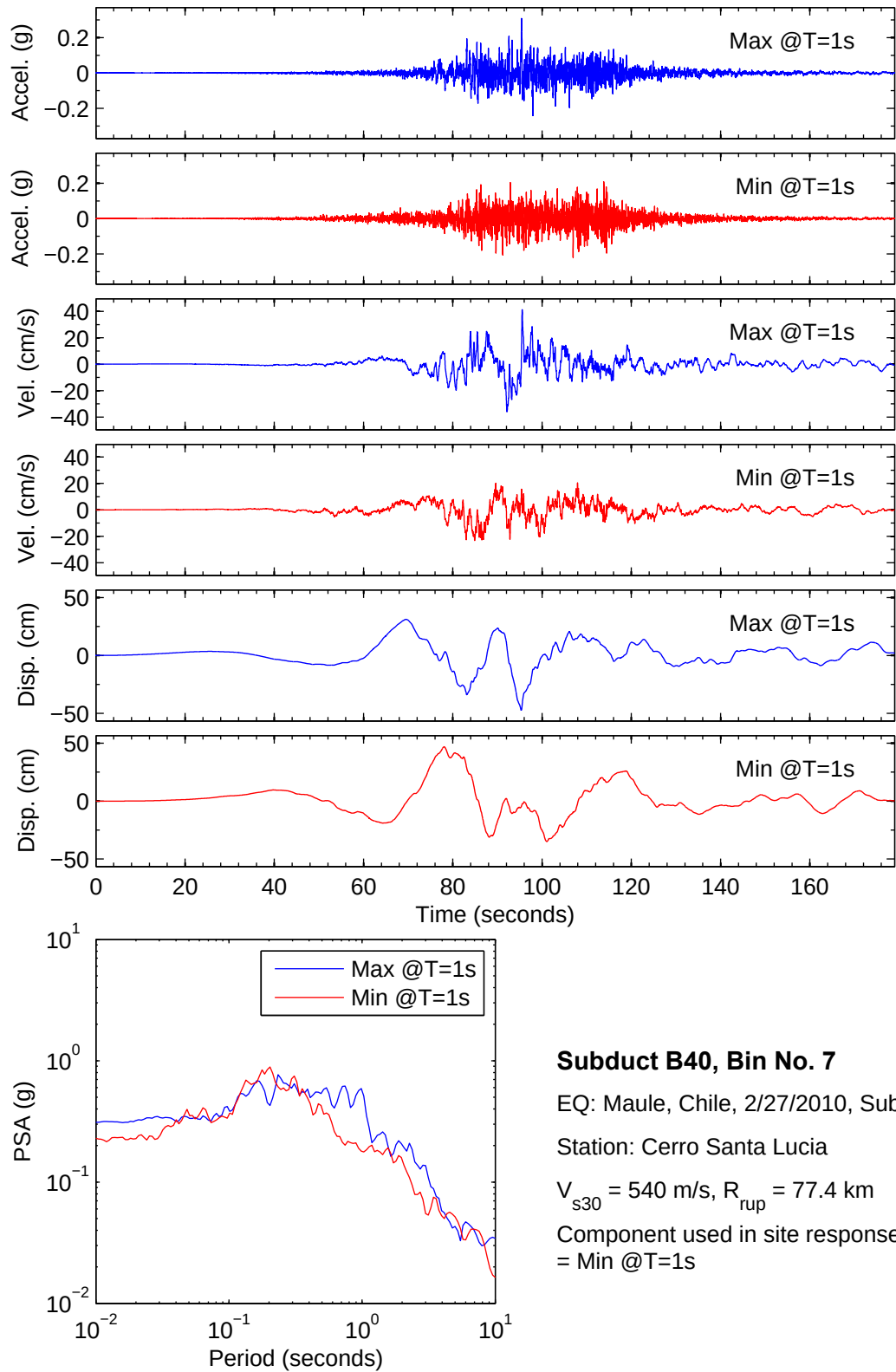


Figure D-68: Input motion Subduct B40, Bin No. 7

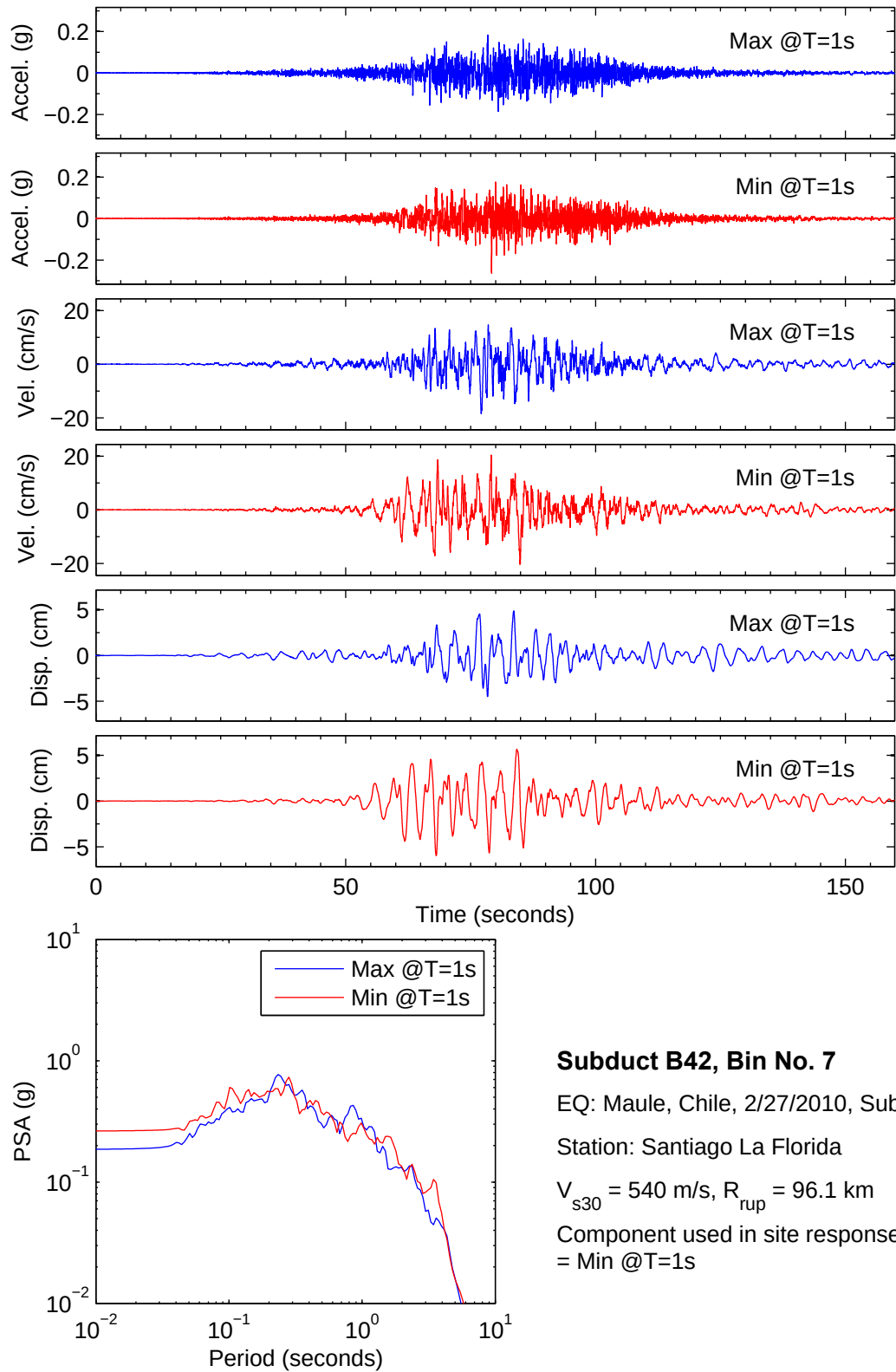


Figure D-69: Input motion Subduct B42, Bin No. 7

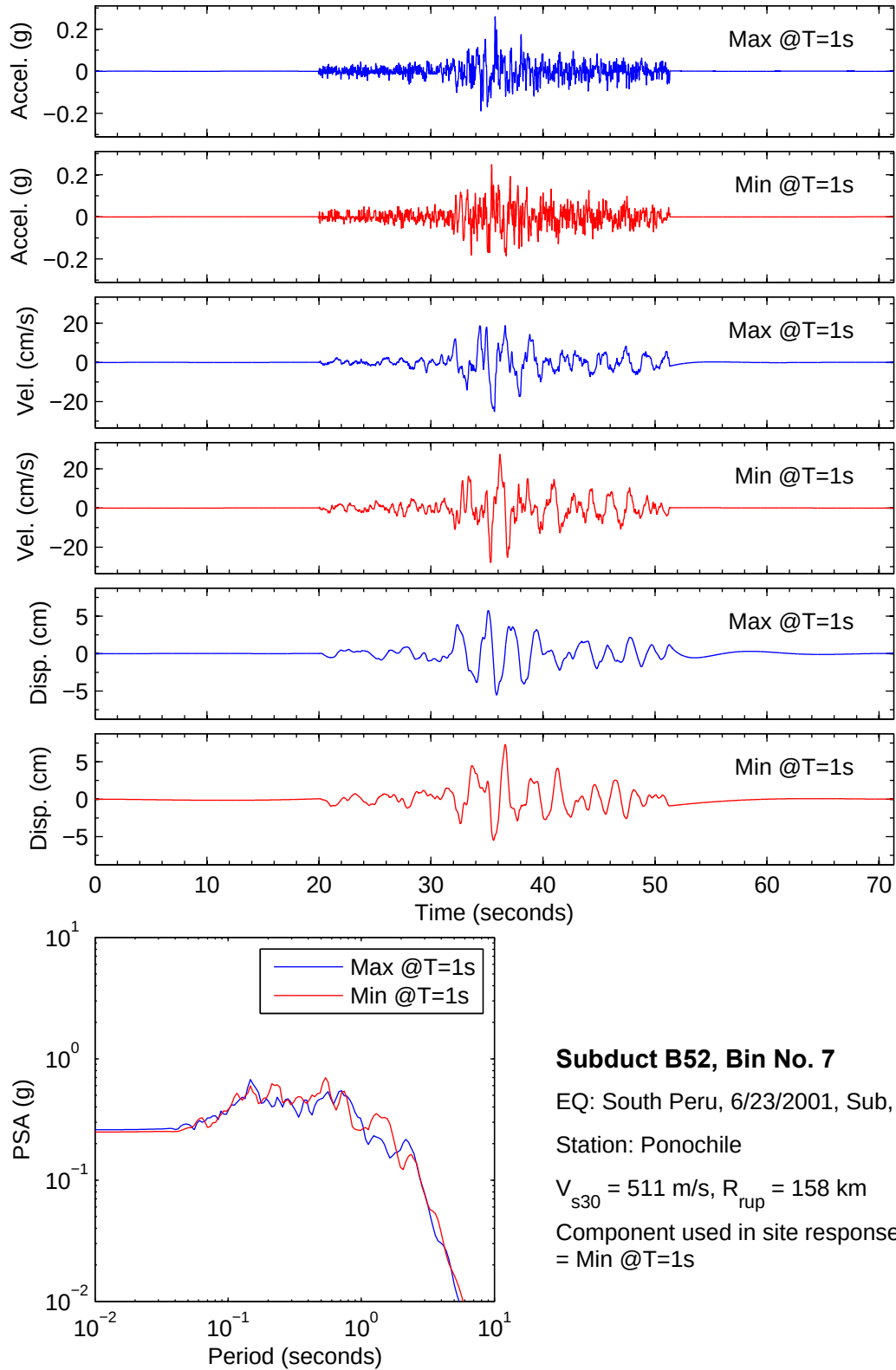


Figure D-70: Input motion Subduct B52, Bin No. 7

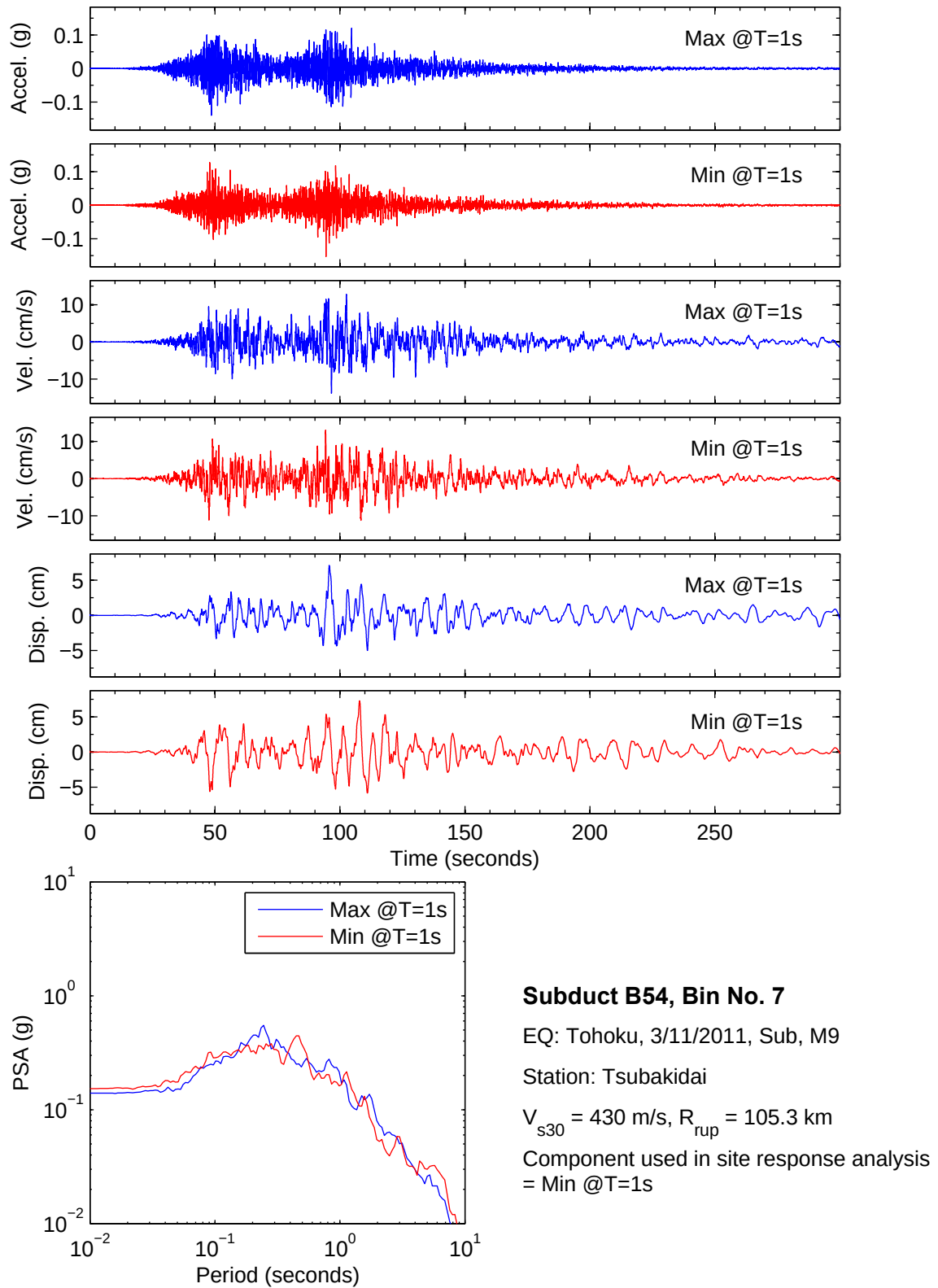


Figure D-71: Input motion Subduct B54, Bin No. 7

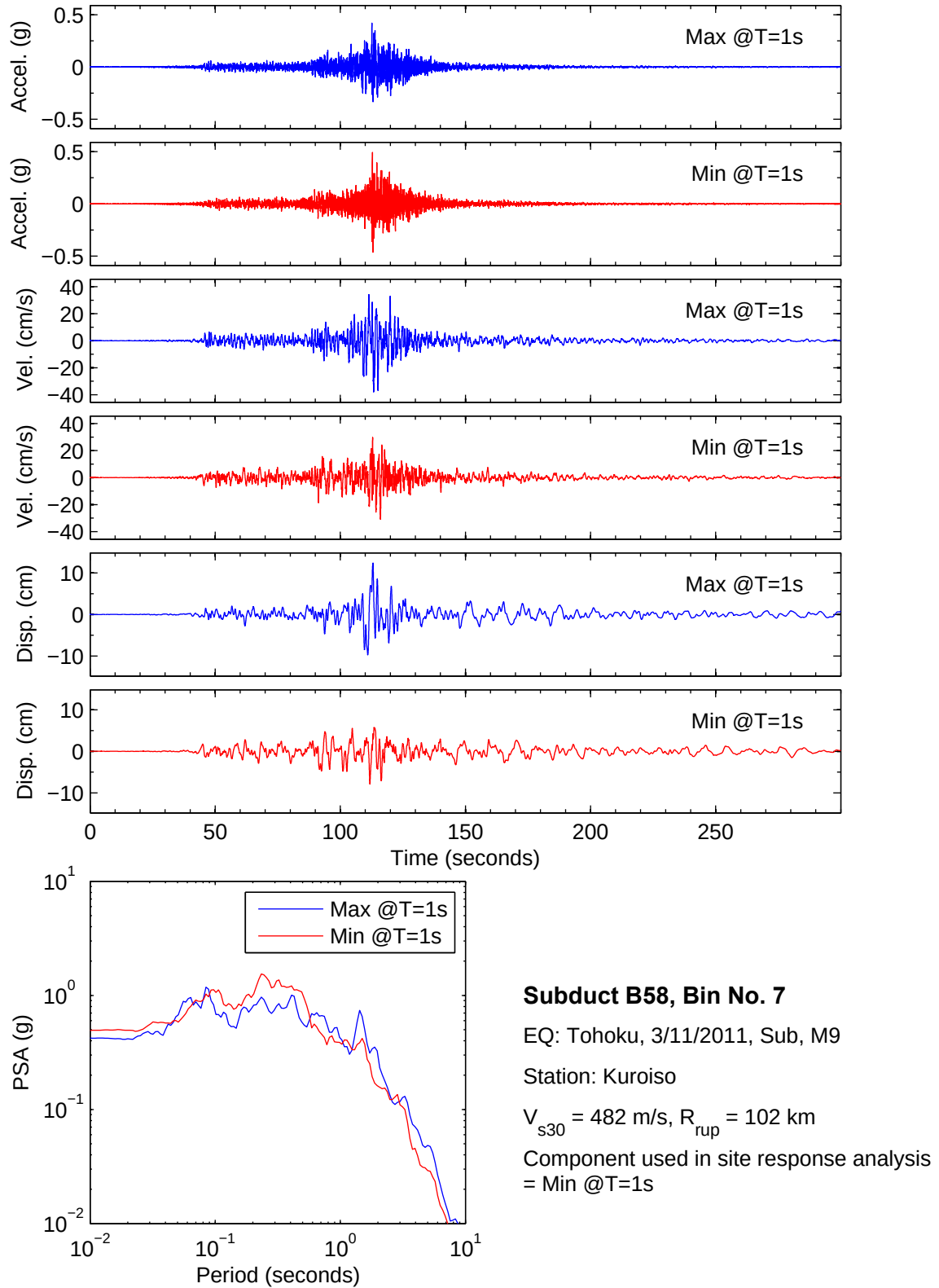


Figure D-72: Input motion Subduct B58, Bin No. 7

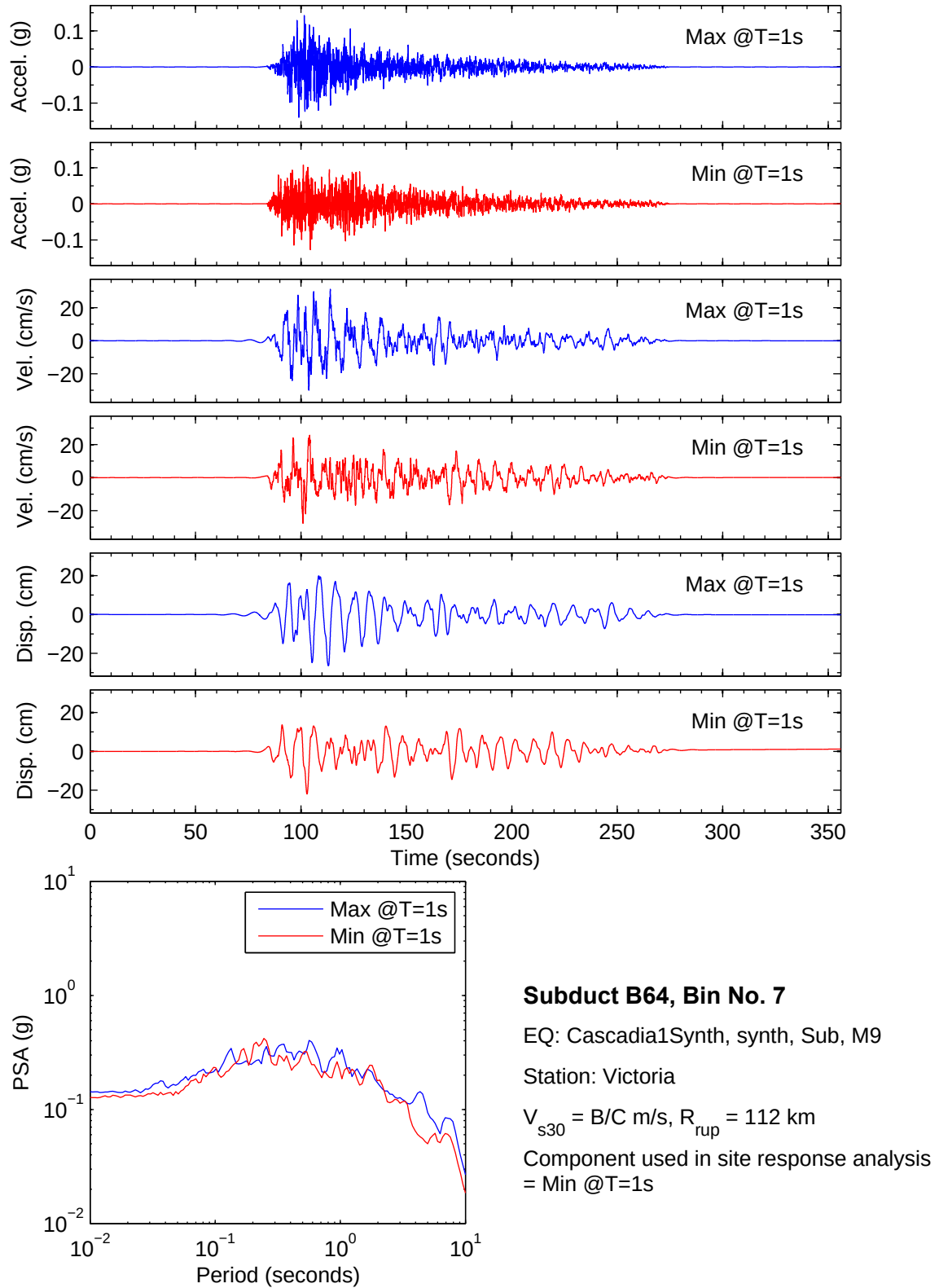


Figure D-73: Input motion Subduct B64, Bin No. 7

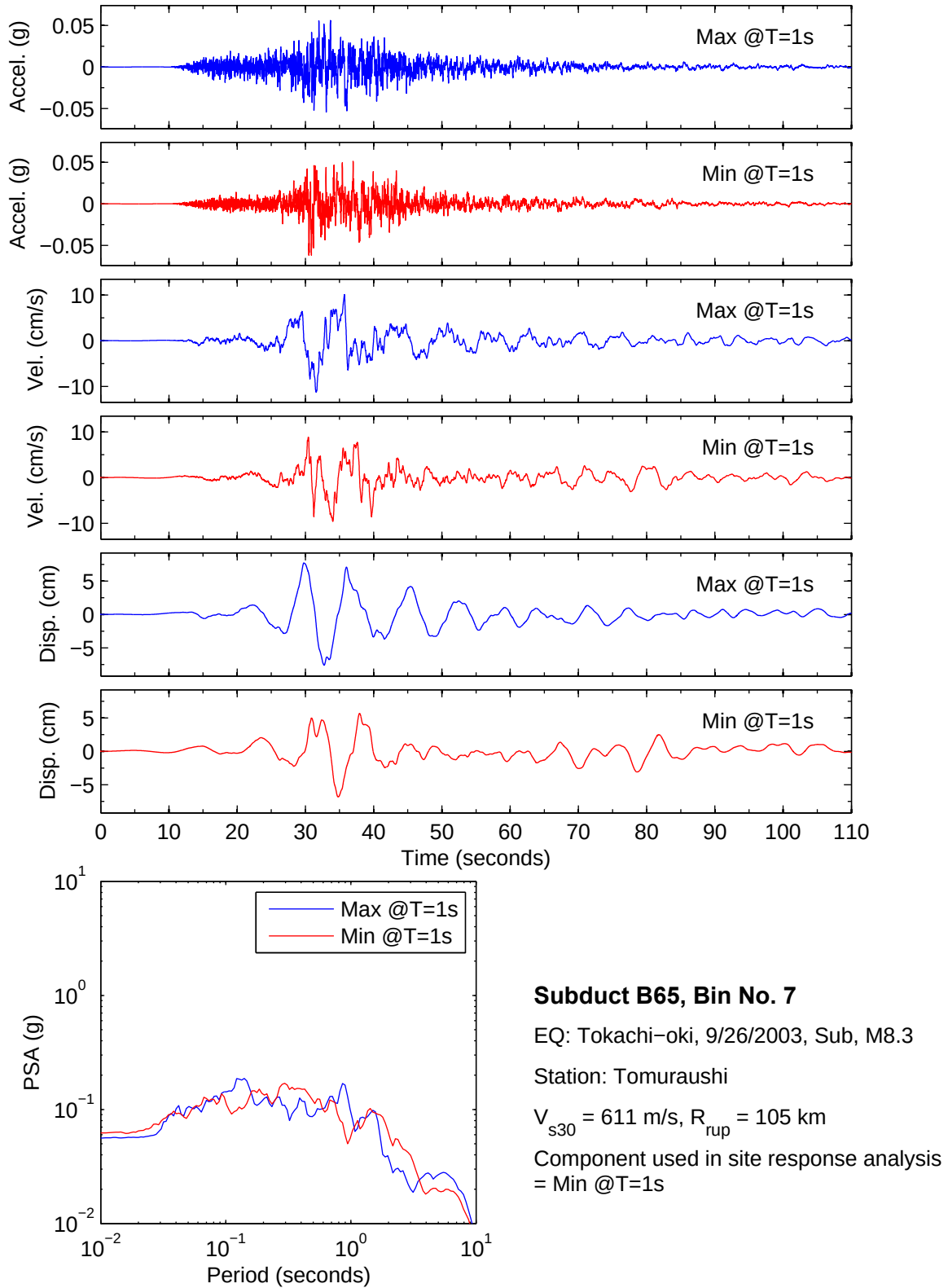


Figure D-74: Input motion Subduct B65, Bin No. 7

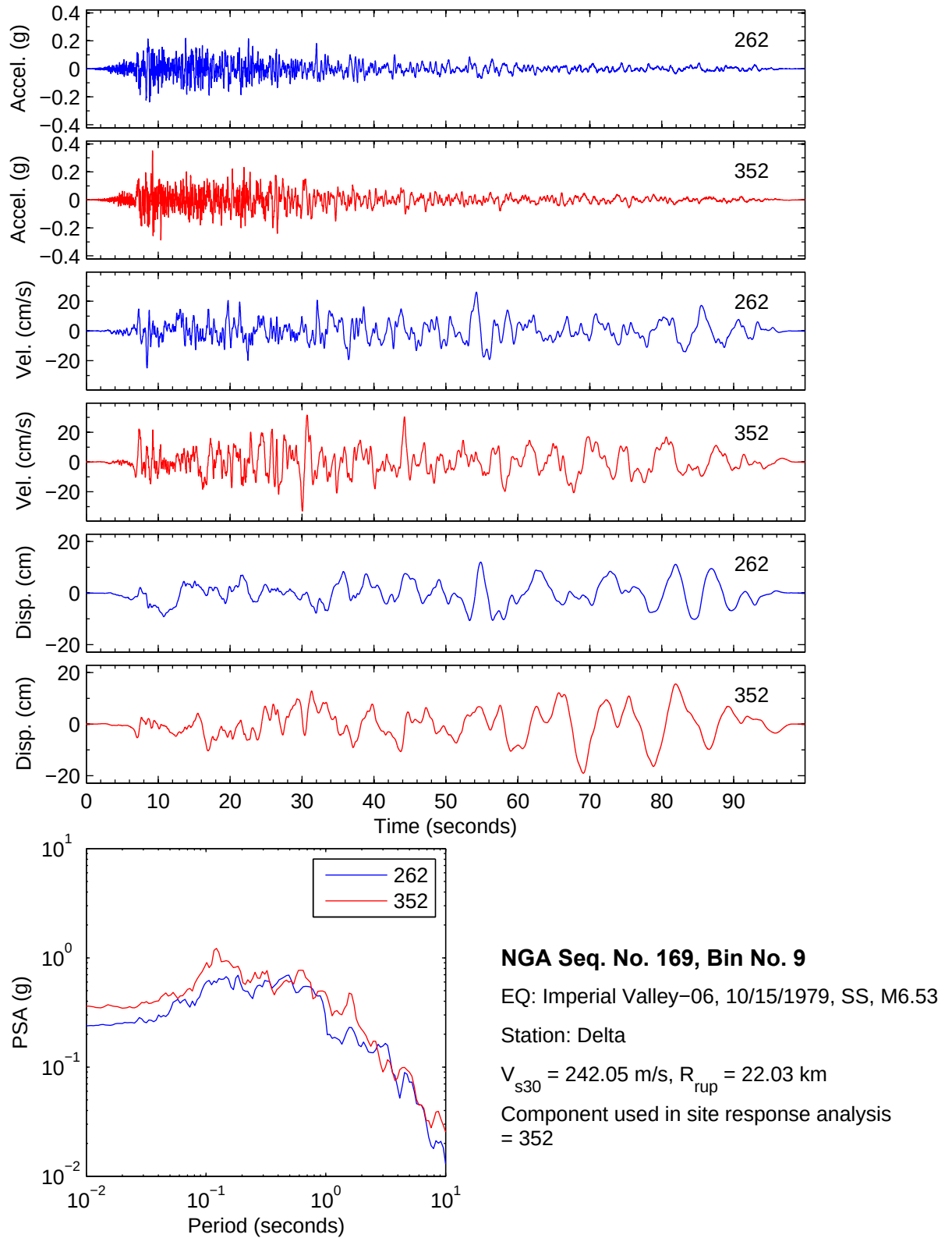


Figure D-75: Input motion NGA Seq. No. 169, Bin No. 9

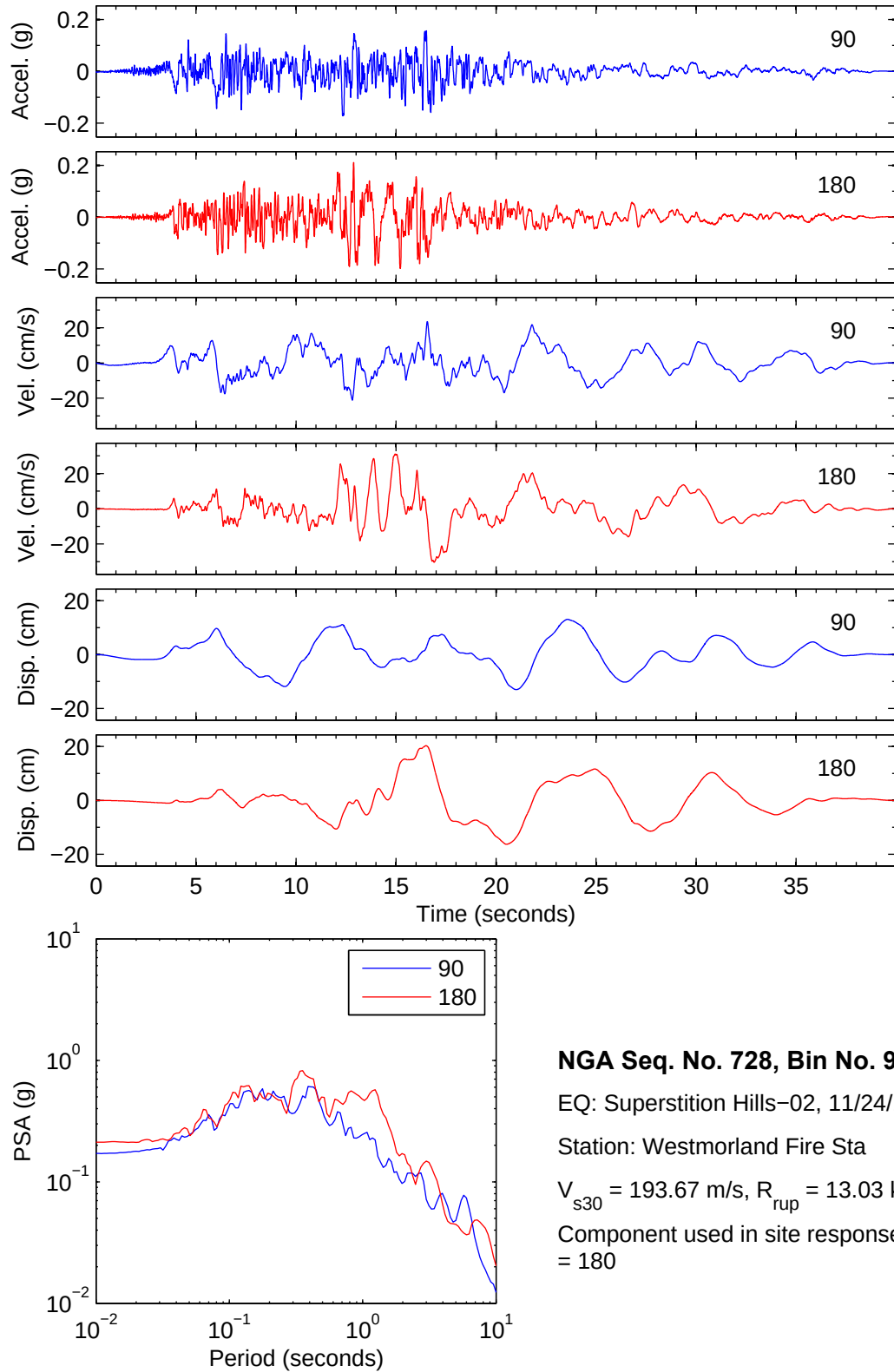


Figure D-76: Input motion NGA Seq. No. 728, Bin No. 9

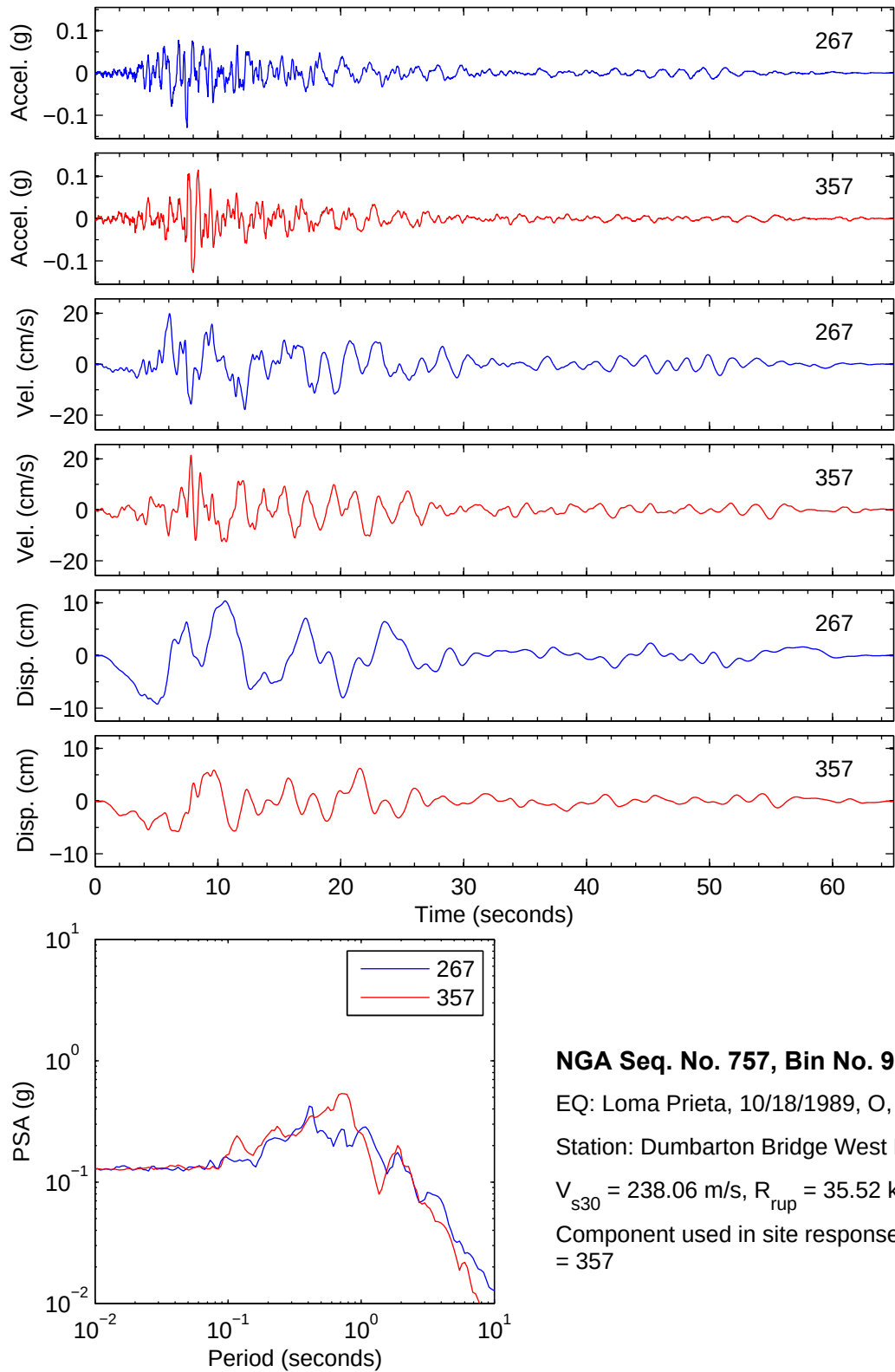


Figure D-77: Input motion NGA Seq. No. 757, Bin No. 9

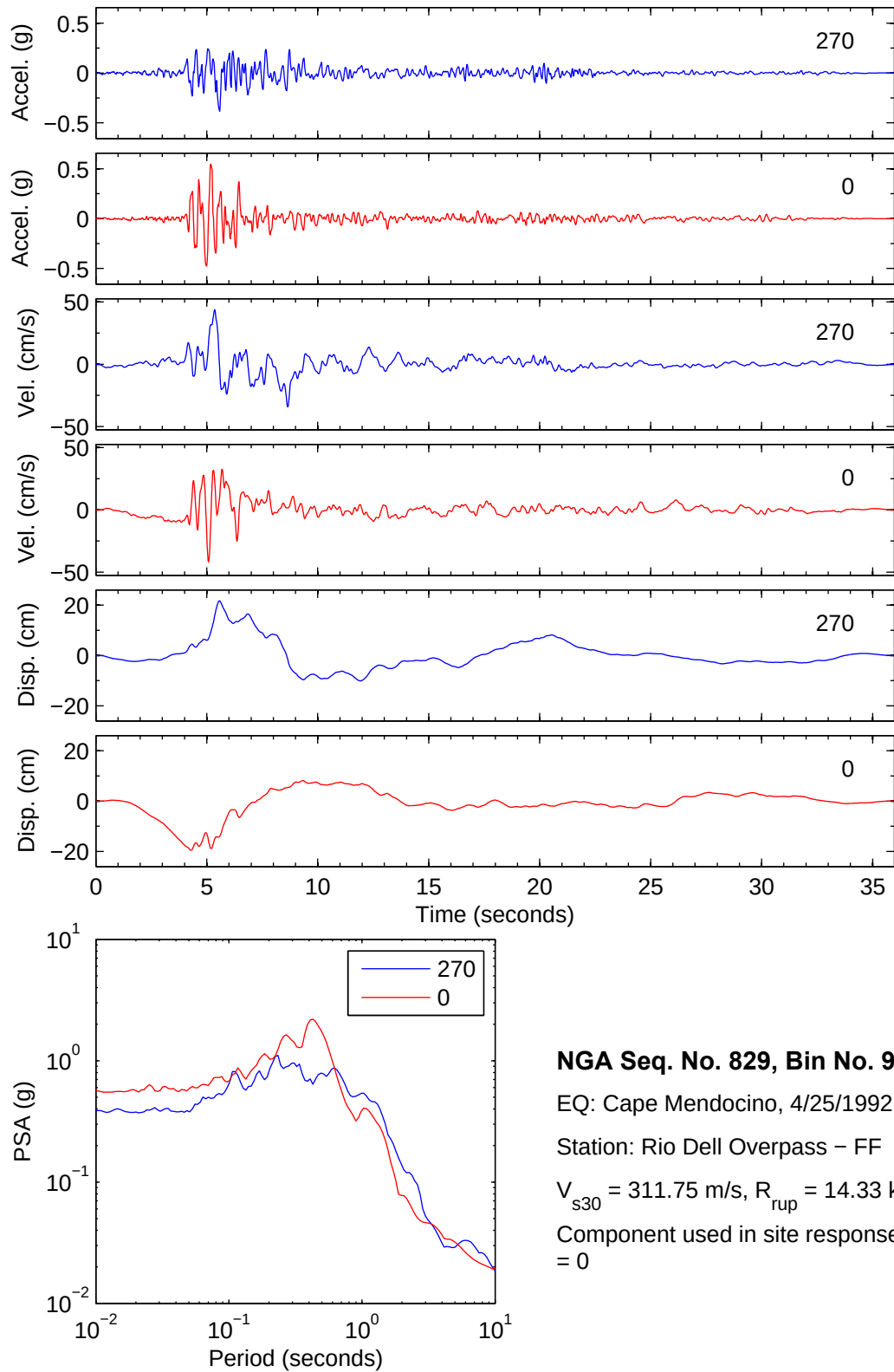


Figure D-78: Input motion NGA Seq. No. 829, Bin No. 9

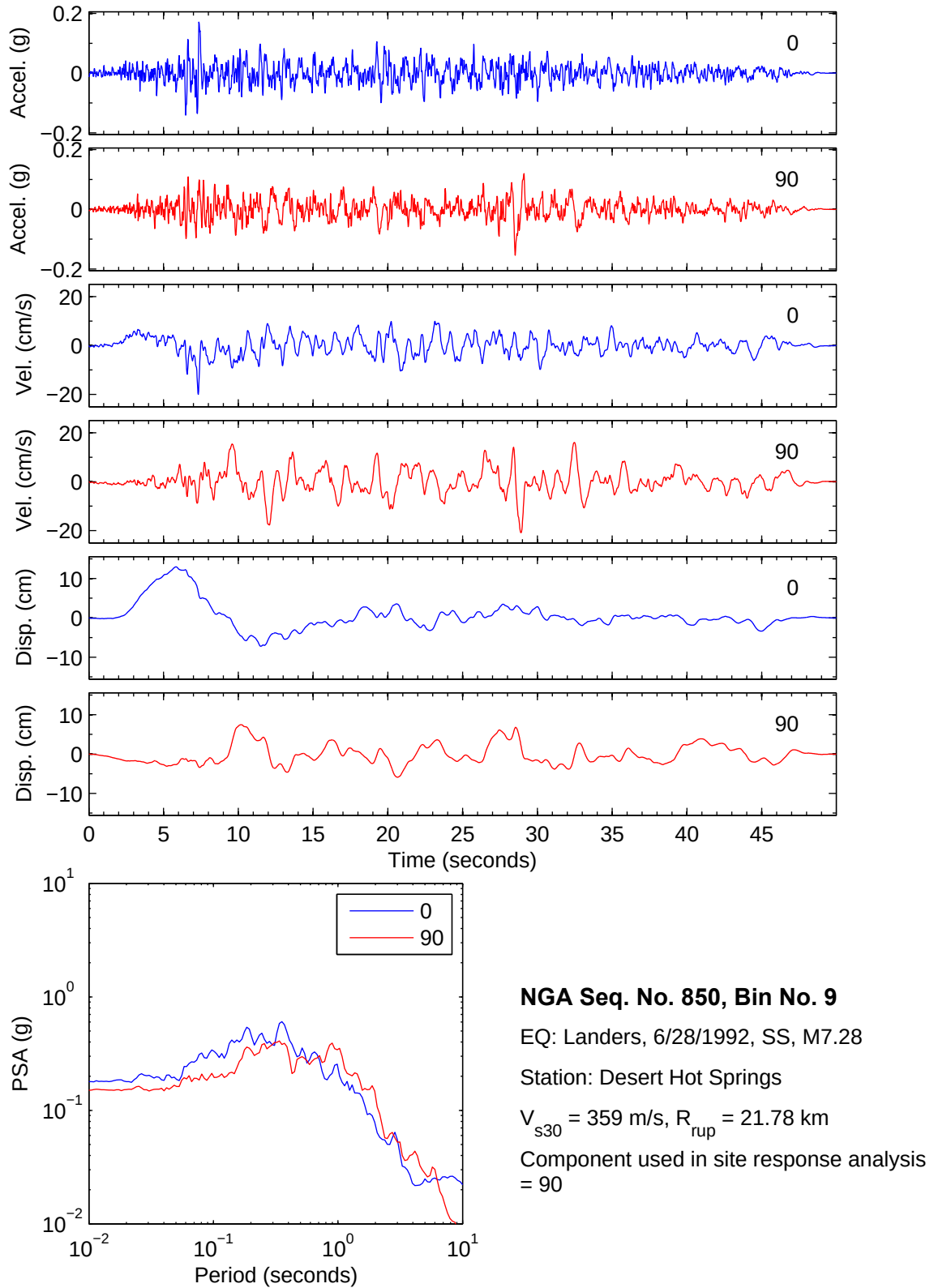


Figure D-79: Input motion NGA Seq. No. 850, Bin No. 9

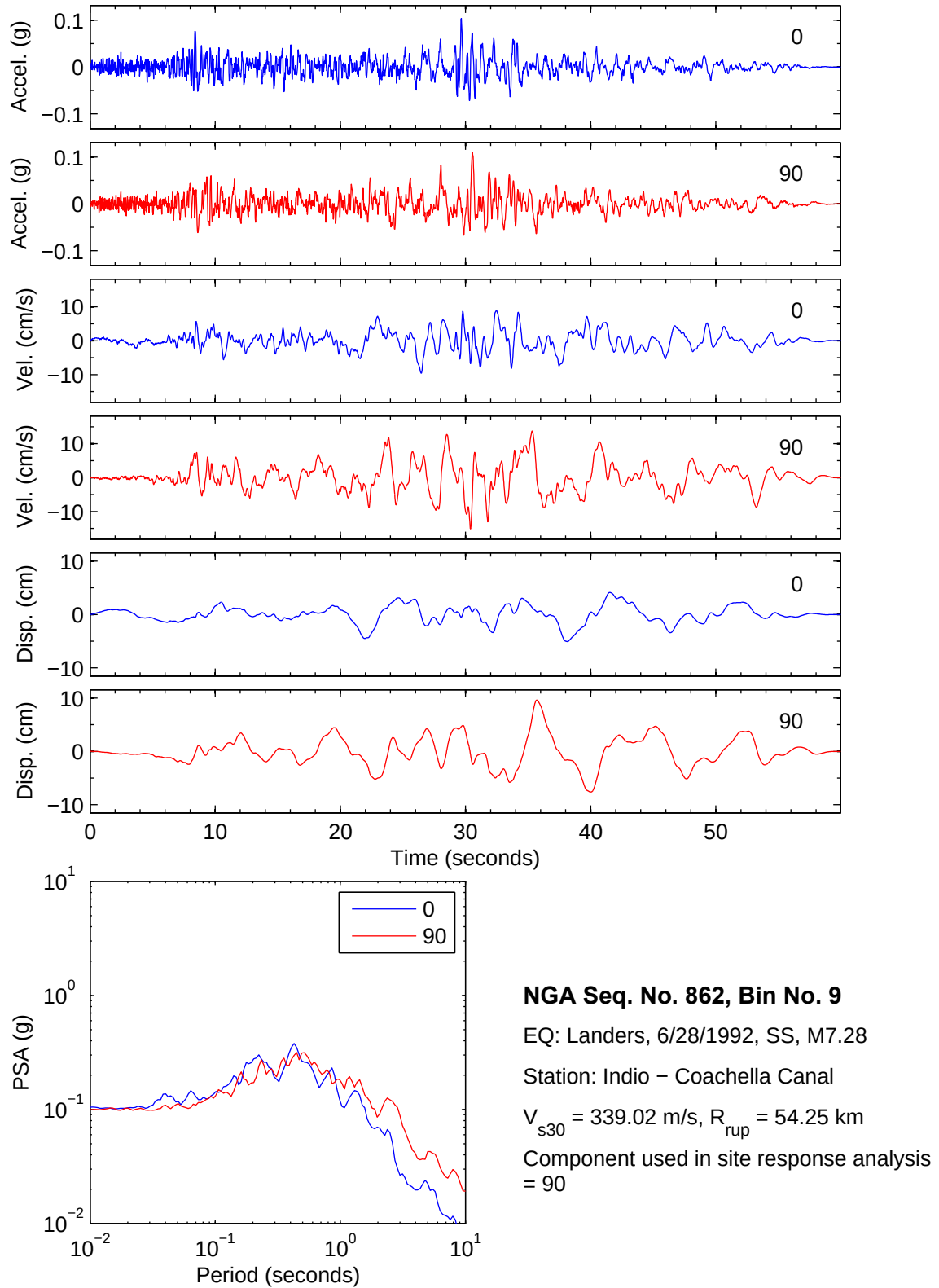


Figure D-80: Input motion NGA Seq. No. 862, Bin No. 9

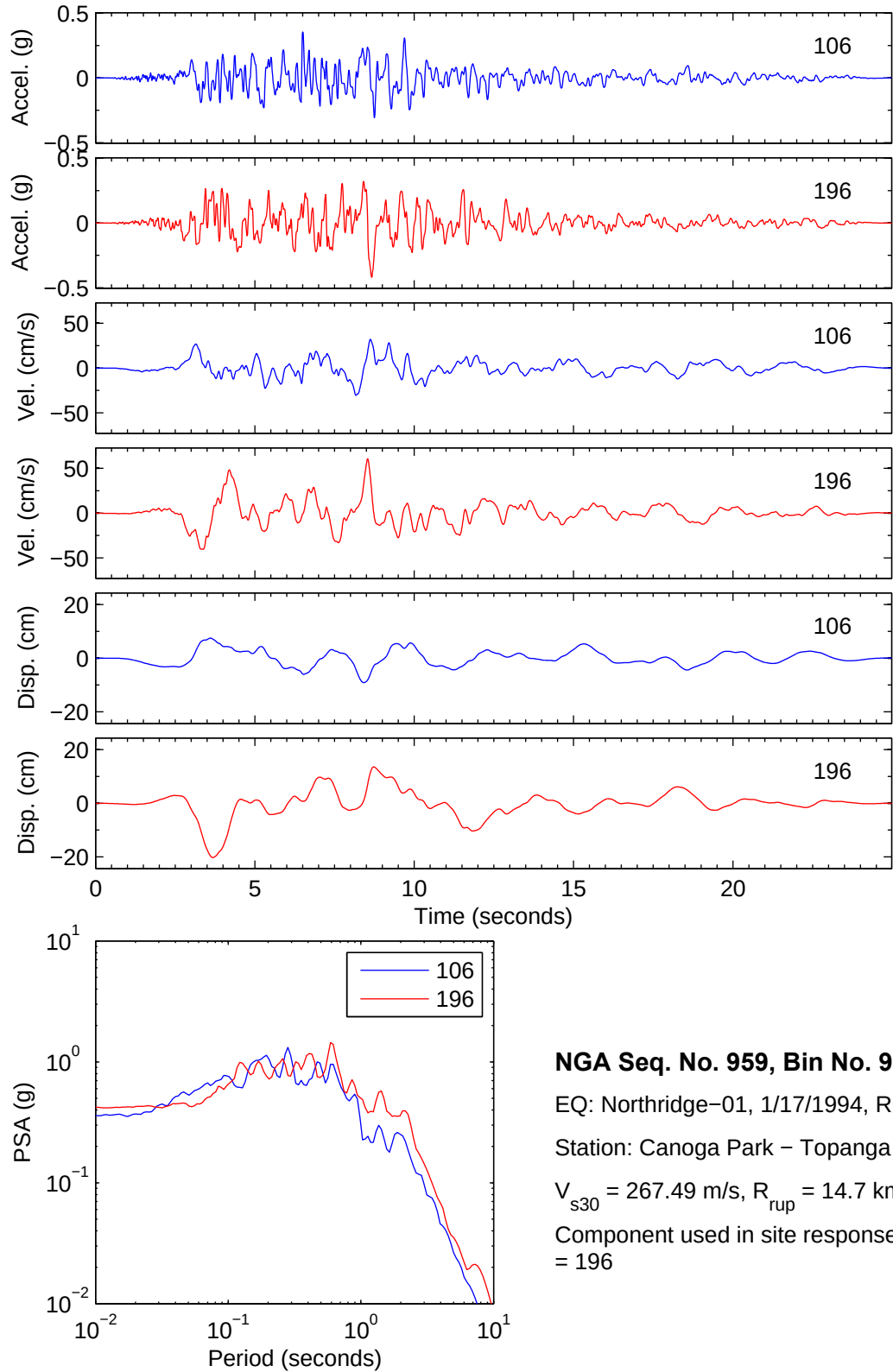


Figure D-81: Input motion NGA Seq. No. 959, Bin No. 9

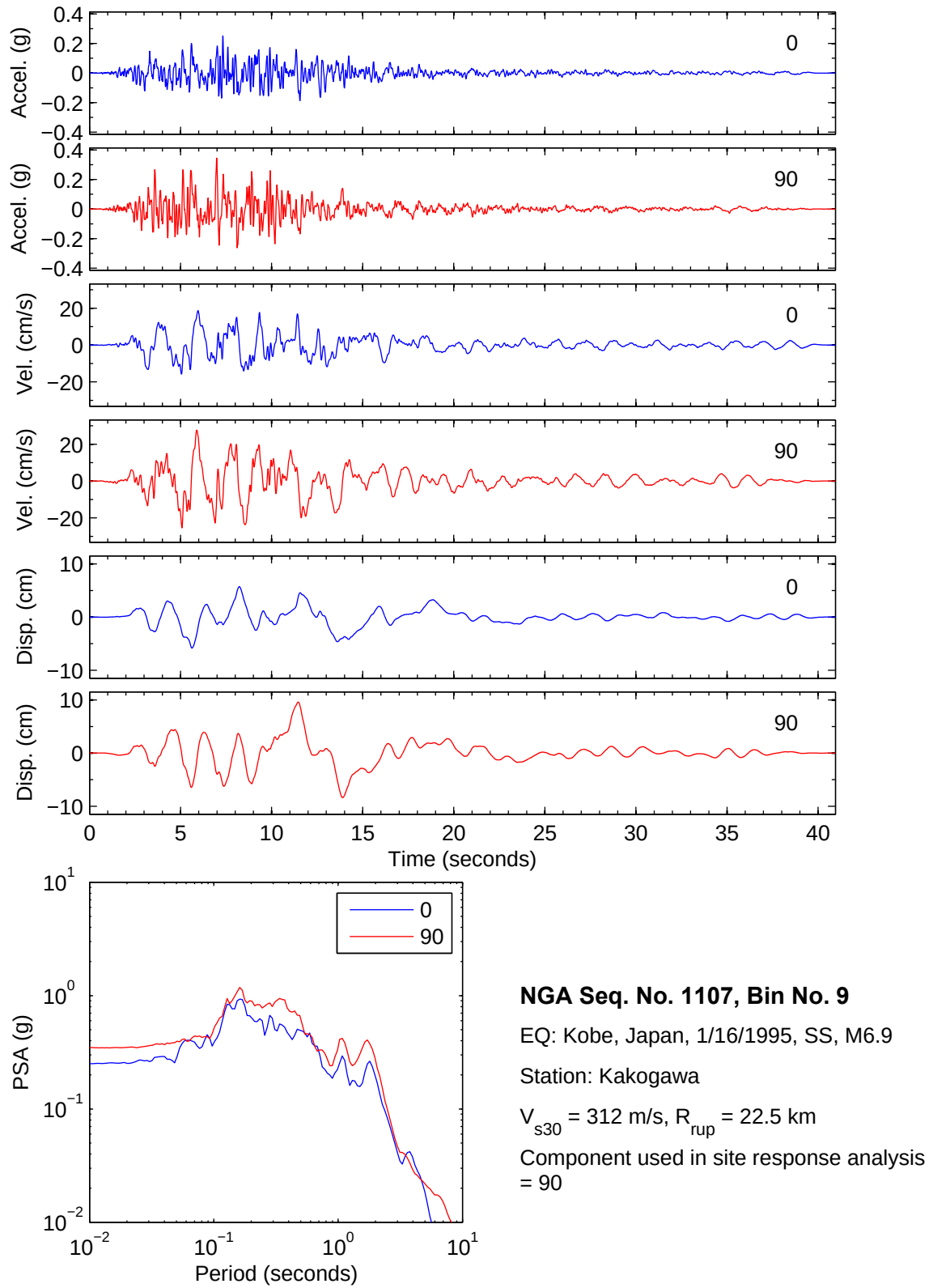


Figure D-82: Input motion NGA Seq. No. 1107, Bin No. 9

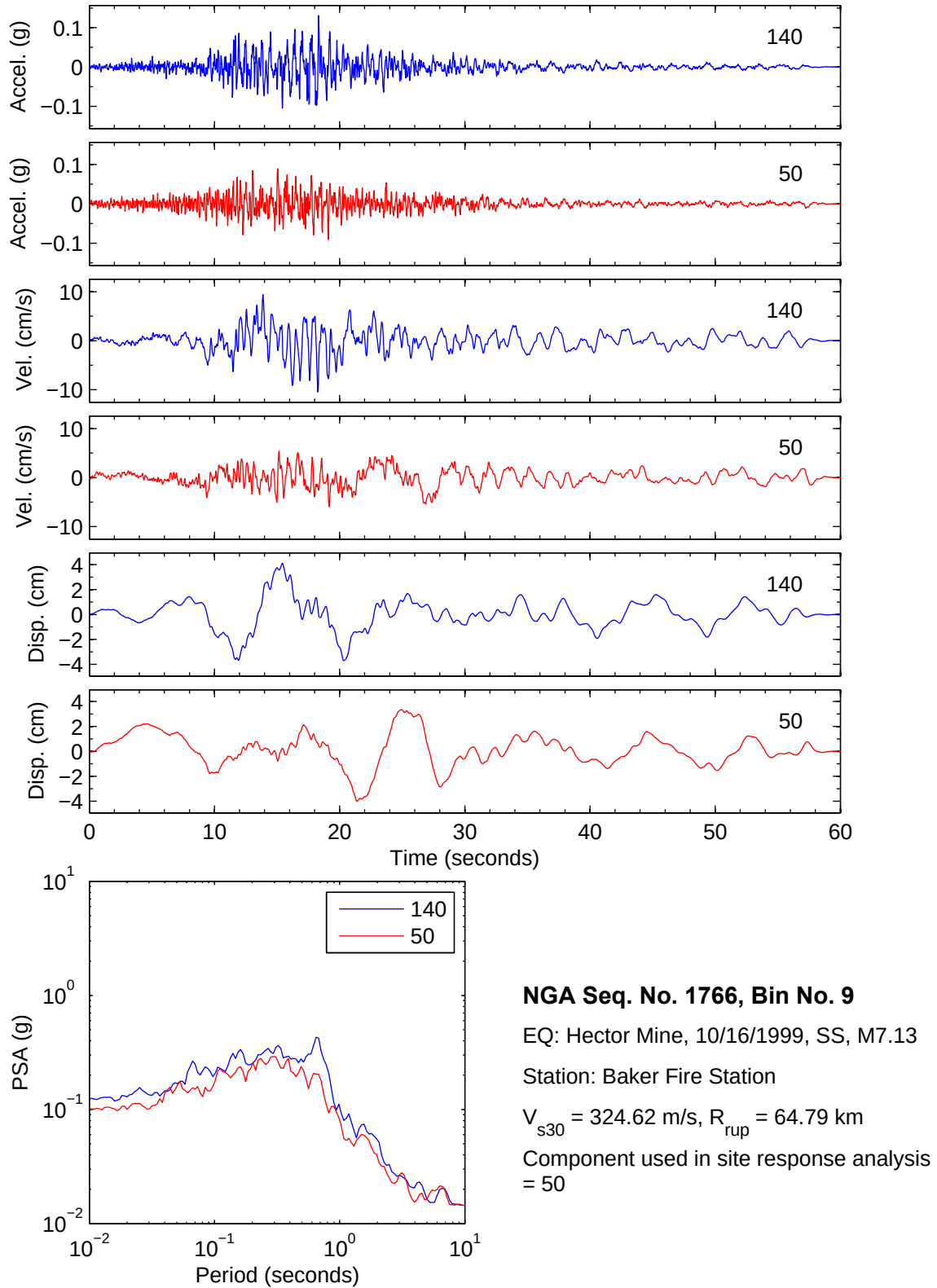


Figure D-83: Input motion NGA Seq. No. 1766, Bin No. 9

Appendix E – Site Response Analysis Parameter Sensitivity

E.1 Introduction

Parametric analyses were performed to evaluate the sensitivity of the results to variations in element size, hydraulic conductivity, vertical excitation, and initial stress anisotropy (K_0) conditions. The following sections present the results of these analyses.

E.2 Element Size

The parametric site response analyses described in Chapter 6 were performed with a 0.5 m vertical element size. To evaluate whether the use of smaller element sizes could affect the results, a series of analyses were performed using the Wildlife Liquefaction Array (WLA) profile and Superstition Hills earthquake 360 degree input motion (Section 5.5.1), with vertical element sizes of 0.5, 0.25 and 0.125 m. The resulting surface response spectra for these three cases are presented in Figure E-1. The results for all three element sizes are identical. Therefore, the element size used of 0.5 m appears to be adequate.

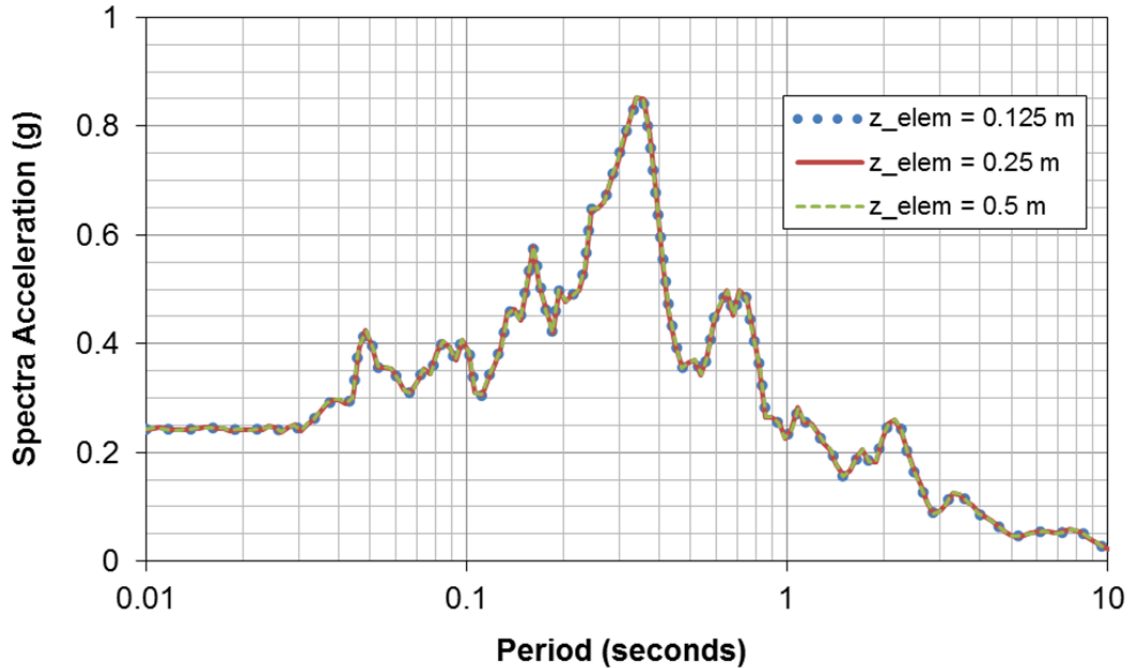


Figure E-1: Response spectra computed for WLA Superstition Hills earthquake 360 component with element sizes of 0.125, 0.25 and 0.5 m.

E.3 Element Size

The parametric site response analyses described in Chapter 6 were performed with a hydraulic conductivity of 1×10^{-5} m/s, which was selected based on a typical value for silty sand to sand (Power 1992). The WLA validation analyses were performed with a hydraulic conductivity of 1×10^{-6} m/s, which is typical of silty sand. To evaluate whether variations in hydraulic conductivity could impact the site response results, a series of analyses were performed using the WLA profile and Superstition Hills earthquake 360 degree input motion (Section 5.5.1) with hydraulic conductivities of 1×10^{-5} , 1×10^{-6} and of 1×10^{-7} m/s. As shown in Figure E-2, the results for all three of these cases are the same.

Therefore, calculated response spectra do not appear to be sensitive to reasonable variations in hydraulic conductivity.

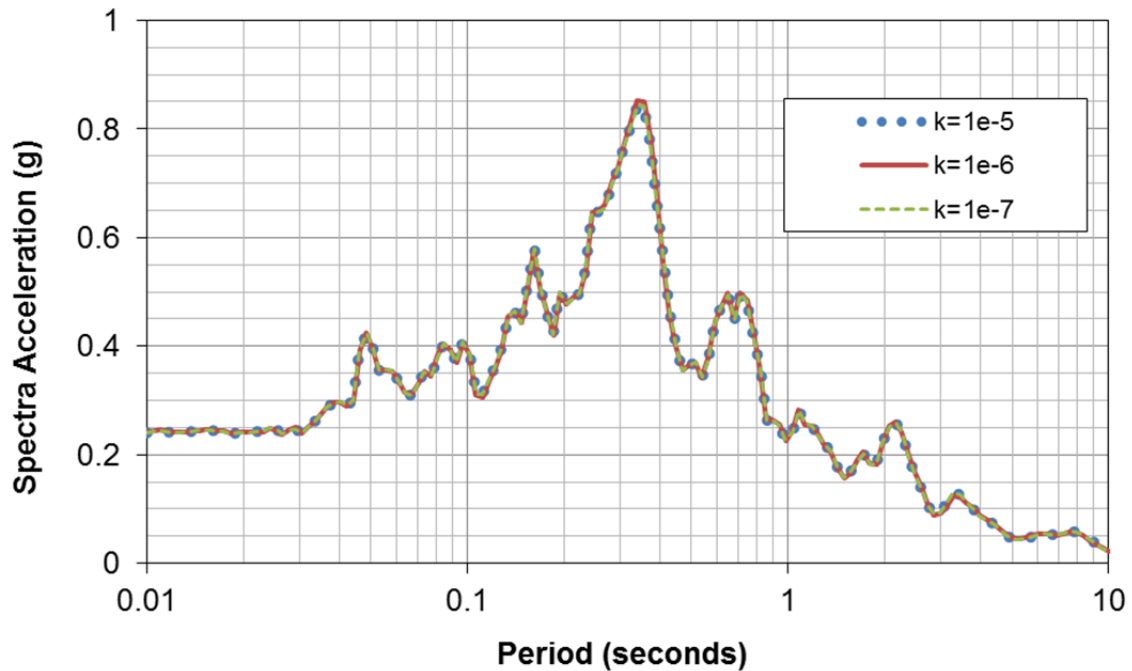


Figure E-2: Response spectra computed for WLA Superstition Hills earthquake 360 component with hydraulic conductivity of 1×10^{-5} , 1×10^{-6} and of 1×10^{-7} m/s.

E.4 Vertical Excitation

The parametric site response analyses described in Chapter 6 were performed with only horizontal excitation. To evaluate whether inclusion of vertical excitation could affect the calculated horizontal surface spectra, an analysis was performed using the WLA profile and Superstition Hills earthquake 360 degree input motion (Section 5.5.1) with vertical excitation included. As with the horizontal motion, the vertical excitation recorded in the down-hole accelerometer from 7.5 feet was applied to the base of the model as a within

motion. Figure E-3 shows that the calculated surface response spectra for the cases with and without the vertical excitation are identical. Therefore, it does not appear necessary to include vertical excitation in the site response results.

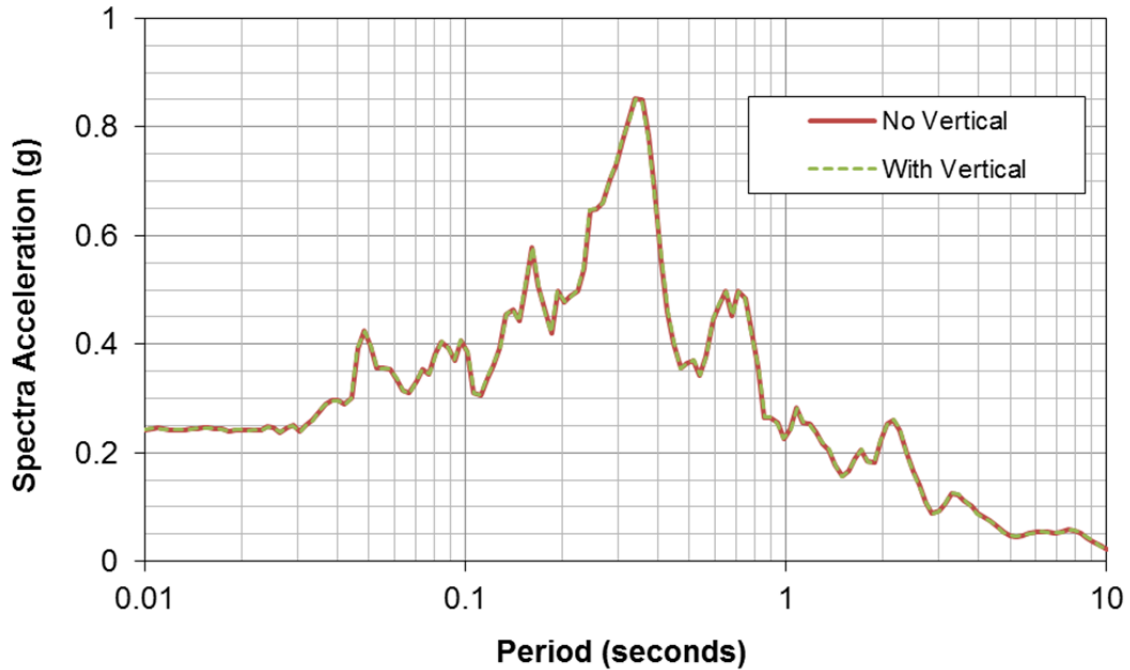


Figure E-3: Response spectra computed for WLA Superstition Hills earthquake 360 component with and without vertical excitation.

E.5 Initial Stress Anisotropy (K_0) Condition

The model calibration work described in Chapter 5 was performed assuming isotropic initial stress conditions ($K_0 = 1$). Typical values of K_0 for normally consolidated liquefiable are commonly estimating using the Jaky (1944) equation, $K_0 = 1 - \sin \phi$, with typical values in the range of 0.3 to 0.55 for sands. The advantages of using the $K_0 = 1$ approach in 1-D nonlinear site response analyses are described in Section 5.3.1. To evaluate any potential

disadvantages of using $K_0 = 1$, a parametric analysis was performed wherein model parameters were calibrated for $Dr = 50\%$ and $\sigma'_{v0} = 1$ atm, with $K_0 = 0.45$ and $K_0 = 1$. Undrained direct simple shear simulations for the two cases are presented in Figure E-4. The shear stress-strain behavior is very similar for the two cases – an even closer match is possible with further refinement of the contraction and dilation parameters. The most significant difference observed in the two cases is in the excess porewater pressure generation. The rate of porewater pressure generation is greater in the first cycle of loading for the $K_0 = 0.45$ case compared to the $K_0 = 1$ case. The rates of porewater pressure generation are similar in the subsequent cycles. As observed in Figure 6-16, differences in AF_{liq} values are insignificant when r_u is less than about 1. Therefore, the results of the study presented in Section 6 are expected to be insensitive to variations in the r_u rate in the first cycle (which is well below $r_u = 1$), and thus also insensitive to variations in the K_0 assumption.

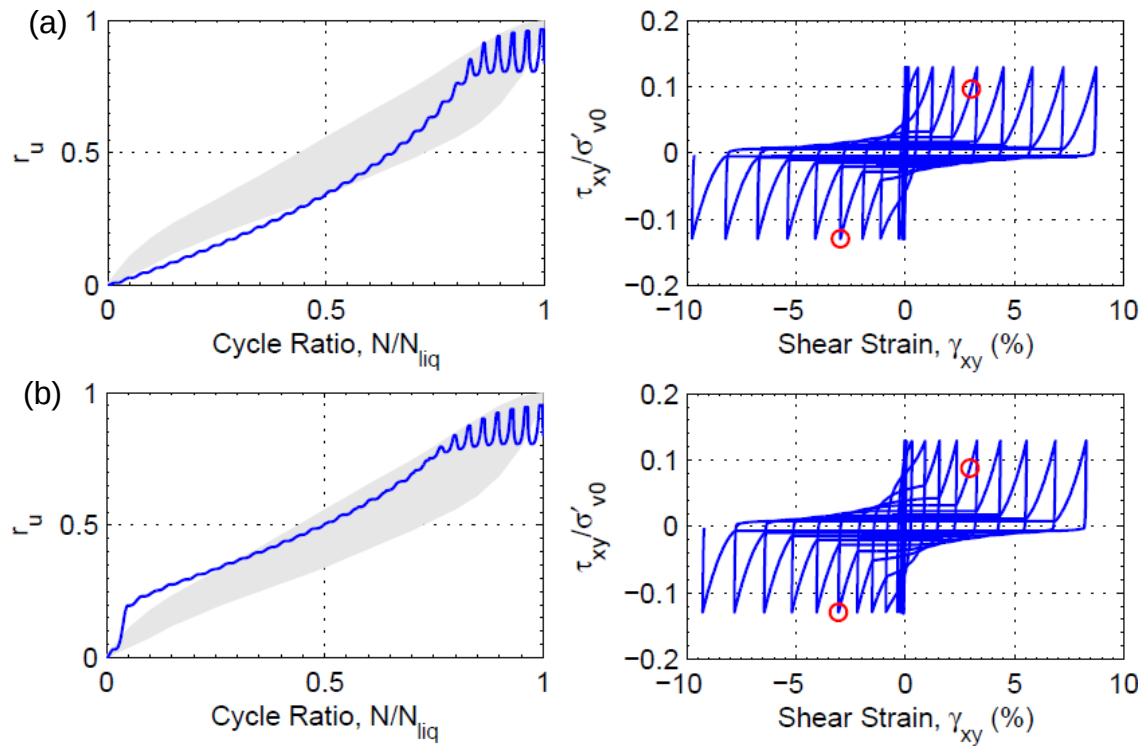


Figure E-4: Comparison of cyclic simple shear simulations for case of (a) $K_0 = 1$ and (b) $K_0 = 0.45$.

E.6 References

- Jaky, J. (1944). "The Coefficient of Earth Pressure at Rest," *Journal of the Society of Hungarian Architects and Engineers*, Budapest, pp 355-358.
- Power, J.P. (1992). *Construction Dewatering, New Methods and Applications*, Second Edition, John Wiley & Sons, Inc., New York.