Bridging Guidelines for the Performance-based Seismic Retrofit of British Columbia School Buildings
Second Edition

PRELIMINARY SITE RESPONSE ANALYSIS FOR BRIDGING GUIDELINES - SECOND EDITION

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1. Introduction

The Bridging Guidelines recommend that a site response analysis be undertaken for all schools founded on Site Class E or Site Class F soils. Several of these site response analyses are now underway to complete Stage 2 feasibility studies or to prepare for commencing final design.

During the development of the 2nd edition of the Bridging Guidelines, the need for site response analysis of Site Class D sites also arose from a unanimous PRC recommendation. Given the potentially high seismic demand at Site Class D sites, the PRC was of the opinion that the Ministry's commitment to promoting cost-effective retrofit solutions would be best served by assessing the surface response of at least 10 Site Class D sites using site response analysis. The data collected from this analysis could then be used to refine the minimum resistance tables in the second edition of the guidelines.

The study reported here is in response to the PRC recommendation. The results presented are preliminary and correspond to schools located on sites C, D and E, rather than just D. Including analyses for different site classes provide valuable insight into the significant effect that the site class can have on the expected performance of the various structural systems considered within the scope of the Bridging Guidelines. It is anticipated that some Site Class D sites may exhibit substantial amplification of the surface ground motion.

The results of this study clearly demonstrate the need of further studies of the influence of site class on the expected response of a school building.

2. Scope of Study (10 school sites)

The scope of the preliminary site response analyses detailed in this report is as follows:

(1) Lateral Shaking

This report is restricted to lateral shaking only. No liquefaction consideration has been introduced into the analyses (eg: reduced shear strength due to pore pressure buildup).

(2) Integrated Soil/Structure Analysis

The analysis of each site is a combination of two analyses. The first analysis is a geotechnical analysis of the soil column through propagation of the input ground motions upwards from firm ground through the overlying soil to the surface. The second analysis is a structural analysis of the response of the building to the these surface ground motions.

(3) School Sites

A following ten school sites were analyzed in this study:

(a) Brighouse Elementary (Richmond)
(b) Garden City Elementary (Richmond)
(c) James Park Elementary (Port Coquitlam)
(d) Langley Fine Arts (Langley)
(e) Lincoln Elementary (Port Coquitlam)
(f) Margaret Jenkins Elementary (Greater Victoria)
(g) Mt. Douglas Secondary (Greater Victoria)
(h) Port Guichon Elementary (Delta)
(i) Victoria High (Greater Victoria)
(j) Willows Elementary (Greater Victoria)

3. Objective of Study

Past efforts have shown that scaling ground motions to NHERP levels for soft soils produces extremely high demands when using those scaled motions for non-linear dynamic analysis. The objective of this study is to conduct a preliminary analytical program to investigate the non-linear response of soft soil profiles taken from existing school locations, and how this response affects the behavior of the structural systems located at the site.

4. Second Edition Site Response Procedure

The 2nd edition of the Bridging Guidelines recommends a site response analysis for building locations underlain by all Site Class E and F soils. Guidelines for site response analysis by qualified geotechnical engineers include:

1. *Confirmation of Site Class:* A geotechnical engineer or professional geologist member of the school seismic retrofit project team confirms that the building is founded on Site Class E or F soils as defined in the Table 4.1.8.4A of the 2005 NBCC.

2. *Design Ground Motions:* UBC will provide the geotechnical engineer with a suite of time histories, and scaling factors, to be used as outcrop motions for the site response analysis. This suite of ground motions are the same used in developing the resistance tables and are recordings of crustal type ground motions on Site Class C soil. There are 10 records in the suite.

3. *Geotechnical Analysis:* The site response analysis shall be conducted using methods of analysis that have appeared in refereed publications and have a track

record of use in local geotechnical practice. Equivalent linear analysis is considered to have reduced reliability for ground motions in excess of 0.4g in softer soils or shear strain amplitudes exceeding 1%-2%. In such cases true nonlinear analyses are preferred.

4. *Outcrop Motion:* The depth at which the outcrop motion is input into the analysis depends on the stratigraphy of the site. The motion should be input where the site properties match the conditions of Site Class C. The outcrop motion should be allowed to propagate both upwards and downwards.

   The analysis for each ground motion is to be performed for a reasonably probable upper and lower bound of soil properties based on the potential for ground motion amplification.

5. *Presentation of Site Response Analysis Results:* The geotechnical engineers send the results of the site response analysis to UBC. The results are to be presented in the form of acceleration time histories for surface lateral shaking for the upper and lower bound soil properties and for the suite of ground motions.

6. *Building Data:* The structural engineer delivers a summary of the relevant building data to UBC to assist UBC in its analysis of the building for the surface ground motions generated from the site response analysis. The building data to be delivered to UBC includes the following:

   i. Seismic Zone.
   ii. List of existing and new LDRSs prototypes.
   iii. List of existing and new diaphragm prototypes and their span lengths.

7. *UBC Analysis:* Using the two new suites of ground motions from site response analysis results, UBC generates new tables of minimum required factored resistances for the specified LDRSs and diaphragms. These tables are forwarded to the structural engineer to use in the assessment or retrofit of the school building located on the site. These tables are used in lieu of the resistance tables found in Sections 3-8 for the site under investigation only. The tables are based on the highest demand from the upper and lower bound ground motion suites.

8. *Liquefaction*: Only the influence of lateral ground shaking is accounted for in the generation of the resistance tables. Liquefaction is not addressed by the 2nd Edition Bridging Guidelines (see Section 1.16).

9. *Database of Site Response Analyses:* UBC reserves the right to use the findings of the site response analyses for future research.

5. Contribution to Bridging Guidelines

The soil type (site class) that the schools are located on is needed to determine the minimum required factored strength from the resistance tables. The 2nd Edition Bridging Guidelines uses the same site classifications as the 2005 NBCC, found in Section 4.1.8.4. The appropriate site properties must be determined for a detailed assessment or retrofit.

The site response analysis results for the ten schools investigated have demonstrated the necessity for refinements to current geotechnical site response practice. These refinements are necessary to obtain reliable results that offer the best prospects for cost-effective structural retrofit solutions.

Given the significance of the preliminary set of results presented here, the current approach incorporated in the Bridging Guidelines may need to be refined in order to account more carefully for the site conditions effects on the structural response of the school buildings. The next phase of the project proposes to produce new criteria for scaling ground motions for soft soil sites.

6. Analytical Methods

a) Site Response Analytical Options

The general methodology for performing a site response analysis is described in Section 4. This general methodology was modified to respond to the need for an expeditious analysis of the first school to be evaluated in accordance with the site response analysis provisions of the Bridging Guidelines. The modified methodology was as follows:

- Rock Ground Motions – UBC provided the geotechnical engineer with a suite of ten Site Class C ground motions with a probability of exceedance of 2% in 50 years.
- Geotechnical Analysis - The geotechnical engineer performed a site response analysis using a well-established non-linear soil dynamic analysis program. The results of this analysis were delivered to UBC in the form of surface acceleration time histories for each of the ten ground motions. The appendices contain the details of the information provided by the geotechnical consultant. The soil properties for the site were provided by the geotechnical engineer.

For this study the nonlinear program DESRA (Lee and Finn, 1978) was used to generate the desired ground motions.

Once the new ground motions are obtained in the surface, a baseline correction is performed in order to avoid large or unreal displacements at the end of each record. The data processing is performed by using SeismoSignal computer program. The time histories for acceleration, velocity and displacements are carefully revised for each of the suites of records for each site. This procedures ends when necessary files for performing nonlinear dynamic analysis are created.

A suite of 10 ground motions have been used in this study. Detailed information of this suite can be found in section 4 of Commentary C of the BG. A summary of the ground motions with its corresponding abbreviations is presented in Table 1.

Table 1: Ground Motion Suite for this study for site Class C (firm soil)

NEW FILE NAME	STATION (No or name)	SOURCE	PROCESSED BY	EARTHQUAKE	DATE MM/DD/YYYY	MAGNITUDE (type)
SO90	Sherman Oaks	COSMOS	CDMG	NORTHRIDGE	01/17/1994	6.6 (ML)
WW235	LA, Wadsworth	COSMOS	USGS	NORTHRIDGE	01/17/1994	6.4 (ML)
WW325	LA, Wadsworth	COSMOS	USGS	NORTHRIDGE	01/17/1994	6.4 (ML)
CC0	Canyon Country	PEER	USC	NORTHRIDGE	01/17/1994	6.6 (ML)
Sara0	Saratoga	PEER	CDMG	LOMA PRIETA	10/18/1989	6.9 (M)
CP196	Canoga Park	PEER	USC	NORTHRIDGE	01/17/1994	6.6 (ML)
CP106	Canoga Park	PEER	USC	NORTHRIDGE	01/17/1994	6.6 (ML)
PK90	Pacoima, CA	COSMOS	CSMIP	NORTHRIDGE	01/17/1994	6.6 (ML)
MD35	Beverly Hills	PEER	USC	NORTHRIDGE	01/17/1994	6.6 (ML)
Gil67	Gilroy	PEER	CDMG	LOMA PRIETA	10/18/1989	6.9 (M)

The next tables show the peak ground acceleration (PGA) in Table 2, peak ground velocity (PGV) in Table 3, and peak ground displacement (PGD) in Table 4, for each of the ground motions used in the nonlinear dynamic analysis and for each of the sites evaluated and presented in this report. Also presented in this tables, are the average and the average plus one standard deviation of the peaks for each site that corresponds to the level of assessment and retrofit defined on the BG-2nd Edition, respectively. For comparison purposes, the peak values for firm soil, i.e. ground motions recorded in a site class C, are also presented for each ground motion. Next to the peak values, the amplification factors defined as the ratio peak value on site to peak value on firm soil are also presented.

Average amplification factors for PGA are lower than one in almost all cases, except two schools in the greater Victoria. However, most of the FA values for PGV and PGD are larger than one. This may lead to de-amplification of the responses in stiffer structures and amplification of the responses for more flexible structures when dynamic analyses of the structures be performed. However, once the structure developed a certain level of yielding (inelastic behaviour) the response will be better captured or limited by velocities or displacements rather than accelerations. Therefore, we may be expecting certain amplification in our overall results.

Figure 1 through Figure 3 show the spectral acceleration and spectral displacement responses of a single-degree-of-freedom system. Although they are elastic responses, we can expect from the NLDA a clear amplification of the responses at periods of the structure of 1 second or longer. This type of flexible structure is expected to be a representative equivalent behaviour when inelastic deformations occur in the structure.

Table 2: Peak ground acceleration (PGA) and Amplification Factors, FA

	SCHOOL	Site Class	CCO		CP106		CP196		GIL67		MD35		PK90		SARA0		SO90		WW235		WW325		Average	
			PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF	PGA g	AF
	LOWE MAINLAND																							
	Firm Ground	C	0.33		0.41		0.32		0.49		0.66		0.26		0.39		0.24		0.40		0.54		0.40	
a	**Brighouse**	E	0.18	**0.5**	0.18	**0.4**	0.18	**0.6**	0.24	**0.5**	0.18	**0.3**	0.17	**0.6**	0.19	**0.5**	0.17	**0.7**	0.18	**0.5**	0.17	**0.3**	0.18	**0.5**
b	**Garden City**	E	0.20	**0.6**	0.22	**0.5**	0.22	**0.7**	0.27	**0.5**	0.20	**0.3**	0.23	**0.9**	0.24	**0.6**	0.25	**1.0**	0.20	**0.5**	0.20	**0.4**	0.22	**0.6**
c	**James Park - total**	D	0.30	**0.9**	0.30	**0.7**	0.32	**1.0**	0.34	**0.7**	0.37	**0.6**	0.31	**1.2**	0.24	**0.6**	0.25	**1.1**	0.31	**0.8**	0.36	**0.7**	0.31	**0.8**
	James Park - pore		0.22	**0.7**	0.18	**0.4**	0.21	**0.6**	0.25	**0.5**	0.20	**0.3**	0.23	**0.9**	0.17	**0.4**	0.17	**0.7**	0.21	**0.5**	0.16	**0.3**	0.20	**0.5**
d	**Langley - upper**	D	0.31	**0.9**	0.32	**0.8**	0.31	**0.9**	0.51	**1.0**	0.39	**0.6**	0.30	**1.1**	0.58	**1.5**	0.28	**1.2**	0.33	**0.8**	0.39	**0.7**	0.37	**1.0**
	Langley - lower		0.26	**0.8**	0.25	**0.6**	0.26	**0.8**	0.33	**0.7**	0.40	**0.6**	0.34	**1.3**	0.29	**0.7**	0.29	**1.2**	0.28	**0.7**	0.35	**0.6**	0.31	**0.8**
e	**Lincoln**	D	0.21	**0.6**	0.24	**0.6**	0.24	**0.7**	0.24	**0.5**	0.26	**0.4**	0.24	**0.9**	0.20	**0.5**	0.21	**0.9**	0.21	**0.5**	0.24	**0.4**	0.23	**0.6**
h	**Port Guichon**	E	0.19	**0.6**	0.18	**0.4**	0.17	**0.5**	0.20	**0.4**	0.19	**0.3**	0.18	**0.7**	0.19	**0.5**	0.18	**0.8**	0.18	**0.5**	0.17	**0.3**	0.18	**0.5**
	GREATER VICTORIA																							
	Firm Ground	C	0.39		0.48		0.38		0.59		0.78		0.31		0.47		0.29		0.47		0.64		0.48	
f	**Margaret Jenkins**	D	0.34	**0.9**	0.32	**0.7**	0.35	**0.9**	0.36	**0.6**	0.40	**0.5**	0.30	**1.0**	0.28	**0.6**	0.36	**1.3**	0.33	**0.7**	0.39	**0.6**	0.34	**0.8**
g	**Mount Douglas - deep**	C/D	0.43	**1.1**	0.61	**1.3**	0.56	**1.5**	0.65	**1.1**	0.66	**0.8**	0.50	**1.6**	0.48	**1.0**	0.53	**1.9**	0.56	**1.2**	0.59	**0.9**	0.56	**1.2**
	Mount Douglas - shallow		0.43	**1.1**	0.59	**1.2**	0.54	**1.4**	0.69	**1.2**	0.66	**0.8**	0.46	**1.5**	0.48	**1.0**	0.53	**1.8**	0.55	**1.2**	0.64	**1.0**	0.56	**1.2**
i	**Victoria**	C	0.38	**1.0**	0.42	**0.9**	0.38	**1.0**	0.86	**1.5**	0.47	**0.6**	1.03	**3.3**	0.75	**1.6**	1.12	**3.9**	0.36	**0.8**	0.41	**0.6**	0.62	**1.5**
j	**Willows**	C	0.26	**0.7**	0.25	**0.5**	0.26	**0.7**	0.33	**0.6**	0.40	**0.5**	0.34	**1.1**	0.29	**0.6**	0.29	**1.0**	0.28	**0.6**	0.35	**0.5**	0.30	**0.7**

PGA: Peak Ground Acceleration *AF: Amplification Factor* *AF = PGA in soil profile / PGA in firm ground*

Table 3: Peak ground velocity (PGV) and Amplification Factors, FA

	SCHOOL	Site Class	CCO		CP106		CP196		GIL67		MD35		PK90		SARA0		SO90		WW235		WW325		Average	
			PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF	PGV cm/s	AF
	LOWE MAINLAND																							
	Firm Ground	C	32.0		39.1		37.3		35.7		38.1		28.7		36.5		34.2		30.5		30.3		34.2	
a	**Brighouse**	E	40.8	**1.3**	34.1	**0.9**	46.4	**1.2**	35.0	**1.0**	33.3	**0.9**	38.8	**1.4**	44.5	**1.2**	43.4	**1.3**	33.1	**1.1**	29.2	**1.0**	37.9	**1.1**
b	**Garden City**	E	41.7	**1.3**	46.0	**1.2**	46.0	**1.2**	34.8	**1.0**	37.5	**1.0**	38.9	**1.4**	43.2	**1.2**	43.6	**1.3**	35.4	**1.2**	28.6	**0.9**	39.6	**1.2**
c	**James Park - total**	D	38.7	**1.2**	45.2	**1.2**	41.6	**1.1**	43.6	**1.2**	38.7	**1.0**	35.4	**1.2**	41.9	**1.1**	43.9	**1.3**	32.8	**1.1**	32.2	**1.1**	39.4	**1.2**
	James Park - pore		33.9	**1.1**	29.9	**0.8**	30.5	**0.8**	34.4	**1.0**	26.9	**0.7**	29.2	**1.0**	36.3	**1.0**	35.4	**1.0**	27.1	**0.9**	19.7	**0.7**	30.3	**0.9**
d	**Langley - upper**	D	51.3	**1.6**	57.5	**1.5**	39.4	**1.1**	55.0	**1.5**	42.6	**1.1**	39.6	**1.4**	56.3	**1.5**	60.4	**1.8**	37.9	**1.2**	40.5	**1.3**	48.0	**1.4**
	Langley - lower		47.9	**1.5**	51.8	**1.3**	36.5	**1.0**	52.9	**1.5**	48.4	**1.3**	40.5	**1.4**	53.9	**1.5**	56.2	**1.6**	37.6	**1.2**	35.3	**1.2**	46.1	**1.3**
e	**Lincoln**	D	24.4	**0.8**	44.7	**1.1**	42.2	**1.1**	53.5	**1.5**	47.9	**1.3**	41.6	**1.5**	50.7	**1.4**	49.2	**1.4**	38.8	**1.3**	30.9	**1.0**	42.4	**1.2**
h	**Port Guichon**	E	38.2	**1.2**	33.7	**0.9**	38.7	**1.0**	32.0	**0.9**	34.7	**0.9**	36.2	**1.3**	40.3	**1.1**	41.6	**1.2**	29.9	**1.0**	26.7	**0.9**	35.2	**1.0**
	GREATER VICTORIA																							
	Firm Ground	C	37.8		46.1		44.1		42.3		45.1		33.8		43.3		40.5		36.1		35.8		40.5	
f	**Margaret Jenkins**	D	50.4	**1.3**	52.2	**1.1**	50.0	**1.1**	50.3	**1.2**	42.7	**0.9**	41.0	**1.2**	53.1	**1.2**	60.2	**1.5**	40.8	**1.1**	43.1	**1.2**	48.4	**1.2**
g	**Mount Douglas - deep**	C/D	50.3	**1.3**	55.7	**1.2**	52.1	**1.2**	51.6	**1.2**	53.9	**1.2**	45.8	**1.4**	54.3	**1.3**	49.5	**1.2**	43.1	**1.2**	42.7	**1.2**	49.9	**1.2**
	Mount Douglas - shallow		44.7	**1.2**	51.4	**1.1**	47.3	**1.1**	45.5	**1.1**	51.9	**1.1**	39.6	**1.2**	49.9	**1.2**	49.6	**1.2**	42.1	**1.2**	40.6	**1.1**	46.3	**1.1**
i	**Victoria**	C	46.6	**1.2**	51.7	**1.1**	43.9	**1.0**	74.6	**1.8**	41.2	**0.9**	78.7	**2.3**	54.6	**1.3**	87.8	**2.2**	37.8	**1.0**	38.9	**1.1**	55.6	**1.4**
j	**Willows**	C	47.9	**1.3**	51.8	**1.1**	36.5	**0.8**	52.9	**1.3**	48.4	**1.1**	40.5	**1.2**	53.9	**1.2**	56.2	**1.4**	37.6	**1.0**	35.3	**1.0**	46.1	**1.1**

PGV: Peak Ground Velocity *AF: Amplification Factor* *AF = PGV in soil profile / PGV in firm ground*

Table 4: Peak ground displacement (PGD) and Amplification Factors, FA

	SCHOOL	Site Class	CCO PGD cm	CCO AF	CP106 PGD cm	CP106 AF	CP196 PGD cm	CP196 AF	GIL67 PGD cm	GIL67 AF	MD35 PGD cm	MD35 AF	PK90 PGD cm	PK90 AF	SARA0 PGD cm	SARA0 AF	SO90 PGD cm	SO90 AF	WW235 PGD cm	WW235 AF	WW325 PGD cm	WW325 AF	Average PGD cm	Average AF
	LOWE MAINLAND																							
	Firm Ground	C	9.5		7.9		11.4		10.0		6.3		8.4		10.3		10.3		9.8		7.1		9.1	
a	**Brighouse**	E	17.7	**1.9**	13.8	**1.7**	20.3	**1.8**	15.3	**1.5**	8.7	**1.4**	19.1	**2.3**	17.4	**1.7**	20.5	**2.0**	13.0	**1.3**	13.0	**1.8**	15.9	**1.7**
b	**Garden City**	E	17.8	**1.9**	20.0	**2.5**	20.0	**1.7**	15.8	**1.6**	8.5	**1.3**	18.5	**2.2**	17.4	**1.7**	22.8	**2.2**	12.6	**1.3**	12.5	**1.8**	16.6	**1.8**
c	**James Park - total**	D	10.2	**1.1**	8.7	**1.1**	12.0	**1.0**	11.5	**1.1**	7.8	**1.2**	9.0	**1.1**	10.8	**1.0**	9.0	**0.9**	10.9	**1.1**	6.9	**1.0**	9.7	**1.1**
	James Park - pore		9.6	**1.0**	8.3	**1.0**	12.7	**1.1**	11.7	**1.2**	6.0	**1.0**	9.0	**1.1**	11.7	**1.1**	9.1	**0.9**	9.9	**1.0**	6.9	**1.0**	9.5	**1.0**
d	**Langley - upper**	D	10.9	**1.2**	9.8	**1.2**	12.8	**1.1**	14.0	**1.4**	10.4	**1.7**	14.1	**1.7**	11.9	**1.2**	12.7	**1.2**	12.1	**1.2**	8.3	**1.2**	11.7	**1.3**
	Langley - lower		11.3	**1.2**	9.8	**1.2**	13.1	**1.1**	14.2	**1.4**	10.2	**1.6**	12.3	**1.5**	16.5	**1.6**	15.2	**1.5**	12.3	**1.3**	8.5	**1.2**	12.3	**1.4**
e	**Lincoln**	D	3.1	**0.3**	10.6	**1.3**	15.1	**1.3**	15.9	**1.6**	10.2	**1.6**	11.0	**1.3**	15.2	**1.5**	12.5	**1.2**	14.7	**1.5**	9.9	**1.4**	11.8	**1.3**
h	**Port Guichon**	E	15.2	**1.6**	11.9	**1.5**	16.6	**1.4**	12.3	**1.2**	7.4	**1.2**	14.4	**1.7**	15.1	**1.5**	15.8	**1.5**	11.0	**1.1**	10.7	**1.5**	13.0	**1.4**
	GREATER VICTORIA																							
	Firm Ground	C	11.2		9.3		13.5		11.9		7.5		9.9		12.2		12.2		11.6		8.4		10.8	
f	**Margaret Jenkins**	D	12.7	**1.1**	10.7	**1.1**	15.3	**1.1**	14.6	**1.2**	10.3	**1.4**	12.2	**1.2**	12.6	**1.0**	10.6	**0.9**	13.7	**1.2**	10.3	**1.2**	12.3	**1.2**
g	**Mount Douglas - deep**	C/D	12.1	**1.1**	10.0	**1.1**	14.1	**1.0**	13.7	**1.2**	9.0	**1.2**	11.5	**1.2**	13.7	**1.1**	13.7	**1.1**	12.8	**1.1**	8.2	**1.0**	11.9	**1.1**
	Mount Douglas - shallow		11.5	**1.0**	10.0	**1.1**	13.8	**1.0**	12.9	**1.1**	8.9	**1.2**	10.8	**1.1**	12.8	**1.0**	12.1	**1.0**	12.2	**1.1**	8.1	**1.0**	11.3	**1.1**
i	**Victoria**	C	11.5	**1.0**	10.5	**1.1**	13.6	**1.0**	71.1	**6.0**	10.3	**1.4**	43.0	**4.4**	31.7	**2.6**	59.0	**4.8**	10.6	**0.9**	8.9	**1.1**	27.0	**2.4**
j	**Willows**	C	11.3	**1.0**	9.8	**1.1**	13.1	**1.0**	14.2	**1.2**	10.2	**1.4**	12.3	**1.2**	16.5	**1.3**	15.2	**1.2**	12.3	**1.1**	8.5	**1.0**	12.3	**1.2**

PGD: Peak Ground Displacement *AF: Amplification Factor* *AF = PGD in soil profile / PGD in firm ground*

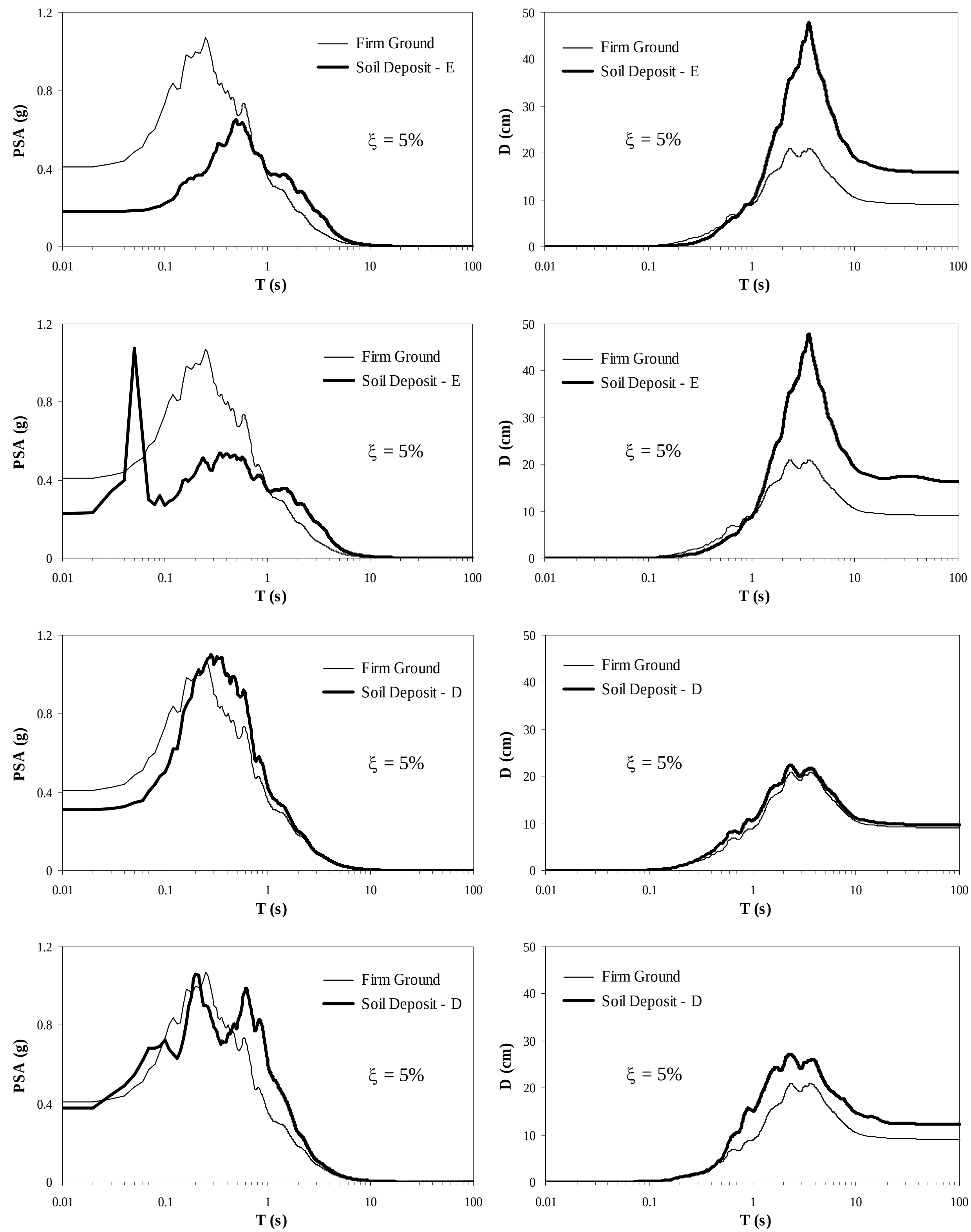

Figure 1: Pseudo Acceleration and Displacement Elastic Response Spectra for Brighouse School, Garden City School, James Park School and Langley School (in descending order)

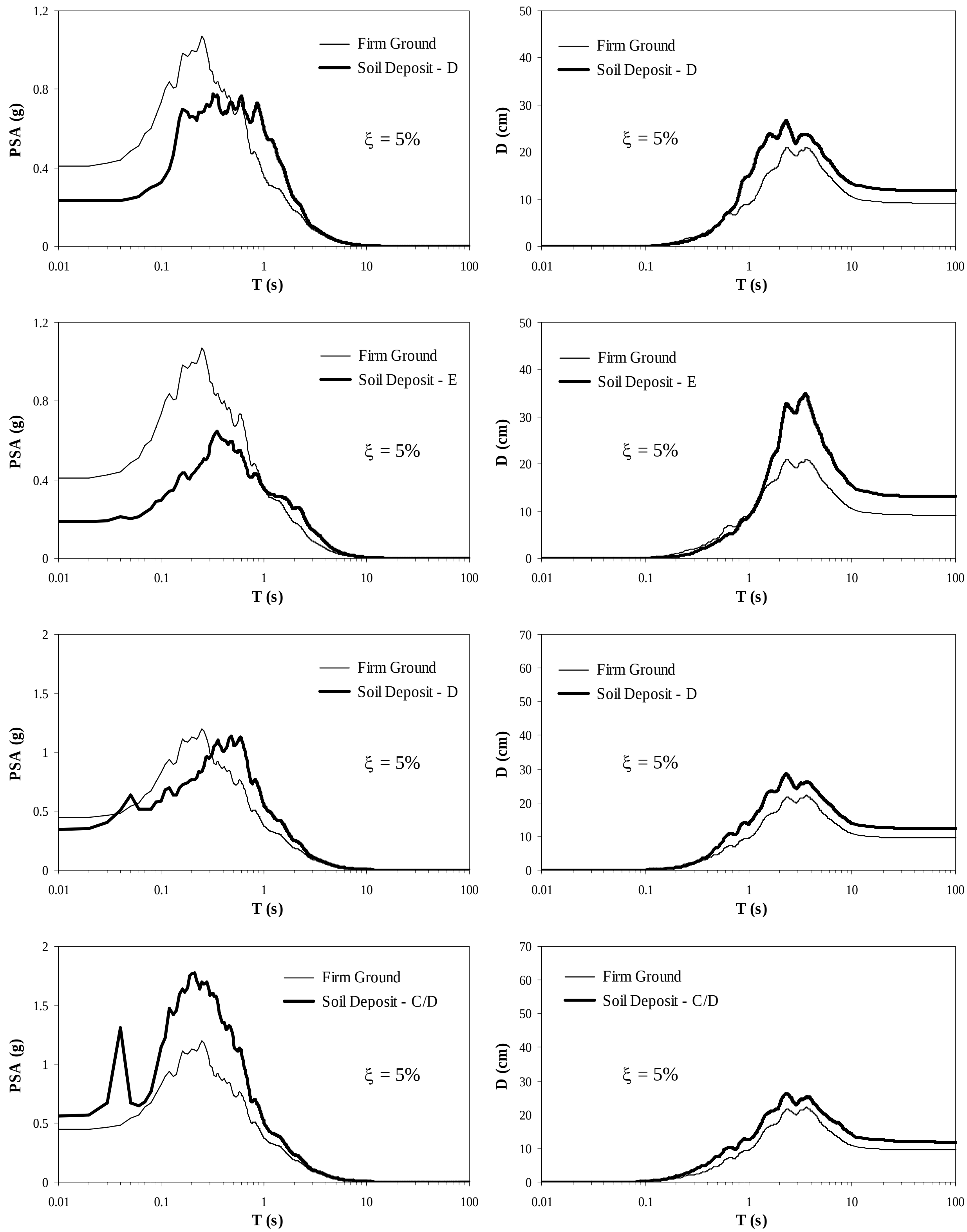

Figure 2: Pseudo Acceleration and Displacement Elastic Response Spectra for Lincoln School, Port Guichon School, Margaret Jenkins School, and Mount Douglas School (in descending order)

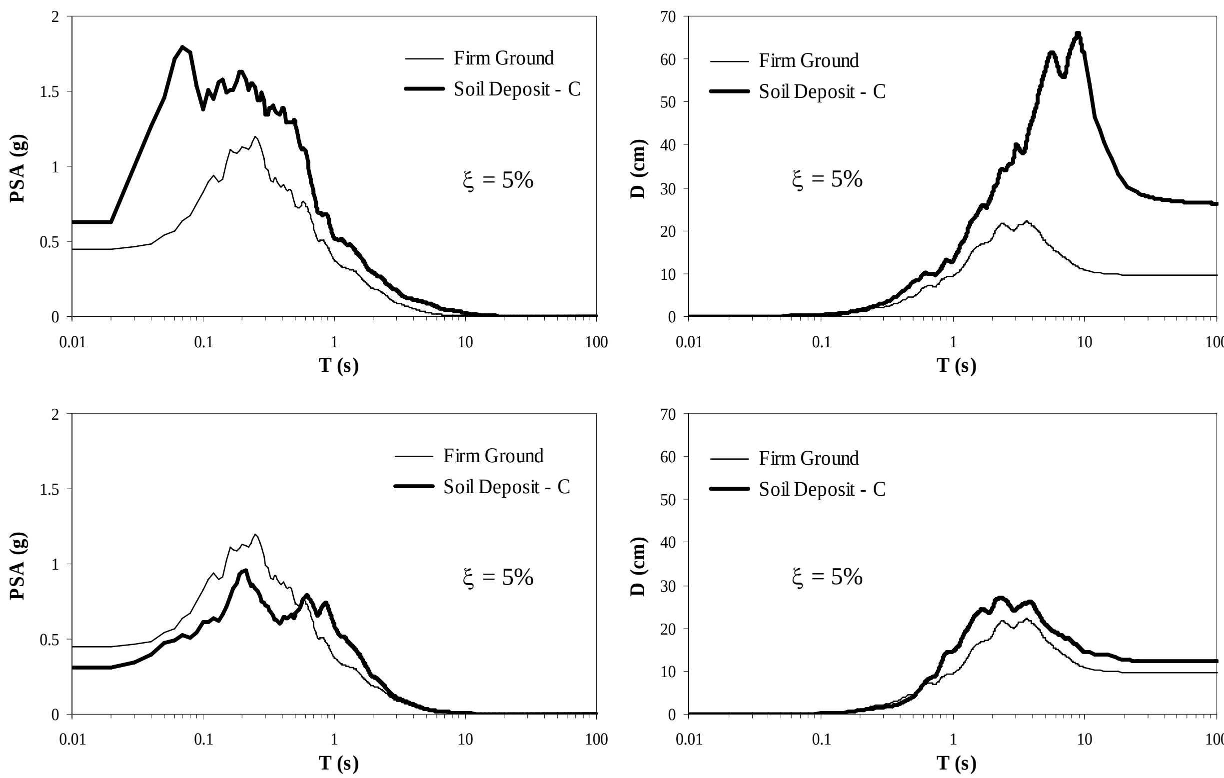

Figure 3: Pseudo Acceleration and Displacement Elastic Response Spectra for Victoria School and Willows School (in descending order)

b) Structural Analysis

The structural analysis corresponds to a nonlinear dynamic analysis (NLDA) for each of the prototypes and using as input ground motions the results obtained from nonlinear dynamic analysis of the column of soil explained on previous section. Details of each prototype with respect to its hysteretic behaviour, backbone curve, and geometric and material properties used in the analysis can be found in Commentary C of the Bridging Guidelines – 2nd Edition.

Each school is defined with a certain prototype according to the structural inspection of each school. The summary of prototypes used for each school as well as location, site class, and values of Rm (%W) are presented in Table 5. The values of Rm corresponds to the minimum required retrofit factored resistance, defined as a percentage of the weight of the structure, obtained from the BG-2nd Edition.

The NLDA is performed by using computer program CANNY which was also used as a parallel tool to evaluate results given in the BG-2nd Edition. For each prototype, maximum inter-story drift values were obtained and then the average and the average plus one standard deviation were defined as the assessment and retrofit limits, respectively. Results for each school and for each of the inter-story drift limits defined in Table 5 are shown in the last part of Appendixes B through L.

Table 5: Information about location, site class, prototypes, and values of Rm (obtained from the BG-2nd edition) for each of the 10 schools

School (City)	Site Class	Prototype	Rm (%W) from BG-2nd Edition					
			1% IDSL	1.5% ISDL	2% IDSL	2.5% IDSL	3% ISDL	4% IDSL
Brighouse Elementary (Richmond)	E	W-1	**33**	**22**	**16**	**12**	**10**	**8**
		M-2	**26**	**21**				
Garden City Elementary (Richmond)	E	W-1	**33**	**22**	**16**	**12**	**10**	**8**
		M-2	**26**	**21**				
James Park Elementary (Port Coquitlam)	D	W-1	**41**	**24**	**18**	**14**	**12**	**9**
		M-2	**31**	**24**				
Langley Fine Arts (Langley)	D	W-1	**41**	**24**	**18**	**14**	**12**	**9**
		W-2		**40**	**25**	**17**	**15**	**11**
		M-2	**26**	**21**				
Lincoln Elemenatry (Port Coquitlam)	D	W-1	**33**	**22**	**16**	**12**	**10**	**8**
		M-2	**26**	**21**				
Margaret Jenkins Elementary (Greater Victoria)	D	W-1	**48**	**25**	**20**	**16**	**13**	**10**
		C-1	**38**	**29**	**24**			
		M-1	**27**	**21**				
Mt. Douglas Secondary (Greater Victoria)	C/D	W-1	**48**	**25**	**20**	**16**	**13**	**10**
		C-1	**38**	**29**	**24**			
Port Guichon Elementary (Delta)	E	W-1	**41**	**24**	**18**	**14**	**12**	**9**
		W-2		**40**	**25**	**17**	**15**	**11**
Victoria High (Greater Victoria)	C	W-1		**25**	**20**	**16**	**13**	**10**
		C-1	**38**	**29**	**24**			
		M-1	**27**	**21**				
Willows Elementary (Greater Victoria)	C	W-1	**48**	**25**	**20**	**16**	**13**	**10**
		C-1	**38**	**29**	**24**			
		M-1	**27**	**21**				

Prototype W-1, Blocked OSB, is repeated for each school which allows that an overall understanding of this work be observed. Table 6 shows two values for each of the schools and its corresponding inter-story drift limit (ISDL). First value is the value of Rm obtained from NLDA and second value is the amplification factor RAF given by the ratio Rm to Rm obtained from Table 5. The RAF shows the average amplification of the responses by conducting a local site analyses. In general, most of the NLDA of local sites give larger responses than those obtained from the BG. However, this observation is highly depending on several other facts such as site class, soil profile, seismic zone, and some other variables included in the nonlinear analyses of the soil columns. More detail in these observations can be found either in section 7 through 10 or in the Appendixes B through L.

Table 6: Minimum required retrofit factored resistance obtained from NLDA of prototype W1 and RAF with respect to the Rm obtained from the BG version 2 for each of the 10 schools

No	School	Site	Rm (RAF)											
	Name	Class	1% ISDL		1.5% ISDL		2% ISDL		2.5% ISDL		3% ISDL		4% ISDL	
1	Brighouse Elementary	E	25	(0.76)	18	(0.82)	15	(0.94)	13	(1.08)	12	(1.20)	12	(1.50)
2	Garden City Elementary	E	21	(0.64)	14	(0.64)	13	(0.81)	12	(1.00)	12	(1.20)	11	(1.38)
3	James Park Elementary	D	45	(1.10)	35	(1.46)	25	(1.39)	20	(1.43)	16	(1.33)	9	(1.00)
4	Langley Fine Arts	D	38	(0.93)	30	(1.25)	26	(1.44)	25	(1.79)	23	(1.92)	18	(2.00)
5	Lincoln Elementary	D	30	(0.91)	23	(1.05)	21	(1.31)	19	(1.58)	18	(1.80)	15	(1.88)
6	Margaret Jenkins Elementary	D	42	(0.88)	33	(1.32)	29	(1.45)	26	(1.63)	22	(1.69)	17	(1.70)
7	Mt. Douglas Secondary	C/D	61	(1.27)	58	(2.32)	37	(1.85)	31	(1.94)	25	(1.92)	17	(1.70)
8	Port Guichon Elementary	E	22	(0.54)	14	(0.58)	13	(0.72)	11	(0.79)	10	(0.83)	9	(1.00)
9	Victoria High	C	--	--	53	(2.12)	39	(1.95)	31	(1.94)	28	(2.15)	24	(2.40)
10	Willows Elementary	C	32	(0.67)	26	(1.04)	23	(1.15)	20	(1.25)	19	(1.46)	16	(1.60)

Rm: Minimum Required Retrofit Factored Resistance
RAF: Rm Amplification Factor (NLDA/BG)
NDLA: Non-linear Dynamic Analysis
BG: Bridging Guidelines
W1: Blocked OSB

7. First Site Response Analysis - Port Guichon Elementary

a) Introduction

The first comprehensive site response analysis, the analysis for Port Guichon Elementary, was used as the benchmark analysis that would guide the analyses of the remaining nine school sites.

b) Preliminary Analysis (SHAKE)

The structural analysis results based on the surface ground motions generated by the SHAKE site response analysis indicate a very high amplification site (refer to Section 8.3) where the retrofit minimum required factored resistance is 21%W for wood frame building prototype W-1. This resistance value is 25% higher than the highest resistance value given in the building code (Site Class D).

c) Refined Analysis (NLDA)

A second analysis for Port Guichon was undertaken where the site response analysis was undertaken using DESRA ..

The structural analysis results based on the surface ground motions generated by the DESRA site response analysis indicate a low amplification site (refer to Section 8.3) where the retrofit minimum required factored resistance is 9%W for wood frame building prototype W-1. This resistance value corresponds to the minimum value given in the Bridging Guidelines (Bridging Guidelines Site Class C value).

d) Conclusion

For this first benchmark site response analysis, our principal conclusion is that a non-linear dynamic site response analysis using a software package such as DESRA yields a more reliable prediction of site amplification.

The remaining nine site response analyses have been conducted using DESRA.

8. Analysis Results

a) Introduction

This section provides details of the site response analysis for the ten selected school sites.

The site response analysis results are presented in three forms as follows:

(1) Surface ground motion amplification

(2) Building minimum retrofit strength requirements

(3) Soil amplification category

b) Surface Ground Motion Amplification

Surface ground motion amplification is presented as either spectral amplification or peak ground motion amplification.

Spectral response amplification graphs for the school sites are given in Appendix B.

The results of the surface peak ground motion amplification are given below (Table 7) with tabulated amplification values corresponding to the average values for the suite of ten input ground motions.

Table 7: Surface peak ground motion amplification

No	**School**	**Site Class**	**Surface Amplification** PGA	PGV	PGD
1	Brighouse Elementary	E	10%	10%	10%
2	Garden City Elementary	E	10%	10%	10%
3	James Park Elementary	D	10%	10%	10%
4	Langley Fine Arts	D	10%	10%	10%
5	Lincoln Elementary	D	10%	10%	10%
6	Margaret Jenkins Elementary	D	10%	10%	10%
7	Mt. Douglas Secondary	C/D	10%	10%	10%
8	Port Guichon Elementary	E	10%	10%	10%
9	Victoria High	C	10%	10%	10%
10	Willows Elementary	C	10%	10%	10%

The surface amplification is calculated as a percentage of the corresponding average ground motion for the Site Class C suite of ten ground motions.

c) Minimum Strength and Soil Amplification Categories

The impact of surface ground motion amplification on the minimum strength requirements for buildings is expressed in the following two forms:

(1) Minimum required factored resistance Rm for the retrofit design of the building

(2) Soil amplification category (defined below)

The soil amplification categories (Table 8) make reference to the following three minimum required factored resistance values in ascending order of magnitude (first value lowest):

Table 8: Surface peak ground motion amplification

Notation	Minimum Required Retrofit Factored Resistance
Rm1	Bridging Guidelines 2nd edition Site Class C value for given seismic zone
Rm2	British Columbia Building Code 2006 Site Class E value for given seismic zone
Rm3	British Columbia Building Code 2006 Site Class D value for given seismic zone

The surface ground motion amplification is divided into six levels (categories) of amplification as defined below (Table 9):

Table 9: Surface ground motion amplification categories

Amplification Category	Severity of Amplification	Minimum Building Strength
SAC1	Very Low	0.75*Rm1 >= Rm
SAC2	Low	Rm1 >= Rm > 0.75*Rm1
SAC3	Medium/Low	0.5(Rm1+Rm2) >= Rm > Rm1
SAC4	Medium	Rm2 >= Rm > 0.5(Rm1+Rm2)
SAC5	High	Rm3 >= Rm > Rm2
SAC6	Very High	Rm > Rm3

The analysis results for the minimum required factored resistance values for retrofitting selected school building prototypes and the associated soil amplification categories are given in the table below (Table 10).

Table 10: Rm values for SAC categories for each school

School	Site Class	Prototype	Rm (%W) SAC2	SAC3	SAC4	SAC5	Site Rm(%W)	SAC	Percent Code
Brighouse	E	W-1	8	10	12	14	12	SAC4	100%
		M-2	21	24	27	33	17	SAC2	64%
Garden City	E	W-1	8	10	12	14	11	SAC4	100%
		M-2	21	24	27	33	14	SAC1	<64%
James Park	D	W-1	8	10	12	14	9	SAC3/5	71%
		M-2	21	24	27	33	27	SAC4	82%
Langley Fine Arts	D	W-1	9	11	13	16	16	SAC5	100%
		W-2	11	15	19	24	24	SAC2	100%
		M-2	24	27	29	36	19	SAC2	67%
Lincoln	D	W-1	8	10	12	14	15	SAC5	107%
		M-2	21	24	27	33	20	SAC2	64%
Margaret Jenkins	D	W-1	10	12	14	17	17	SAC5	100%
		C-1	24	25	26	31	15	SAC2	77%
		M-1	21	27	32	39	16	SAC1	<54%
Mt. Douglas	C/D	W-1	10	12	14	17	16	SAC5	100%
		C-1	24	25	26	31	29	SAC5	100%
Port Guichon	E	W-1	9	11	13	16	9	SAC2	69%
		W-2	11	15	19	24	12	SAC2	69%
Victoria	C	W-1	10	12	14	17	24	SAC6	155%
		C-1	24	25	26	31	32	SAC5	112%
		M-1	21	27	32	39	38	SAC5	107%
Willows	C	W-1	10	12	14	17	16	SAC6	100%
		C-1	24	25	26	31	15	SAC5	53%
		M-1	21	27	32	39	19	SAC5	54%

The site Rm values are the Rm values determined from the structural analysis of the building above its foundations in response to surface ground motions generated by the site response analysis propagated up from the underlying "firm ground".

The percent code values are the Rm values for the site SAC category expressed as a percantage of the corresponding code values for the given Site Class.

The prototype LDRSs listed in the above table are as follows:

(a) C-1: Moderately ductile concrete shearwalls
(b) M-1: Sliding masonry
(c) M-2: Reinforced masonry in flexure/shear
(d) W-1: Blocked OSB
(e) W-2: Unblocked OSB

The amplification results for prototype W-1 at James Park Elementary are classified as SAC3/5 because the site Rm value at the drift limit of 4.0% corresponds to SAC3 whereas the site values for drifts < 4% are closer to SAC5.

9. Assessment of Results

Our overall lateral shaking conclusions from the analysis results given in Section 8 are as follows:

(1) Deep Soft Soil Sites

The three Site Class E sites with deep soil columns above firm ground (at least 150 metres) exhibit low surface amplification (SAC2). The retrofit of buildings on these sites can be designed to the lowest resistance values given in the Bridging Guidelines (Bridging Guidelines Site Class C values).

(2) Site Class D Sites

The five Site Class D sites exhibit high amplification (SAC5) for wood frame building prototypes. Wood frame building retrofits need to be designed for the highest resistance requirements given in the building code (Site Class D).

For building prototypes with lower drift limits (masonry and concrete), the structural demand is less than that for wood frame buildings. With few exceptions, masonry and concrete building retrofits can be designed for the lowest resistance values given in the Bridging Guidelines (Bridging Guidelines Site Class C values).

(3) Site Class C Sites

The results for Victoria High School exhibit unusually high amplification and require closer scrutiny in future research.

The results for Willows Elementary seem to be similar to the results for the majority of the Site Class D sites - high amplification for wood frame prototypes and low amplification for concrete and masonry prototypes.

10. Future Research

A combined APEGBC/UBC proposal has been submitted to the British Columbia Ministry of Education to continue this research beyond the recent completion of the second edition of the Bridging Guidelines. If the APEGBC/UBC proposal is funded by the British Columbia Ministry of Education, the priorities for on-going site response analysis are as follows:

(1) Deconvolution

The impact of deconvolution on the site response analysis of shallower sites needs to be investigated.

(2) Very High Amplification Sites

For life safety considerations, the soil column characteristics of very high amplification sites need to be determined. Basin edge effects and topographic amplification will be considered in the identification of zones of potentially very high amplification.

(3) Large Database

The database of site response analyses needs to be expanded to at least 50 sites to ensure greater reliability of results.

(4) Subduction Ground Motion

Site response analysis needs to be expanded to include a suite of subduction ground motions for the assessment and retrofit of south Vancouver Island schools.

(5) Soil Amplification Criteria

To reduce dependency on site response analysis, a set of site classification criteria needs to be developed for identifying zones of different soil amplification severity.

11. Conclusion

The principal conclusions arising from this preliminary site response analysis study are as follows:

(1) NEHRP

NEHRP site classification criteria for Site Class D sites seem to be a reliable indicator of high amplification sites for wood frame buildings.

The NEHRP criteria do not appear to be a reliable indicator of the severity of site amplification for other site class/form of construction combinations.

(2) Future Research

The preliminary conclusions given in (1) above necessitate future research (detailed in Section 10) to provide a more precise basis for predicting site amplification characteristics.

(3) Short Term Strategy

Until the future research in (2) is completed, site response analyses should be conducted for all school retrofit sites.

Appendices:

A. Selected Publication Extracts

B. Detailed Results for Brighouse Elementary School

C. Detailed Results for Garden City Elementary School

D. Detailed Results for James Park Elementary School

E. Detailed Results for Langley Fine Arts School

F. Detailed Results for Lincoln Elementary School

G. Detailed Results for Margaret Jenkins Elementary School

H. Detailed Results for Mount Douglas Secondary School

I. Study of the Ground Response Analysis in Port Guichon using Pro-Shake Program

J. Detailed Results for Port Guichon School

K. Detailed Results for Victoria High School

L. Detailed Results for Willows Elementary School

RECOMMENDED PROCEDURES FOR IMPLEMENTATION OF DMG SPECIAL PUBLICATION 117 GUIDELINES FOR ANALYZING AND MITIGATING LIQUEFACTION IN CALIFORNIA

Organized through the
Southern California Earthquake Center
University of Southern California

RECOMMENDED PROCEDURES
FOR IMPLEMENTATION OF
DMG SPECIAL PUBLICATION 117
GUIDELINES FOR ANALYZING AND MITIGATING LIQUEFACTION HAZARDS IN CALIFORNIA

Implementation Committee:

G.R. Martin and M. Lew

Co-chairs and Editors

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Committee Members

March 1999

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Cover page photograph of L.A. County Juvenile Hall, Sylmar, California damaged by liquefaction during the San Fernando earthquake of February 9, 1971, was provided by Jack Meehan, Structural Engineer.

Title page photograph of Marine Research Facility at Moss Landing, California, damaged by liquefaction during the Loma Prieta earthquake of October 17, 1989, was provided by Prof. T. L. Youd of Brigham Young University.

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data. However, note that the five-foot sampling interval used for the SPT lacks the ability to pick up the significant variations in blow counts with depth, typical of interbedded sedimentary stratigraphy.

As discussed in the NCEER Workshop Proceedings, increased field performance data have become available at liquefaction sites investigated with CPT in recent years. Those data have facilitated the development of CPT-based liquefaction resistance correlations. These correlations allow direct calculation of CRR, without the need to convert CPT measurements to equivalent SPT blow counts and then applying SPT criteria.

Figure 7.5 shows a chart developed by Robertson and Wride (Youd and Idriss, 1997) for determining liquefaction strengths for clean sands (fines content, FC, less than or equal to 5%) from CPT data. The chart, which is only valid for magnitude 7.5 earthquakes, shows calculated cyclic stress ratios plotted as a function of corrected and normalized CPT resistance, q_{c1N}, from sites where liquefaction effects were or were not observed following past earthquakes. A curve separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction. Dashed curves showing approximate cyclic shear strain potential, _$_c$, as a function of q_{c1N} are shown to emphasize that cyclic shear strain and ground deformation potential of liquefied soils decrease as penetration resistance increases.

The NCEER Workshop Proceedings provide an explicit commentary on how the new Robertson and Wride CPT procedure should be used for liquefaction evaluations. Although there is not complete consensus about this procedure, it is recommended by this Implementation Committee that the method be used with care; a parallel borehole should be drilled to verify soil types and liquefaction resistances estimated from the CPTs.

7.2 Use of Site-Specific Response Analyses

For critical projects, the use of non-linear site specific one dimensional ground response analyses may be warranted to assess the liquefaction potential at a site. For these analyses, acceleration time histories representative of the seismic hazard at the site are used to define input ground motions at an appropriate firm ground interface at depth. One common approach is to use the equivalent linear total stress computer program SHAKE (Idriss and Sun, 1992) to determine maximum earthquake induced shear stresses at depth for use with the simplified procedure described above, in lieu of using the mean values of r_d shown in Figure 7.3.

In general, equivalent linear analyses are considered to have reduced reliability as ground shaking levels increase to values greater than about 0.4g in the case of softer soils, or where maximum shear strain amplitudes exceed 1 to 2 percent. For these cases, true non-linear site response programs may be used, where non-linear shear stress-shear strain models (including failure criteria) can replicate the hysteretic soil response over the full time history of earthquake loading. The computer program DESRA-2, originally developed by Lee and Finn (1978), is perhaps the most widely recognized non-linear one dimensional site response program. Other non-linear programs include MARDES (Chang et al., 1991), D-MOD (Matasovic, 1993) and SUMDES (Li et al., 1992).

The application of the DESRA-2 code in an effective stress mode, where time histories of pore water pressure increase are computed during ground shaking, is described for example by Finn et al. (1977) and Martin et al. (1991). The latter paper describes a comparison between the simplified method for evaluating liquefaction potential and an effective stress site response analysis for a particular site.

Two-dimensional and three-dimensional response analyses can also be performed.

7.3 Hazard Assessment

The report on liquefaction assessment at a given site should include drill hole logs, field and corrected SPT blow counts, and classification test results, if SPT tests are performed. If CPT tests are performed, field and normalized CPT data (tip resistance, sleeve friction, and friction ratio) should be provided. The CPT data also should be interpreted to estimate soil behavior types. Values of $(N_1)_{60}$ and/or q_{c1N} required to resist liquefaction for a factor of safety equal to 1.0 should be determined as shown in the example on Figure 7.6. In that figure, CPT data were converted to equivalent values of $(N_1)_{60}$ at one-foot intervals. The site liquefaction potential should be evaluated for a specific design earthquake magnitude and peak ground acceleration and the evaluation should be repeated for the other CPT soundings across the site (Martin et al., 1991).

In using such data to evaluate mitigation needs and to establish appropriate factors of safety for analyses, four principal liquefaction-related potential hazards need to be considered:

1. Flow slides or large translational or rotational site failures mobilized by existing static stresses (i.e., the site static factor of safety drops below unity (1.0) due to low strengths of liquefied soil layers).

2. Limited lateral spreads of the order of feet or less triggered and sustained by the earthquake ground shaking.

3. Ground settlement.

4. Surface manifestation of underlying liquefaction.

Each of those hazards and their potential should be addressed in the site report, along with mitigation options, if appropriate. Specific guidelines on each of the hazards are discussed in the subsections that follow.

In evaluating the need to address the above hazards, an acceptable factor of safety needs to be chosen. Often the acceptable factor of safety is chosen arbitrarily. The CDMG guidelines (Special Publication 117) suggest a minimum factor of safety of 1.3 when using the CDMG ground motion maps, with a caveat that if lower values are calculated, the severity of the hazard should be evaluated. Clearly, no single value can be cited in a guideline, as considerable judgment is needed in weighing the many factors involved in the decision. Several of those factors are noted below:

1. The type of structure and its vulnerability to damage. As discussed in Section 8.3, structural mitigation solutions may be more economical than ground remediation.

2. Levels of risk accepted by the owner or governmental regulations associated with questions related to design for life safety, limited structural damage, or essentially no damage.

APPENDIX B

DETAILED RESULTS FOR BRIGHOUSE ELEMENTARY SCHOOL

CONTENTS

(1) <u>Trow Associates Ltd. Report (SHAKE Analysis)</u>

(a) Covering letter dated January 26, 2007

Trow

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January 26, 2007 Reference: 061-02483

School District 38 (Richmond).
7811 Granville Avenue
Richmond, BC V6Y 3E3

Via e-mail EThorleifson@richmond.sd38.bc.ca

Attention: Mr. Eric Thorliefson

Ground Response Analyses for Phase 2 Seismic Assessment of Garden City and Brighouse Elementary Schools, Richmond, B.C

Dear Sir:

Further to your authorization and the subsequent meeting with UBC, Bush Bohlman & Partners, and CWMM Consulting Engineers held at UBC on November 28, 2006, Trow Associates Inc. has completed the ground response analyses for the subject two elementary school sites.

At the meeting in UBC we have concluded that the ground response analyses may be carried out using the commercially available computer program ProSHAKE, a derivative of program SHAKE. The program computes the response of a semi-infinite horizontal layered soil deposit overlying a uniform half-space subjected to vertically propagating shear waves using equivalent linear methods.

Subsoil conditions at both sites were obtained based on the following information:

1. Test hole and laboratory testing data from the site investigation completed in December 2006, including one SCPT and one CPT both to 30m depth, and two auger holes to 6.1m for each school (Test hole logs and laboratory results will be attached to the geotechnical report);
2. Deep test hole FD96-1 by Geological Survey of Canada, containing shear wave velocity log and bore hole log (GSC Open File 3622 and 3563);
3. Top of the Pleistocene sediments (TILL-LIKE) was assumed as the elastic half space (Site Class C).

Key soil parameters required for the analyses are the low-strain shear modulus (Gmax), initial damping, and shear modulus reduction and damping curves as a function of shear strain. Gmax is calculated using the shear wave velocity measured from SCPT. The following shear modulus reduction and damping curves were used in the SHAKE analyses:

1. shear modulus reduction curve for sand was assumed to be the upper bound curve of Seed;
2. damping curve for sand was assumed as the lower bound of Seed;
3. shear modulus reduction and damping curves for silt were assumed to be curves proposed by Vucetic for Plasticity Index tested or assumed.

Ground motions provided in the "Commentary to the Bridging Guidelines, Second Edition" were used as input motion at the outcrop of TILL-LIKE deposit (Site Class

C). Digital data were subsequently provided to us by UBC. A total of ten earthquake records were analyzed. The scaling factor for the ground motions for Zone 4 as shown in Table C.4-2 of the Commentary was used to scale ground motions to seismic demands on Site Class C. Acceleration and velocity spectra of the each of the ten record were obtained for 5% damping and the mean acceleration and velocity response spectra were calculated as shown in the attached Figure 1 and 2. For comparison, the proposed NBCC 2005 spectral acceleration and velocity values for Richmond are shown in the figures.

Earthquake motions at the ground surface were obtained from the SHAKE analyses. Spectral acceleration and velocity at the ground surface for the ten analyses and the mean acceleration and velocity spectra, all obtained for 5% damping, are shown in Figures 3 and 4 respectively. The ratio of spectral accelerations at the ground surface to that at the outcrop of the TILL-LIKE deposit is shown in Figure 5.

Digital earthquake data at the ground surface will be provided to UBC only for their input to structural analyses.

Discussion:

The above analyses were carried out using a derivative of the program SHAKE. For high levels of shaking where the soil is approaching failure on some cycles, such as the case for the 2475 year event in Richmond, the "equivalent linear method" used in SHAKE tends to over-estimate the response. Analyses conducted for local bridge structures indicated the non-linear hysteretic analyses gave spectral accelerations approximately 20% less than SHAKE for the 2475 year event, while giving similar response for the 475 year event. Analyses using a non-linear hysteretic model are available and could be carried out in addition to the SHAKE analyses, if requested.

We trust the above will meet your present requirements. Please contact the undersigned if you have any questions, or require further assistance.

Yours truly,
TROW ASSOCIATES INC.

Mark Qian, P. Eng.
Senior Geotechnical Engineer

Reviewed by:

Ernest Naesgaard, P. Eng.
Principal Geotechnical Engineer

Enclosure for each school:

Response Spectrum of Input Motions (acceleration, velocity)
Response Spectrum of Ground Surface Motions (Acceleration, Velocity)
Spectral Acceleration Ratio
ProSHAKE Report
Digital Data Files for Earthquake at the Ground Surface

cc:

TBG Seismic Consultants Ltd. – Graham Taylor, gwt@tbgsc.bc.ca
UBC Civil Eng. Dept. – Carlos Ventura, ventura@civil.ubc.ca
KMBR Architects – Cristina Marghetti, cmarghetti@kmbr.com
CWMM Consulting Engineers – John Papadakis, jpapadakis@cwmm.com
Bush Bohlman & Partners – Rob Hall, rhall@bushbohlman.com

(1) <u>Trow Associates Ltd. Report (SHAKE Analysis)</u>

(b) ProShake report

ProShake Report

Data File: C:\PROSHAKE\RMDSCHL\BRIGHO~1\BH-5.DAT

Soil Profile

Profile Name: Seismic Assessment of Brighouse Elementary School
Water Table: 1.00 m
Number of Layers: 37

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Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve	Mod. Parameter	Damp. Parameter
1	SILT	1.00	18.00	14.87	90.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
2	SILT	1.00	18.00	14.87	90.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
3	SILT	1.00	18.00	14.87	90.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
4	SILTY SAND	2.00	18.50	29.95	126.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
5	SAND	2.00	19.00	37.97	140.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
6	SAND	5.00	19.00	48.37	158.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
7	SAND	3.25	19.00	69.94	190.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
8	SILT	1.25	18.00	69.79	195.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
9	SILTY SAND	1.25	19.00	81.42	205.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
10	sandy SILT	1.25	18.50	79.28	205.00	Vucetic - Dobry	Vucetic - Dobry	5.00	5.00
11	SAND	1.00	19.00	81.42	205.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
12	sandy SILT	2.75	18.00	82.49	212.00	Vucetic - Dobry	Vucetic - Dobry	5.00	5.00
13	SILT	3.25	18.00	77.14	205.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
14	SAND	2.00	19.00	89.56	215.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
15	SILT	2.00	18.00	84.84	215.00	Vucetic - Dobry	Vucetic - Dobry	15.00	15.00
16	SILT	4.50	18.00	80.94	210.00	Vucetic - Dobry	Vucetic - Dobry	15.00	15.00
17	SAND	7.00	19.00	99.83	227.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
18	SILT	8.50	18.00	87.23	218.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
19	SAND	10.50	19.00	111.60	240.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
20	SILT	8.50	18.00	154.36	290.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
21	SILT	9.00	18.00	133.81	270.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
22	SILT	13.00	18.00	182.12	315.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
23	SAND	12.00	19.00	302.29	395.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
24	SILT	7.00	18.00	286.38	395.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
25	SILT	11.00	18.00	224.84	350.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
26	SILT	12.00	18.00	301.06	405.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
27	SILT	13.00	18.00	355.35	440.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
28	SILT	19.00	18.00	265.04	380.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
29	SILT	11.00	18.00	414.13	475.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
30	SAND	6.00	19.00	409.96	460.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
31	SILT	5.50	18.00	388.38	460.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
32	SILT	32.50	18.00	301.06	405.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
33	SILT	15.50	18.00	458.87	500.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
34	SILT	34.50	18.00	496.31	520.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
35	SILT	12.50	18.00	585.93	565.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
36	SILT	17.50	18.00	696.48	616.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
37	TILL	0.00	22.00	1,295.75	760.00	Linear	Linear		1.00

Input Motion

Number of Motions: 10

Numeber of Iterations: 5
Strain Ratio: 0.65
Tolerance: 5.00%

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File Name	No of Acc. Values	Max. Acc. (g)	Time Step (sec)	Cuttoff Freq. (Hz)	No of Fourier Terms	Layer	Outcrop
C:\PROSHAKE\RMDSCHL\SHAKER~1\SO90.EQ	3000	0.268	0.020	25.00	4096	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\WW235.EQ	5000	0.345	0.005	50.00	8192	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\WW325.EQ	5716	0.514	0.005	50.00	8192	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CC0.EQ	1999	0.329	0.010	50.00	2048	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\SARA0.EQ	2000	0.404	0.020	25.00	2048	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CP196.EQ	2499	0.276	0.010	50.00	4096	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CP106.EQ	2499	0.371	0.010	50.00	4096	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\PK90.EQ	2000	0.253	0.020	25.00	2048	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\MD35.EQ	2398	0.665	0.010	50.00	4096	37	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\GIL67.EQ	2000	0.498	0.020	25.00	2048	37	Yes

Output Locations

Error! Bookmark not defined.

Layer No	Depth (m)	Outcrop
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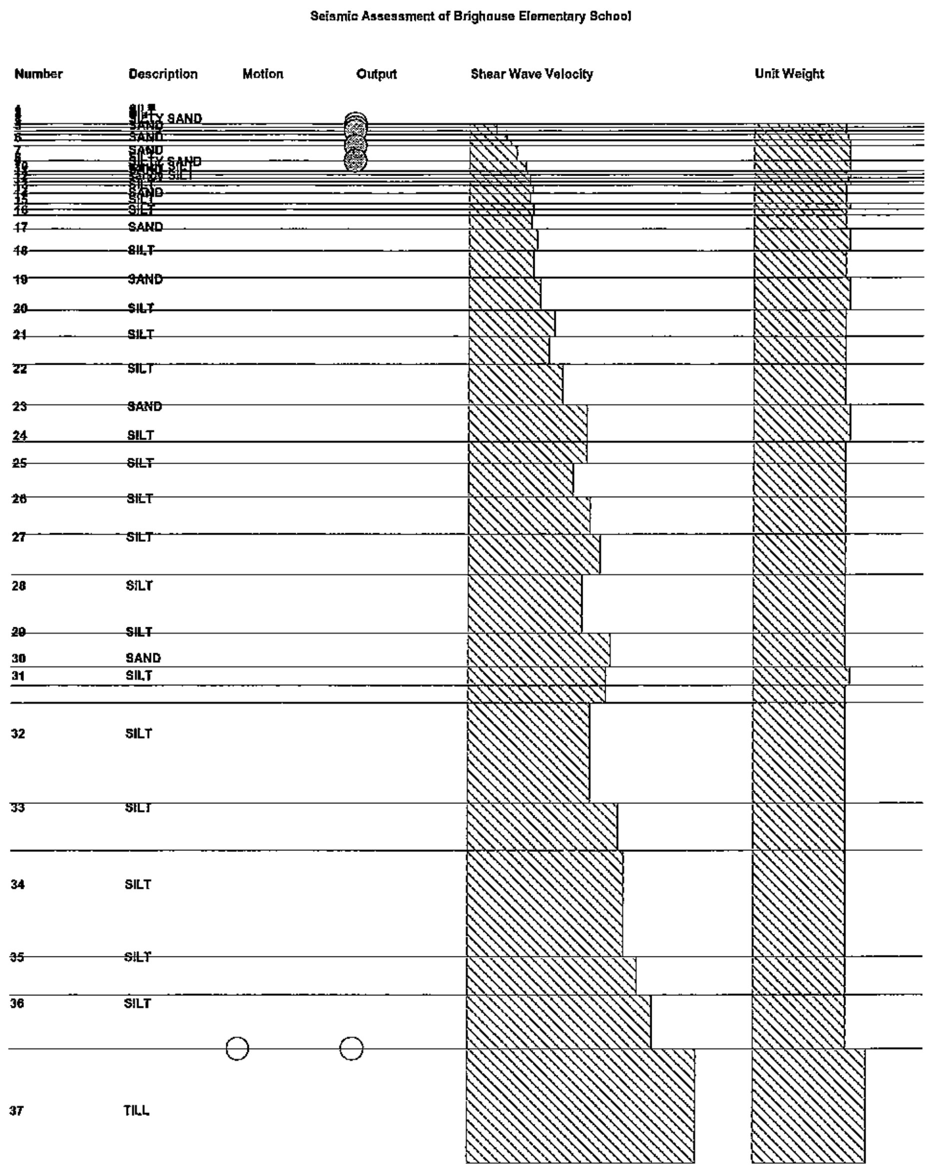
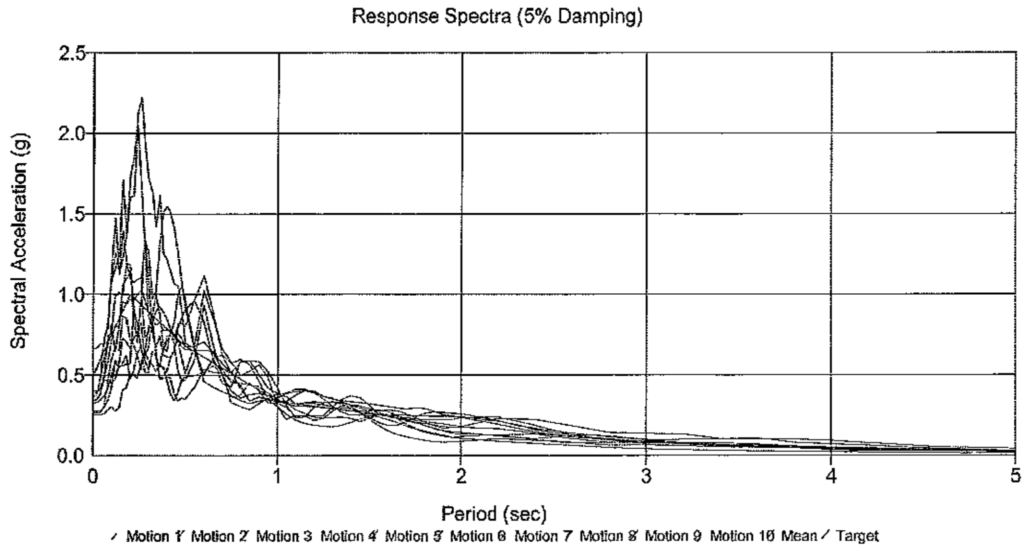
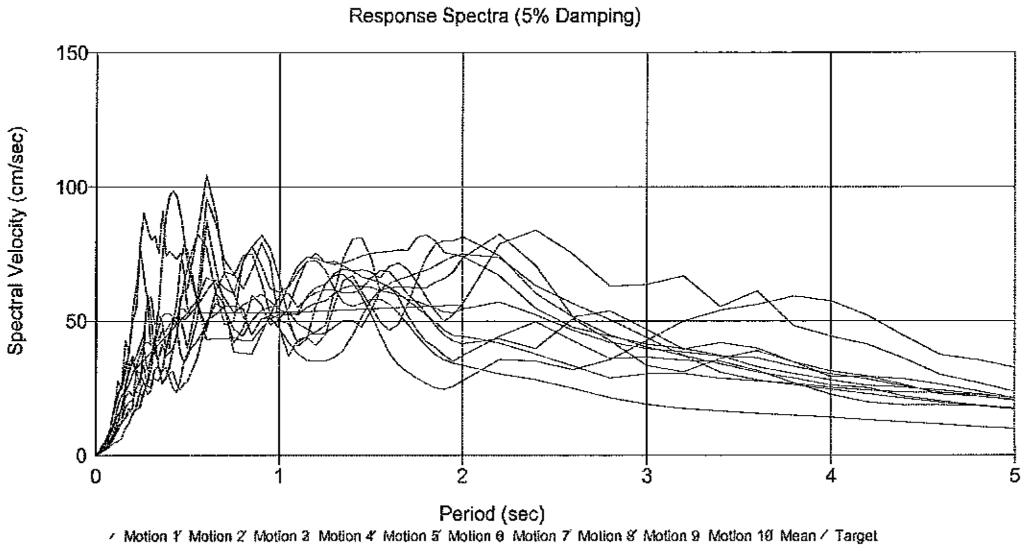
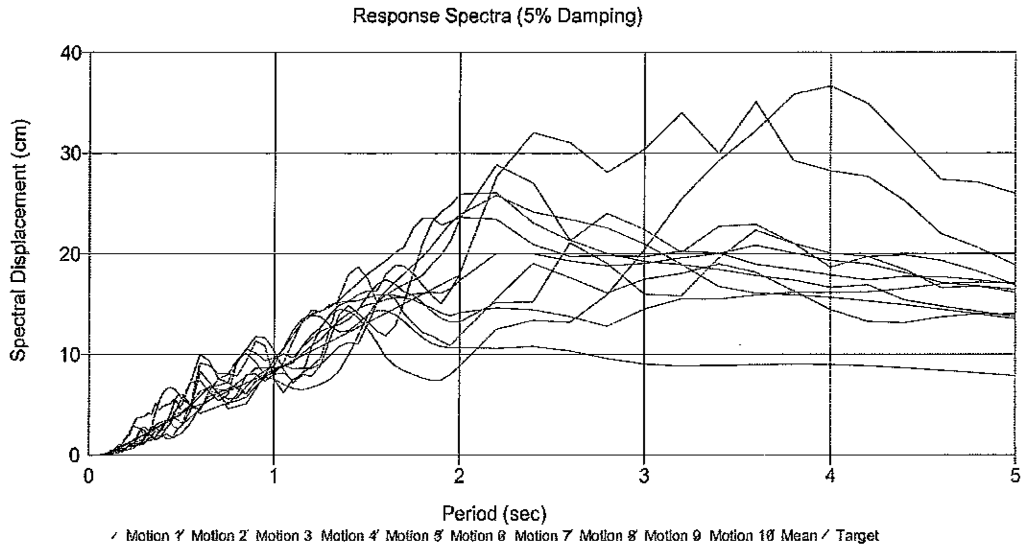
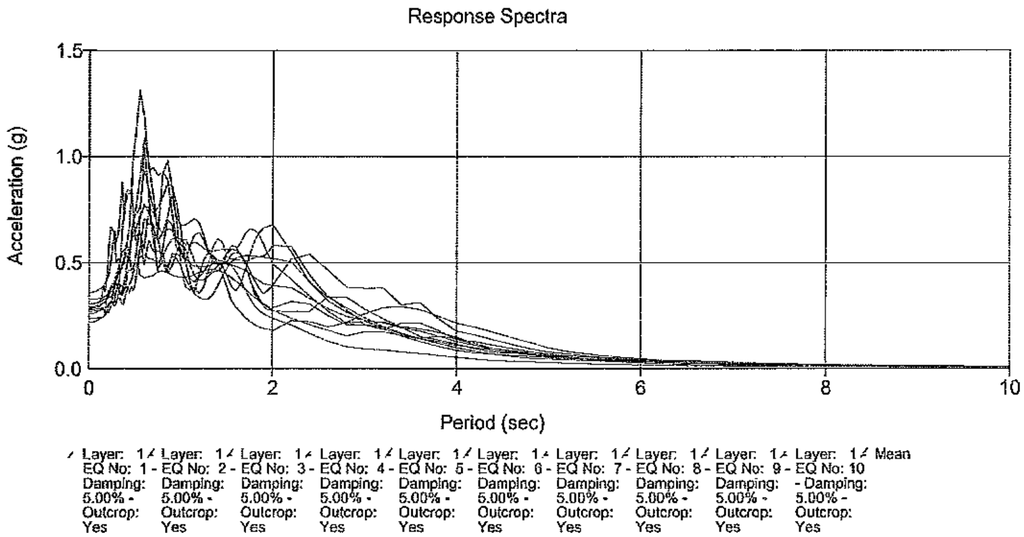
3	2.00	No
6	7.00	No
7	12.00	No
37	300.00	Yes

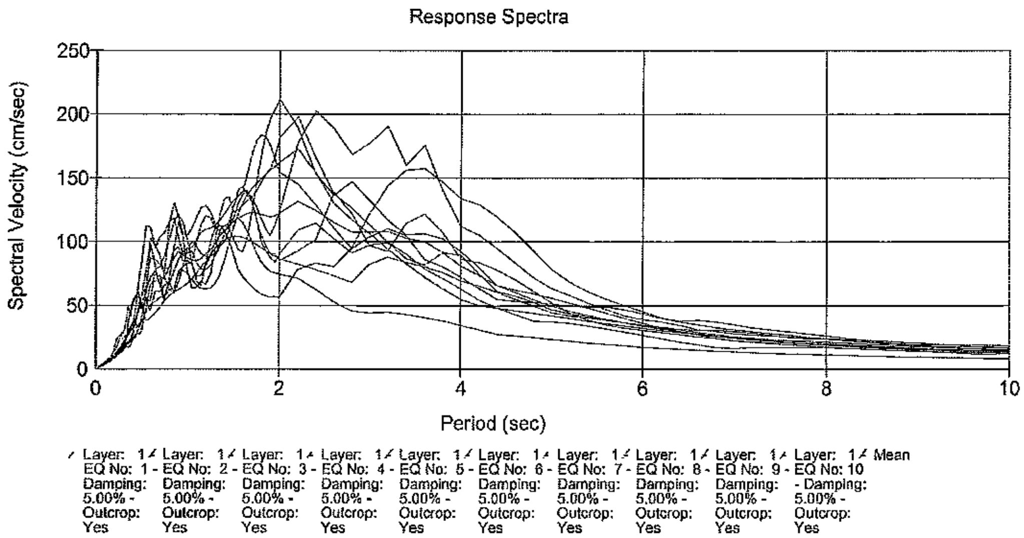

Seismic Assessment of Brighouse Elementary School
Number
Description
Motion
Output
Shear Wave Velocity
Unit Weight
17 SAND
18 SILT
19 SAND
20 SILT
21 SILT
22 SILT
23 SAND
24 SILT
25 SILT
26 SILT
27 SILT
28 SILT
29 SILT
30 SAND
31 SILT
32 SILT
33 SILT
34 SILT
35 SILT
36 SILT
37 TILL

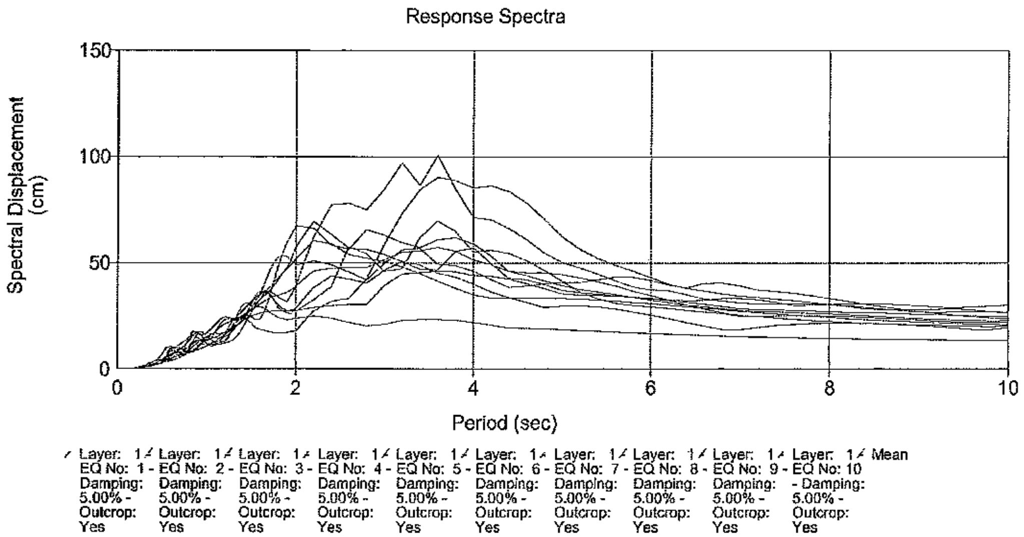

Response Spectra (5% Damping)
Spectral Acceleration (g)
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

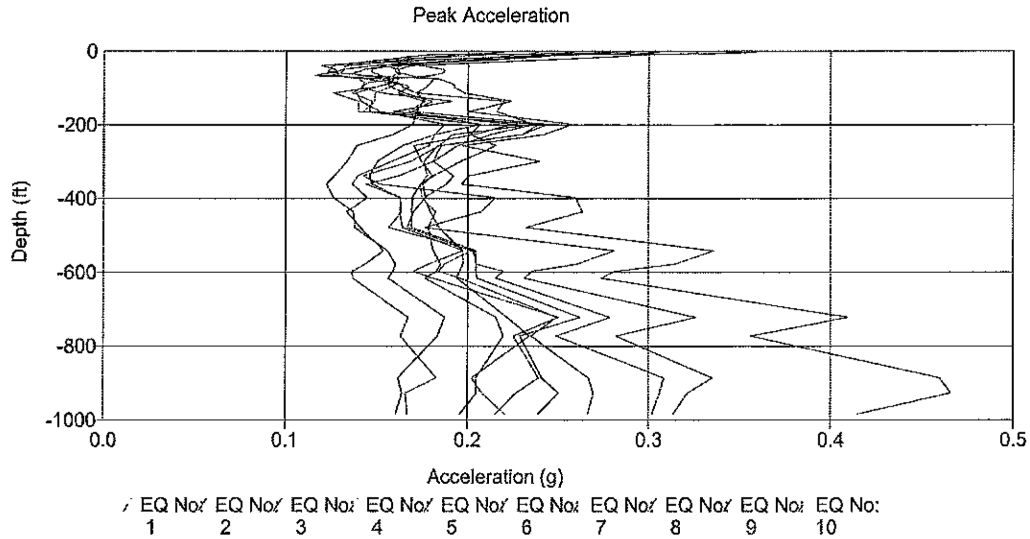

Response Spectra (5% Damping)
Spectral Velocity (cm/sec)
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

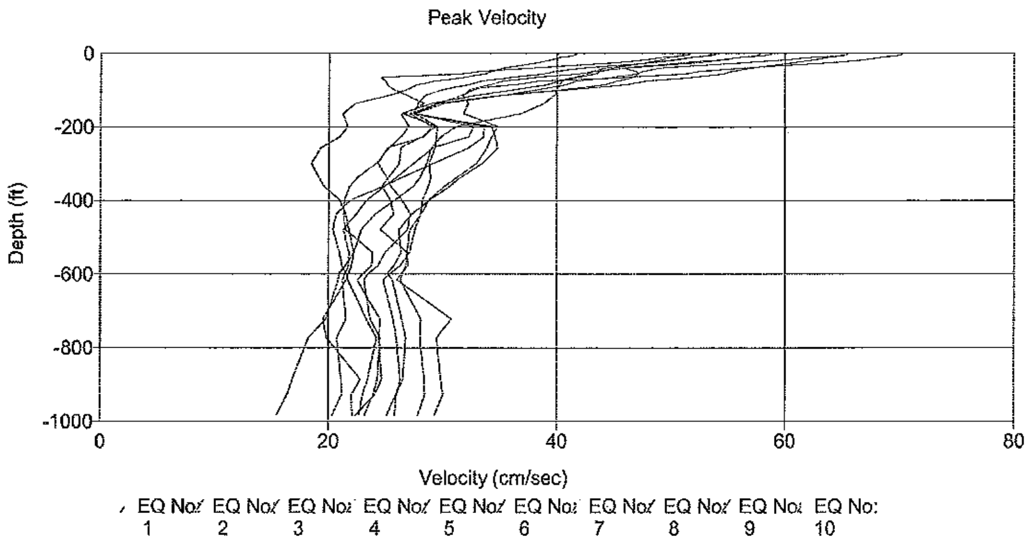

Response Spectra (5% Damping)
Spectral Displacement (cm)
40
30
20
10
0
0
1
2
3
4
5
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

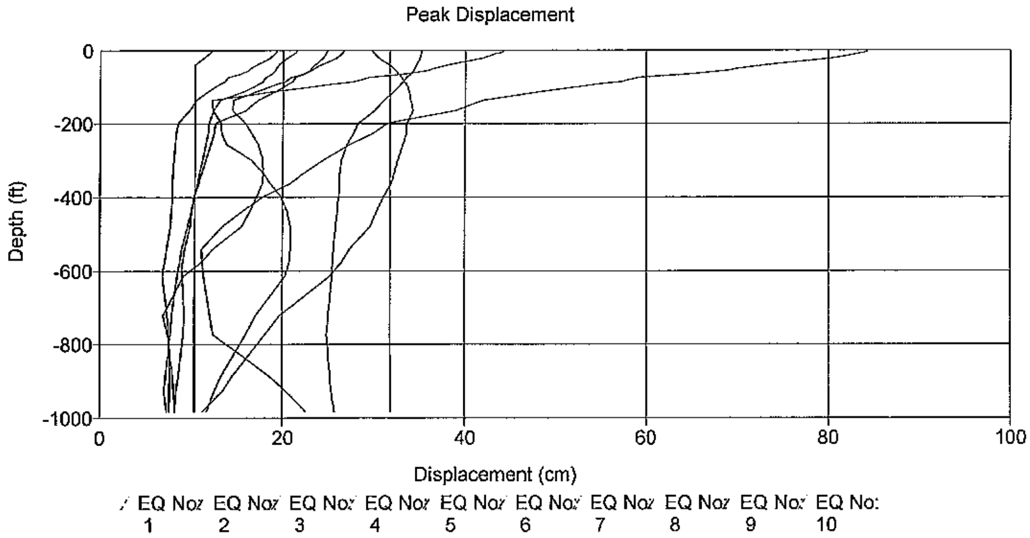

Response Spectra
Acceleration (g)
1.5
1.0
0.5
0.0
0
2
4
6
8
10
Period (sec)
Layer: 1 EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

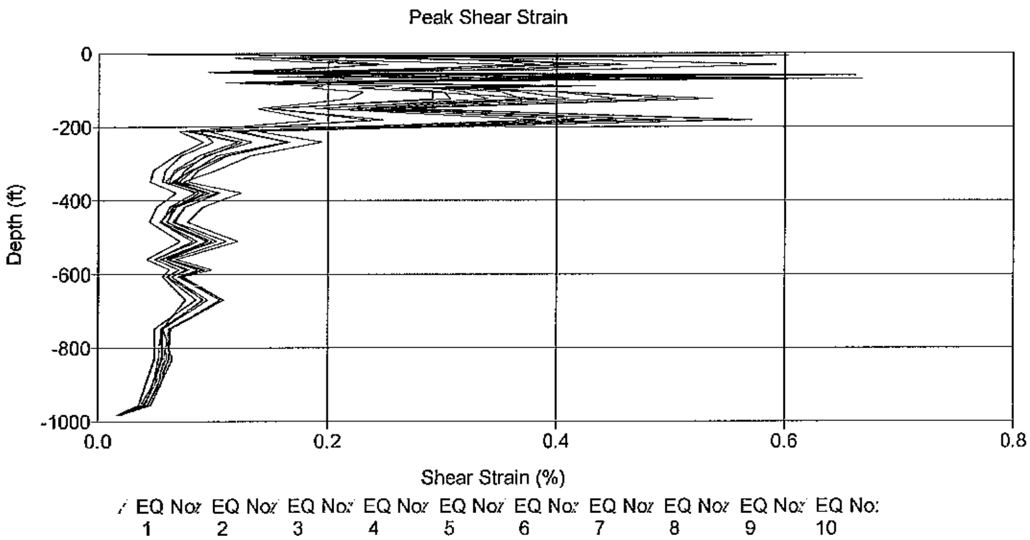

Response Spectra
Spectral Velocity (cm/sec)
250
200
150
100
50
0
0
2
4
6
8
10
Period (sec)
Layer: 1 - EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

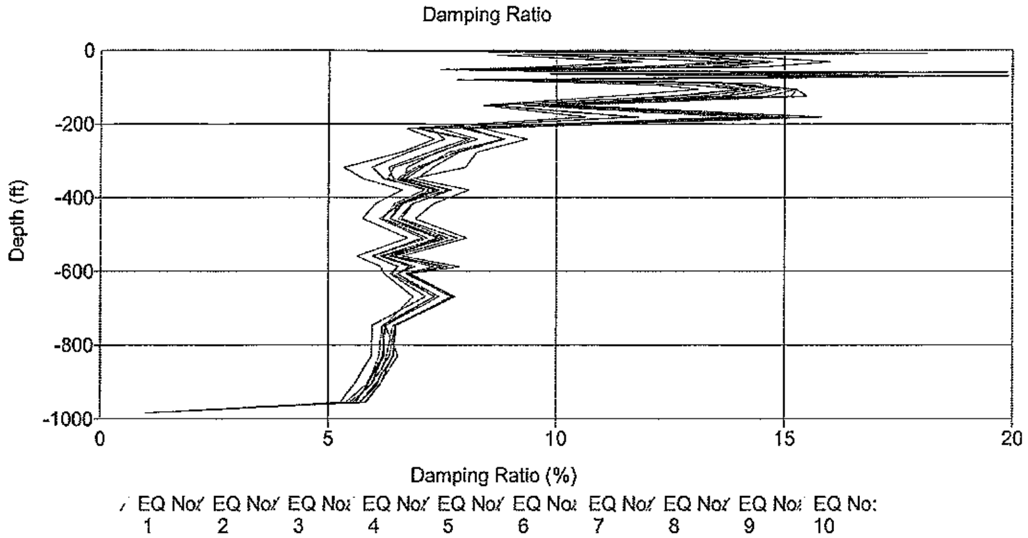

Response Spectra
Spectral Displacement (cm)
150
100
50
0
0
2
4
6
8
10
Period (sec)
Layer: 1 - EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

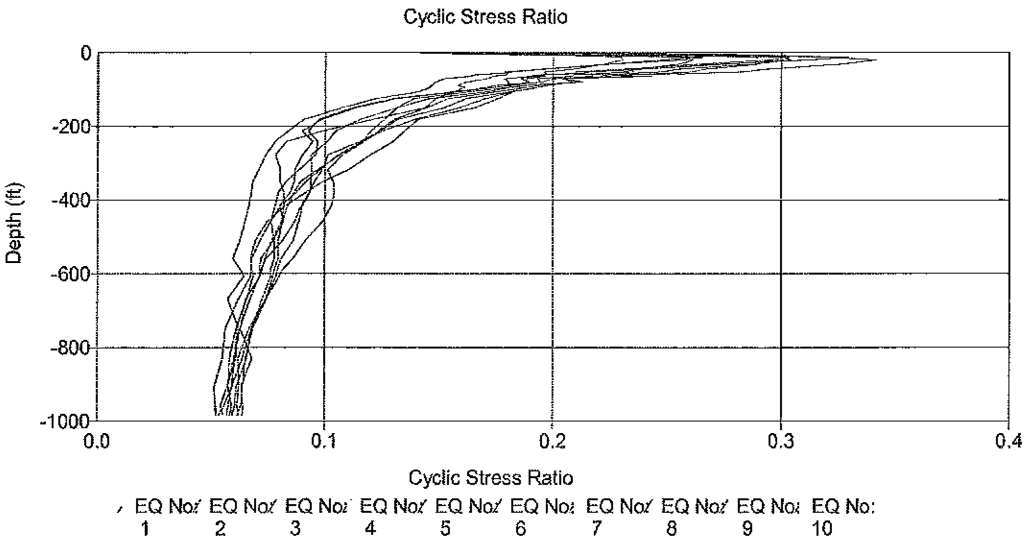

Peak Acceleration
0
-200
-400
-600
-800
-1000
Depth (ft)
0.0
0.1
0.2
0.3
0.4
0.5
Acceleration (g)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

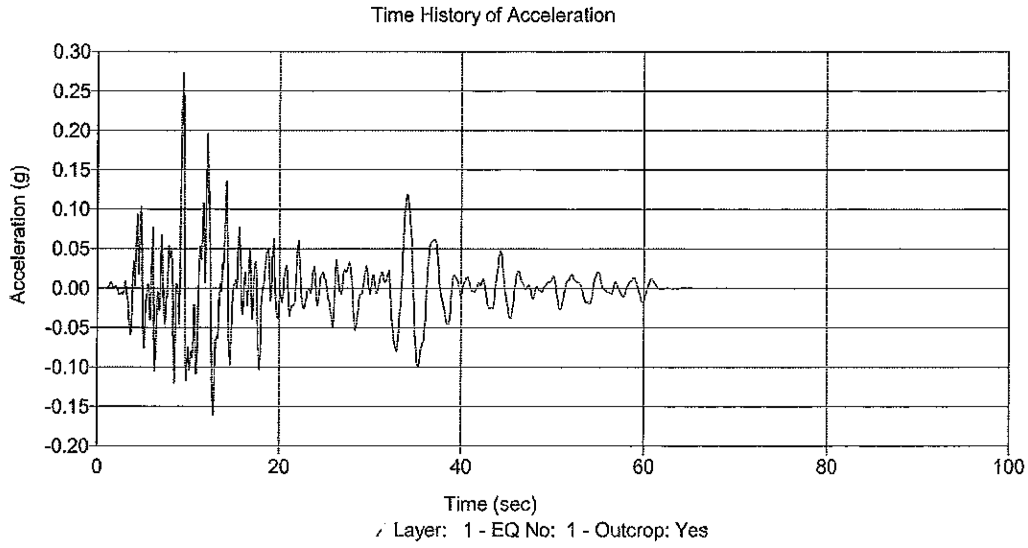

Peak Velocity
0
-200
-400
-600
-800
-1000
Depth (ft)
0
20
40
60
80
Velocity (cm/sec)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

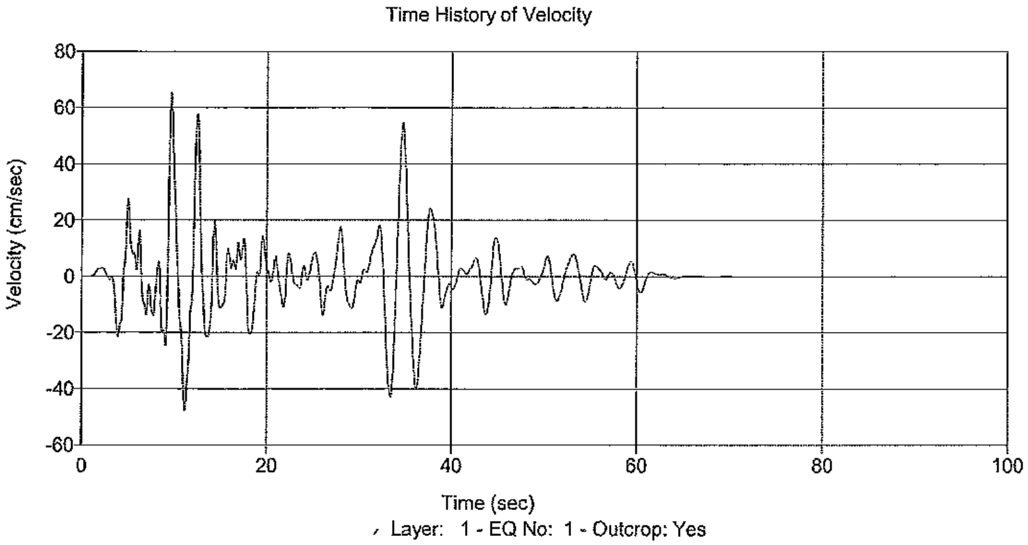

Peak Displacement
Depth (ft)
0
-200
-400
-600
-800
-1000
0
20
40
60
80
100
Displacement (cm)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

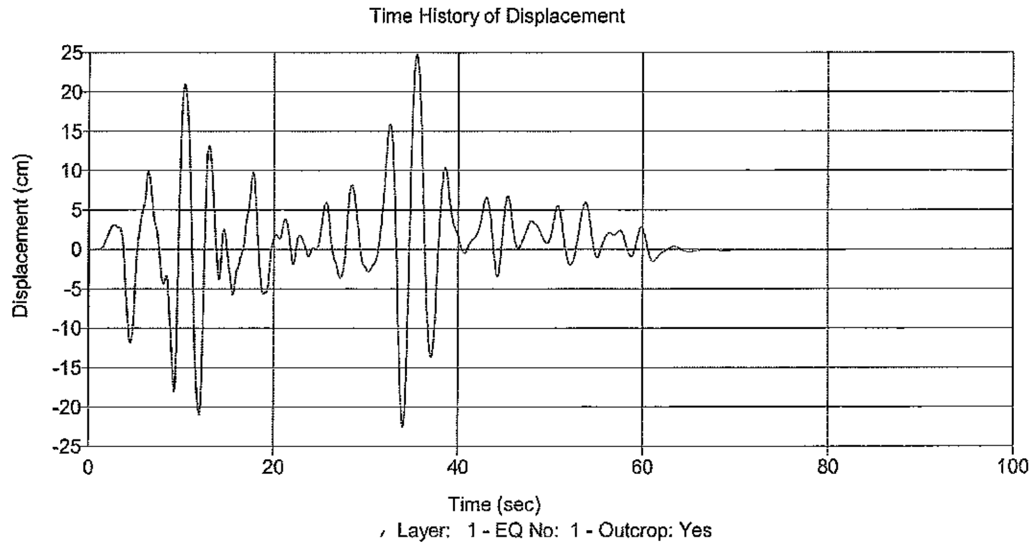

Peak Shear Strain
Depth (ft)
0
-200
-400
-600
-800
-1000
0.0
0.2
0.4
0.6
0.8
Shear Strain (%)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

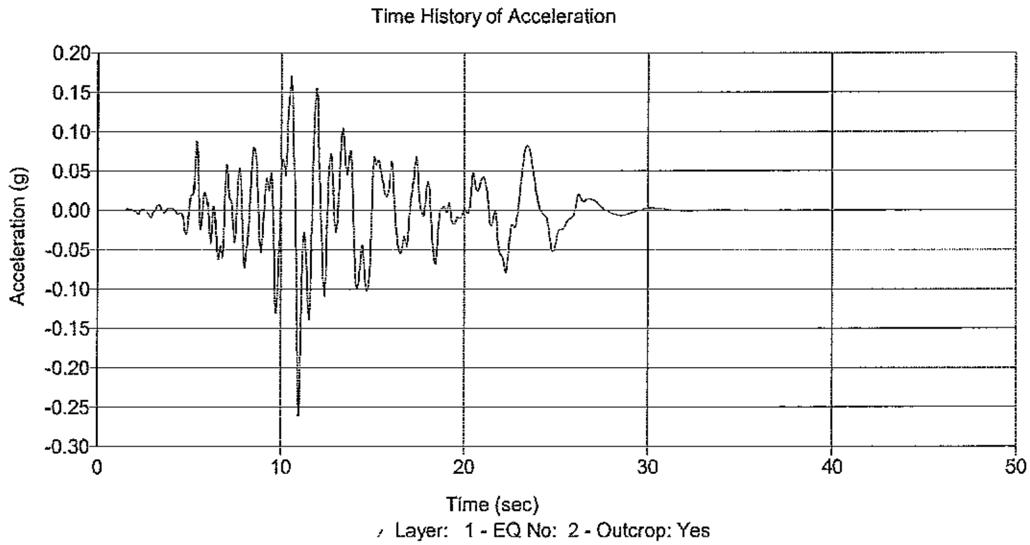

Damping Ratio
Depth (ft)
0
-200
-400
-600
-800
-1000
0
5
10
15
20
Damping Ratio (%)
EQ No: 1 EQ No: 2 EQ No: 3 EQ No: 4 EQ No: 5 EQ No: 6 EQ No: 7 EQ No: 8 EQ No: 9 EQ No: 10

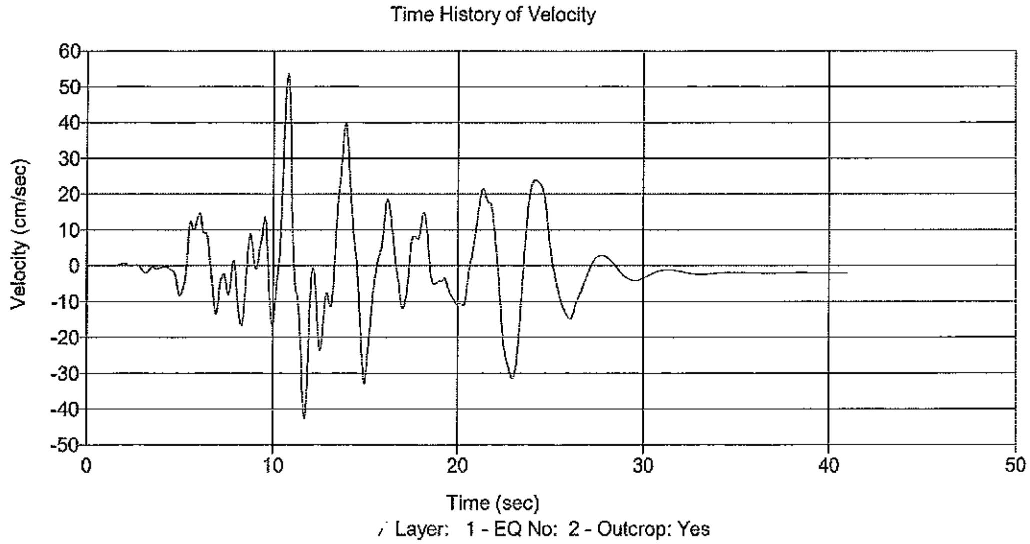

Cyclic Stress Ratio
Depth (ft)
0
-200
-400
-600
-800
-1000
0.0
0.1
0.2
0.3
0.4
Cyclic Stress Ratio
EQ No: 1 EQ No: 2 EQ No: 3 EQ No: 4 EQ No: 5 EQ No: 6 EQ No: 7 EQ No: 8 EQ No: 9 EQ No: 10

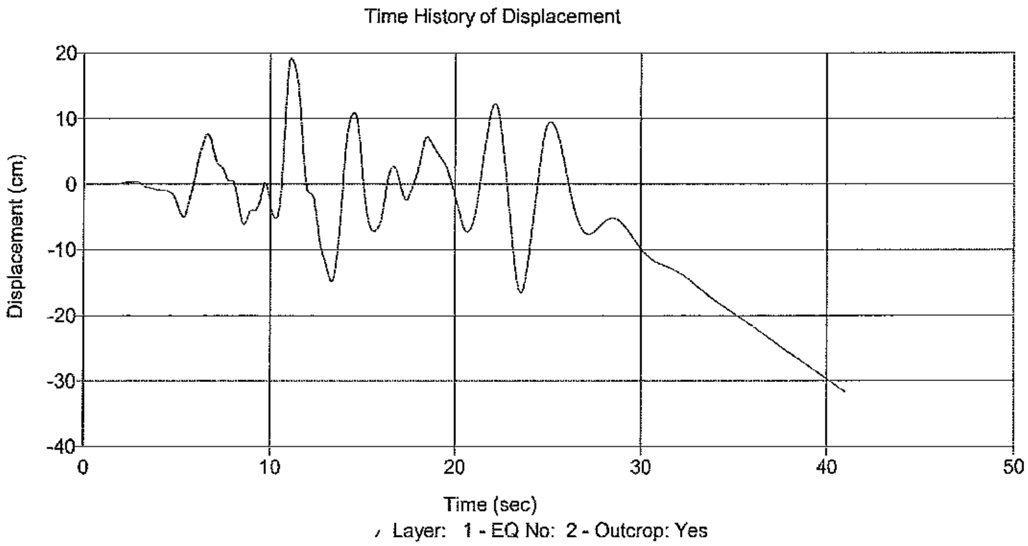

Time History of Acceleration
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
Acceleration (g)
0
20
40
60
80
100
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

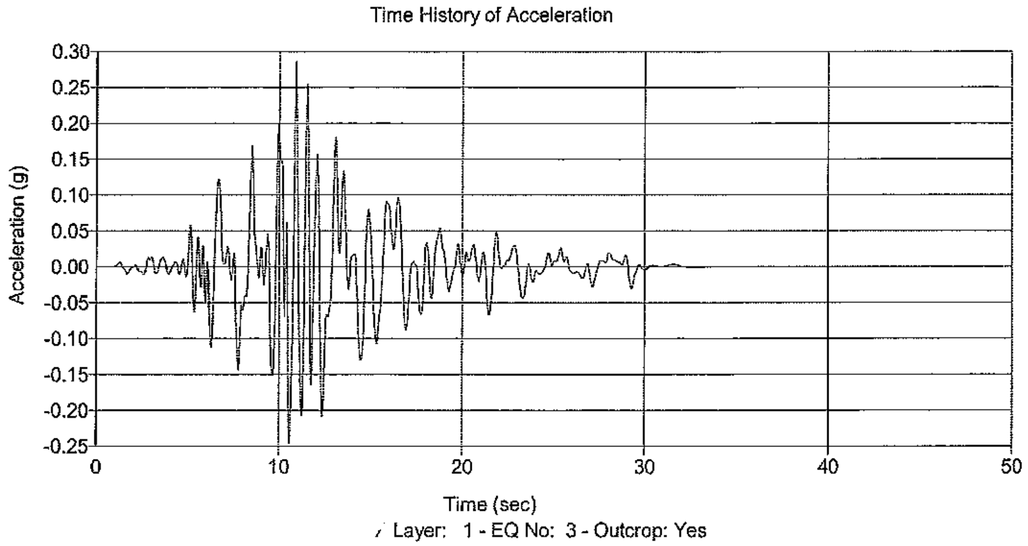

Time History of Velocity
80
60
40
20
0
-20
-40
-60
Velocity (cm/sec)
0
20
40
60
80
100
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

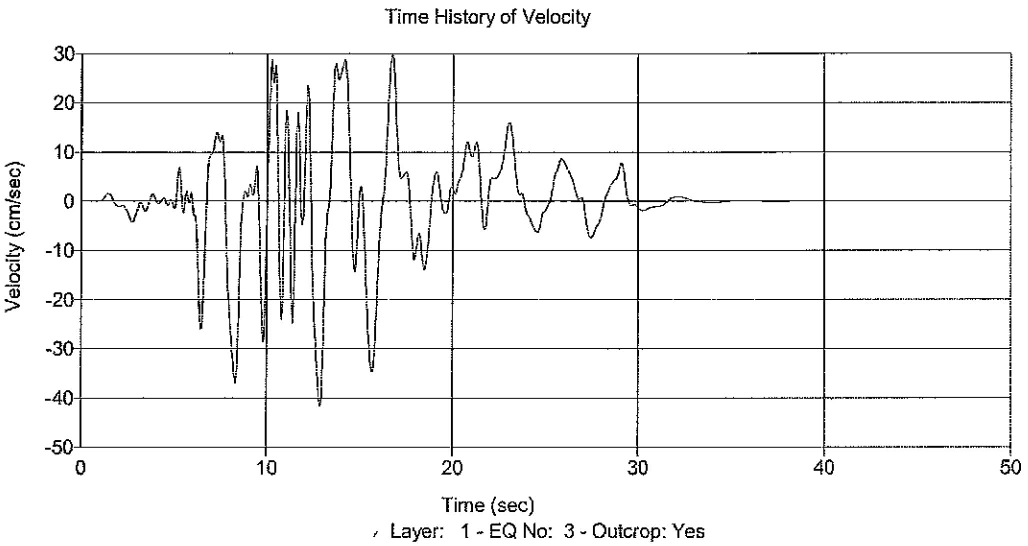

Time History of Displacement
Displacement (cm)
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

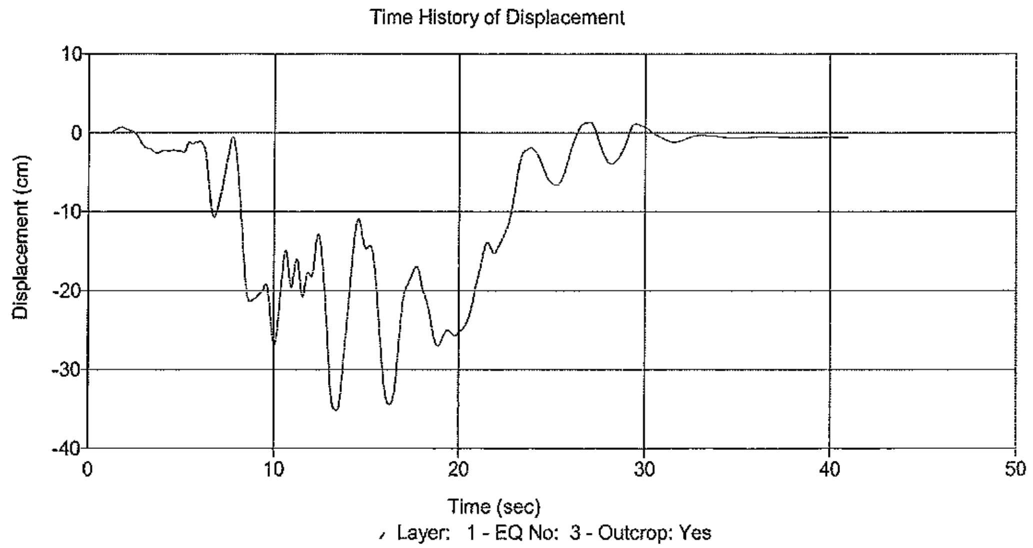

Time History of Acceleration
Acceleration (g)
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes

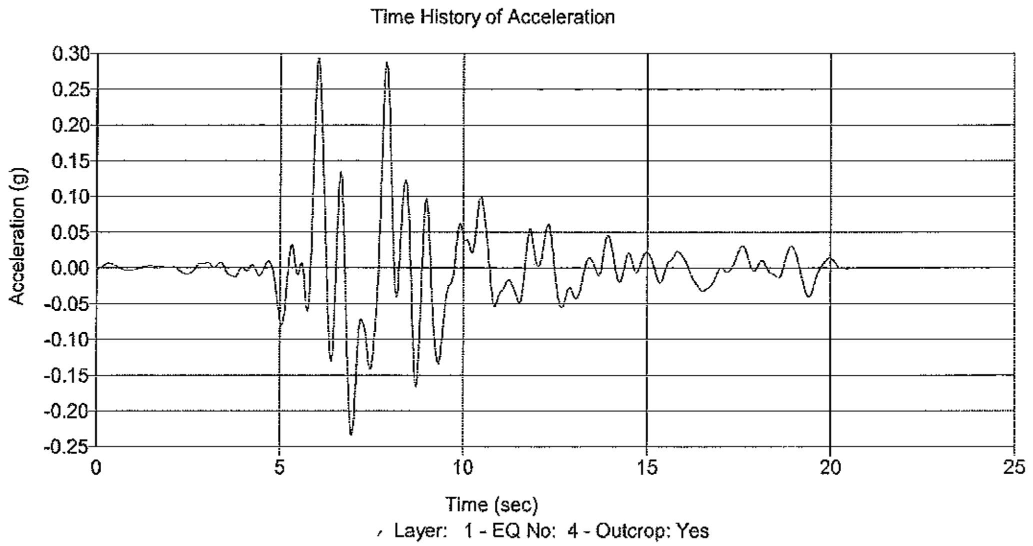

Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes

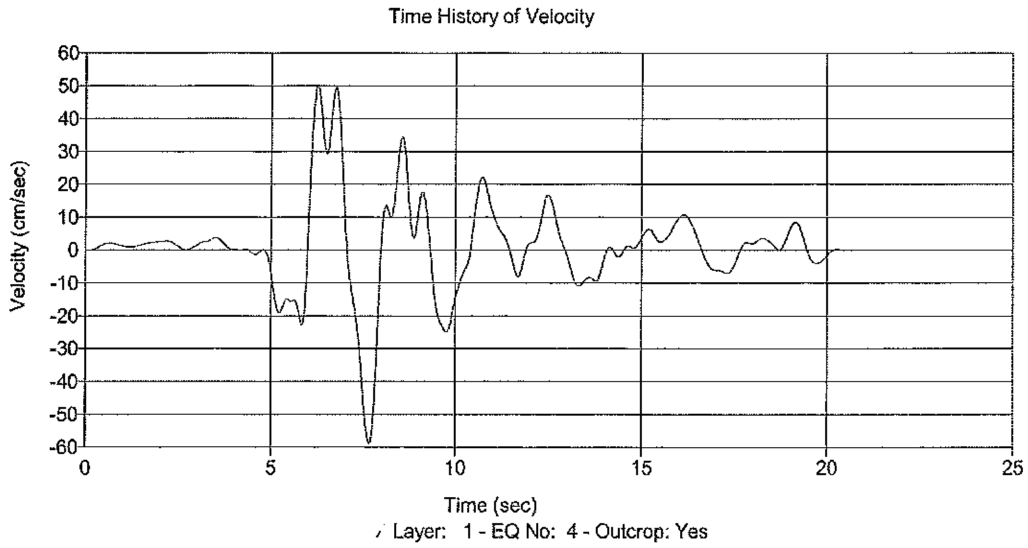

Time History of Displacement
Displacement (cm)
20
10
0
-10
-20
-30
-40
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes


Time History of Acceleration
Acceleration (g)
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 3 - Outcrop: Yes

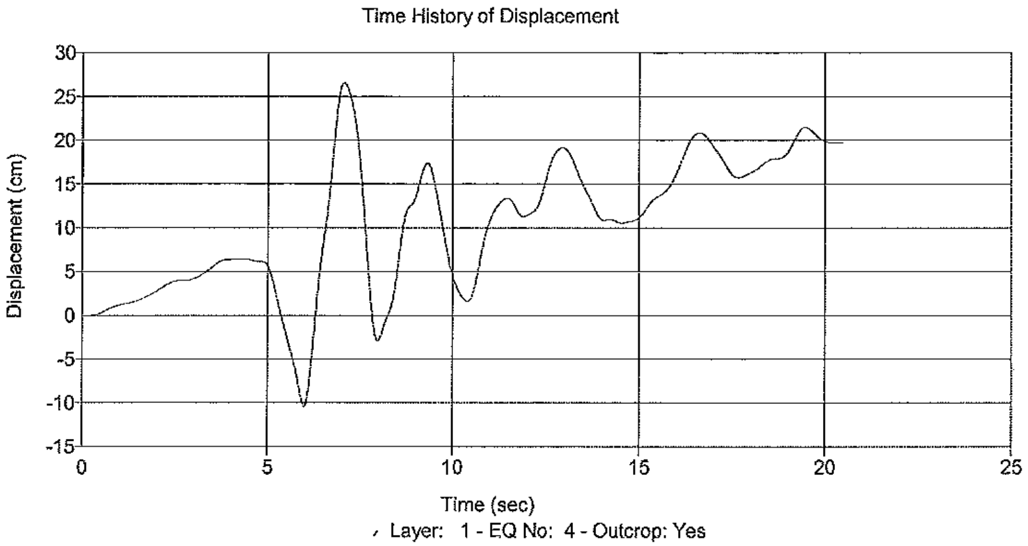

Time History of Velocity
Velocity (cm/sec)
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 3 - Outcrop: Yes


Time History of Displacement
10
0
-10
-20
-30
-40
Displacement (cm)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 3 - Outcrop: Yes

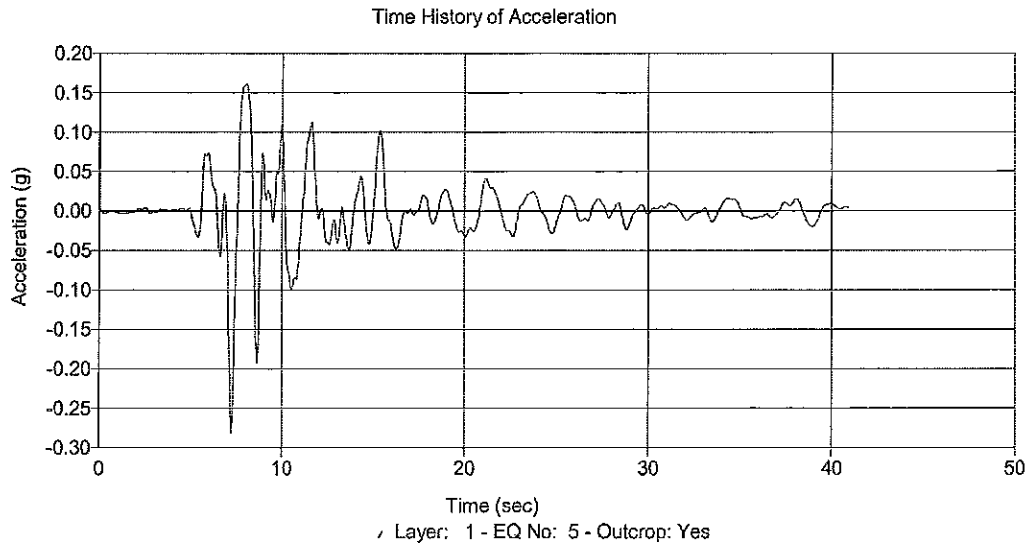

Time History of Acceleration
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
Acceleration (g)
0
5
10
15
20
25
Time (sec)
Layer: 1 - EQ No: 4 - Outcrop: Yes

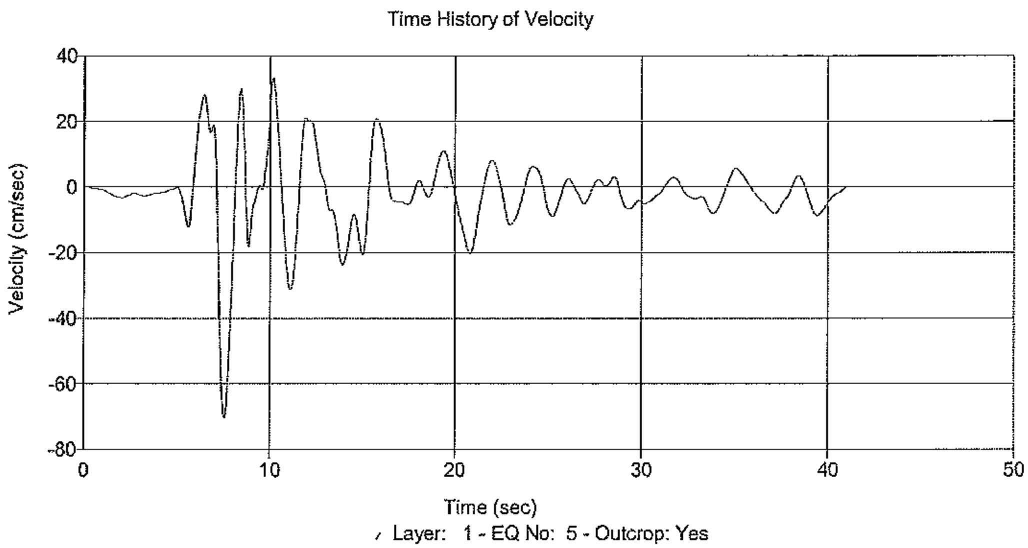


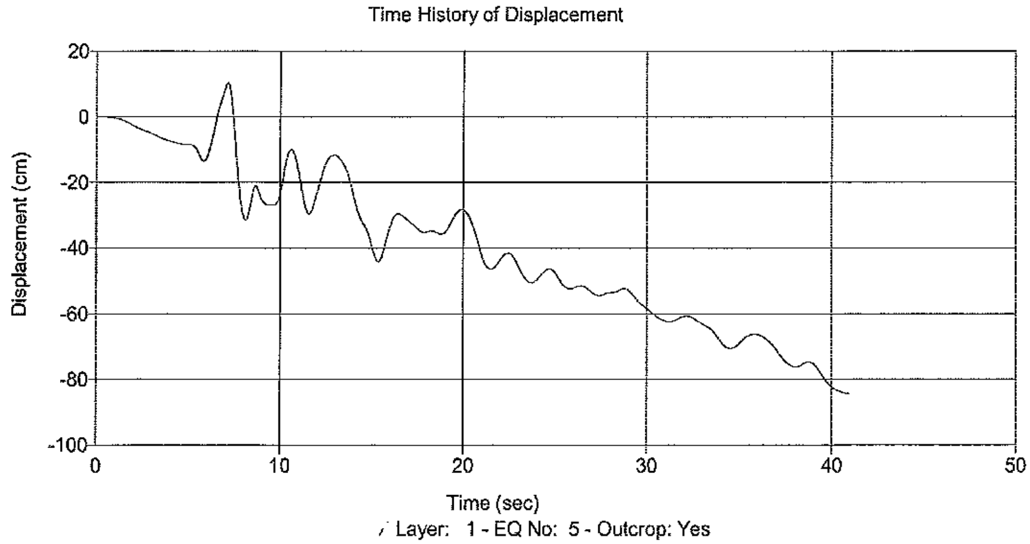
Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0
5
10
15
20
25
Time (sec)
Layer: 1 - EQ No: 4 - Outcrop: Yes

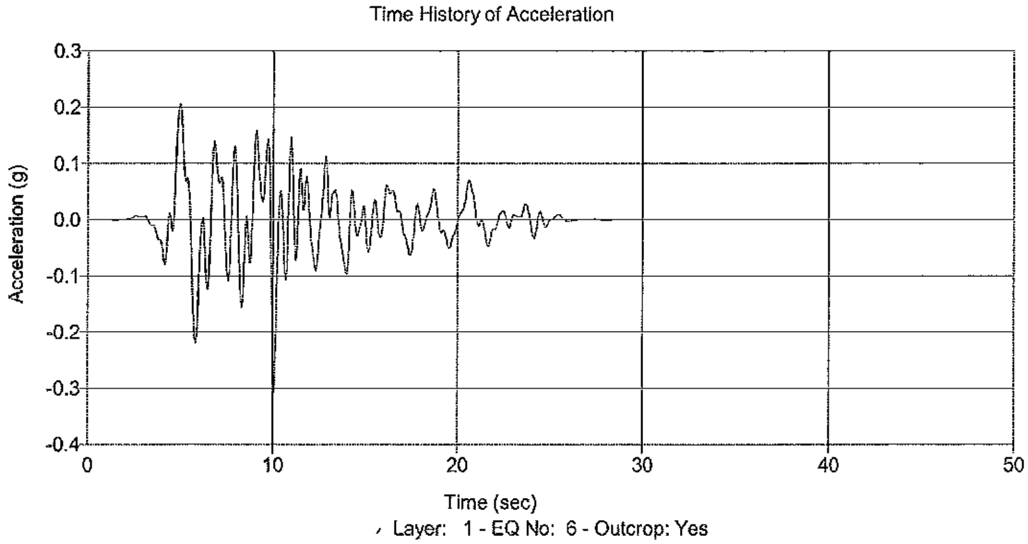


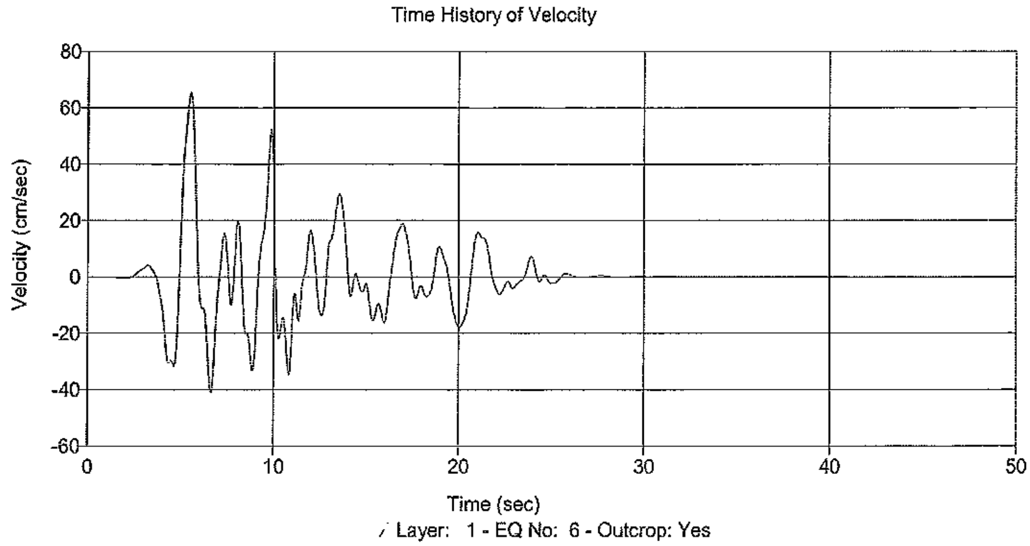
Time History of Displacement
Displacement (cm)
30
25
20
15
10
5
0
-5
-10
-15
0
5
10
15
20
25
Time (sec)
Layer: 1 - EQ No: 4 - Outcrop: Yes

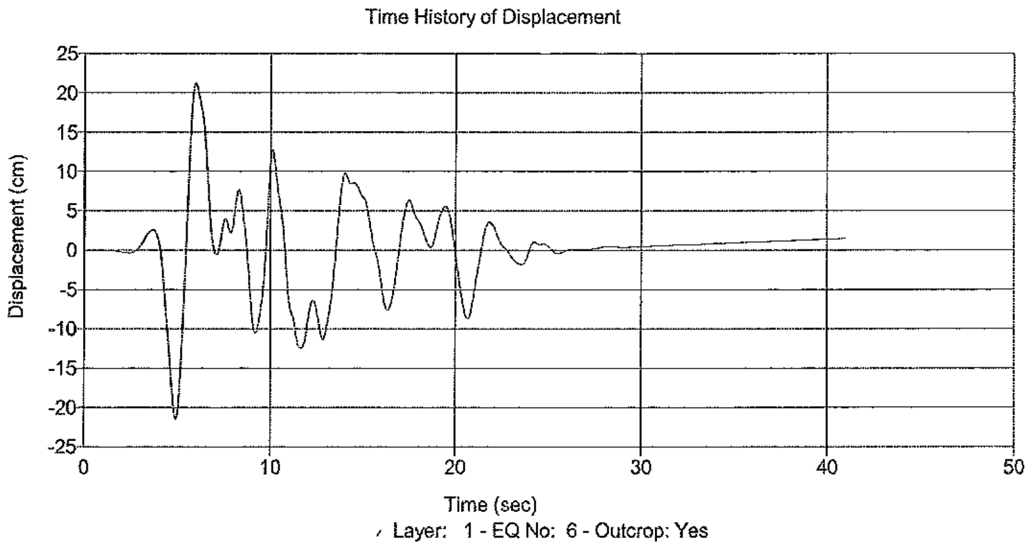

Time History of Acceleration
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
-0.30
Acceleration (g)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

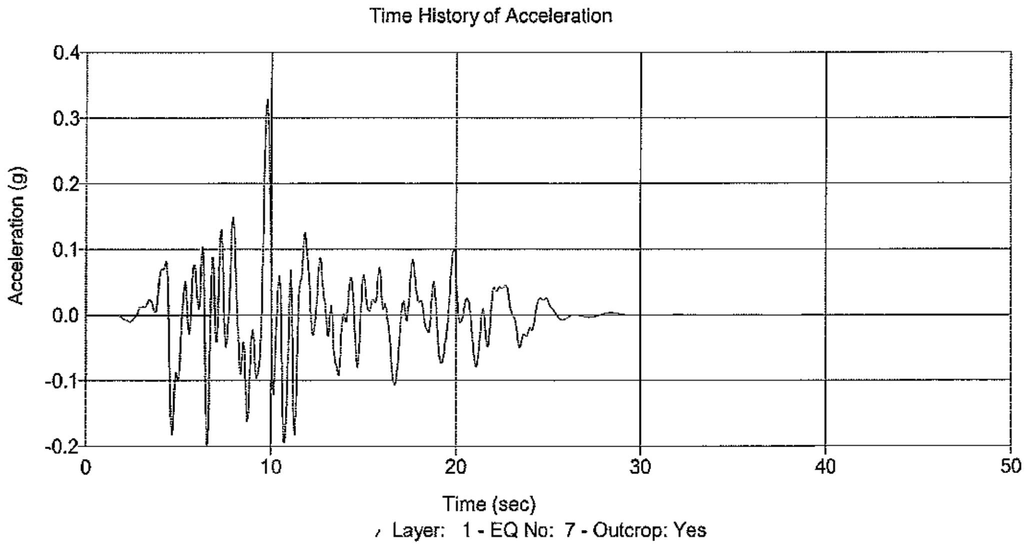

Time History of Velocity
40
20
0
-20
-40
-60
-80
Velocity (cm/sec)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

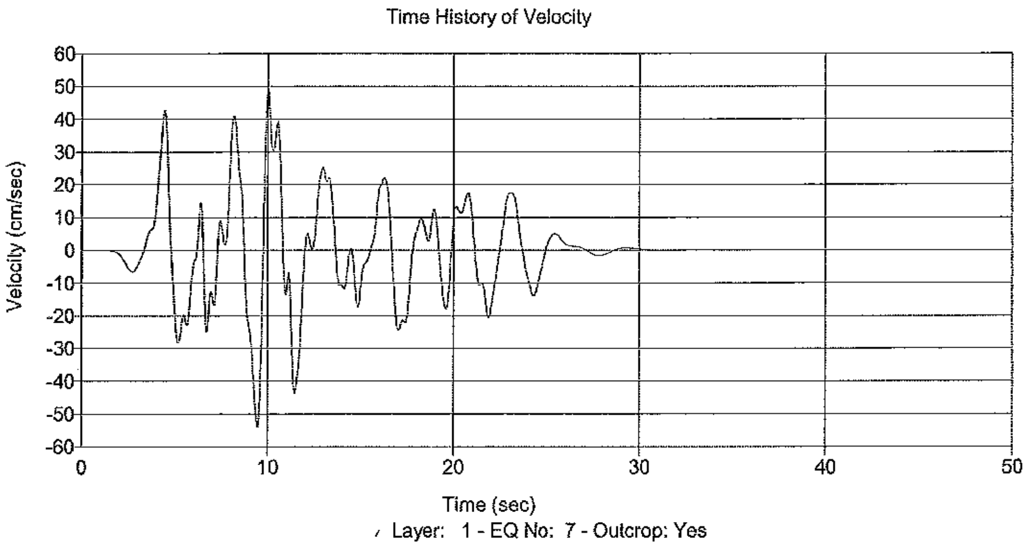

Time History of Displacement
Displacement (cm)
20
0
-20
-40
-60
-80
-100
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

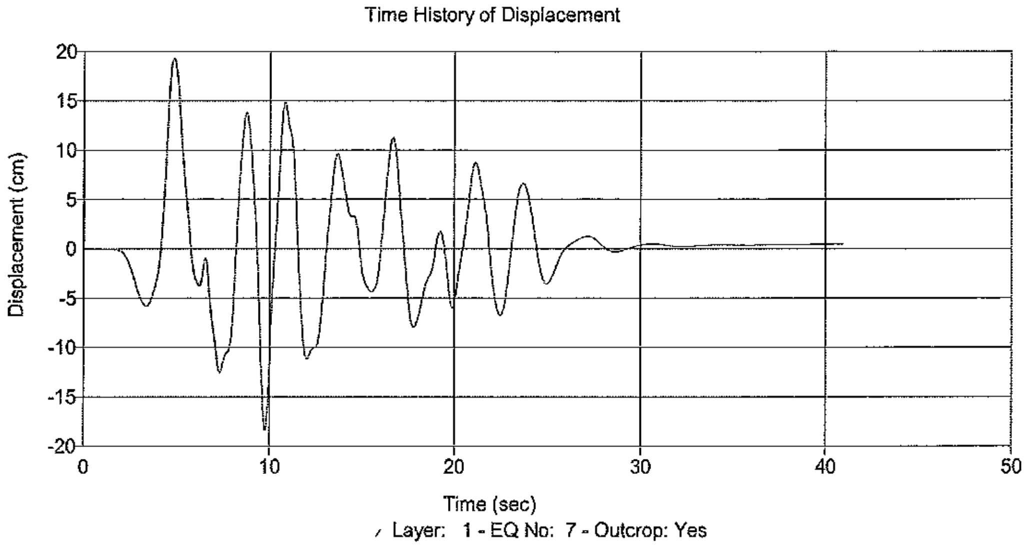

Time History of Acceleration
Acceleration (g)
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes

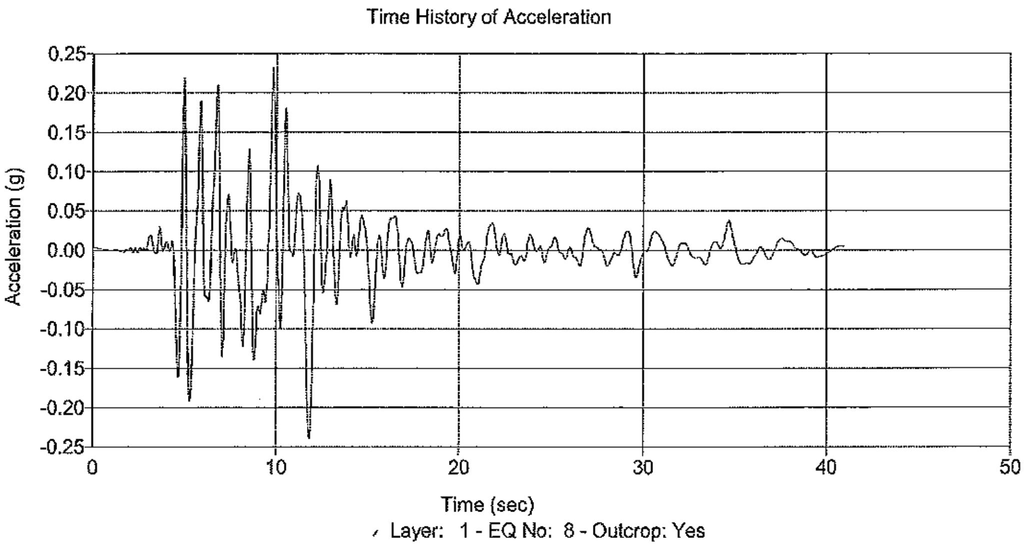

Time History of Velocity
80
60
40
20
0
-20
-40
-60
Velocity (cm/sec)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes

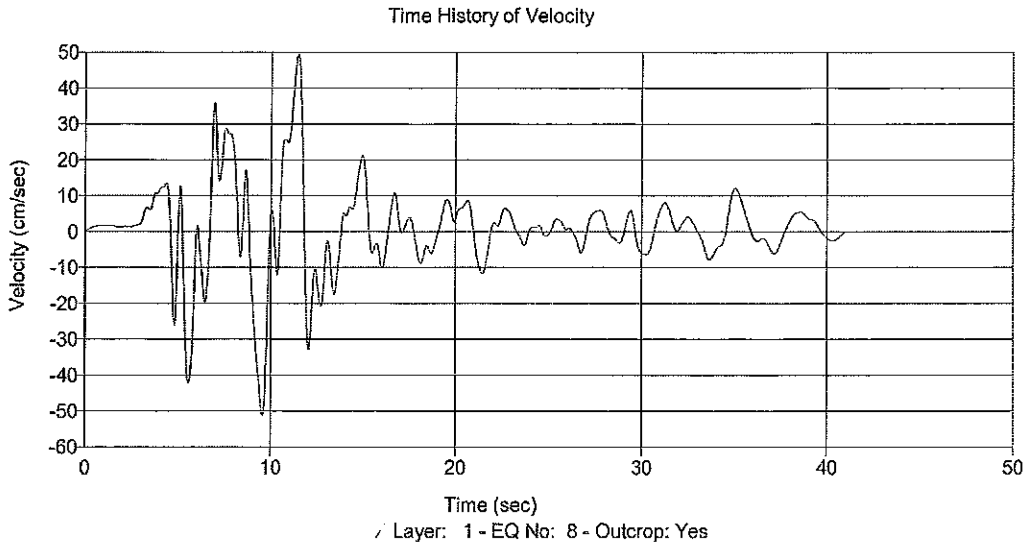

Time History of Displacement
25
20
15
10
5
0
-5
-10
-15
-20
-25
Displacement (cm)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes

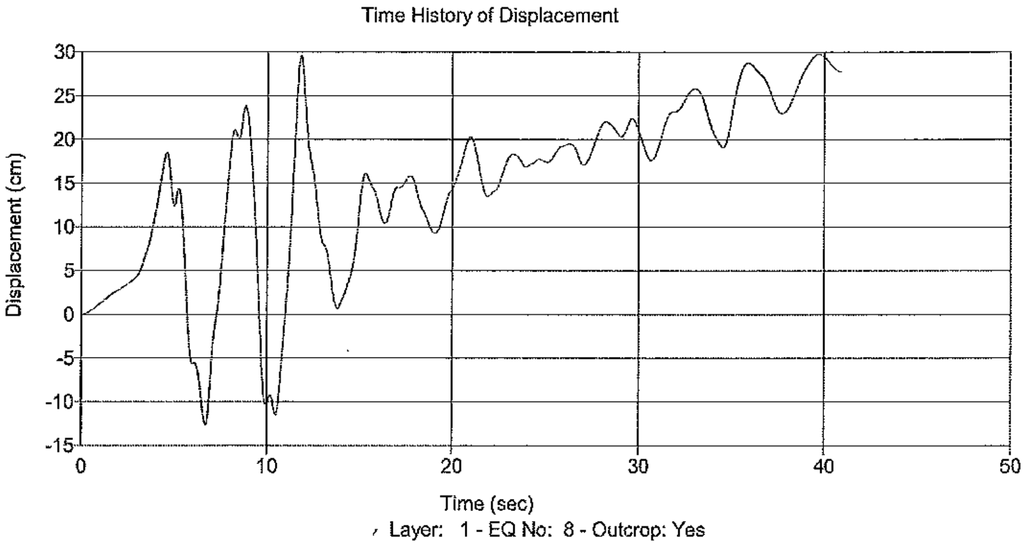

Time History of Acceleration
Acceleration (g)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 7 - Outcrop: Yes

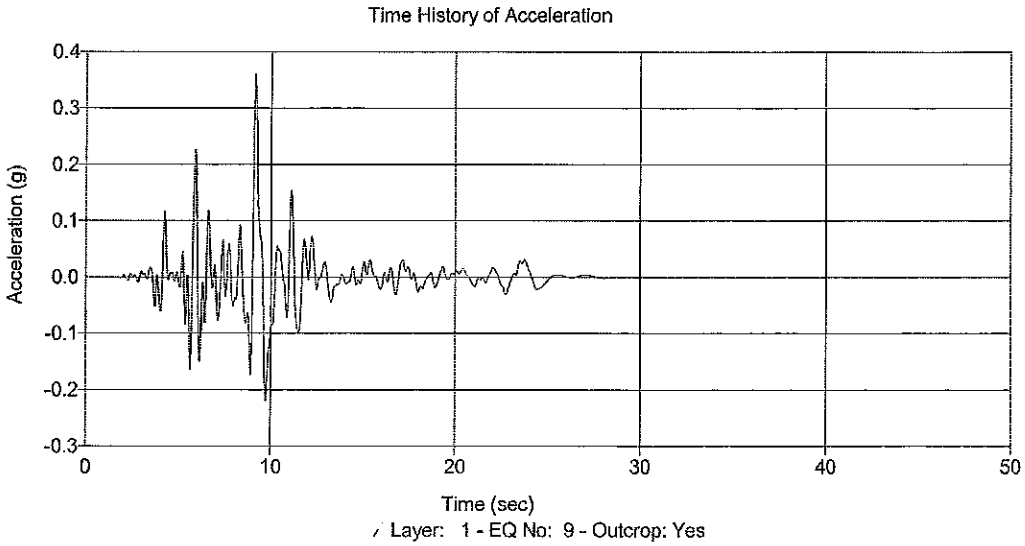

Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 7 - Outcrop: Yes

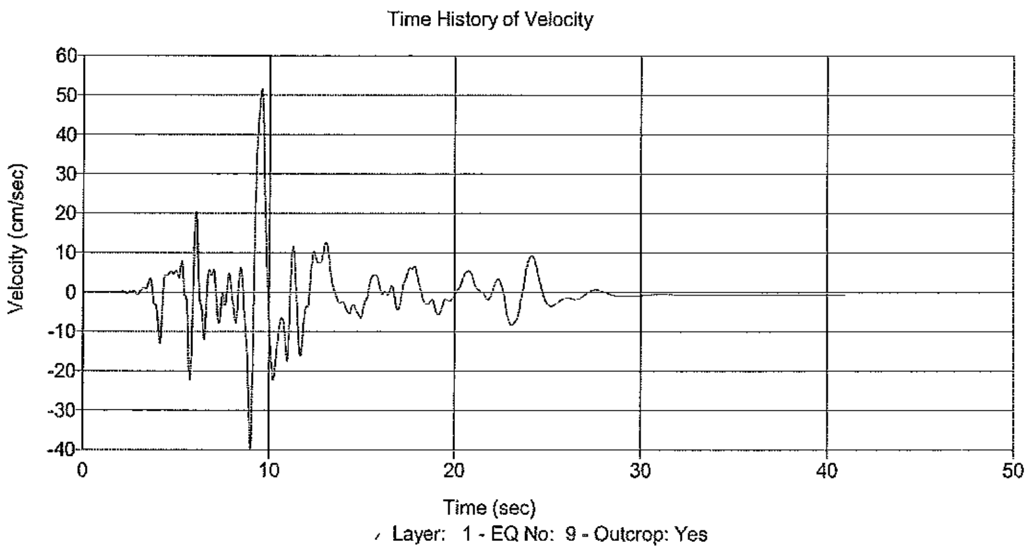

Time History of Displacement
Displacement (cm)
20
15
10
5
0
-5
-10
-15
-20
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 7 - Outcrop: Yes

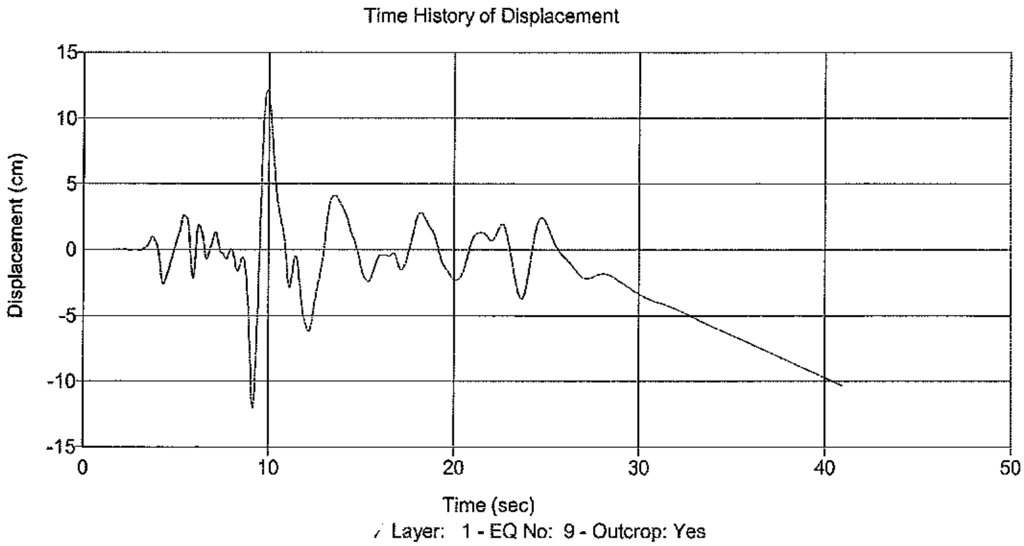

Time History of Acceleration
Acceleration (g)
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 8 - Outcrop: Yes

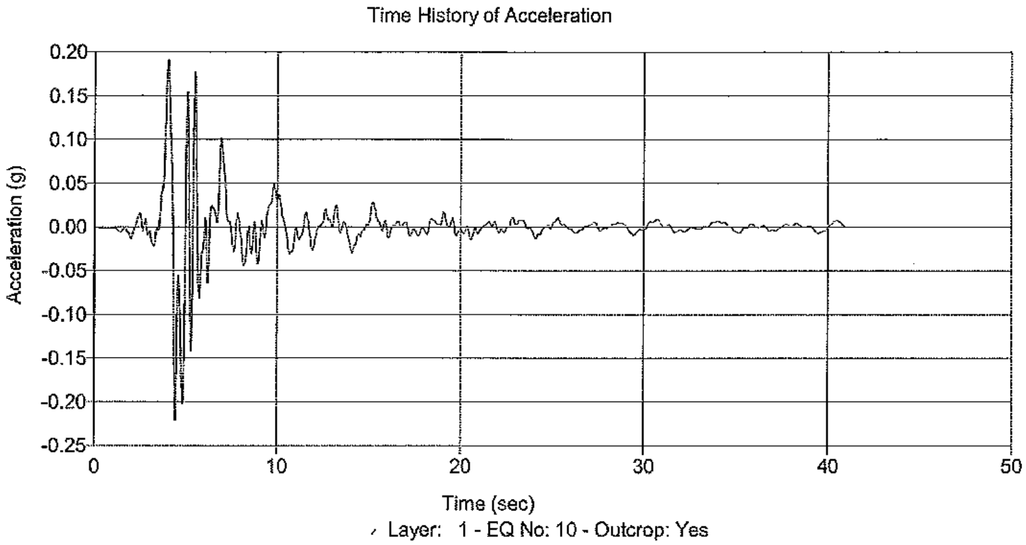

Time History of Velocity
Velocity (cm/sec)
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 8 - Outcrop: Yes


Time History of Displacement
Displacement (cm)
30
25
20
15
10
5
0
-5
-10
-15
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 8 - Outcrop: Yes

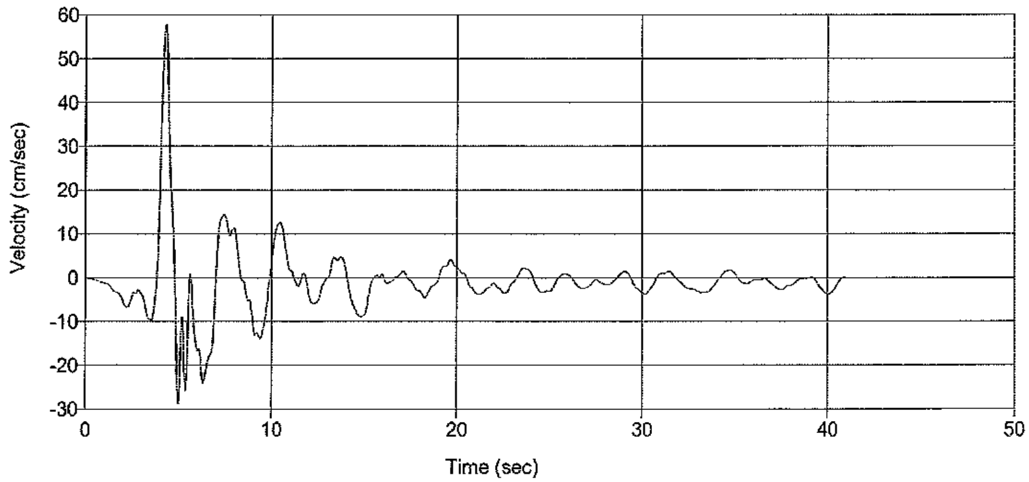

Time History of Acceleration
Acceleration (g)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 9 - Outcrop: Yes


Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 9 - Outcrop: Yes


Time History of Displacement
Displacement (cm)
15
10
5
0
-5
-10
-15
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 9 - Outcrop: Yes

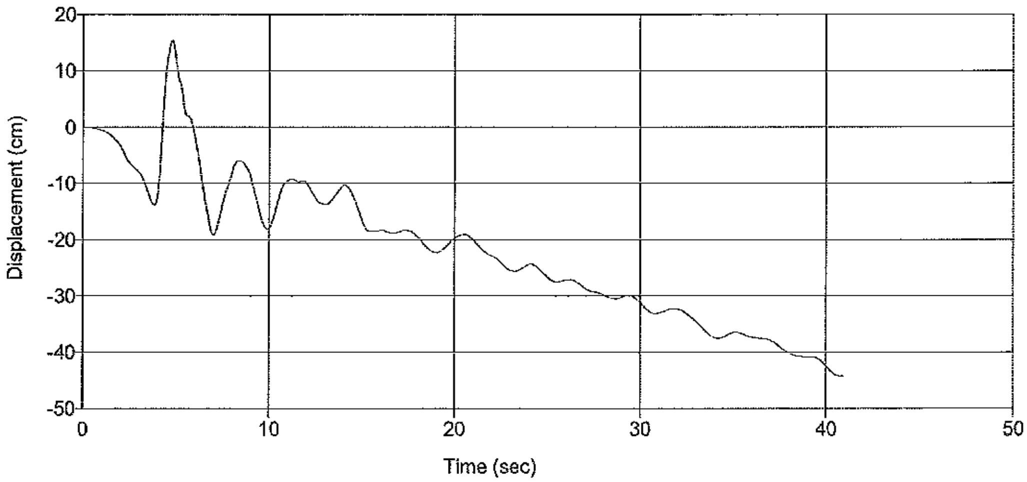

Time History of Acceleration
Acceleration (g)
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 10 - Outcrop: Yes

Time History of Velocity



Layer: 1 - EQ No: 10 - Outcrop: Yes

Time History of Displacement



Layer: 1 - EQ No: 10 - Outcrop: Yes

(2) MEG Consulting Ltd. Report (DESRA Analysis)

(a) E-mail dated March 21, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: gwt@tbgsc.bc.ca
Sent: Wednesday, March 21, 2007 4:02 PM
Subject: Site Response Analysis Results for Brighouse School Using DESRA-2C

Graham,

Using the same soil layering model and shear wave velocity/soil density distribution as used by Trow for their SHAKE analysis for the soil profile at Brighouse Elementary (with firm ground considered to be at the 300m depth) and the same suite (10) of firm ground input motions, I have run the nonlinear site response program DESRA-2C to compute near surface (1.0m depth) ground motions. The shear strength profile used in the analysis is given in the attached Excel spreadsheet, along with other soil layer properties.

I have also attached computed cyclic stress ratios and peak ground accelerations versus depth in the Excel file "CSR & Amax", as well as computed peak spectral accelerations and velocities versus structural period in the Excel file "spectra". Finally, the computed acceleration time histories at the 1.0 m depth are attached.

I have used drained strengths in the near surface silt/sand fill, undrained strengths in the underlying near surface silts (including a strain rate factor of 1.1) judged to be representative of cyclic simple shear strengths and estimated based on available cpt data, drained shear strengths on the horizontal plane in the underlying sands based on estimated friction angles and K0 values from the cpt data, and dynamic undrained strengths (including a strain rate factor of 1.1) in the deeper interbedded clay/silt/sand deposits (considered to be representative of approximately normally consolidated simple shear strengths at larger depths; over-consolidated undrained strengths near the surface of the silt based on the available cpt data).

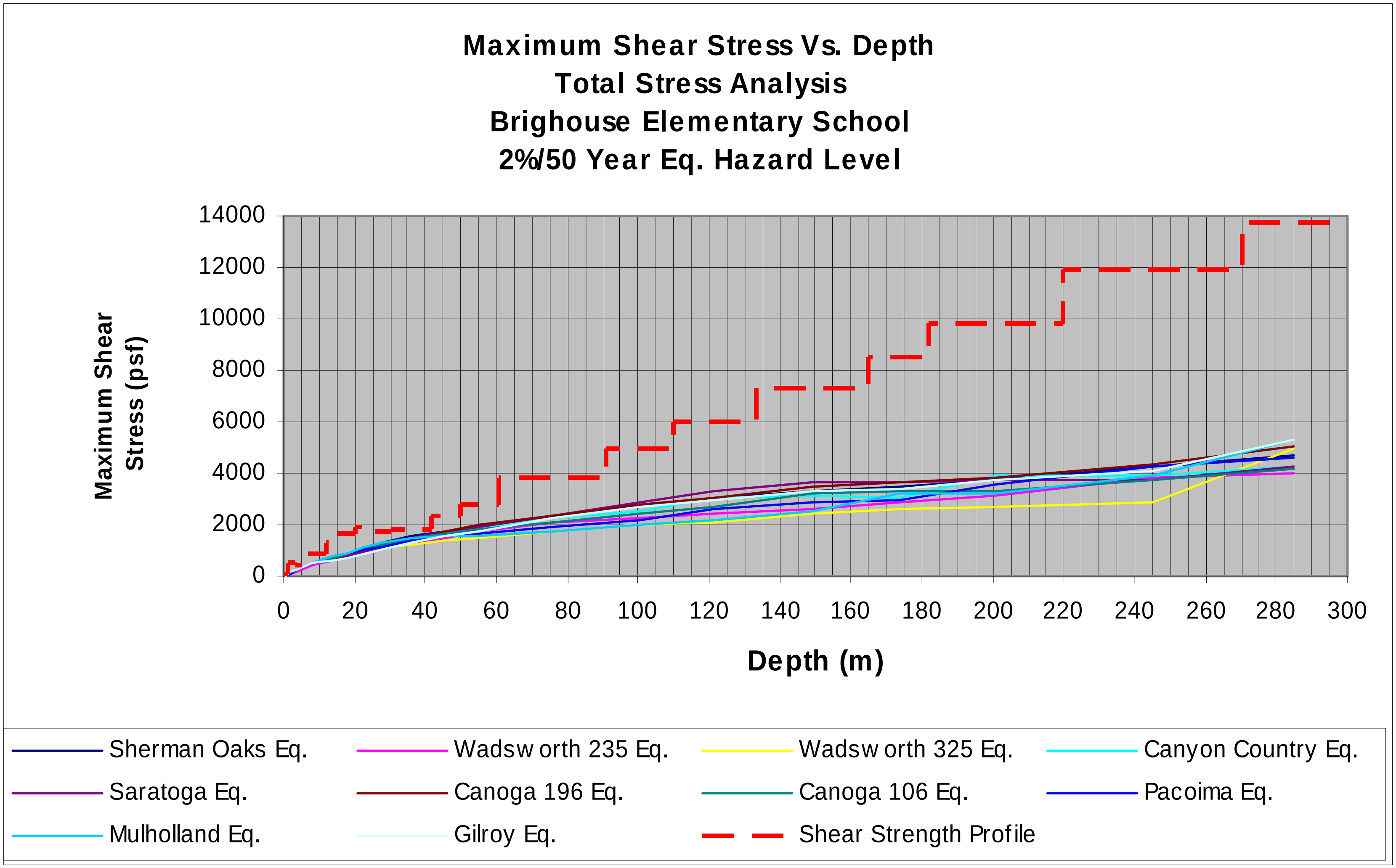
I have passed my strength estimates on to Mark Qian at Trow and would appreciate any comments he may have.

Regards,

Blair Gohl, Ph.D., P.Eng.
Principal
MEG Consulting Ltd.

(2) MEG Consulting Ltd. Report (DESRA Analysis)

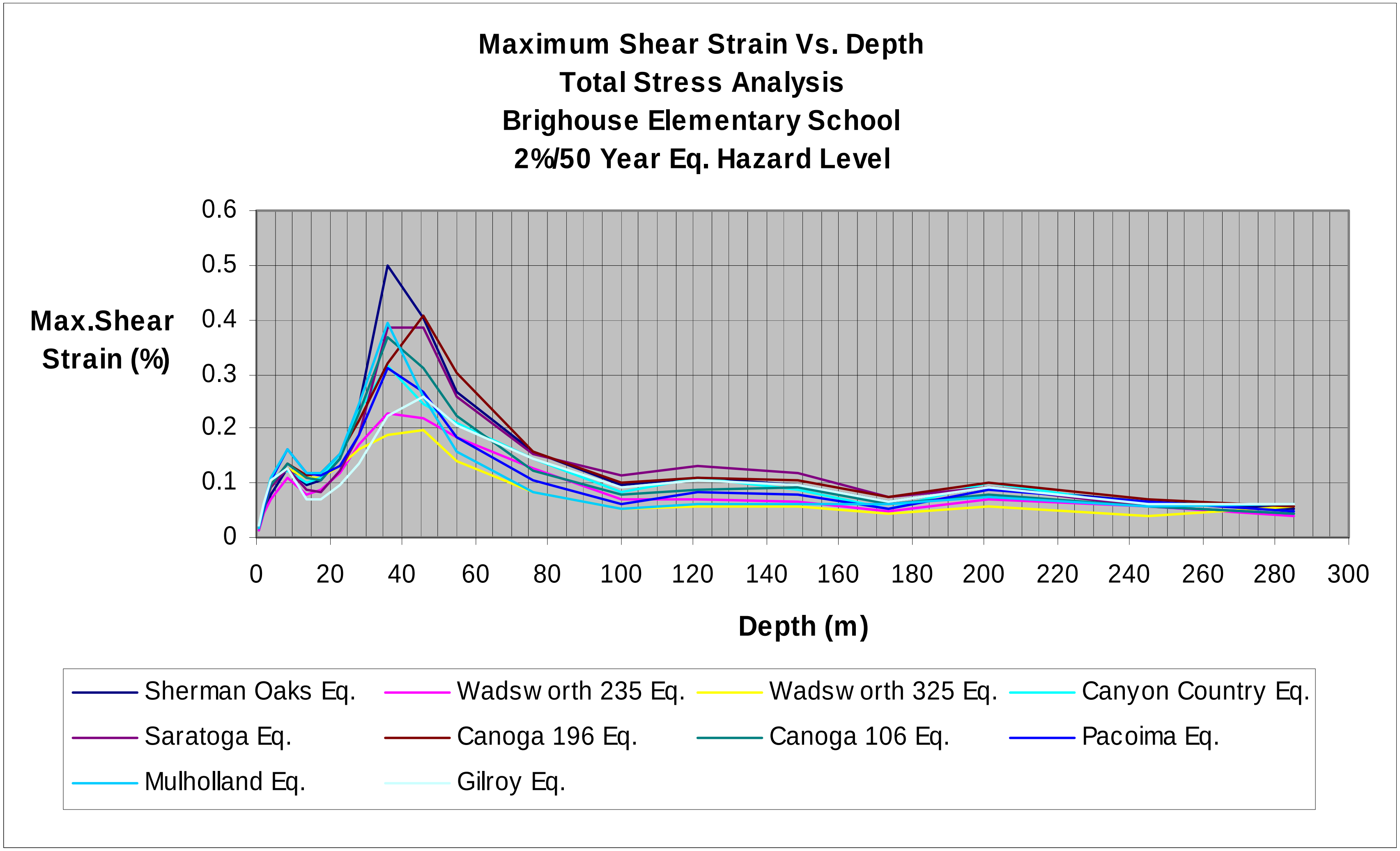
(b) Graph - maximum shear stress versus depth



Maximum Shear Stress Vs. Depth
Total Stress Analysis
Brighouse Elementary School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

(2) MEG Consulting Ltd. Report (DESRA Analysis)

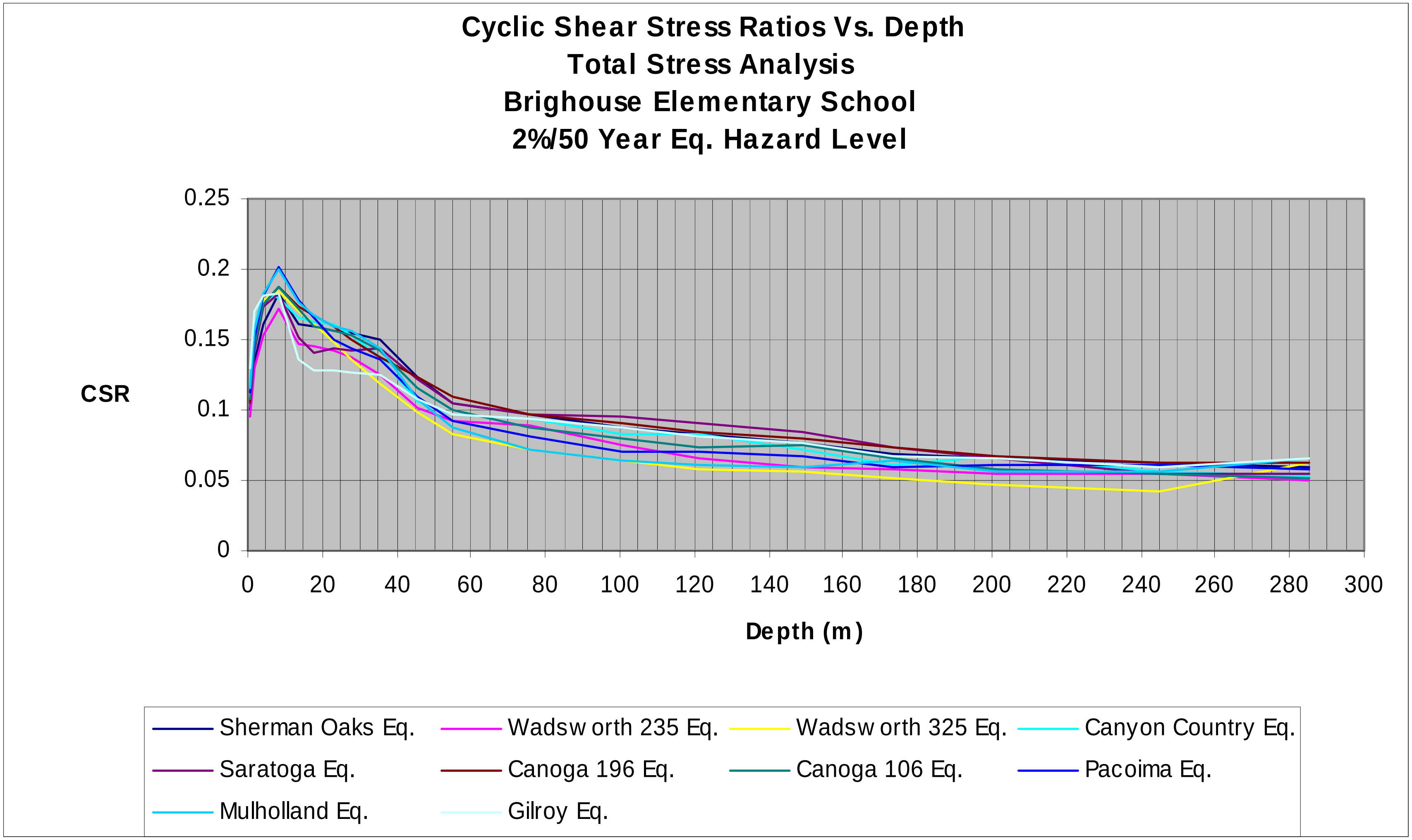
(c) Graph - maximum shear strain versus depth



Maximum Shear Strain Vs. Depth
Total Stress Analysis
Brighouse Elementary School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

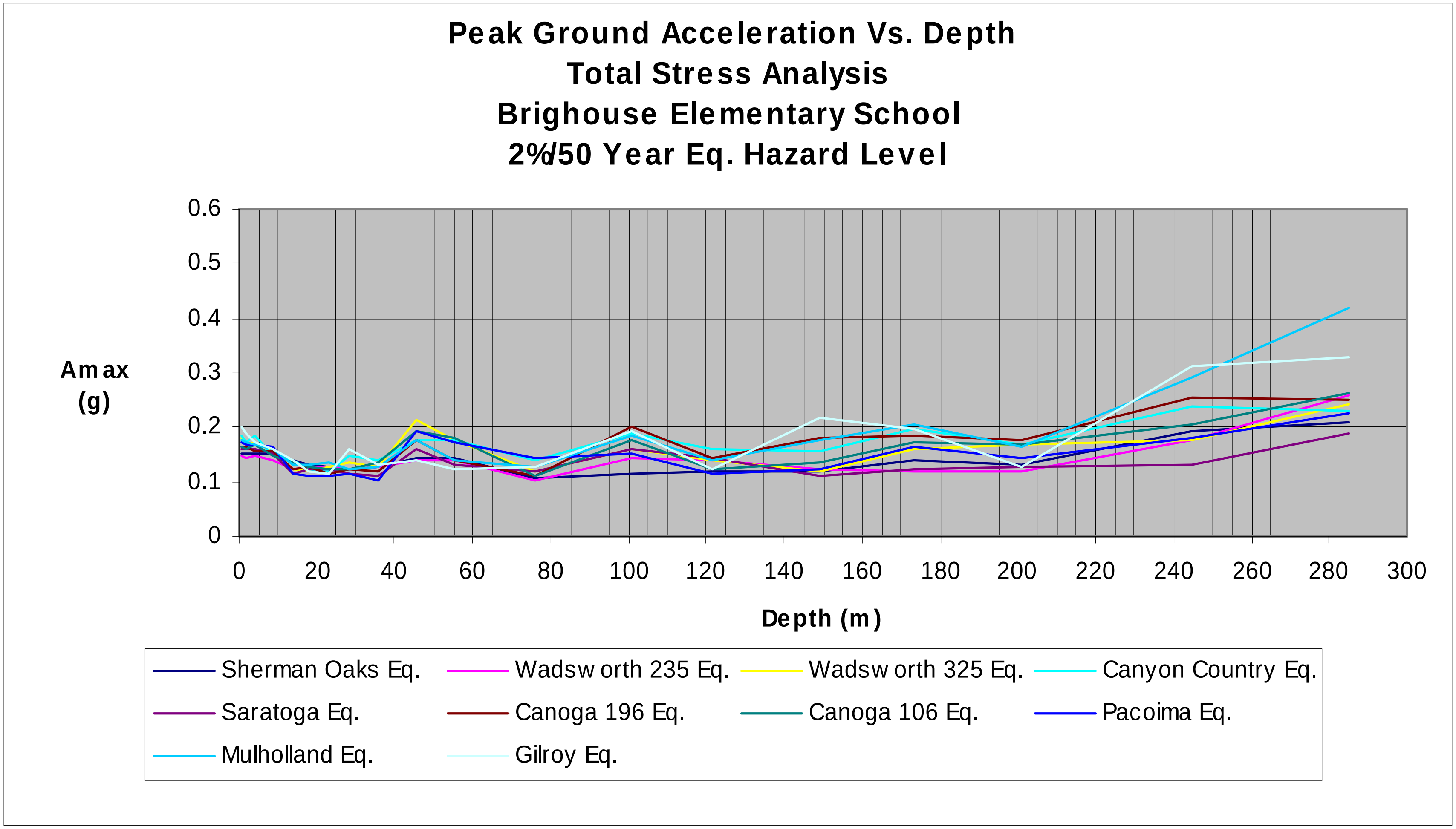
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Brighouse Elementary School
2%/50 Year Eq. Hazard Level
CSR
0.25
0.2
0.15
0.1
0.05
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

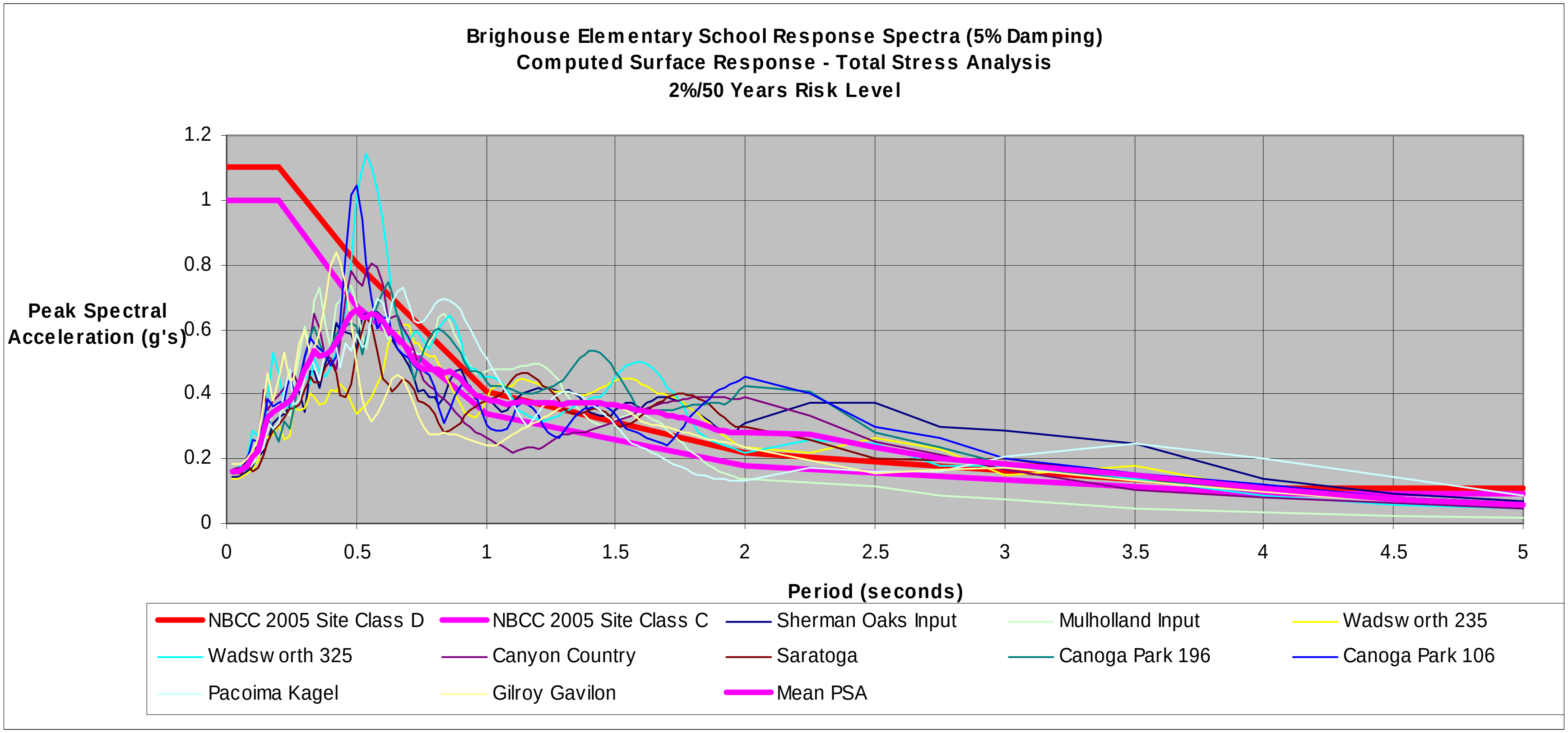
(2) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Brighouse Elementary School
2%/50 Year Eq. Hazard Level
Amax (g)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

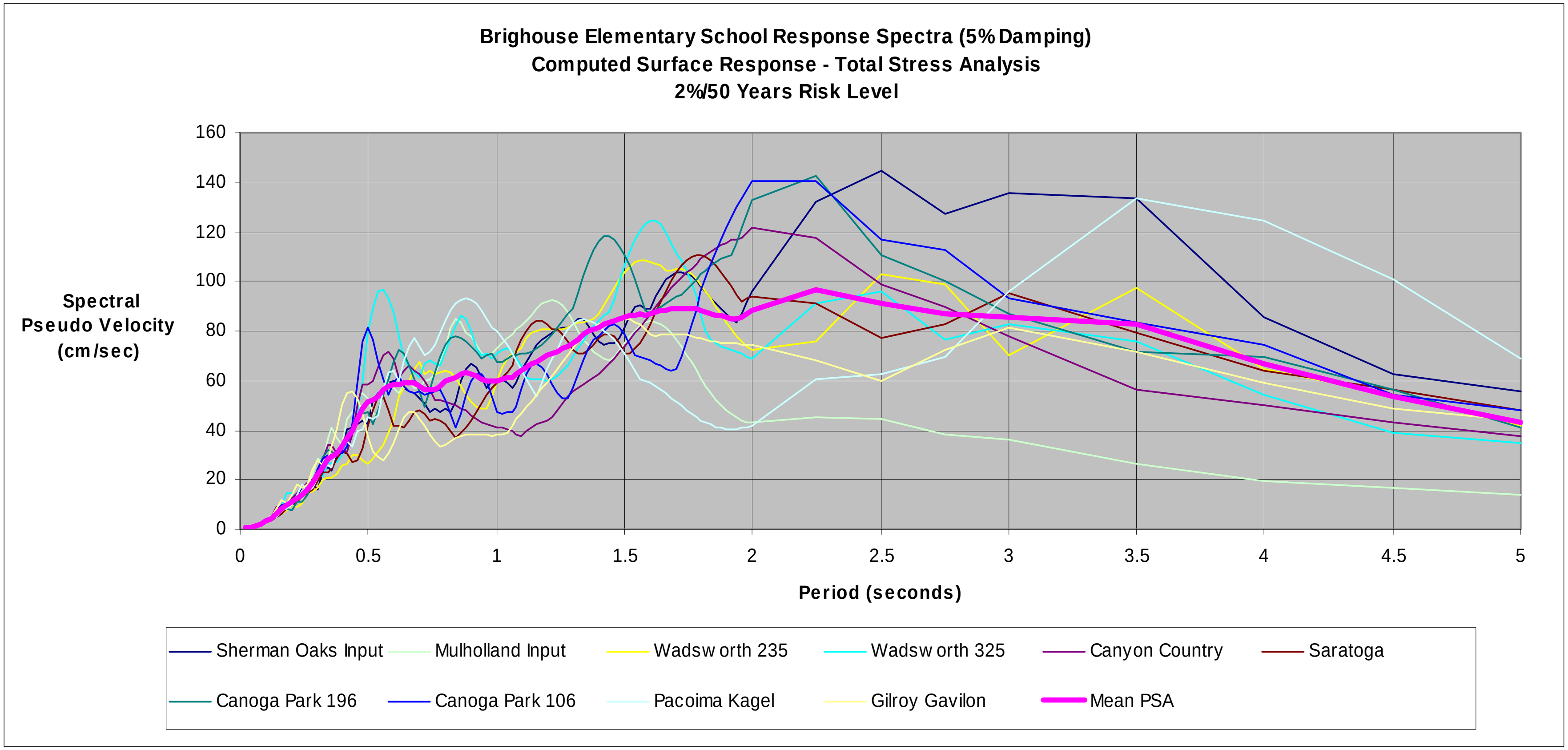
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(f) Graph - surface acceleration response spectra


Brighouse Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSA

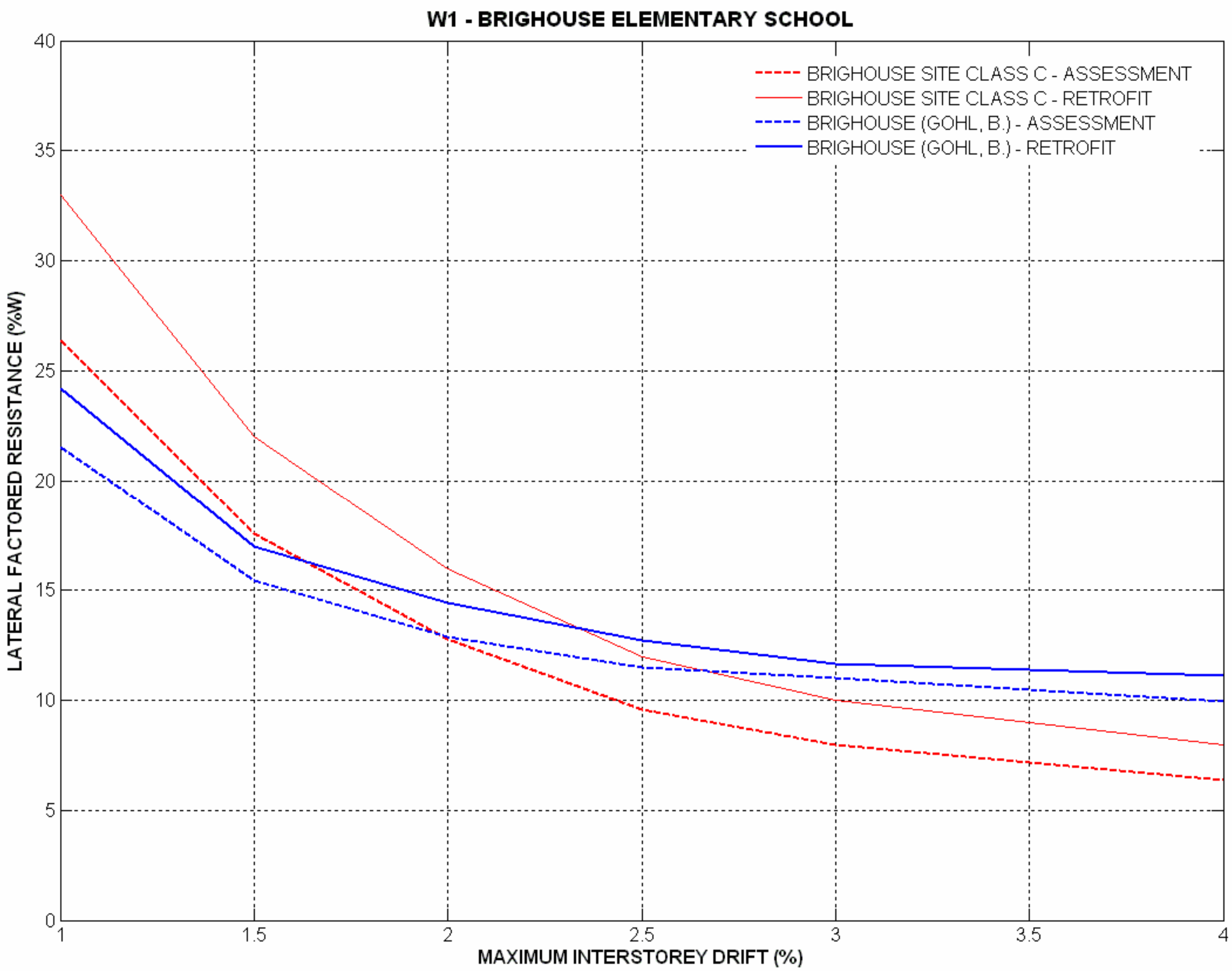
(2) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(g) Graph - surface velocity response spectra


Brighouse Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral
Pseudo Velocity
(cm/sec)
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSA

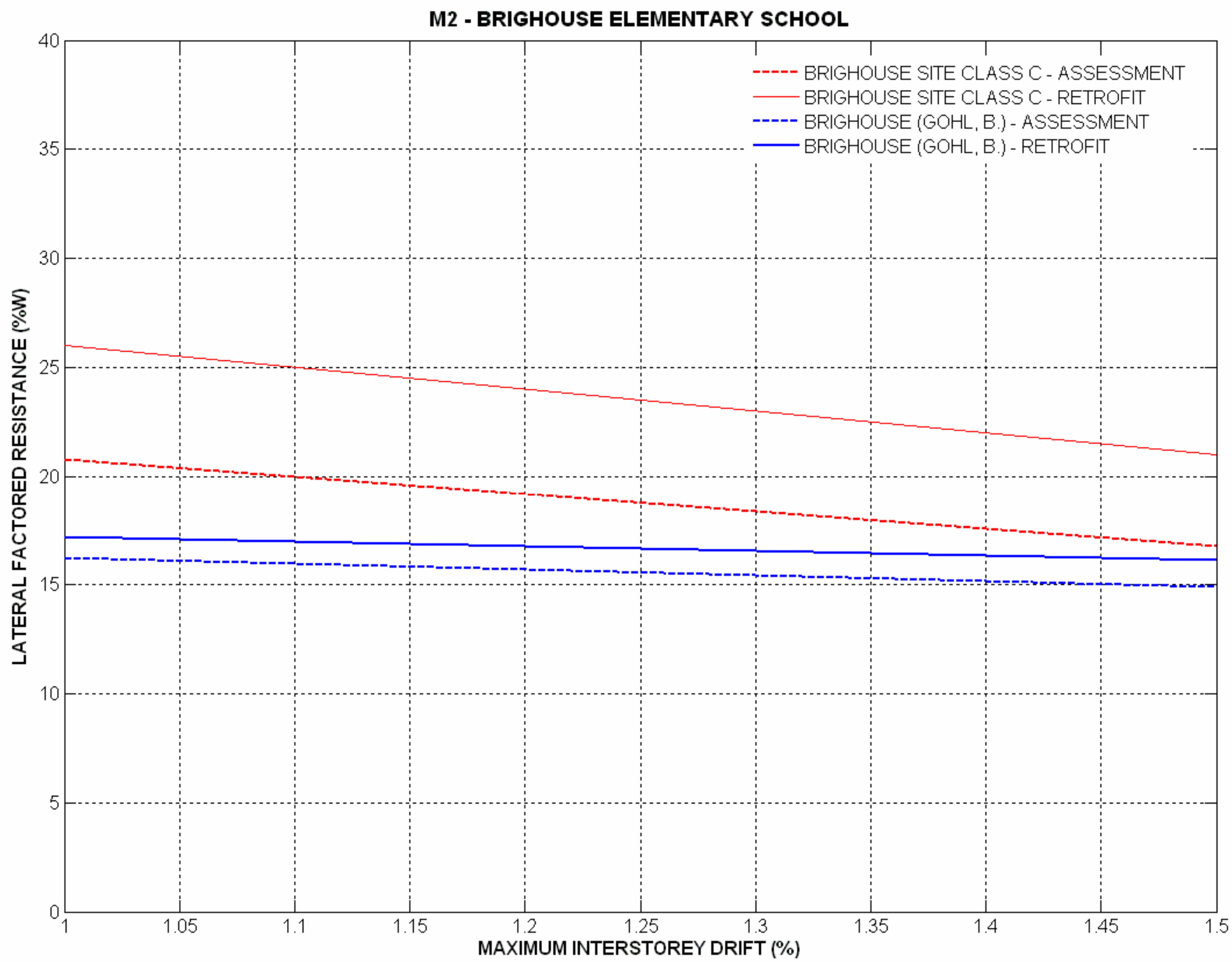
(3) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1


W1 - BRIGHOUSE ELEMENTARY SCHOOL
BRIGHOUSE SITE CLASS C - ASSESSMENT
BRIGHOUSE SITE CLASS C - RETROFIT
BRIGHOUSE (GOHL, B.) - ASSESSMENT
BRIGHOUSE (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.5
2
2.5
3
3.5
4

(3) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for M-2



M2 - BRIGHOUSE ELEMENTARY SCHOOL
BRIGHOUSE SITE CLASS C - ASSESSMENT
BRIGHOUSE SITE CLASS C - RETROFIT
BRIGHOUSE (GOHL, B.) - ASSESSMENT
BRIGHOUSE (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX C

DETAILED RESULTS FOR GARDEN CITY ELEMENTARY SCHOOL

CONTENTS

(1) <u>Trow Associates Ltd. Report (SHAKE Analysis)</u>

(a) ProShake report

ProShake Report

Data File: C:\PROSHAKE\RMDSCHL\GARDEN~1\GC-5.DAT

Soil Profile

Profile Name: Seismic Assessment of Garden City Elementary School, Richmond, BC
Water Table: 1.00 m
Number of Layers: 34

Error! Bookmark not defined.

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve	Mod. Parameter	Damp. Parameter
1	SILT	1.00	18.00	8.99	70.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
2	SILTY SAND/SANDY SILT	1.00	18.00	8.99	70.00	Vucetic - Dobry	Vucetic - Dobry	5.00	5.00
3	SILTY SAND/SANDY SILT	1.50	18.00	18.35	100.00	Vucetic - Dobry	Vucetic - Dobry	5.00	5.00
4	SILTY SAND/SANDY SILT	2.20	18.50	27.16	120.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
5	SILTY SAND	2.30	18.50	61.12	180.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
6	SAND	3.00	19.00	62.77	180.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
7	SAND	4.00	19.00	85.44	210.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
8	SAND	2.00	19.00	111.60	240.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
9	SANDY SILT	1.20	18.50	99.79	230.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
10	SAND	4.20	19.00	93.77	220.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
11	SANDY SILT	5.10	18.50	91.30	220.00	Vucetic - Dobry	Vucetic - Dobry	10.00	10.00
12	SILT	2.50	18.00	80.94	210.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
13	SILT	4.50	18.00	80.94	210.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
14	SAND	7.00	19.00	99.83	227.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
15	SILT	8.50	18.00	87.23	218.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
16	SAND	10.50	19.00	111.60	240.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
17	SILT	8.50	18.00	154.36	290.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
18	SILT	9.00	18.00	133.81	270.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
19	SILT	13.00	18.00	182.12	315.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
20	SAND	12.00	19.00	302.29	395.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
21	SILT	7.00	18.00	286.38	395.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
22	SILT	11.00	18.00	224.84	350.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
23	SILT	12.00	18.00	301.06	405.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
24	SILT	13.00	18.00	355.35	440.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
25	SILT	19.00	18.00	265.04	380.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
26	SILT	11.00	18.00	414.13	475.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
27	SAND	6.00	19.00	409.96	460.00	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Lower Bound		
28	SILT	5.50	18.00	388.38	460.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
29	SILT	32.50	18.00	301.06	405.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
30	SILT	15.50	18.00	458.87	500.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
31	SILT	34.50	18.00	496.31	520.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
32	SILT	12.50	18.00	585.93	565.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
33	SILT	17.50	18.00	696.48	616.00	Vucetic - Dobry	Vucetic - Dobry	30.00	30.00
34	TILL	0.00	22.00	1,295.75	760.00	Linear	Linear		1.00

Input Motion

Number of Motions: 10
Numeber of Iterations: 5
Strain Ratio: 0.65
Tolerance: 5.00%

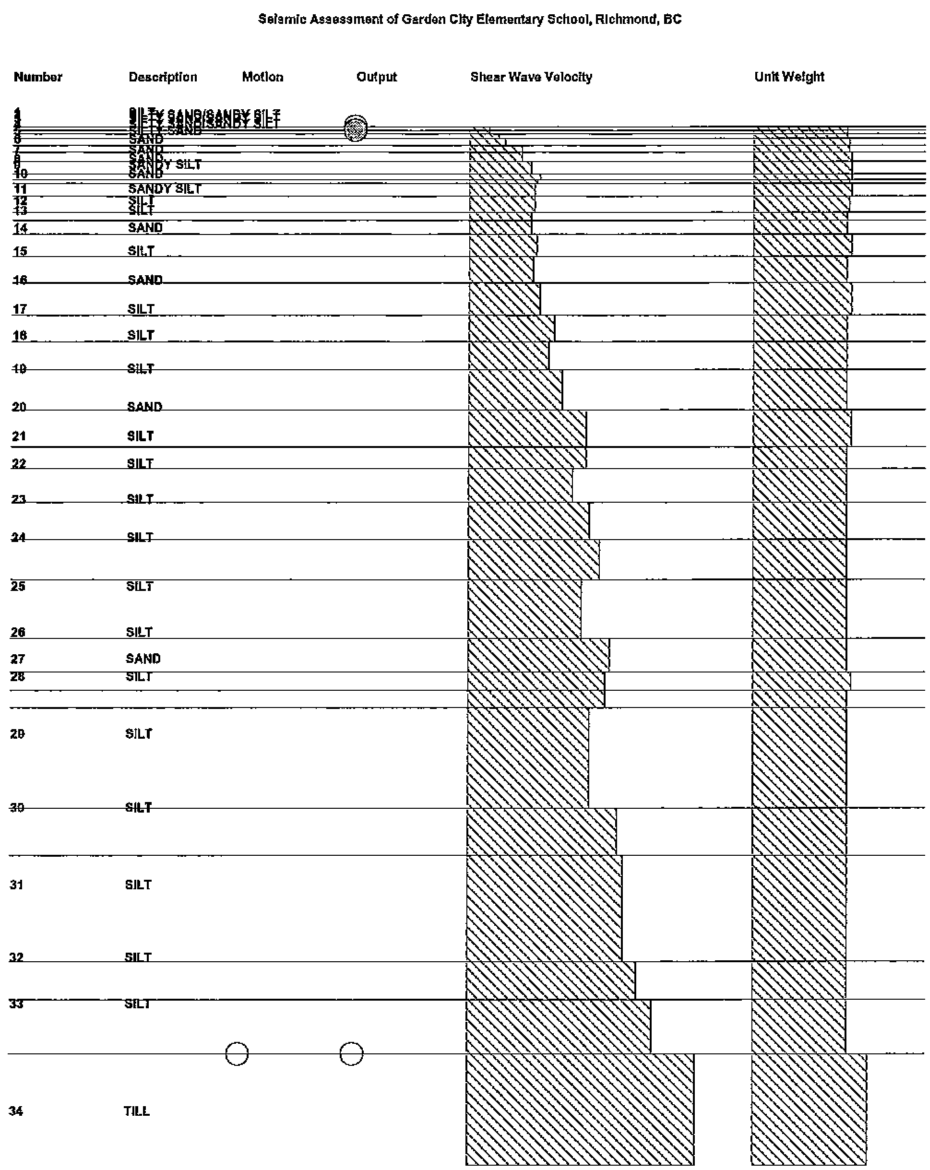
Error! Bookmark not defined.

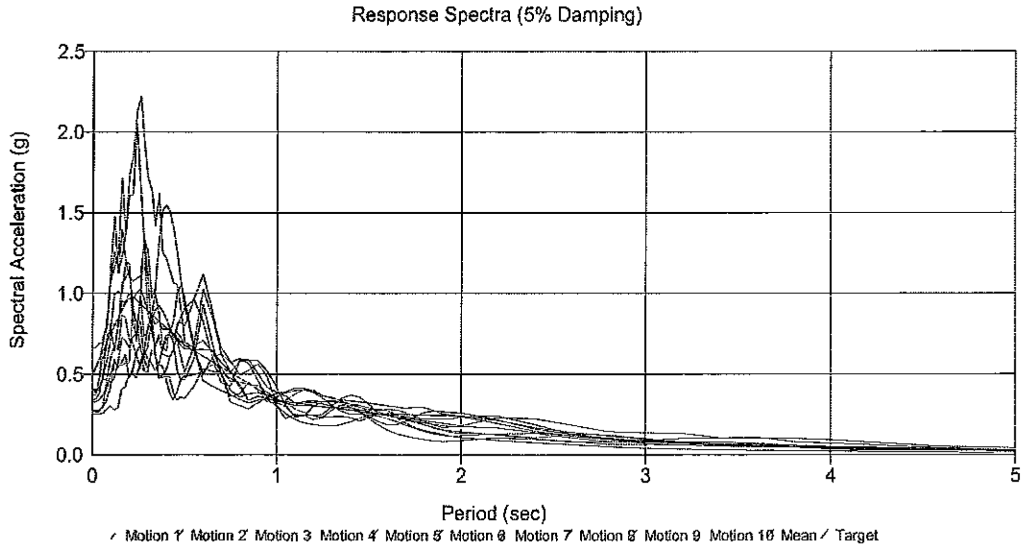
File Name	No of Acc. Values	Max. Acc. (g)	Time Step (sec)	Cuttoff Freq. (Hz)	No of Fourier Terms	Layer	Outcrop
C:\PROSHAKE\RMDSCHL\SHAKER~1\SO90.EQ	3000	0.268	0.020	25.00	4096	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\WW235.EQ	5000	0.345	0.005	50.00	8192	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\WW325.EQ	5716	0.514	0.005	50.00	8192	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CC0.EQ	1999	0.329	0.010	50.00	2048	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\SARA0.EQ	2000	0.404	0.020	25.00	2048	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CP196.EQ	2499	0.276	0.010	50.00	4096	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\CP106.EQ	2499	0.371	0.010	50.00	4096	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\PK90.EQ	2000	0.253	0.020	25.00	2048	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\MD35.EQ	2398	0.665	0.010	50.00	4096	34	Yes
C:\PROSHAKE\RMDSCHL\SHAKER~1\GIL67.EQ	2000	0.498	0.020	25.00	2048	34	Yes

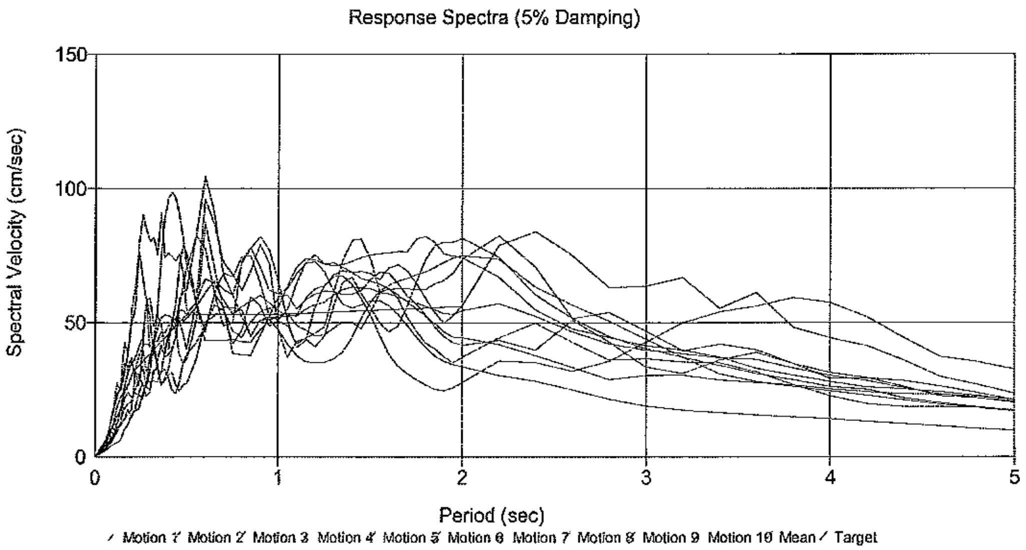
Output Locations

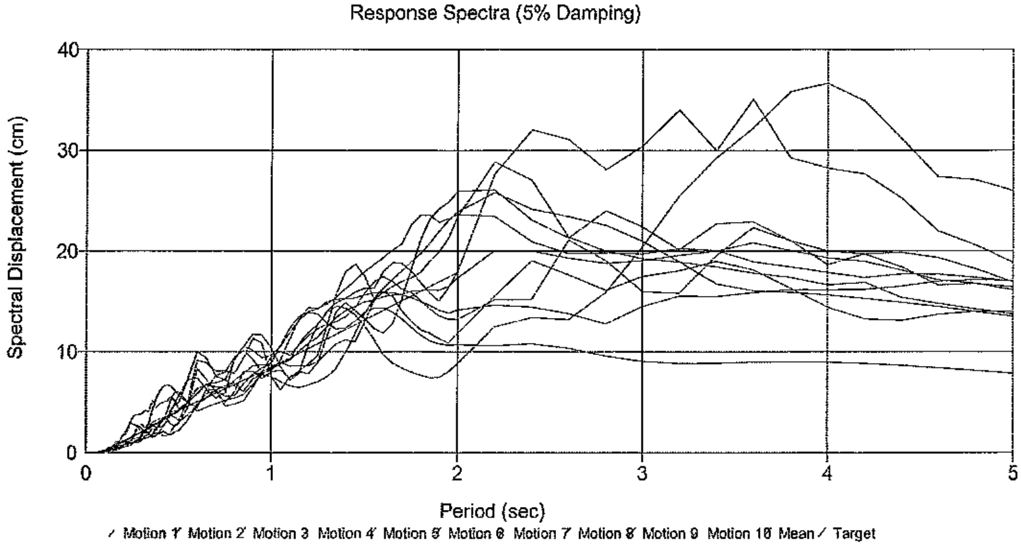
Error! Bookmark not defined.

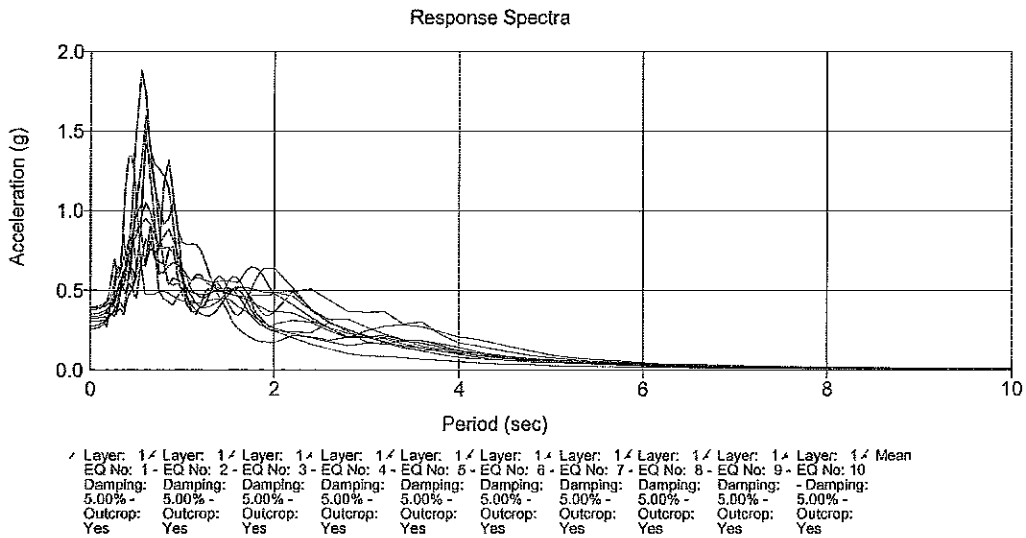
Layer No	Depth (m)	Outcrop
1	0.00	Yes
2	1.00	No
34	300.00	Yes

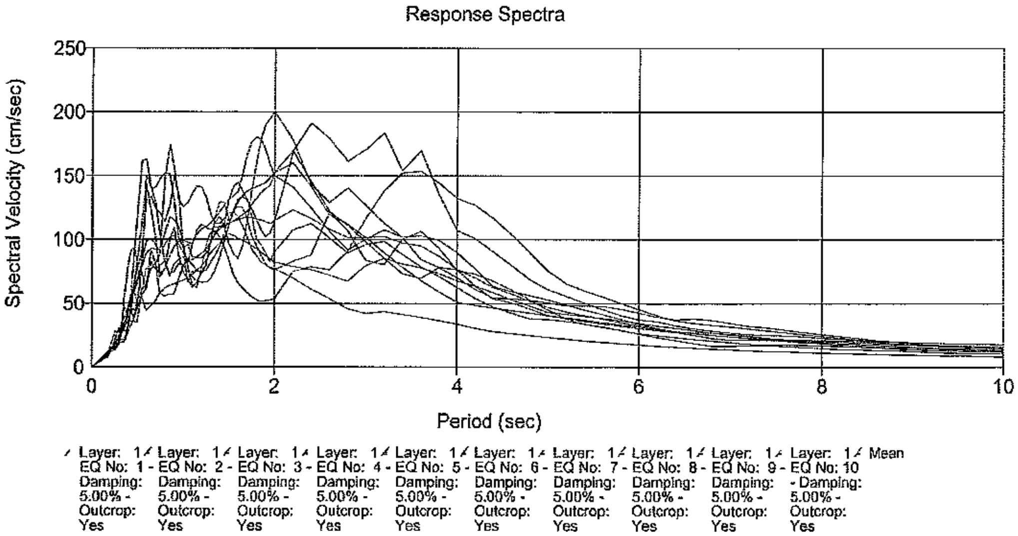

Seismic Assessment of Garden City Elementary School, Richmond, BC
Number
Description
Motion
Output
Shear Wave Velocity
Unit Weight
SAND
SANDY SILT
11 SANDY SILT
14 SAND
15 SILT
16 SAND
17 SILT
18 SILT
19 SILT
20 SAND
21 SILT
22 SILT
23 SILT
24 SILT
25 SILT
26 SILT
27 SAND
28 SILT
29 SILT
30 SILT
31 SILT
32 SILT
33 SILT
34 TILL

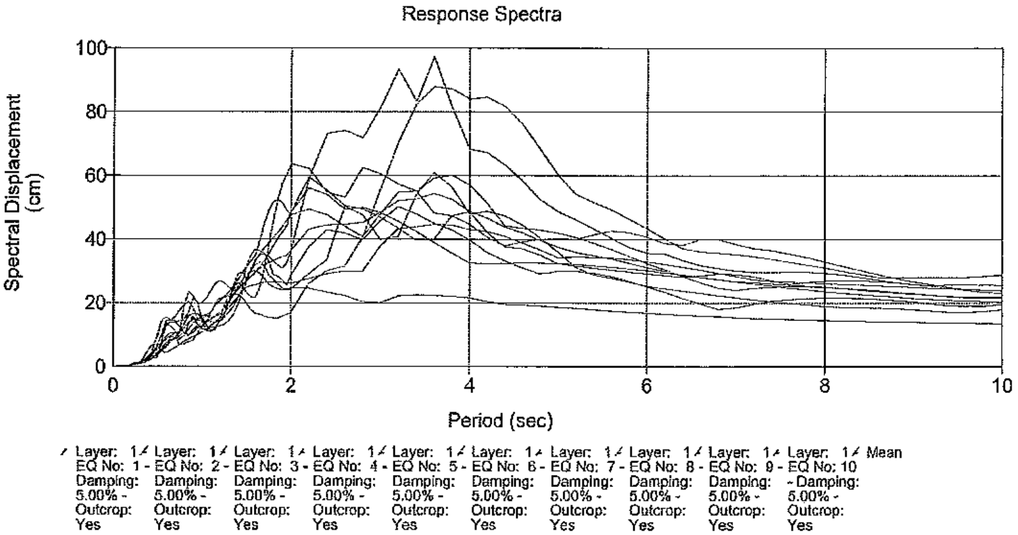

Response Spectra (5% Damping)
Spectral Acceleration (g)
2.5
2.0
1.5
1.0
0.5
0.0
0
1
2
3
4
5
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

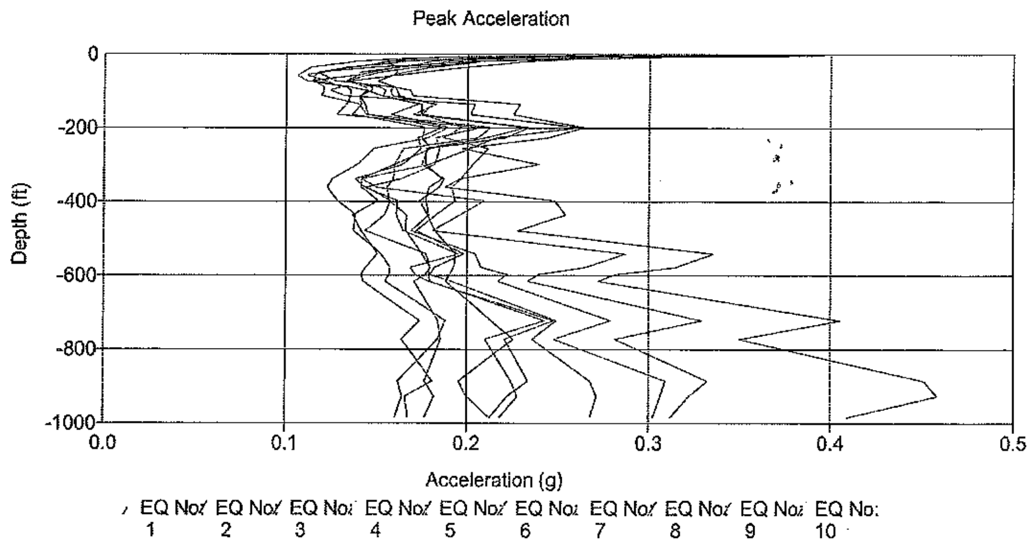

Response Spectra (5% Damping)
Spectral Velocity (cm/sec)
150
100
50
0
0
1
2
3
4
5
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

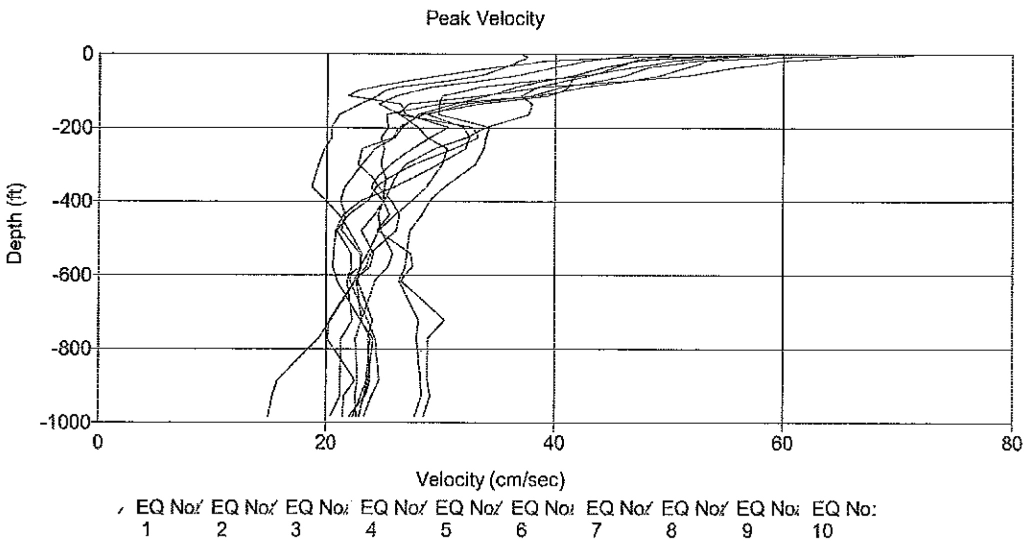

Response Spectra (5% Damping)
Spectral Displacement (cm)
Period (sec)
Motion 1 Motion 2 Motion 3 Motion 4 Motion 5 Motion 6 Motion 7 Motion 8 Motion 9 Motion 10 Mean Target

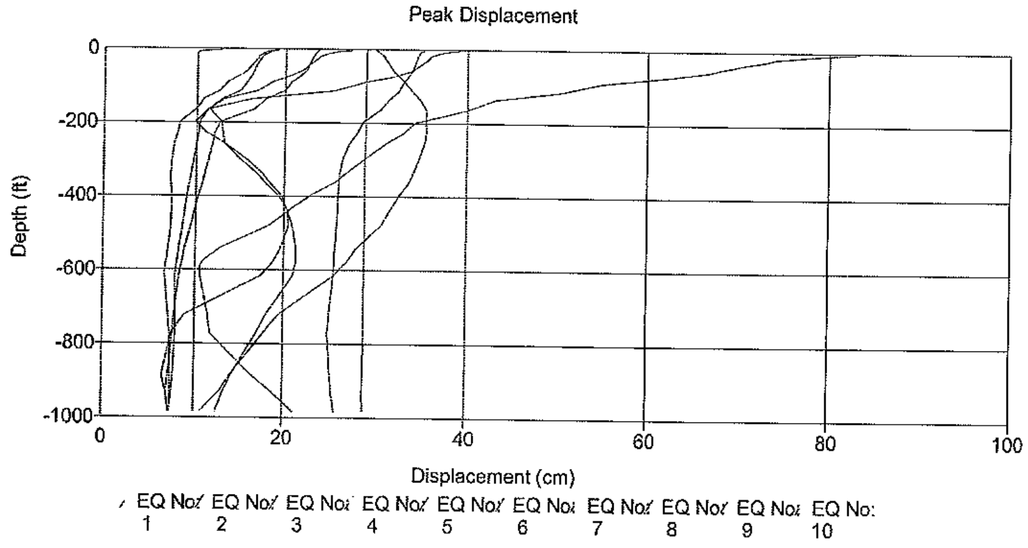

Response Spectra
Acceleration (g)
Period (sec)
Layer: 1 - EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

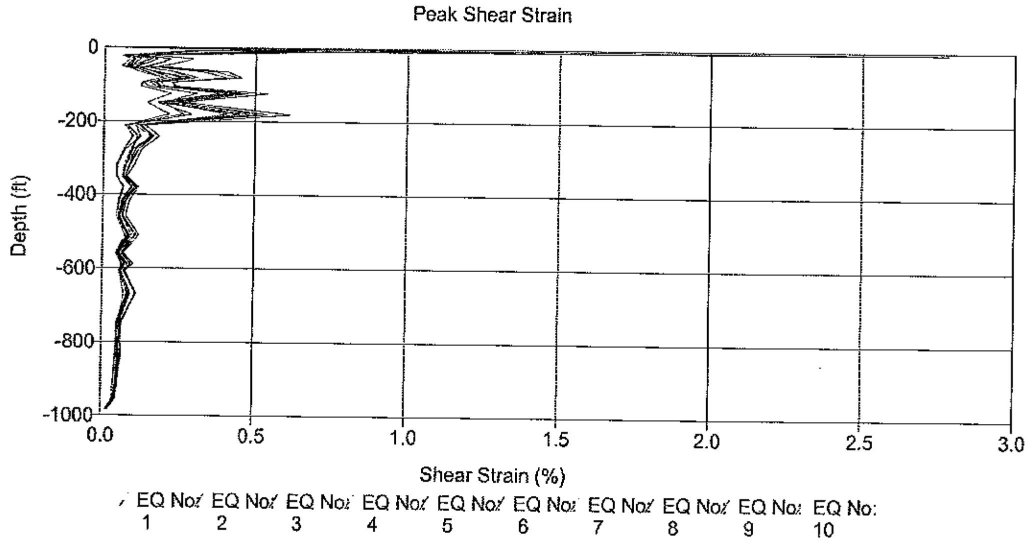

Response Spectra
Spectral Velocity (cm/sec)
0
50
100
150
200
250
0
2
4
6
8
10
Period (sec)
Layer: 1 - EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

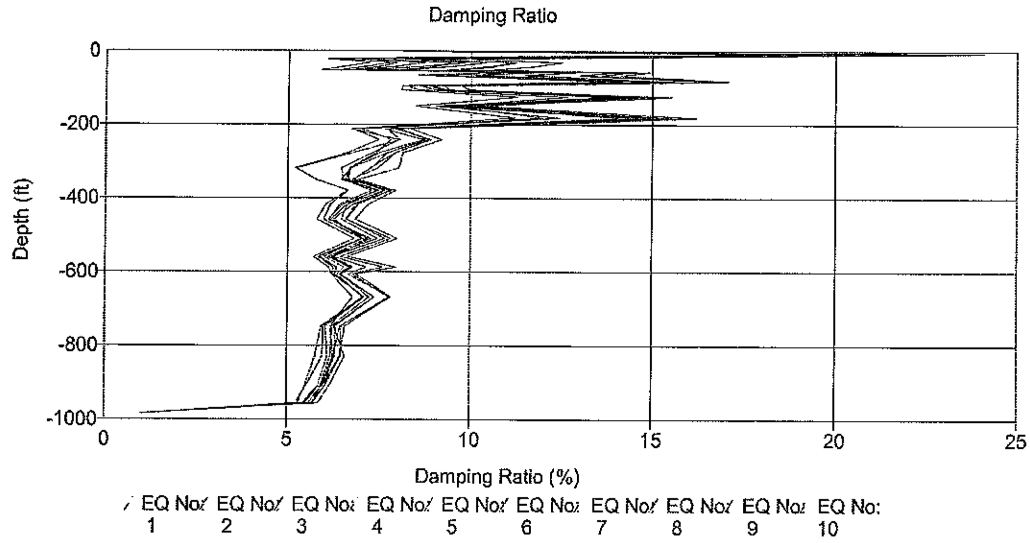

Response Spectra
Spectral Displacement (cm)
0
20
40
60
80
100
0
2
4
6
8
10
Period (sec)
Layer: 1 - EQ No: 1 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 2 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 3 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 4 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 5 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 6 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 7 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 8 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 9 - Damping: 5.00% - Outcrop: Yes
Layer: 1 - EQ No: 10 - Damping: 5.00% - Outcrop: Yes
Mean

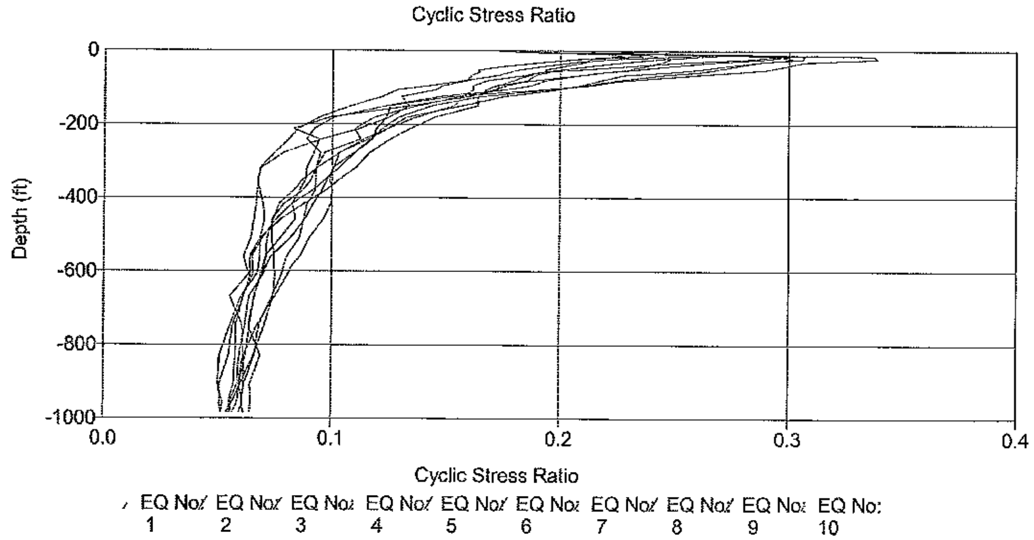

Peak Acceleration
Depth (ft)
0
-200
-400
-600
-800
-1000
0.0
0.1
0.2
0.3
0.4
0.5
Acceleration (g)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

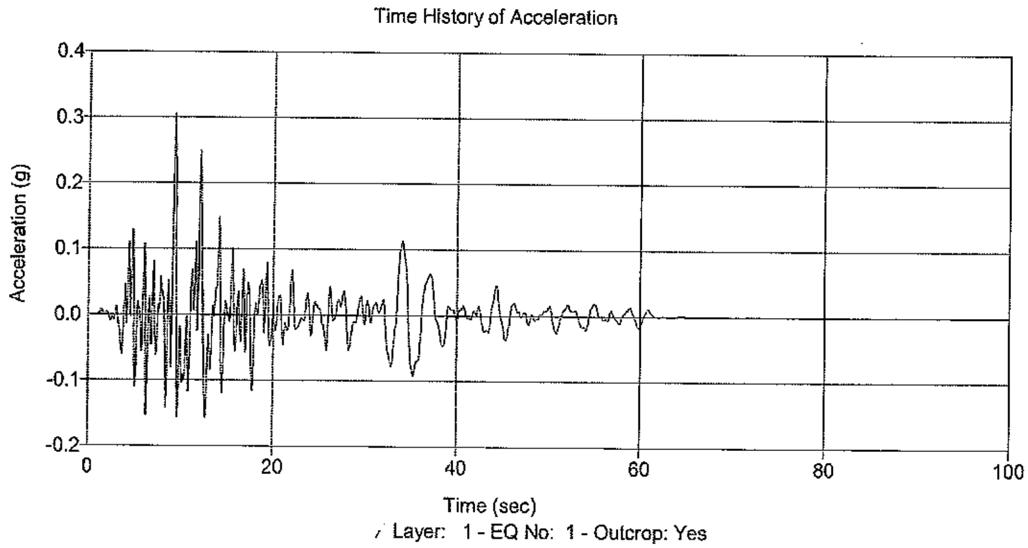

Peak Velocity
Depth (ft)
0
-200
-400
-600
-800
-1000
0
20
40
60
80
Velocity (cm/sec)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

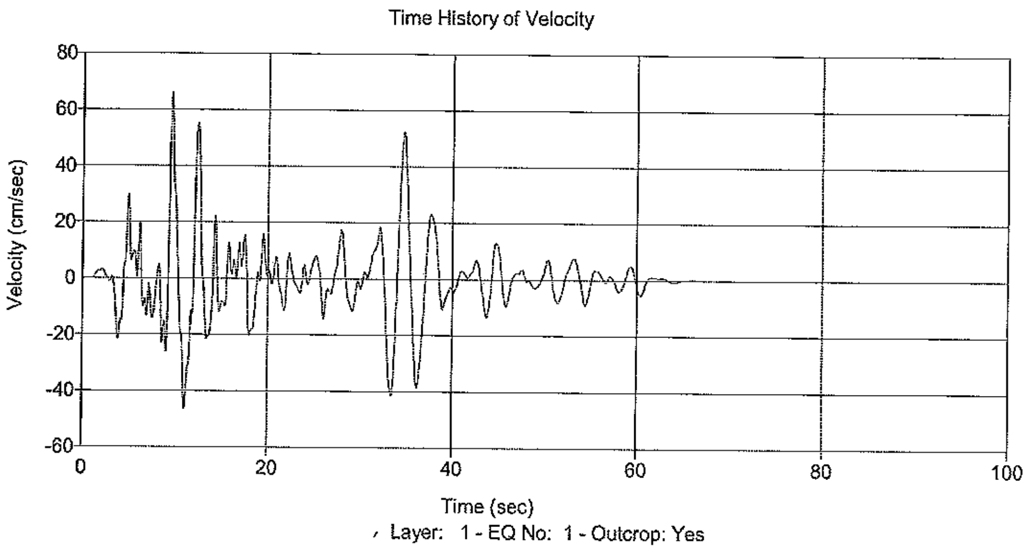

Peak Displacement
Depth (ft)
0
-200
-400
-600
-800
-1000
0
20
40
60
80
100
Displacement (cm)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

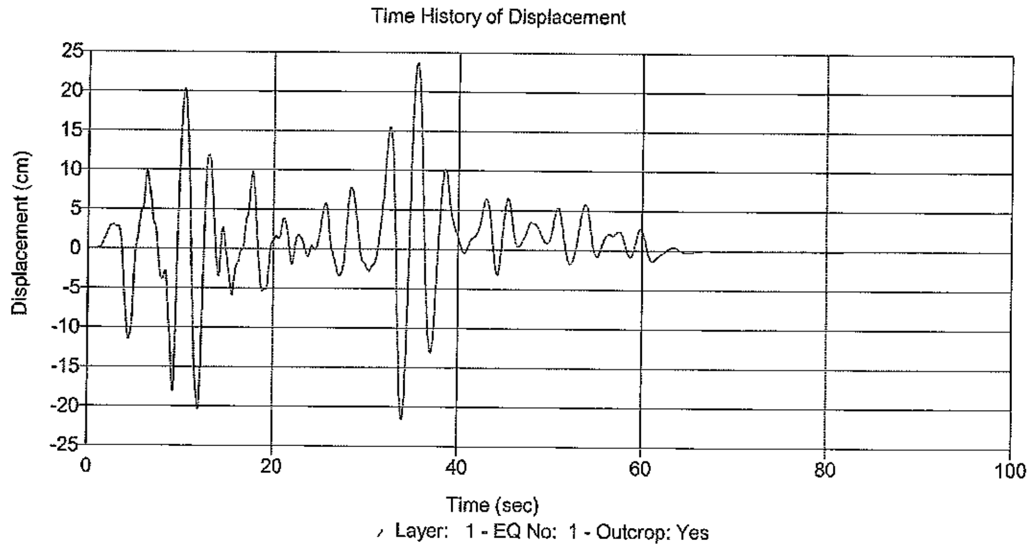

Peak Shear Strain
Depth (ft)
0
-200
-400
-600
-800
-1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Shear Strain (%)
EQ No: 1
EQ No: 2
EQ No: 3
EQ No: 4
EQ No: 5
EQ No: 6
EQ No: 7
EQ No: 8
EQ No: 9
EQ No: 10

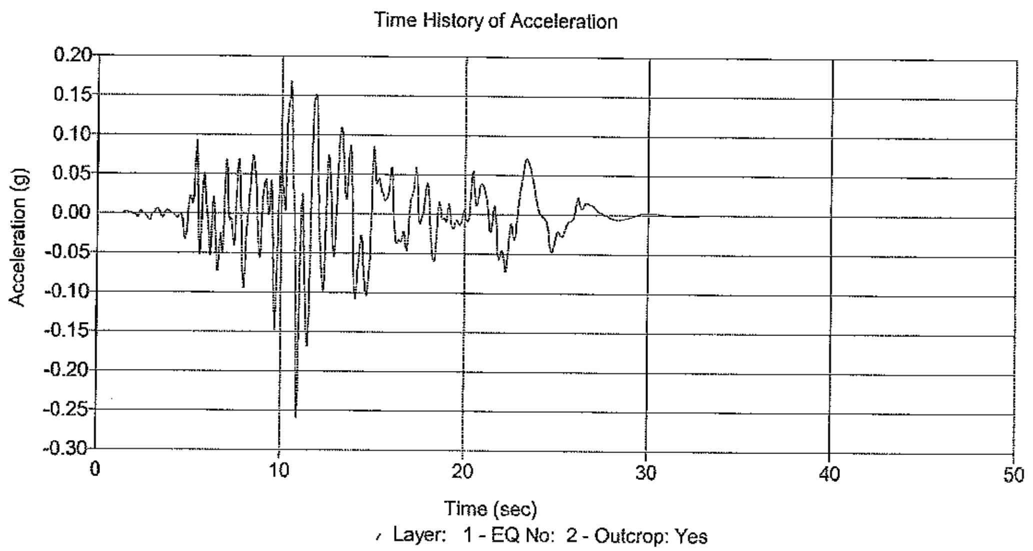

Damping Ratio
0
-200
-400
-600
-800
-1000
Depth (ft)
0
5
10
15
20
25
Damping Ratio (%)
EQ No/ 1
EQ No/ 2
EQ No: 3
EQ No/ 4
EQ No/ 5
EQ No: 6
EQ No/ 7
EQ No/ 8
EQ No: 9
EQ No: 10

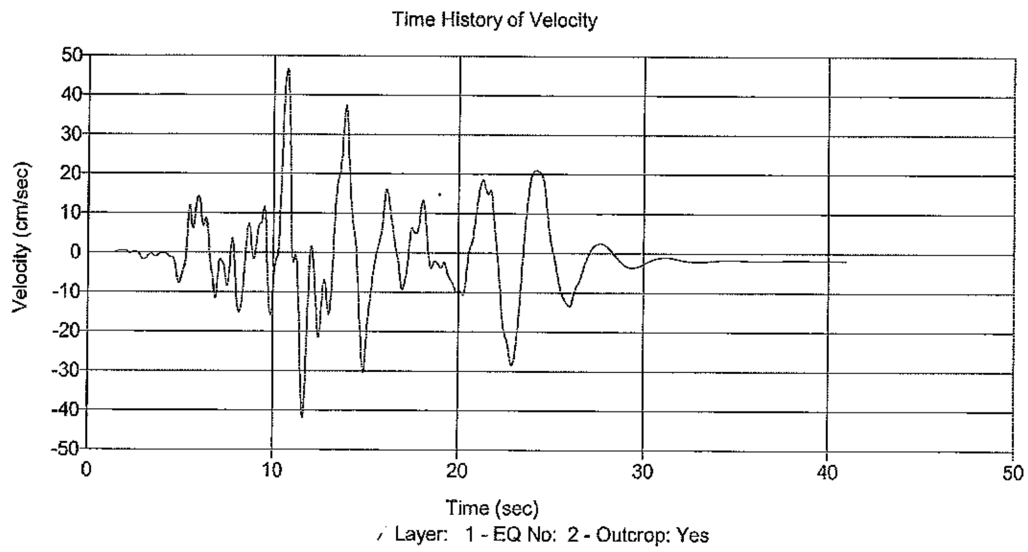

Cyclic Stress Ratio
0
-200
-400
-600
-800
-1000
Depth (ft)
0.0
0.1
0.2
0.3
0.4
Cyclic Stress Ratio
EQ No/ 1
EQ No/ 2
EQ No: 3
EQ No/ 4
EQ No/ 5
EQ No: 6
EQ No/ 7
EQ No/ 8
EQ No: 9
EQ No: 10

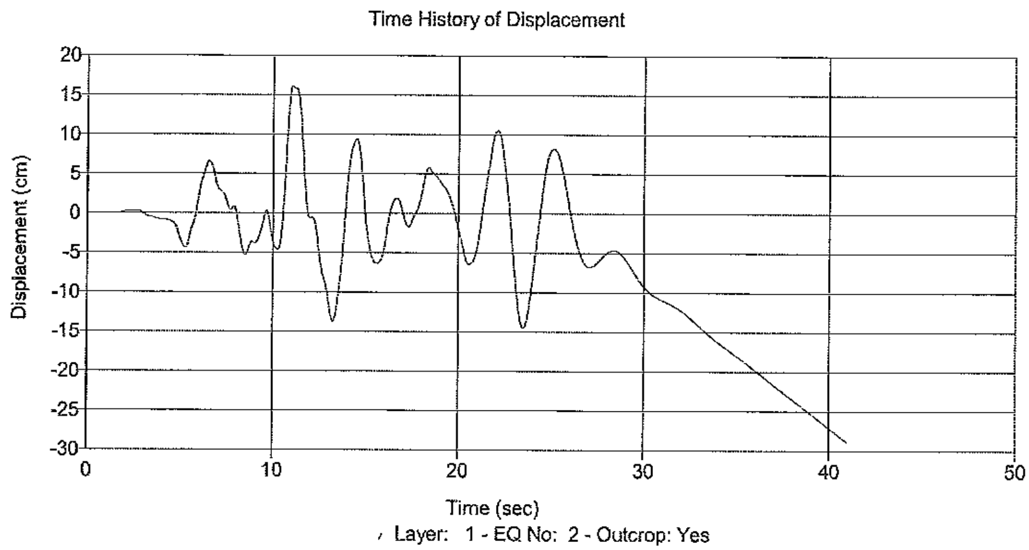

Time History of Acceleration
Acceleration (g)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
0
20
40
60
80
100
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

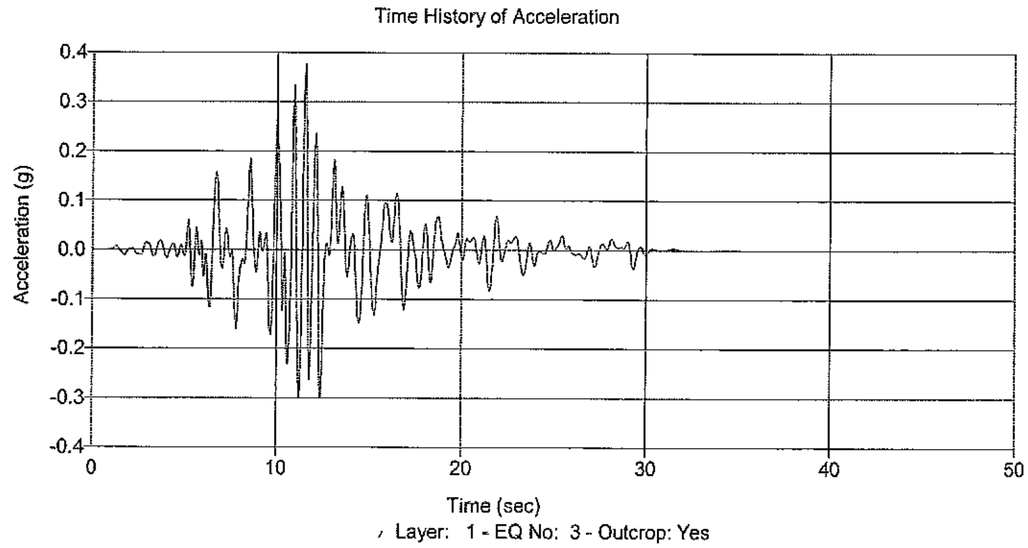

Time History of Velocity
Velocity (cm/sec)
80
60
40
20
0
-20
-40
-60
0
20
40
60
80
100
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

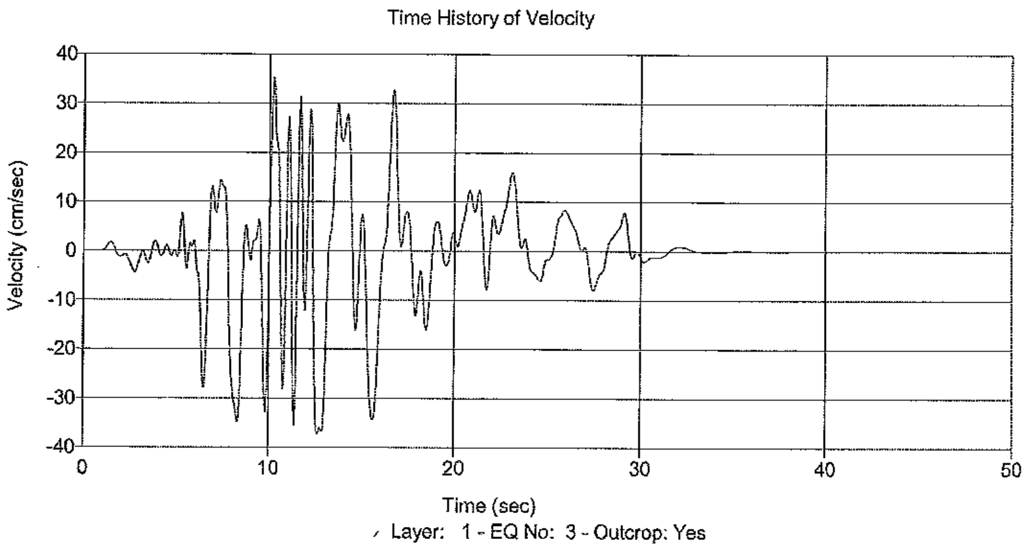

Time History of Displacement
Displacement (cm)
25
20
15
10
5
0
-5
-10
-15
-20
-25
0
20
40
60
80
100
Time (sec)
Layer: 1 - EQ No: 1 - Outcrop: Yes

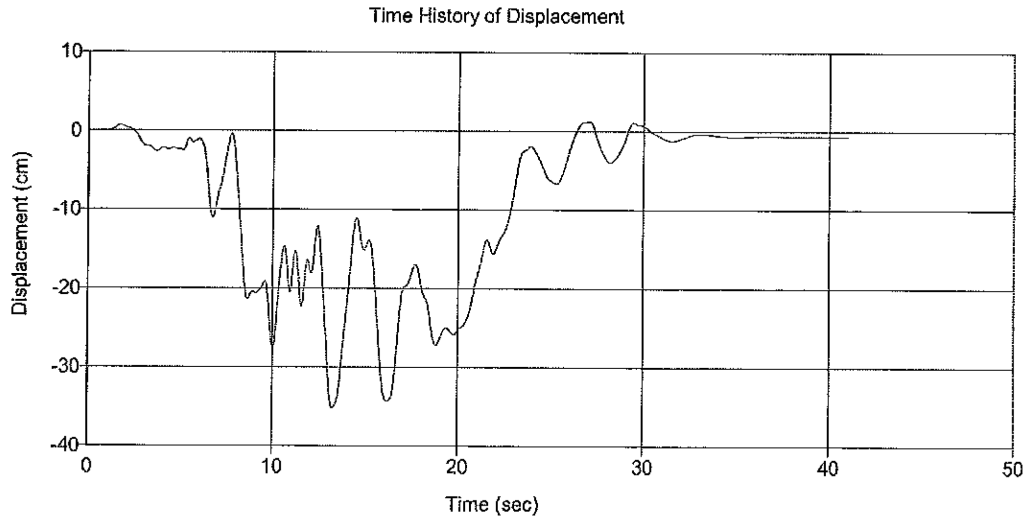

Time History of Acceleration
Acceleration (g)
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
-0.30
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes


Time History of Velocity
Velocity (cm/sec)
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes

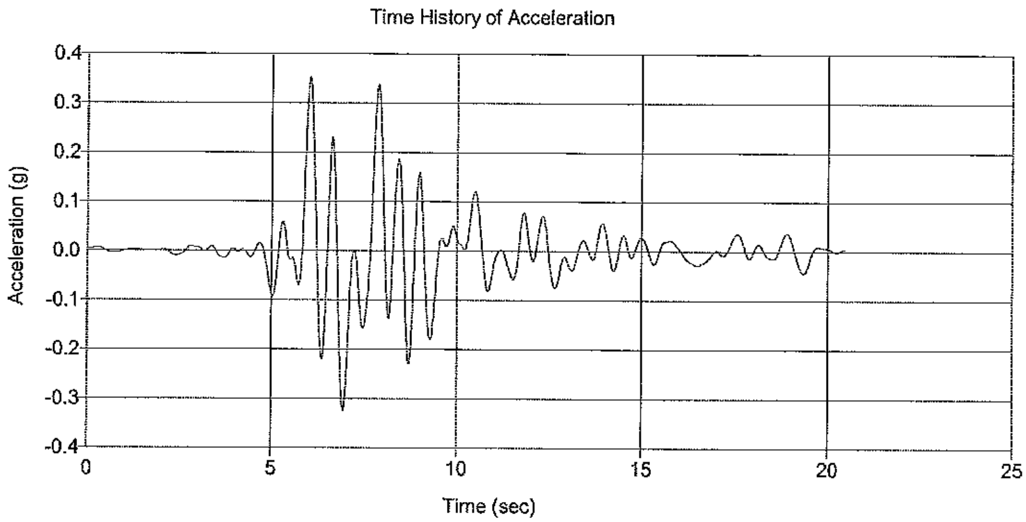

Time History of Displacement
Displacement (cm)
20
15
10
5
0
-5
-10
-15
-20
-25
-30
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 2 - Outcrop: Yes


Time History of Acceleration
Acceleration (g)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 3 - Outcrop: Yes

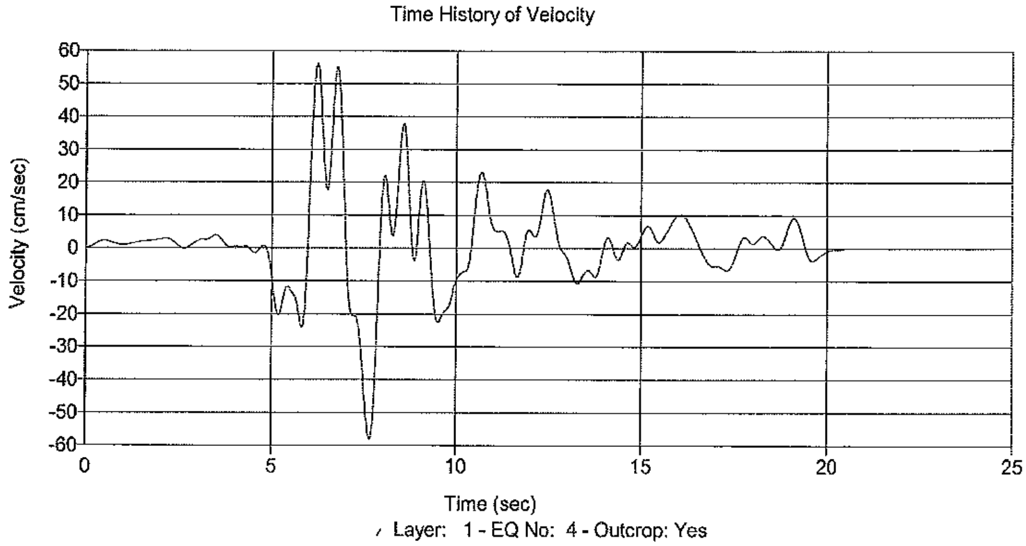

Time History of Velocity
Velocity (cm/sec)
40
30
20
10
0
-10
-20
-30
-40
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 3 - Outcrop: Yes

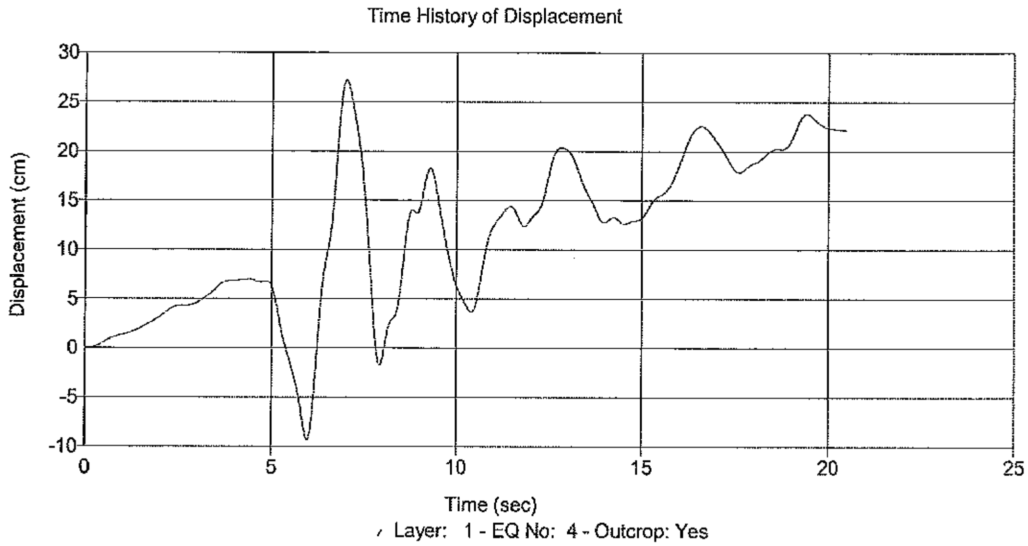


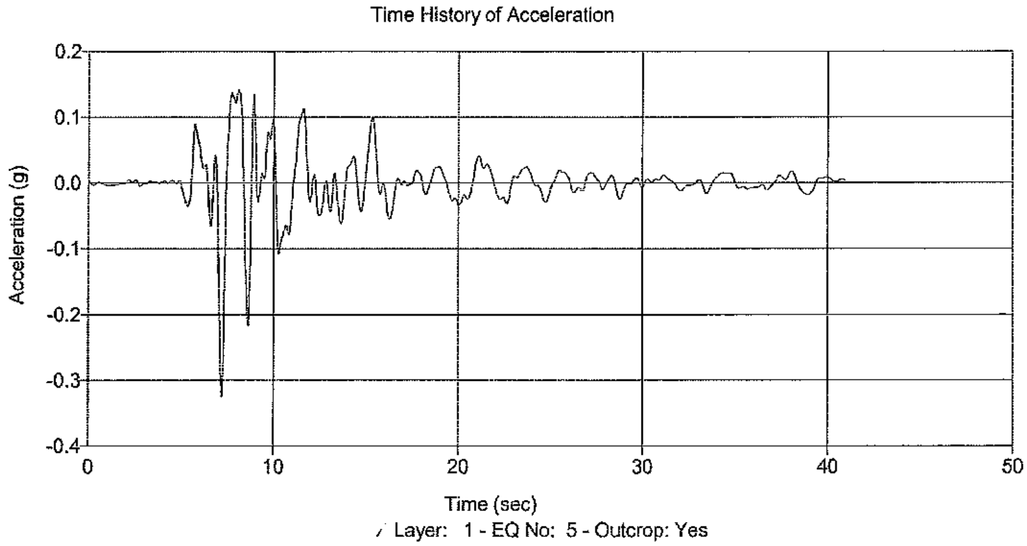
Layer: 1 - EQ No: 3 - Outcrop: Yes

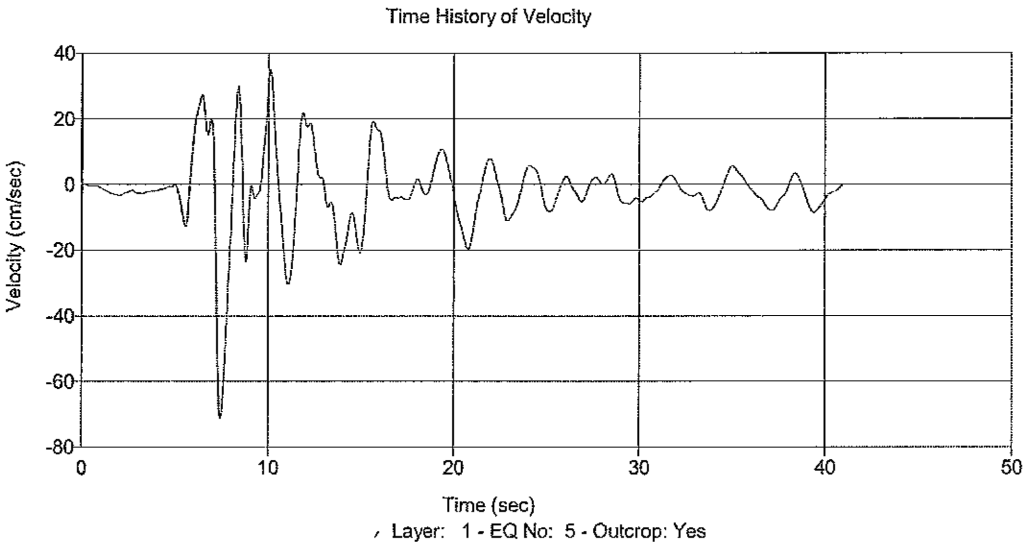


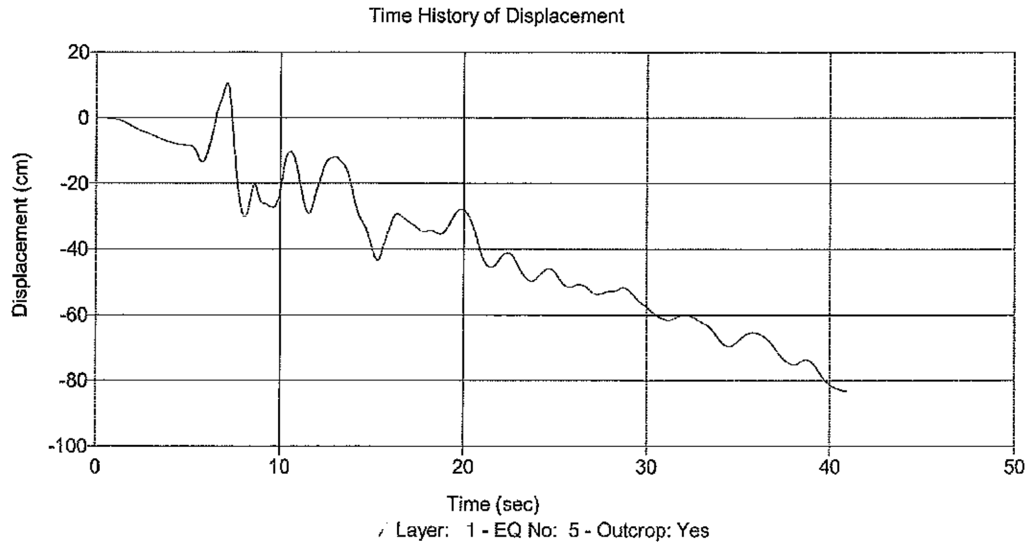
Layer: 1 - EQ No: 4 - Outcrop: Yes

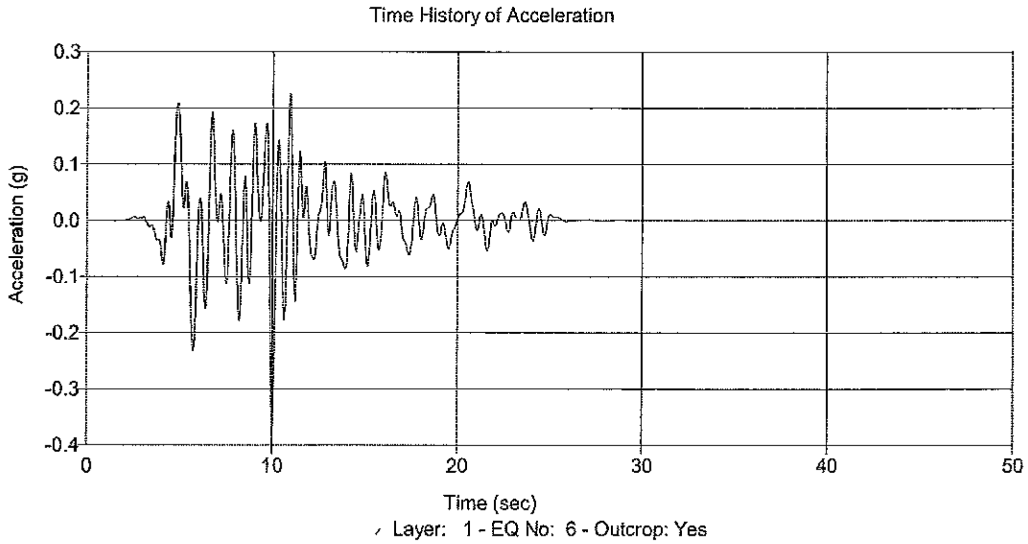

Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0
5
10
15
20
25
Time (sec)
Layer: 1 - EQ No: 4 - Outcrop: Yes

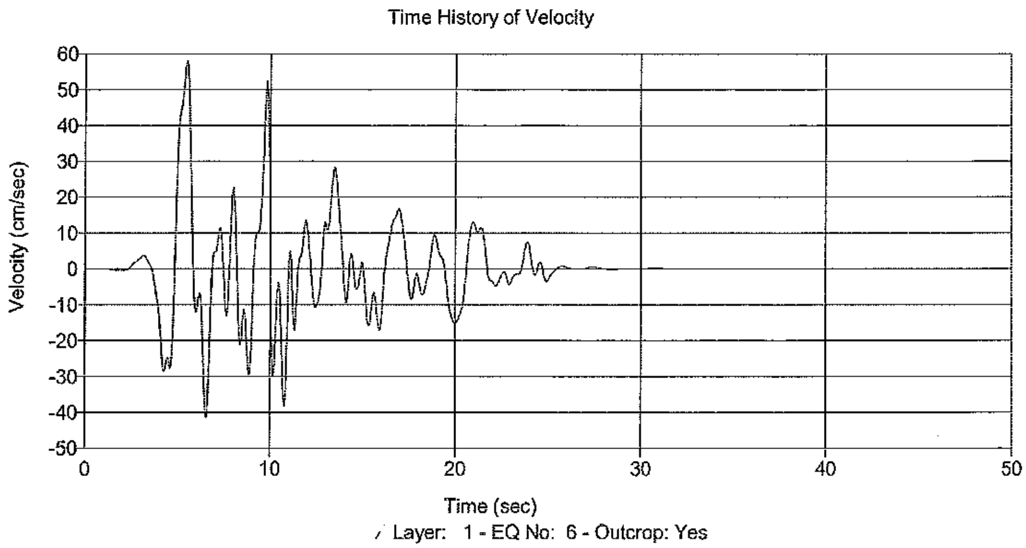

Time History of Displacement
Displacement (cm)
30
25
20
15
10
5
0
-5
-10
0
5
10
15
20
25
Time (sec)
Layer: 1 - EQ No: 4 - Outcrop: Yes

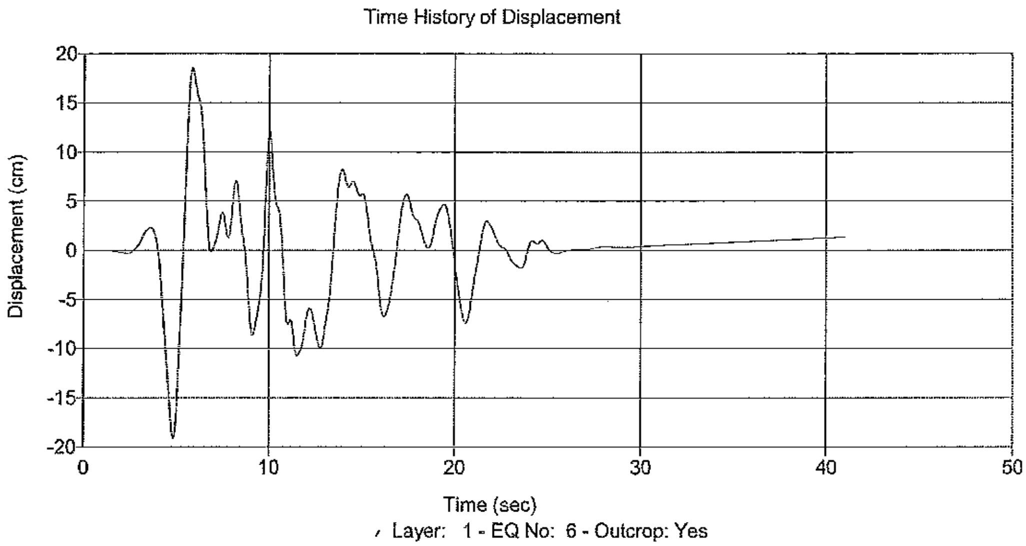

Time History of Acceleration
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
Acceleration (g)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

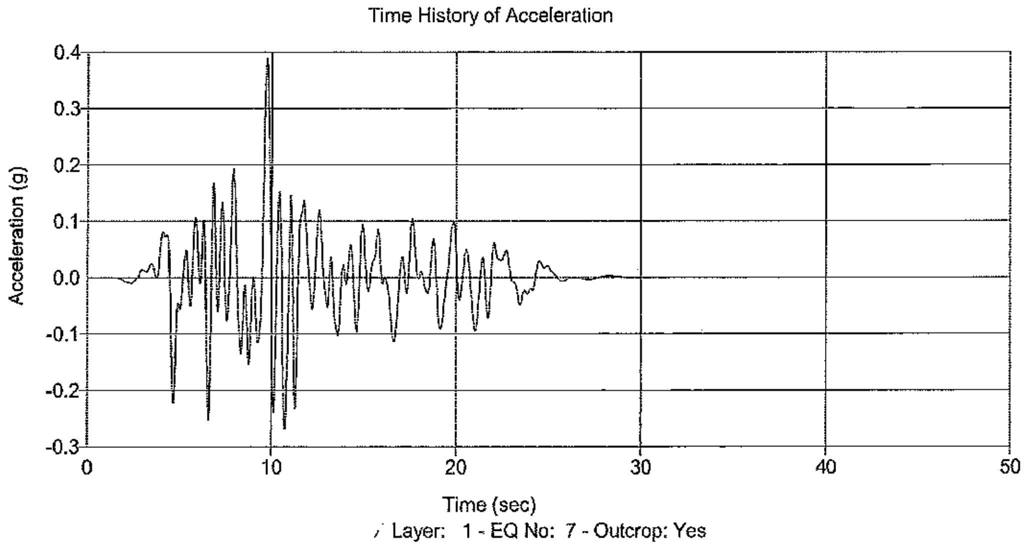

Time History of Velocity
40
20
0
-20
-40
-60
-80
Velocity (cm/sec)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

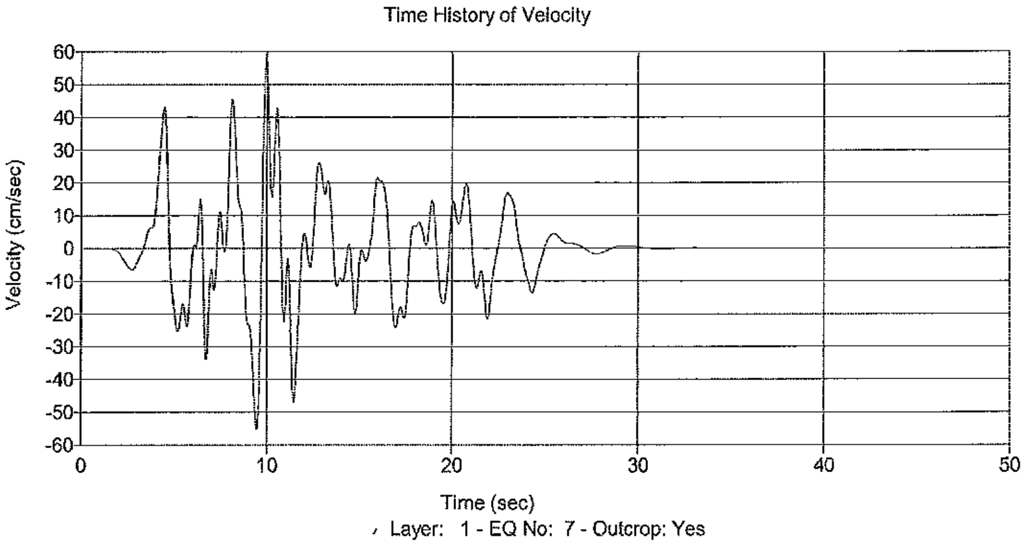

Time History of Displacement
Displacement (cm)
20
0
-20
-40
-60
-80
-100
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 5 - Outcrop: Yes

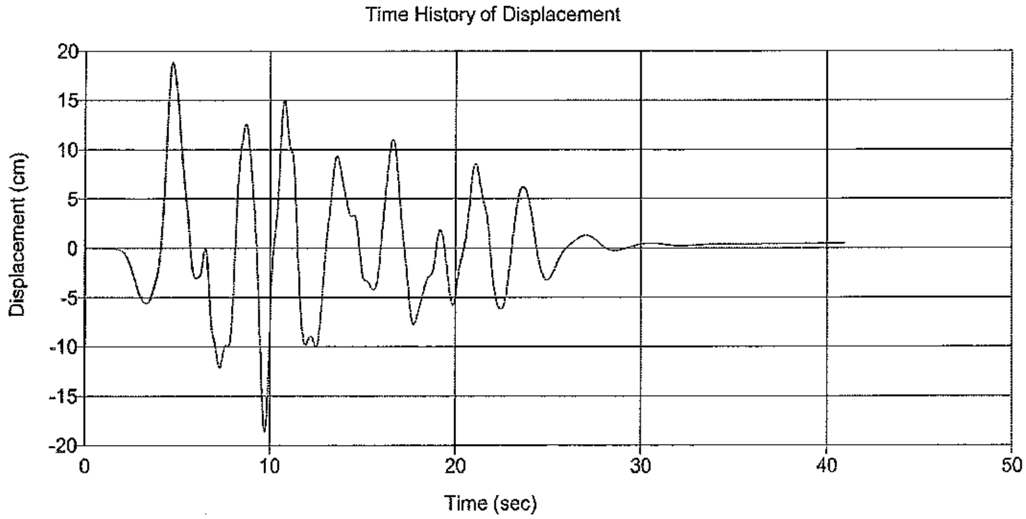

Time History of Acceleration
Acceleration (g)
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes


Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes

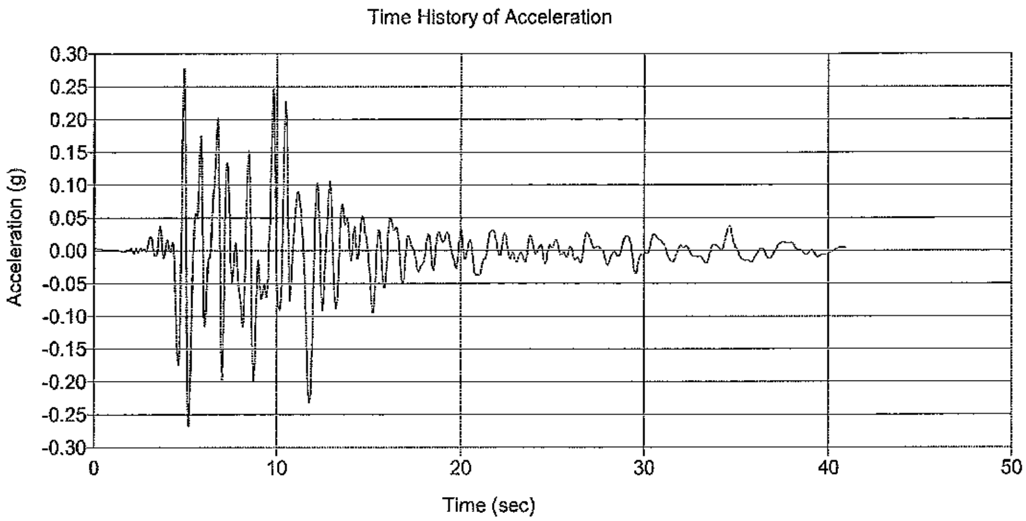

Time History of Displacement
Displacement (cm)
20
15
10
5
0
-5
-10
-15
-20
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 6 - Outcrop: Yes


Time History of Acceleration
Acceleration (g)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 7 - Outcrop: Yes

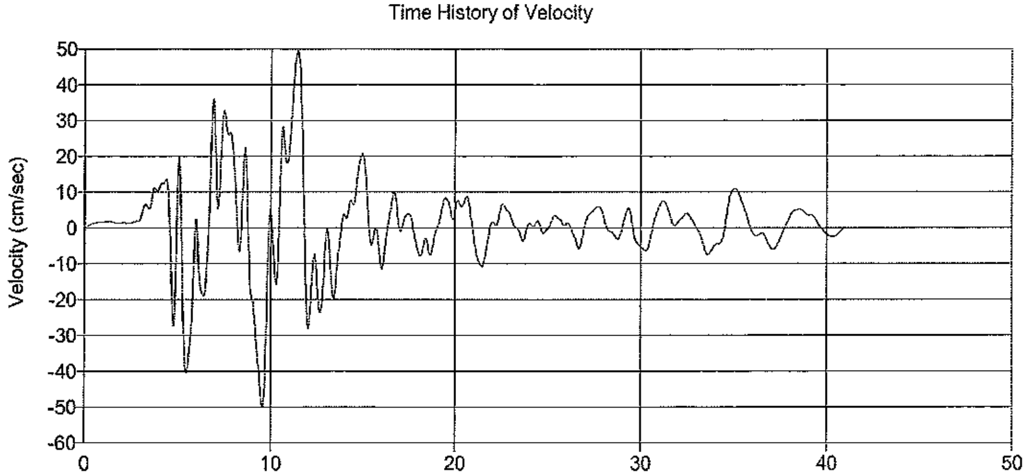

Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 7 - Outcrop: Yes



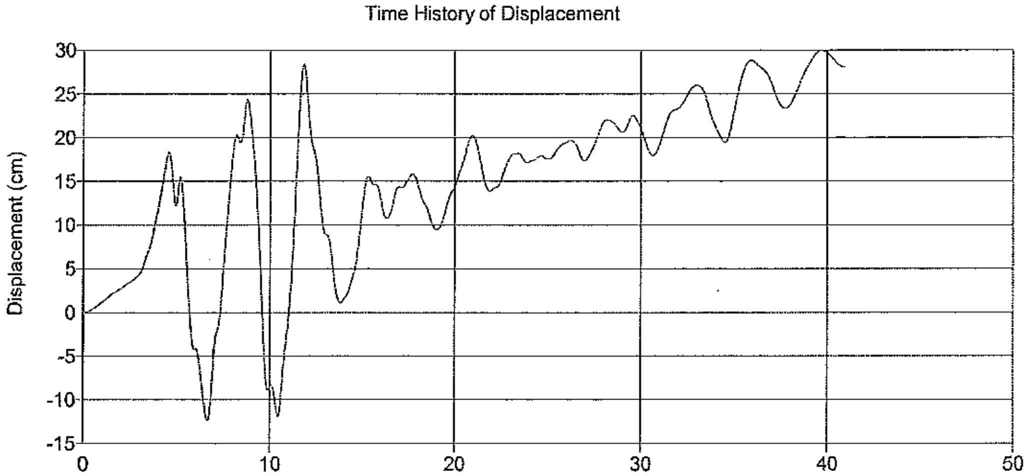
Layer: 1 - EQ No: 7 - Outcrop: Yes



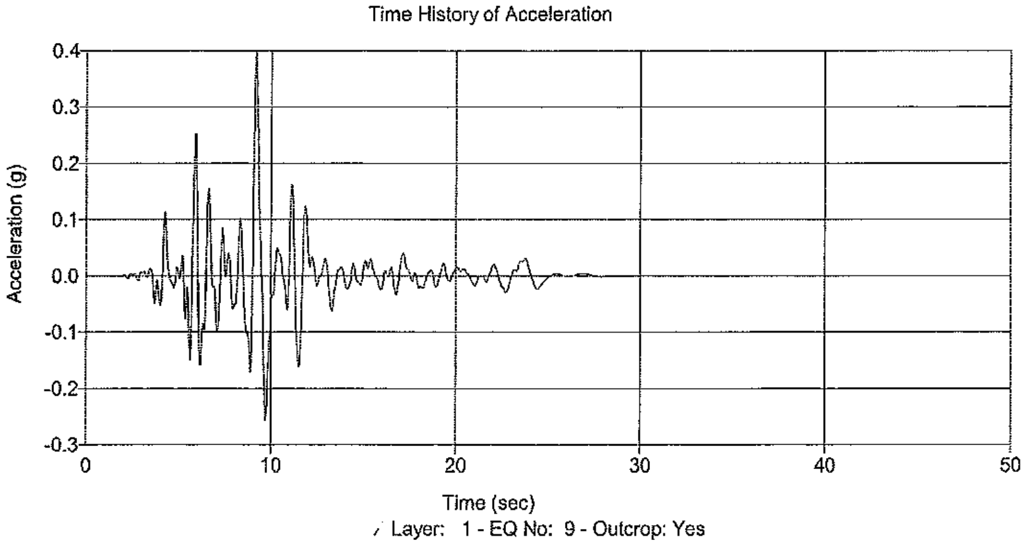
Layer: 1 - EQ No: 8 - Outcrop: Yes

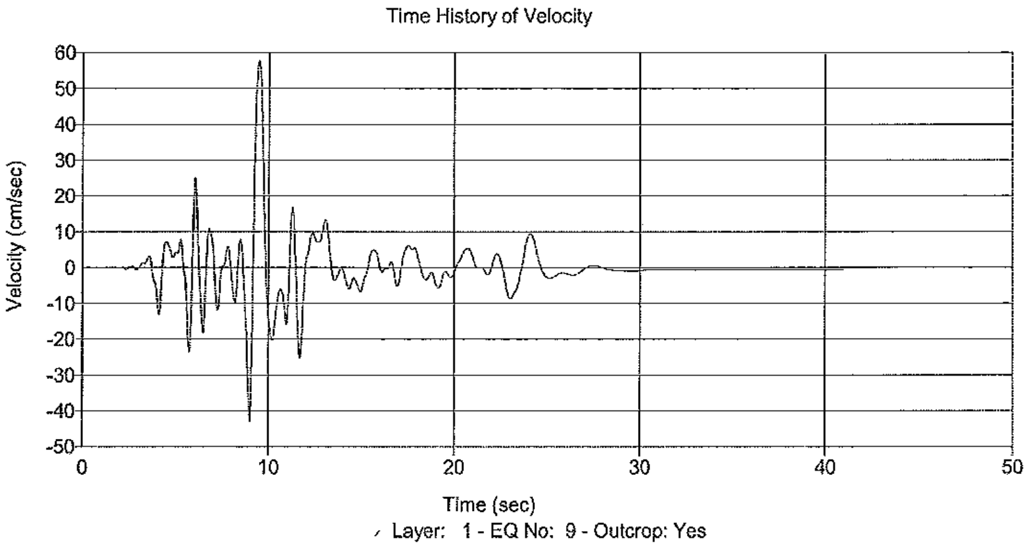


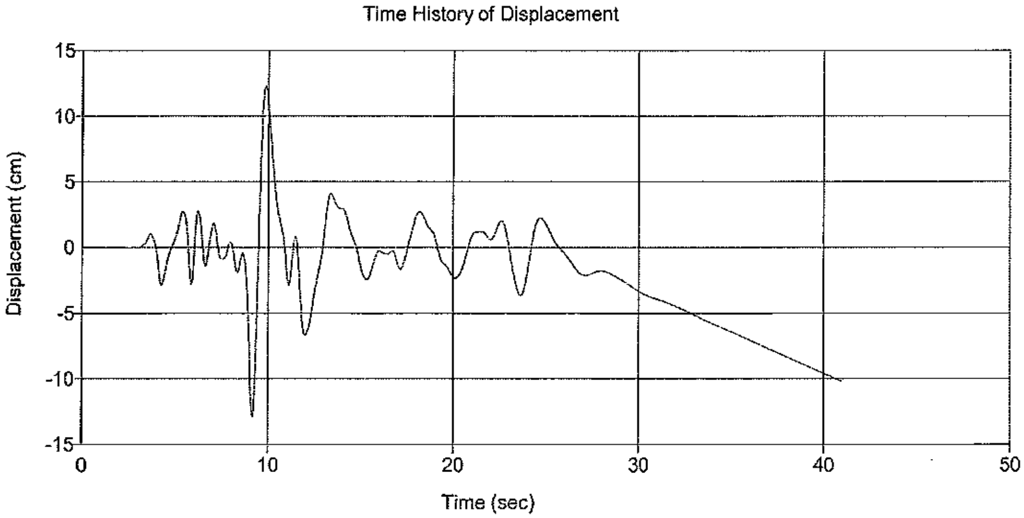
Time (sec)
Layer: 1 - EQ No: 8 - Outcrop: Yes



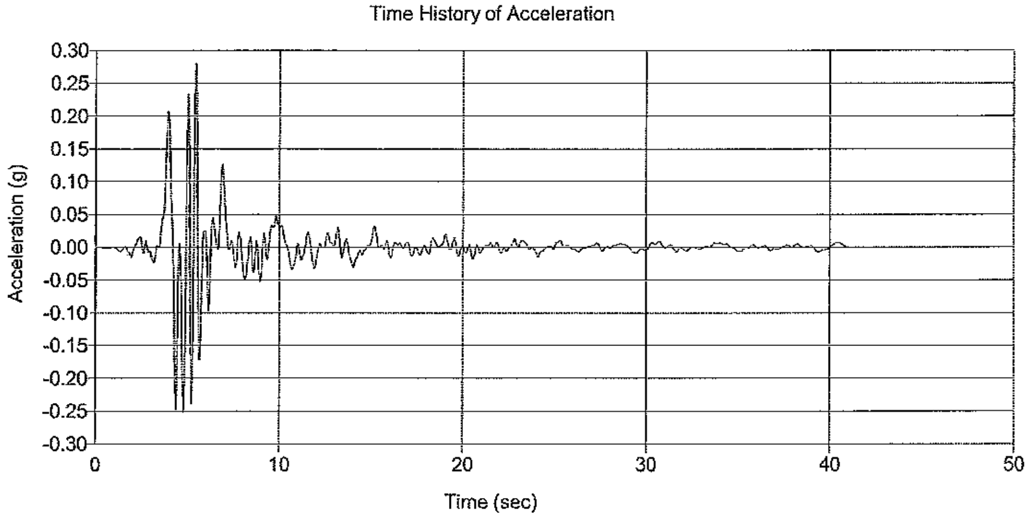
Time (sec)
Layer: 1 - EQ No: 8 - Outcrop: Yes


Time History of Acceleration
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
Acceleration (g)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 9 - Outcrop: Yes

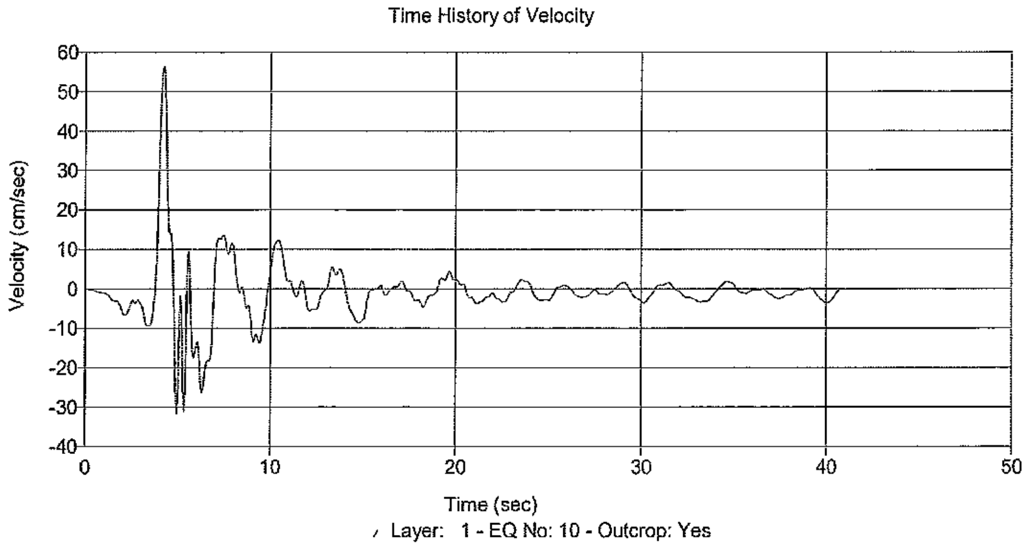

Time History of Velocity
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
Velocity (cm/sec)
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 9 - Outcrop: Yes

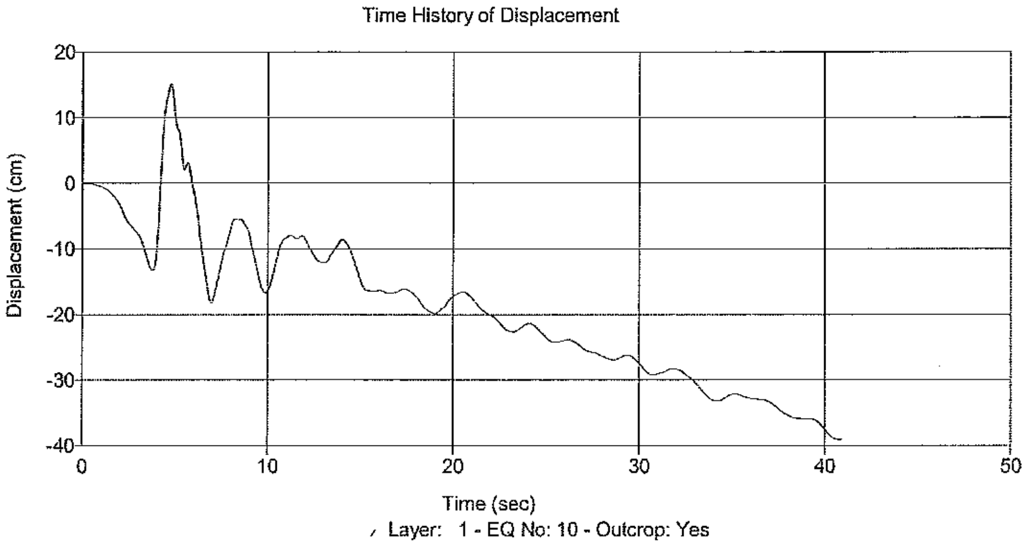


Layer: 1 - EQ No: 9 - Outcrop: Yes



Layer: 1 - EQ No: 10 - Outcrop: Yes


Time History of Velocity
Velocity (cm/sec)
60
50
40
30
20
10
0
-10
-20
-30
-40
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 10 - Outcrop: Yes


Time History of Displacement
Displacement (cm)
20
10
0
-10
-20
-30
-40
0
10
20
30
40
50
Time (sec)
Layer: 1 - EQ No: 10 - Outcrop: Yes

(2) MEG Consulting Ltd. Report (DESRA Analysis)

(a) E-mail dated March 20, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: gwt@tbgsc.bc.ca
Sent: Tuesday, March 20, 2007 4:09 PM
Subject: Site Response Analysis Results for Garden City Elementary

Graham,

Using the same soil layering model and shear wave velocity/soil density distribution as used by Trow for their SHAKE analysis for the soil profile at Garden City Elementary (with firm ground considered to be at the 300m depth) and the same suite (10) of firm ground input motions, I have run the nonlinear site response program DESRA-2C to compute near surface (1.0m depth) ground motions. The shear strength profile used in the analysis is given in the attached Excel spreadsheet, along with other soil layer properties.

I have also attached computed cyclic stress ratios and peak ground accelerations versus depth in the Excel file "CSR & Amax", as well as computed peak spectral accelerations and velocities versus structural period in the Excel file "spectra". Finally, the computed acceleration time histories at the 1.0 m depth are attached.

I have used undrained strengths in the near surface silts (including a strain rate factor of 1.1) judged to be representative of cyclic simple shear strengths, drained shear strengths on the horizontal plane in the underlying sands based on estimated friction angles and K0 values, and dynamic undrained strengths (including a strain rate factor of 1.1) in the deeper interbedded clay/silt/sand deposits representative of approximately normally consolidated simple shear strengths.

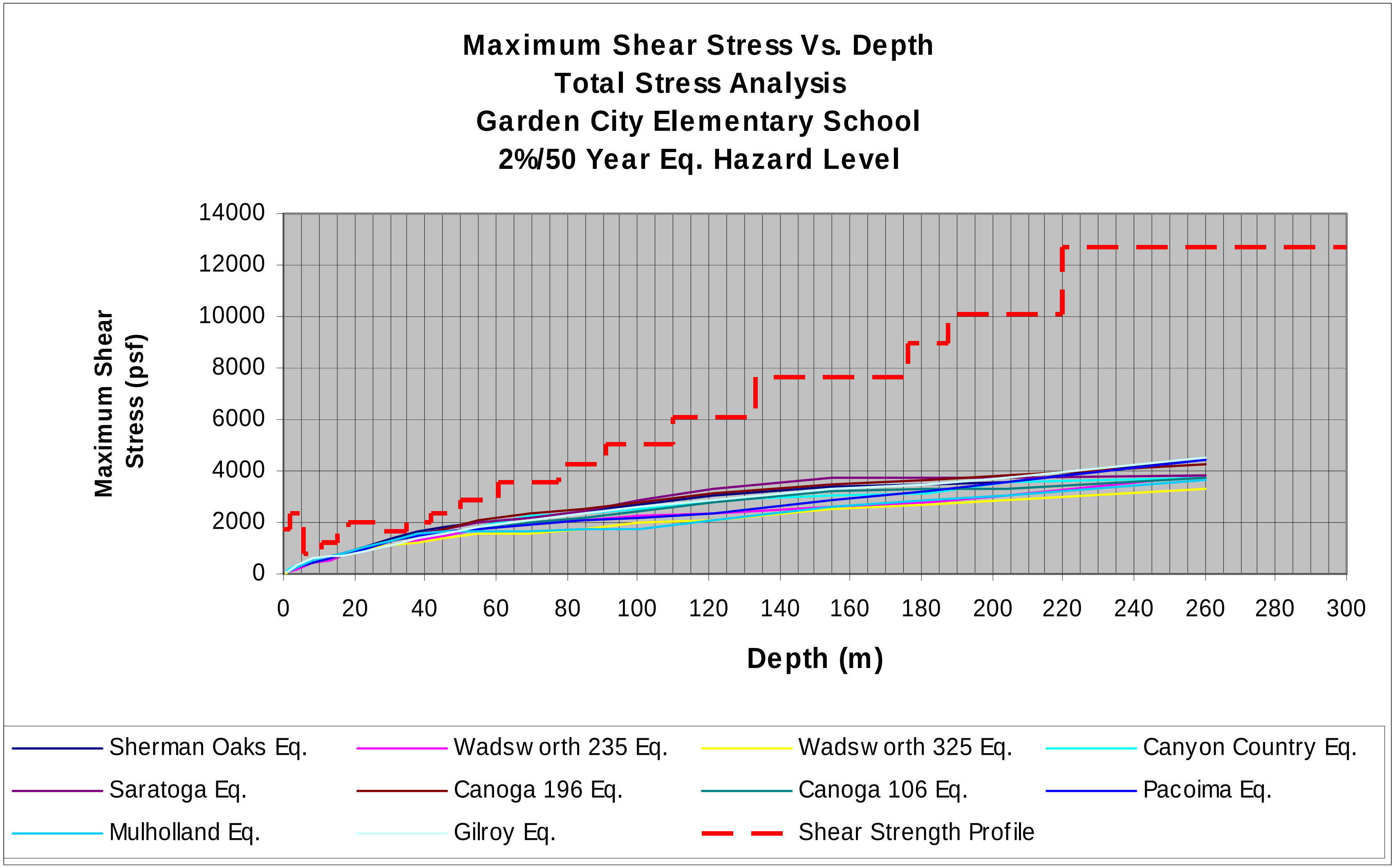
I will proceed with analysis of the remaining 4 school sites. Please advise if a more formal report is required for each school site, and the results of your ongoing structural response analysis.

Regards,

Blair Gohl, Ph.D., P.Eng.
Principal
MEG Consulting Ltd.

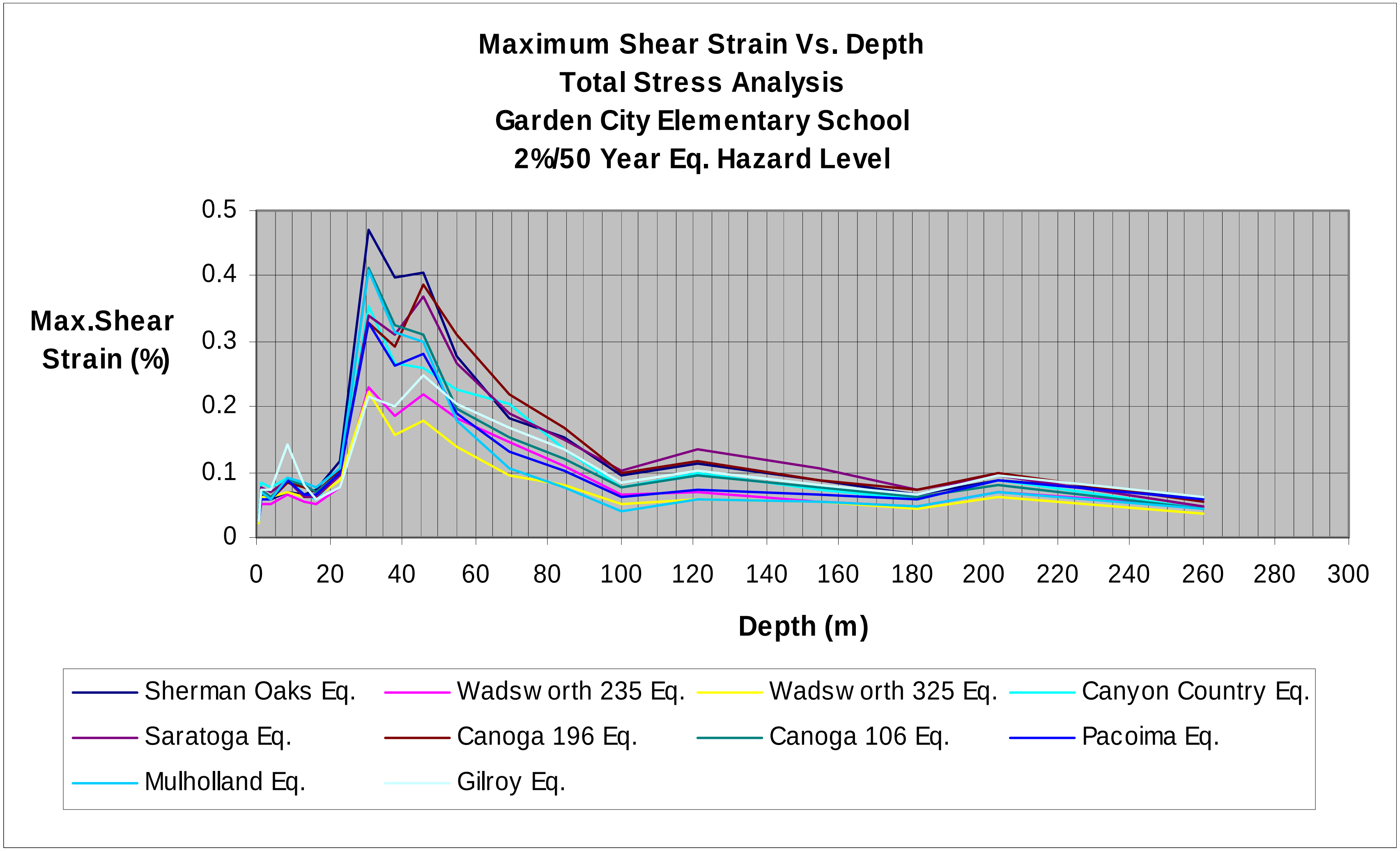
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Garden City Elementary School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

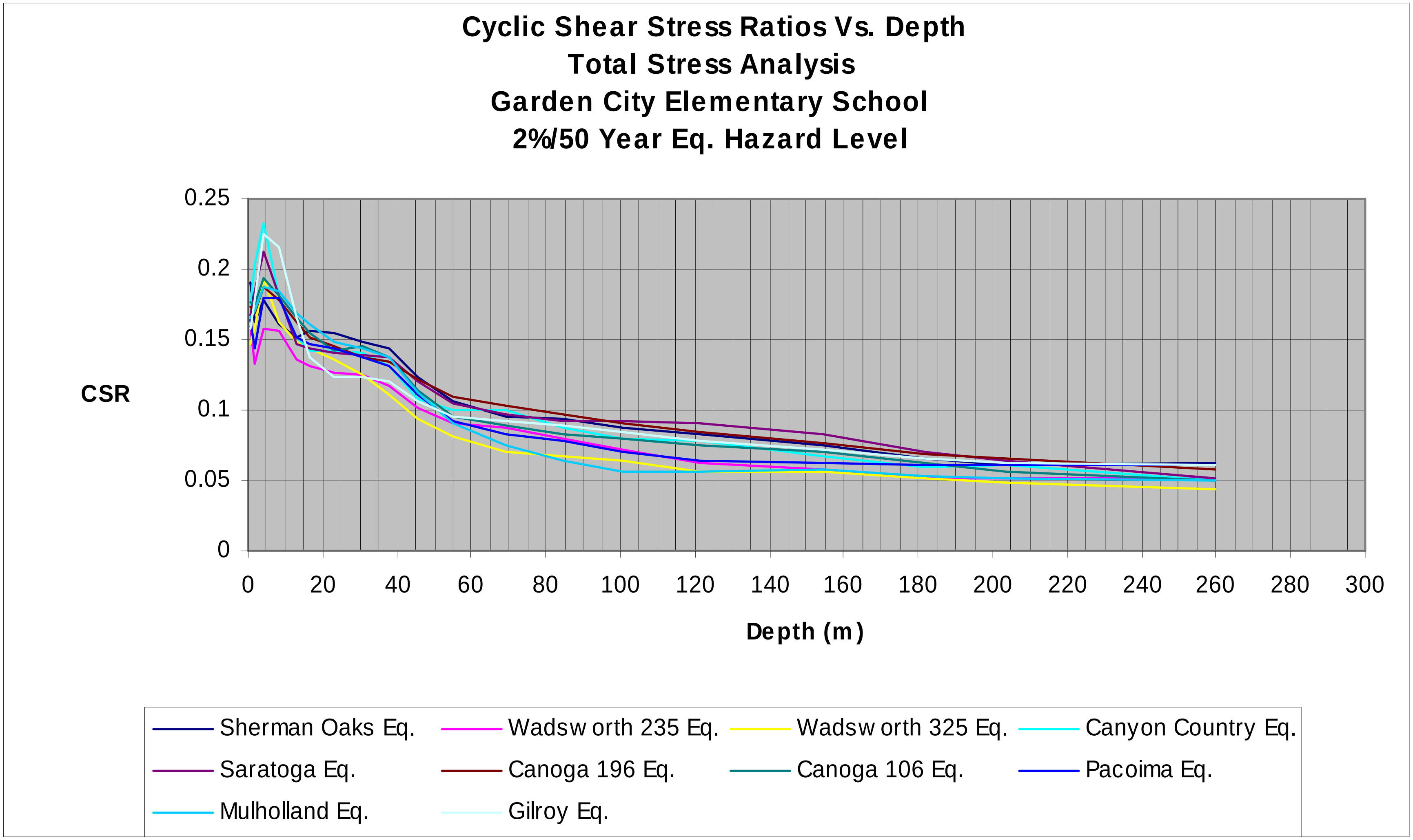
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Garden City Elementary School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
0.5
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

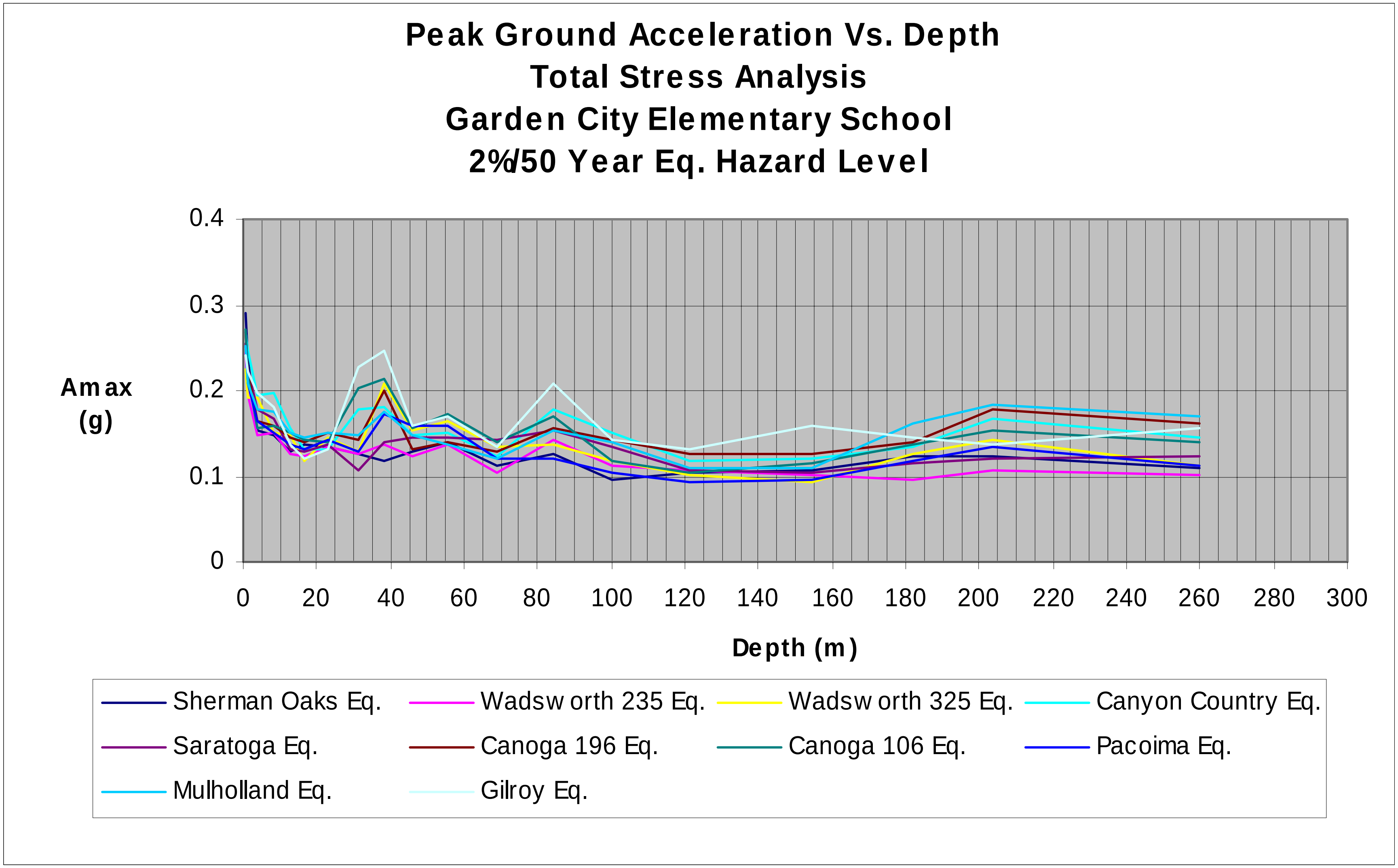
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Garden City Elementary School
2%/50 Year Eq. Hazard Level
CSR
0.25
0.2
0.15
0.1
0.05
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

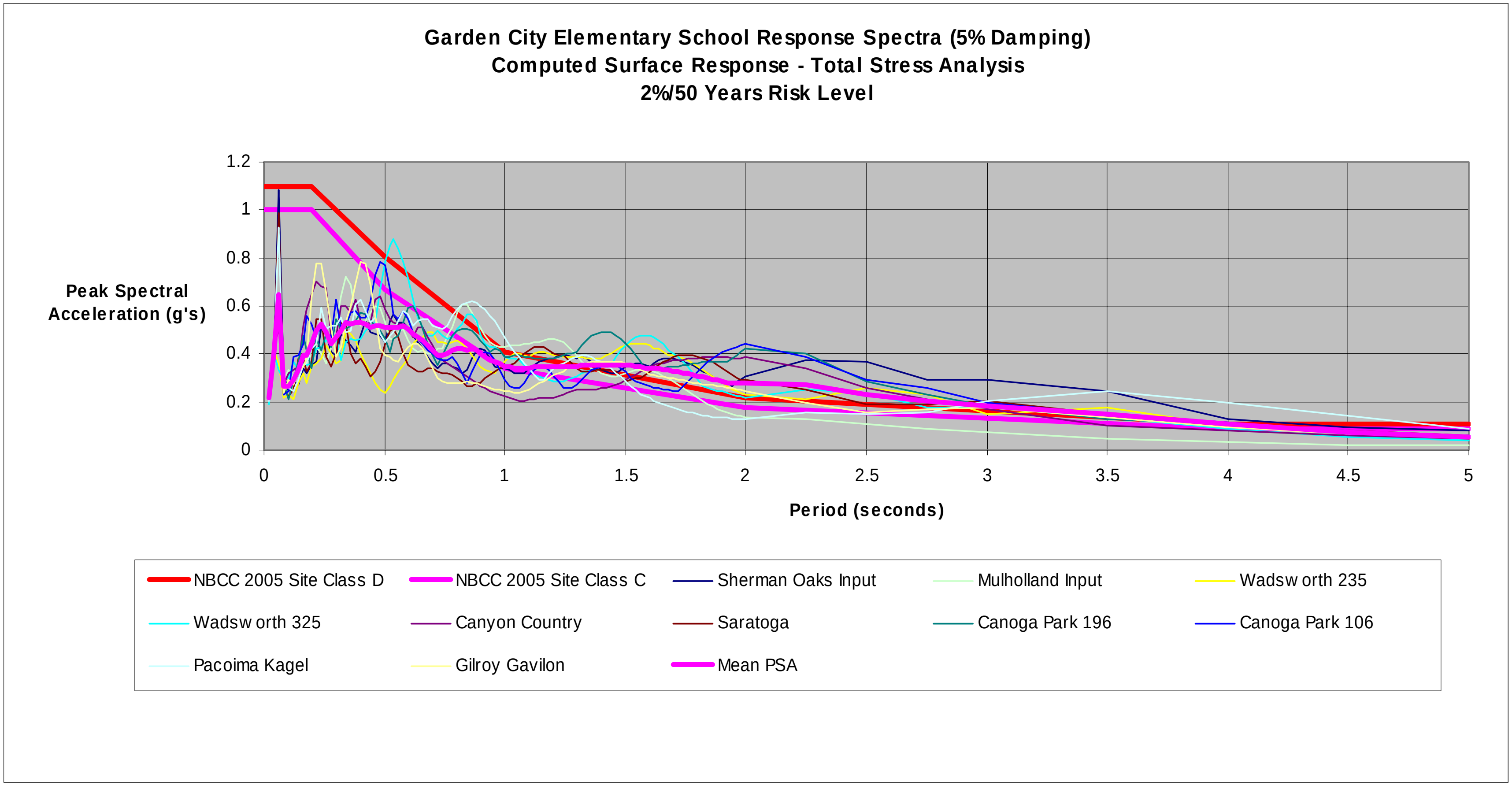
(2) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Garden City Elementary School
2%/50 Year Eq. Hazard Level
Amax (g)
0.4
0.3
0.2
0.1
0
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

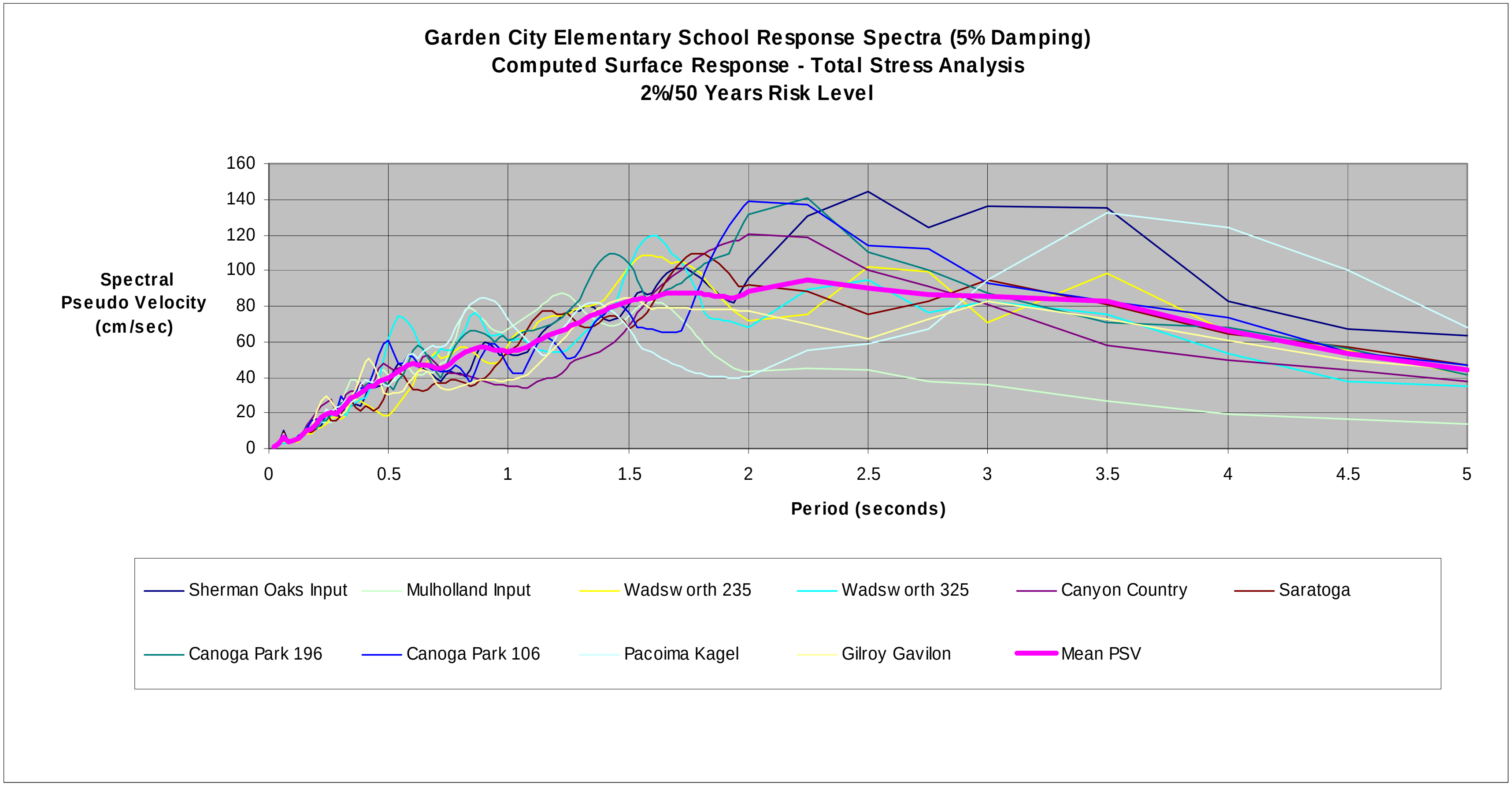
(2) MEG Consulting Ltd. Report (DESRA Analysis)

(f) Graph - surface acceleration response spectra


Garden City Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSA

(2) MEG Consulting Ltd. Report (DESRA Analysis)

(g) Graph - surface velocity response spectra


Garden City Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
0
20
40
60
80
100
120
140
160
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSV

(2) MEG Consulting Ltd. Report (DESRA Analysis)

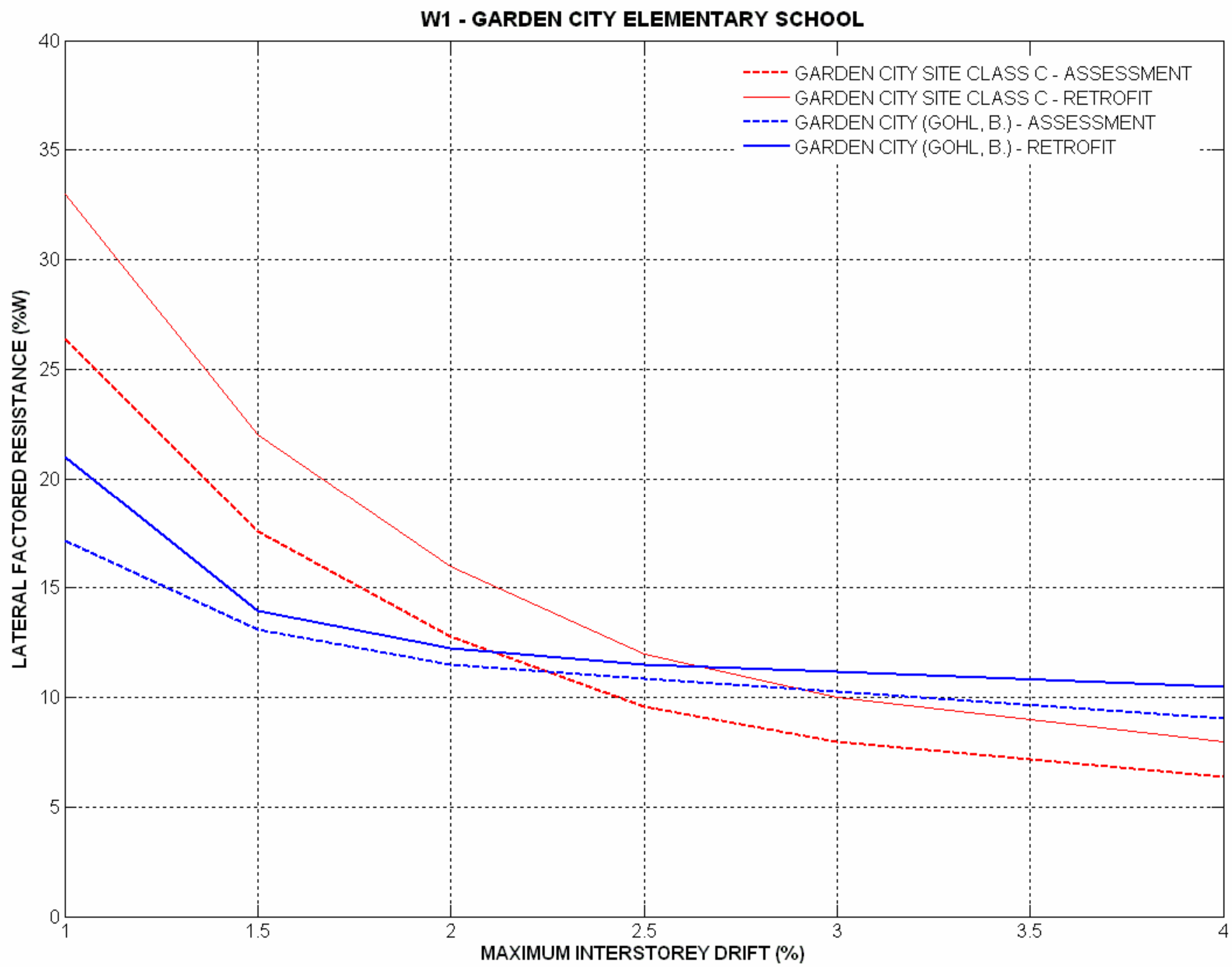
(h) DESRA soil column properties

Nonlinear Site Response Analysis: Dynamic Soil Properties for Garden City School

Layer No.	Soil Type	Thk. (ft)	Unit Weight (pcf)	Vs (ft/sec)	Gmax (psf)	Drained Sand shear strength (psf)	Undrained Shear Strength (psf)
1	silt	3.28	114.4954	229.6	187818.3486		1781.608
2	silt/sand	3.28	114.4954	229.6	187818.3486		1760.099
3	silt/sand	12.136	117.6758	360.8	476679.5617		2305.904
4	silty sand	17.384	120.8563	590.4	1310895.413	823.2858	
5	sand	13.12	120.8563	688.8	1784274.312	1248.72	
6	sandy silt	10.496	120.8563	770.8	2234388.863	1578.088	
7	sandy silt	30.504	120.8563	721.6	1958251.172	2034.673	
8	silt	22.96	114.4954	688.8	1690365.138		1650.754
9	sand	22.96	120.8563	744.56	2084849.683		2000.207
10	silt	27.88	114.4954	715.04	1821608		2384.947
11	sand	34.44	120.8563	787.2	2330480.734		2862.078
12	silt	57.4	114.4954	918.4	3005093.578		3550.931
13	silt	42.64	114.4954	1033.2	3803321.56		4268.435
14	sand/silt	62.32	120.8563	1295.6	6312730.148		5075.803
15	silt	75.44	114.4954	1230	5390194.954		6118.417
16	silt	141.04	114.4954	1410.4	7087267.89		7671.051
17	sand/silt	37.72	120.8563	1508.8	8561279.918		8986.182
18	silt	106.6	114.4954	1328.4	6287123.394		10054.3
19	silt	262.4	114.4954	1804	11594908.26		12700.84

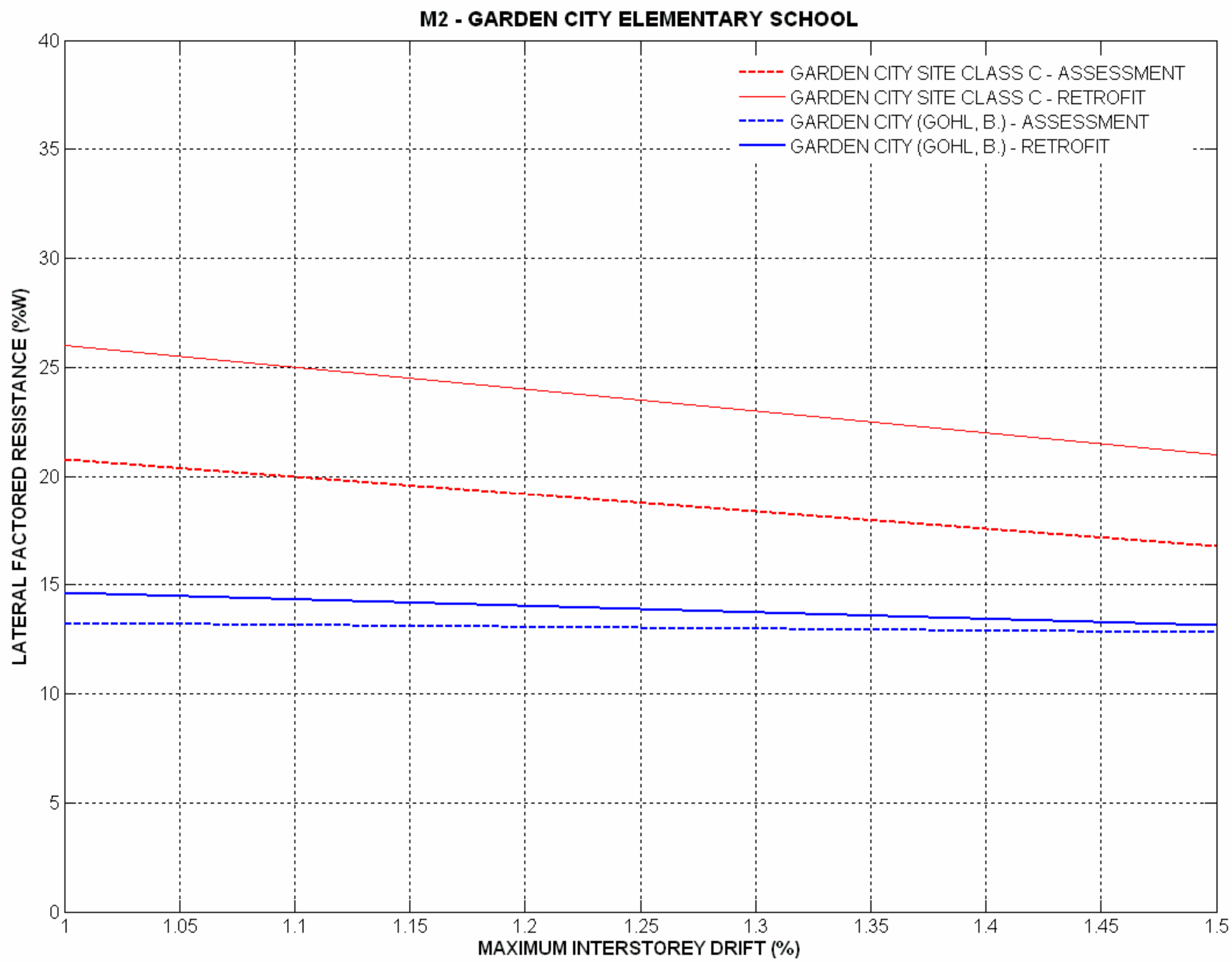
(3) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1


W1 - GARDEN CITY ELEMENTARY SCHOOL
GARDEN CITY SITE CLASS C - ASSESSMENT
GARDEN CITY SITE CLASS C - RETROFIT
GARDEN CITY (GOHL, B.) - ASSESSMENT
GARDEN CITY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.5
2
2.5
3
3.5
4

(3) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for M-2


M2 - GARDEN CITY ELEMENTARY SCHOOL
GARDEN CITY SITE CLASS C - ASSESSMENT
GARDEN CITY SITE CLASS C - RETROFIT
GARDEN CITY (GOHL, B.) - ASSESSMENT
GARDEN CITY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX D

DETAILED RESULTS FOR
JAMES PARK ELEMENTARY SCHOOL

CONTENTS

(1) Pacific Geodynamics Inc. Report (DESRA Analysis)

(a) Project draft report dated December 24, 2006

(2) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1 (total stress)

(b) Graph - lateral factored resistance versus drift for M-2 (total stress)

(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis)</u>

(a) Project draft report dated December 24, 2006

14 Sherwood Place,
Delta, B.C. V4L 2C7 CANADA
(604) 943-0350 fax (604)943-6190
Email: pgigohl@dccnet.com

December 24, 2006

Pomeroy Consulting Engineers Ltd.
Suite 400 – 6450 Roberts Street
Burnaby, B.C.
V5G 4E1

Attention: Mr. Peter Kiddie, P.Eng.

Re: James Park Elementary School
1761 Westminster Avenue, Port Coquitlam, B.C.
DRAFT Report on Geotechnical Aspects of Seismic Design

Dear Sirs,

This **draft** report summarizes geotechnical analyses and recommendations prepared by Pacific Geodynamics Inc. (PGI) pertaining to seismic retrofit design of James Park Elementary School. It is intended that these recommendations be used to facilitate seismic structural design of the school facilities to be carried out by Pomeroy Engineering Consultants Ltd. **The present report is submitted in draft form to permit comments to be received from all parties (Pomeroy Engineering, Coquitlam School District 43 and GeoPacific Consultants) involved prior to the report being finalized.**

The following work tasks were carried out during the present study:

- Review of existing geotechnical information for the site provided by GeoPacific Consultants
- Assistance to GeoPacific in planning and conducting additional geotechnical site investigation, including electronic cone penetration testing, Becker drill penetration testing, and downhole seismic testing
- Carrying out nonlinear, one dimensional site response analyses of earthquake wave propagation at the site to assess cyclic shear stresses versus depth, ground surface acceleration response, liquefaction triggering potential of predominantly sand, silt and sand and gravel soils present at the site, and ground deformation (vertical and lateral) potential
- Review of potential methods of ground improvement for the foundation soils to mitigate soil liquefaction potential, reduce post-seismic ground deformations, and provide foundation underpinning.

EXECUTIVE SUMMARY

Geotechnical engineering analysis of the earthquake response of the James Park Elementary School site, Port Coquitlam, B.C. has been carried out by Pacific Geodynamics Inc., working in conjunction with Pomeroy Consulting Engineers Ltd. and GeoPacific Consultants. Seismic input motions representative of a 2% probability of exceedance in 50 years were considered, consistent with seismic design provisions of the 2005 National Building Code of Canada.

Available geotechnical drill hole and geophysical data for the site indicate that the upper 15m or so of the soil profile contains locally loose granular soils which are potentially liquefiable under design levels of seismic shaking. The upper 6m of the soil profile contains extensive thicknesses of liquefiable material. The occurrence of liquefaction in looser granular soil layers will result in post-seismic settlements. These will adversely impact the school building since building footings are expected to settle differentially. Analysis indicates potential ground settlements of up to 400 mm (for the loosest soil conditions considered likely for the site) and differential vertical settlements of up to 350 mm in absence of ground improvement. Differential settlements will occur because of soil variability across the site. Available geotechnical data for the site indicates a wide range of granular soil density conditions, which exacerbates the potential for post-seismic differential settlement.

Under level ground conditions, cyclic softening of the soils above the 15 m depth could result in post-seismic lateral ground displacements of up to 50 to 60 mm (depending on the input ground motion). Since the magnitude of lateral ground movement could also vary over the site, it is recommended that building footings be tied together horizontally to minimize the effects of differential lateral ground displacement.

Seismic wave propagation analyses carried out to model the effects of earthquake shaking at the site assumed vertical shear wave propagation. Seismic input motions were applied at the 20 m depth which were judged to be representative of “firm ground” consistent with the provisions of the 2005 NBCC. Energy absorbing boundaries at the base of the model were considered. The available drill hole and geophysics data were used to construct a soil layer model (with estimates of soil shear strength, dynamic stiffness and pore water pressure generation characteristics). The input motions were then propagated upwards through the overburden soils. Computed ground surface response spectra (5% damping case) and surface acceleration time histories were provided to Pomeroy Engineering, working in conjunction with the University of British Columbia, who used these to calculate seismic shear levels to be used in structural retrofit design of the school building

In view of the potential for significant vertical and lateral ground movements under the building envelope due to the presence of the problematic soils above the 15 m depth, two approaches are considered viable to provide adequate structural safety to the school building. One would involve a purely structural retrofit in which footings are tied together so that relative lateral movements are minimized. In addition, floor slab thickening will likely be required so that differential vertical settlements over the building envelope are reduced due to the structural rigidity of the slab. In effect, a “raft slab” would be created. Bearing pressures under the slab will also need to be reduced. Preliminary analysis for relatively small pad footings (up to 2m in width) indicates that maximum values of 35 kPa should be used in design so that bearing failure is not induced in the event of soil liquefaction. This value should be checked if larger raft slab foundations are used, once details of foundation layout are known more precisely.

As an alternate to the creation of an effective raft slab, ground improvement may be considered to stabilize the soils above the 6 m depth. Compaction grouting (CG) is judged to be most feasible inside the school building due to the limited headroom environment. The CG technique is expected to mitigate the liquefaction susceptibility of looser granular soil layers through a densification process, as well as reinforcing siltier soil layers, by creating soil-grout columns that are expanded out into the soil medium. The use of CG is expected to underpin building footings and reduce post-seismic settlements of footings. Tying footings together would still be recommended to minimize independent lateral movements.

If economic analysis indicates that either the structural retrofit (raft slab) or CG method are feasible from a cost point of view, further analysis of the dynamic interaction between the vibrating ground and the soil-cement columns (including densification of the surrounding ground within a particular grid of CG columns) is recommended. Field testing using Becker Density Testing is also recommended to demonstrate the densification effect of the method, both within a test section prior to production densification, and during production densification.

1.0 SITE DESCRIPTION

The school site is located in Port Coquitlam at 1761 Westminster Avenue. The school is located on a relatively flat piece of land directly north of Westminster Avenue. An aerial view of the school is shown in Figure 1. The property is bordered on the west by an existing grass field, to the east by residential homes, and a gravel field plus James Park Annex to the north. Parking is located directly west of the building with a landscaped area fronting the school on Westminster Avenue.

2.0 ADDITIONAL FIELD INVESTIGATION

In addition to the previous auger hole and dynamic cone penetration testing carried out at the site under the supervision of GeoPacific Consultants (as summarized in their report to School District 43 dated May 11, 2006), the following additional geotechnical site investigation was carried out during the present study:

- Electronic cone penetration testing (cpt) at 3 locations adjacent to auger holes AH-1, AH-2 and AH-3 (see Figure 1). Maximum depths of penetration of 3.25m were achieved due to the high gravel content of the subsoils which restricted cone advance. Downhole seismic testing was also carried out at each cpt hole location.
- Open casing, Becker drilling involving the advance of a 168 mm OD open end, double walled casing using a percussive air hammer. Five open Becker holes (BH06-1 to BH06-5) were advanced at the locations shown in Figure 1. During casing advance, air was forced down the annulus of the casing resulting in soil samples being blown up the inside of the casing for collection by a field technician supplied by GeoPacific and logging of the soil strata encountered. It was found that the open end Becker casing was able to be driven through dense sand and gravel strata encountered at the site to maximum depths of 25.0 m at the BH06-1 location. At some locations, casing advance was halted at the surface of a dense gravel layer located at the 15m depth. The presence of this dense gravel layer made drilling to larger depths difficult. Four open end Becker holes (BH-2 to BH-5) were advanced to depths varying between 15.8 m and 20.4 m.
- Closed end, Becker Penetration Testing at all locations excluding BH06-2 location involving the advance of the above casing but fitted with a closed end drill bit. Energy measurements of the air hammer used to drive the casing were made by PGI at the BH06-1 and BH06-4 locations by attaching strain gauges and accelerometers to the top of the casing. Strain gauge/accelerometer output recorded during selected impacts of the air hammer were recorded using a high speed data acquisition system, and this data further processed to compute the input energy applied to the top of the casing. The input energy of the Becker hammer is important in order to correct Becker blow counts (number of blows required of the hammer to advance the casing 0.3m) to a standard hammer energy efficiency (3.3 kN-m) for typical Becker drills. From this corrected blow count, correlations between corrected Becker blow count (Nbc) and Standard Penetration Test blow count (N_{60}) were made. The latter is commonly used within geotechnical engineering practise to infer relative density and liquefaction resistance of granular soils (sands, gravels and non-plastic silts). The energy measurements are summarized in Figure 2, which plots average energy per blow versus blow count (number of blows required to advance the casing a distance of 0..3 m). The figure indicates that a reasonable average energy of 2.7 kJoules was applied to the casing based on all the measurements.
- A downhole seismic survey carried out at the BH06-1 location by Frontier Geosciences. This required grouting in a PVC casing after completion of Becker drilling. Geophones were then be placed down the casing and used to detect shear wave and compressive wave arrivals created by impacting a shear beam at the ground surface. Details of the testing methodology are provided in the Frontier Geosciences report presented in Appendix A. The casing was placed to a maximum depth of 25.0 m.

The electronic cone penetration testing and downhole seismic testing at the cpt hole locations were carried out by Dynamic Drilling Inc. The results of this testing are presented in Appendix B. The Becker drilling and soils logging was carried out under the full time supervision of GeoPacific Consultants Ltd. Pacific Geodynamics was on site during closed end advance of BH06-1 and BH06-4 and and carried out the Becker hammer energy measurements. Drill hole logs prepared by GeoPacific are presented in Appendix C.

3.0 GENERAL SOIL AND GROUNDWATER CONDITIONS

We have reviewed the May 11, 2006 geotechnical report prepared by GeoPacific Consultants for the site as well as the results of more recent drilling and seismic surveys in order to determine a reasonable average soil stratigraphy to be used in analysis of earthquake wave propagation at the site. The May 11, 2006 GeoPacific auger hole and dynamic cone penetration test data is presented in Appendix D.

Available surficial geology maps for the area (reference Geological Survey of Canada Map 1484A) indicate the site is underlain by post-glacial lowland and mountain stream deltaic, channel fill and overbank sediments up to 15m thick. The latter overlie glaciated Capilano Sediments consisting of raised deltaic and channel fill sand to cobbly gravels up to 15m thick deposited by glacial meltwater streams. These sediments were deposited in the Pleistocene epoch during the last glaciation. The deltaic and channel fill materials are commonly underlain by silt/clay materials.

The above surficial geology is generally consistent with soil conditions encountered at the site. Based on the drilling and electronic cone penetration testing carried out at the site, the following average soil profile is inferred to exist under the school building:

- Variable density (loose to dense), granular fills consisting typically of sand or sand and gravel with thicknesses of 2m or less
- Variable density (typically loose to compact), fine sand or sand and silt mixtures with fines contents of up to 50% (based on auger hole data and inferred fines contents from cpt interpretation), interbedded with compact to very dense, coarse grained sand and gravel extending down to the 5 to 6m depth
- Compact to dense, interbedded sand, gravel, or sand and gravel mixtures with occasional loose to compact zones extending to about the 15m depth
- Compact to very dense, interbedded, sand, gravel or sand and gravel mixtures extending to the maximum depth (25m) of the Becker drill holes

The groundwater table was encountered at the 1.5 to 3.3m depth during the various field investigations.

The above granular soil profile has strong density variations which will impact footing bearing capacity and amplify post-seismic differential settlements across the building footprint in absence of ground improvement. Seismically induced liquefaction triggering is considered to be of greatest concern in the soils above the 6m depth, but local zones of deeper liquefaction may occur at larger depths, as discussed in Section 7.

The electronic cone penetration test (cpt), dynamic cone penetration test (dcpt) and Becker blow count information (closed end and open end) have been used to infer equivalent Standard Penetration Test N_{60} values corrected to a vertical effective pressure of 1 atmosphere (termed an $N_{1,60}$ value) for the various granular soil layers. The $N_{1,60}$ values have been used to estimate relative densities and cyclic liquefaction resistance during earthquake loading following methods used in standard geotechnical engineering practice. The N_{60} values have been estimated using the following procedures:

- Assuming a ratio of cone tip resistance (in units of bars where 1 bar = 100 kPa) to N_{60} of 6 in predominantly clean sand or sand and gravel layers.
- Assuming a 1:1 correspondence between dcpt blow count and N_{60} for shallow depths
- Pro-rating a measured closed end Becker blow count by the ratio of average input hammer energy (= 2.7 kJ based on Figure 2) to a recommended standard input energy of 3.3 kJ based on Sy and Campanella (1994)
- Assuming a corrected, closed end Becker blow count Nbc equal to 1.5 times N_{60} based on previous correlations between Becker hammer blow counts (corrected to a standard energy level of 3.3 kN-m) and Standard Penetration N_{60} values carried out in the Fraser River Delta in predominantly sand subsoils (Sy and Campanella, 1994)
- Assuming an average ratio between a closed end Becker blow count and an open end Becker blow count of 1.0 for open end blow counts less than 10 and a ratio of 1.2 for blow counts greater than 10 based on the correlation shown in Figure 3 established from data at the site. The ratios adopted are judged to be appropriately conservative given the large scatter in the data. This ratio reflects the influence of soil stresses on the tip of a closed end casing which increase driving resistance and blow counts relative to an open end casing. This ratio will vary depending on the relative contributions of external casing friction and casing tip resistance, which is site specific and depth dependent. Gravel content of the soil will also influence the ratio. For low blow counts in loose sandy soils, dynamic soil liquefaction at the casing tip likely occurs during Becker casing advance, resulting in very small casing tip resistance and a blow count ratio of 1.0.
- Using the relationship $N_{1,60} = C_N N_{60}$ where C_N is a stress level correction factor. At shallow depths, C_N should not exceed a value of 1.7 (Idriss and Boulanger, 2006).

Inferred $N_{1,60}$ values versus depth in the granular soil layers are plotted in Figure 4 based on the dcpt and cpt data, Figure 5 based on the open end Becker penetration data, and Figure 6 based on the closed end Becker penetration data. No correction for estimated fines content has been made to convert these blow count values to “equivalent clean sand” values. Higher silt contents are present in the upper 5m or so of the soil profile. Figures 4 to 6 indicate a predominance of loose to compact soils above the 6m depth ($N_{1,60}$ less than 30), interbedded with denser materials. Higher blow counts with $N_{1,60}$ values in excess of 30 reflect denser gravel layers and are indicated by the closed end Becker blow count data, as well as the dcpt and cpt data. In the latter case, particle size effects may indicate misleadingly high blow counts.

Shear wave velocities (Vs) versus depth derived from downhole seismic methods are shown plotted in Figure 7. Lower velocities of around 120 m/sec are indicated in the upper 4m of the soil profile at the test locations, indicative of the lower densities of the granular soils and their high seismic liquefaction potential.

4.0 “FIRM GROUND” SEISMIC INPUT MOTIONS

It is necessary to define input earthquake motions at “firm ground” level in order to carry out analysis of earthquake wave propagation for a particular site. These input motions will depend on seismic risk levels being considered for design. In the case of James Park Elementary School, a seismic risk level having a 2% probability of being exceeded in 50 years has been adopted, consistent with the provisions of the 2005 National Building Code of Canada (NBCC).

The Geologic Survey of Canada (2003) report defining seismic ground motion parameters to be considered throughout Canada for the above seismic risk level states that “firm ground” is defined by materials having shear wave velocities in the range of 360 to 750 m/sec. Thus input earthquake motions selected for the study were placed at the 20 m depth where the soil materials had shear wave velocities in excess of 500 m/sec based on the downhole seismic profiling. The 2005 NBCC defines these firm ground conditions as “Site Class C” soil conditions, representative of very dense soil, or soft rock.

The earthquake input motions (specified as horizontal accelerations versus time, termed an accelerogram) selected for seismic wave propagation analysis were supplied by the University of British Columbia Dept. of Civil Engineering (UBC) and the Transit Bridge Group (TBG) who are actively engaged in seismic research pertaining to seismic design of school structures. The input motions were recorded at the ground surface during previous earthquakes at a variety of sites in California on soil conditions considered representative of Site Class C soils. The input firm ground motions were scaled from the original accelerograms so that after scaling their peak spectral velocity (PSV) averaged over the 0.5 to 1.5 second period range matched a target PSV (= 55.4 cm/sec) specified by the Geologic Survey of Canada (2003) for the Port Coquitlam area. The input accelerograms adopted for the present study and the scaling factors applied to the original accelerograms are presented in Table 1. The peak firm ground acceleration (PGA) and average peak spectral velocity (PSV) over the 0.5 to 1.5 second period range prior to scaling for each input motion are also presented in the table. Elastic response spectra computed for 5% structural damping after scaling of each accelerogram are shown in Figure 8.

Table 1
Input Firm Ground Motions

INPUT ACCELEROGRAM	SCALE FACTOR	PSV (cm/sec)	PGA (g's)
(1) Sherman Oaks – 105E	1.25	44.1	0.214
(2) Wadsworth - 235E	1.14	48.4	0.303
(3) Wadsworth - 325E	1.32	42.1	0.389
(4) Canyon Country – 0E	0.83	67.0	0.396
(5) Saratoga - 0E	0.8	69.2	0.504
(6) Canoga Park - 196E	0.71	77.8	0.434
(7) Canoga Park - 106E	1.06	52.3	0.350
(8) Pacoima Kagel – 90E	0.84	66.3	0.301
(9) 12520 Mulholland Dr. - 35E	1.13	49.2	0.588
(10) Gilroy Gavilon College - 67E	1.4	39.7	0.356

The above earthquakes have been recorded during earthquakes with magnitudes in the range of 6.5 to 7.5 and are considered representative of earthquake magnitudes likely to affect the school site for the seismic risk levels being considered.

5.0 SITE RESPONSE ANALYSIS – GENERAL METHODOLOGIES

Earthquake wave propagation at the site was assumed due predominantly to vertically propagating shear waves for purposes of assessing soil liquefaction and deformation potential and estimating horizontal inertial forces (base shear) acting on the school superstructure. This assumption is consistent with seismic design practice in the Vancouver Lower Mainland. The one dimensional analysis program DESRA2C developed by Lee and Finn (1978) was used for this purpose. The program models a column of soil elements subject to seismic base excitation. The nonlinear, cyclic shear stress-shear strain response of individual soil elements at a particular depth are modeled, including the effects of pore water pressure generation if specified. Level ground conditions are assumed, i.e. without the effects of imbalanced shear stress acting within a soil element due to sloping ground conditions or the effects of local stresses caused by building footings. A level ground condition is judged to be a reasonable assumption over the James Park Elementary School site.

Where pore pressure generation is not considered in softening a soil element's stress-strain "backbone" curve, this is termed a "total stress" analysis. Use of a total stress analysis leads to maximum prediction of shear stresses, accelerations and inertial forces transmitted to the school structure through the soil profile. Where softening of the soil stress-strain backbone curve occurs over time due to pore pressure generation, this is termed an “effective stress” analysis (since soil shear strength depends on total stress less pore pressure, termed effective stress). Use of an “effective stress” analysis results in lower transmission of shear stresses and accelerations through the soil profile, and lower transmitted inertial forces to the school structure. However, since greater softening of the foundation soils occurs relative to the total stress analysis, larger foundation soil deformations (vertical and lateral) develop. An average soil layering profile was considered in the analysis, as summarized in Table 2.

Based on the measured shear wave velocity profiles (Vs), the small strain shear stiffness of a particular soil element (Gmax) was computed as $Gmax = \Delta Vs^2$ where Δ is the total mass density of the soil at a particular depth. The maximum shear strength of a soil element was computed based on the assumption of fully drained strengths (using estimated peak friction angles, lateral stress K_0 coefficients, and vertical effective stresses. A summary of soil properties used in total stress, site response analysis is presented in Table 2.

A summary of soil properties used in effective stress (with pore pressure generation), site response analysis is presented in Table 3.

Pore pressure generation constants used in the DESRA2C analysis were calibrated for each soil layer to achieve a specified degree of positive pore pressure normalized with respect to the effective vertical stress at the depth being considered (termed a pore pressure ratio, PPR). A PPR value of 1.0 implies that complete soil liquefaction has occurred. Ten effective cycles of shaking at a specified cyclic shear stress level divided by the vertical effective stress (or cyclic stress ratio, CSR) were used in each calibration analysis. The number of effective cycles of shaking is considered representative of the number of equivalent cycles of shaking for a magnitude 7earthquake. For granular soil layers, the critical CSR value to cause complete liquefaction (PPR = 1.0) was based on correlations between $N_{1,60}$ and liquefaction triggering considering magnitude 7 earthquakes, with appropriate accounting for the estimated fines content of the material (Seed et al, 2003; Idriss and Boulanger, 2006).

Table 2 - Site Response Analysis (Total Stress Analysis)

Layer No.	Soil Type	Layer Thk. (m)	Vs (m/sec)	Total Unit Weight (kN/cu.m.)	Gmax (MPa)	Inferred** $N_{1,60}$	Shear Strength* (kPa)
1	Sand (Fill)	1.5	125	18	28.7	10	5
2	Silty Sand	1.5	135	18	33.4	5	13
3	Silty Sand	2.0	150	18	41.3	5	20
4	S&G	3	300	22	201.8	50	36
5	S&G	2	300	21	192.7	30	45
6	S&G	2	330	21	233.1	20	51
7	S&G	3	330	20	222.0	10	58
8	S&G	5	500	22	560.7	50	89

* Based on the assumption of drained shear strengths. Assume groundwater table at 1.5m depth below existing ground surface.

** Values selected at lower bound to mid-range of values shown in Figures 4 to 6

S&G = sand and gravel

It is important to note that the seismic input motions specified by the UBC research group and the Geologic Survey of Canada are considered to be representative of motions occurring at the ground surface on a firm ground “outcrop”. Since firm ground representative of Site Class C conditions occurs at relatively large depth (20 metres), then some accounting for seismic wave energy dissipation into deeper materials below the 20 m depth must be made. Application of an interior seismic excitation combined with consideration of an energy absorbing bottom boundary reduces the effective seismic energy transmitted to the overlying soil layers. An energy absorbing bottom boundary was used in all DESRA2C analyses presented herein based on the theory presented by Lee and Finn. The energy absorption characteristics of the lower boundary was based on an average shear wave velocity of 700 m/sec, consistent with the geophysical testing for the site.

Table 3 - Site Response Analysis (Effective Stress Analysis)

Layer No.	Soil Type	Layer Thk. (m)	Vs (m/sec)	Total Unit Weight (kN/cu.m.)	Gmax (MPa)	Inferred*** $N_{1,60}$	Shear Strength (kPa)	Pore Pressure Generation
1	Sand (Fill)	1.5	125	18	28.7	10	5	No
2	Silty Sand	1.5	135	18	33.4	5	13	Yes (PPR = 1.0 for CSR = 0.11)**
3	Silty Sand	2.0	150	18	41.3	5	20	Yes (PPR = 1.0 for CSR = 0.11)**
4	S&G	3	300	22	201.8	50	36	No
5	S&G	2	300	21	192.7	30	45	Yes (PPR = 1.0 for CSR = 0.5)
6	S&G	2	330	21	233.1	20	51	Yes (PPR = 1.0 for CSR = 0.23)
7	S&G	3	330	20	222.0	10	58	Yes (PPR = 1.0 for CSR = 0.14)
8	S&G	5	500	22	560.7	50	89	No

* Based on the assumption of drained shear strengths. Assume groundwater table at 1.5m depth below existing ground surface.
** Pore pressure generation estimates based on an average fines content of 15%.
*** Values selected at lower bound to mid-range of values shown in Figures 4 to 6
S&G = sand and gravel

6.0 SITE RESPONSE ANALYSIS – TOTAL STRESS ANALYSIS RESULTS

Site response analysis results using total stress approaches are presented in the following figures:

Figure 9 – Peak ground surface acceleration versus depth.

Figure 10 – Cyclic stress ratios (CSR) versus depth. The CSR at a particular depth is computed as 0.65 times the peak cyclic shear stress, divided by the vertical effective overburden stress, consistent with geotechnical engineering practice.

Figure 11 – Peak cyclic shear strain on the horizontal plane versus depth.

Figure 12 – Elastic response spectra (peak spectral acceleration versus structural building period for 5% structural damping). The latter were obtained from computed horizontal accelerations at

the 0.75m depth (the approximate base depth of footings used to support the school structure) using the theory derived from a single degree of freedom oscillator. The computed spectra are compared with generic spectra provided in the 2005 NBCC for Site Class C and Site Class D soils.

The figures present analysis results for all 10 seismic input motions.

Computed acceleration time histories at the 0.75m depth have been provided to the UBC/TBG research group for further input into a structural model used to compute seismic base shears transmitted to buildings representative of those at James Park Elementary School. We understand that this seismic base shear information will be provided separately to Pomeroy.

Ratios of the peak ground acceleration near the bottom of the soil column model (19.9m depth) used in DESRA2C to the peak input base acceleration are presented in Table 4 for each seismic input motion. The ratios are in the range of 0.77 to 0.93, showing the effect of the energy absorbing bottom boundary used in the model to reduce some of the input seismic energy transmitted to the overburden soils.

Table 4
DESRA2C Analysis Results (Total Stress Analysis)
Ratios of Peak Accel. Near Base of Model (Amax) to Peak Input Base Accel. (PGA)

INPUT ACCELEROGRAM	PGA (g's)	Amax (g's)	Amax/PGA
(1) Sherman Oaks – 105E	0.267	0.225	0.84
(2) Wadsworth - 235E	0.345	0.332	0.96
(3) Wadsworth - 325E	0.513	0.406	0.79
(4) Canyon Country – 0E	0.329	0.305	0.93
(5) Saratoga - 0E	0.403	0.334	0.83
(6) Canoga Park - 196E	0.308	0.279	0.90
(7) Canoga Park - 106E	0.371	0.286	0.77
(8) Pacoima Kagel – 90E	0.253	0.221	0.87
(9) 12520 Mulholland Dr. - 35E	0.664	0.576	0.87
(10) Gilroy Gavilon College - 67E	0.498	0.437	0.88

Examination of the above figures leads to the following observations:

- De-amplification of ground accelerations from the base of the soil column through the overlying denser materials to about the 10 m depth
- Slight amplification of ground accelerations above the 10 m depth due to the limited shear strength of the near surface soils
- Peak ground surface accelerations in the range of 0.26 to 0.37 g
- Peak CSR's in the range of 0.18 to 0.27 below the water table (assumed at the 1.5m depth). The CSR values are used to estimate liquefaction triggering potential for the granular soil layers below the water table based on correlations between CSR and $N_{1,60}$ (Seed et al, 2003; Idriss and Boulanger, 2006).
- Computed response spectra (on average) show broad agreement for structural periods greater than 1.0 second with the 2005 NBCC Site Class D design spectrum. Class D soils are defined in the 2005 NBCC as "stiff soils" and have characteristics similar to those that exist at the site below about the 6m depth. Higher and lower spectral response is indicated for some of the input earthquake records relative to the site Class D spectrum for periods less than 1.0 second. The ongoing work by the UBC/TBG research

group, however, indicates that consideration of peak spectral velocity over the period range of most interest (0.5 to 1.5 seconds) provides a better measure of seismic base shear transmitted to school building structures and building damage potential. Their analysis of base shears to be considered in design using computed near surface acceleration time histories is considered to supercede base shears computed using traditional modal analysis and acceleration response spectra. The latter are provided strictly for comparison with 2005 NBCC design spectra.

7.0 SOIL LIQUEFACTION TRIGGERING

Based on the computed cyclic stress ratios in the range of 0.18 to 0.27 above the 20 m depth, localized loose to compact sand or sand and gravel layers with $N_{1,60}$ values less than about 22 could undergo seismic liquefaction. This is based on liquefaction triggering curves for clean sands (fines contents less than 5%) provided by Idriss and Boulanger (2006) based on a design earthquake magnitude 7.0. The required $N_{1,60}$ values in clean granular soils required to prevent liquefaction triggering under the design ground motions are shown plotted versus depth in Figures 4 to 6. Examination of these figures (where inferred $N_{1,60}$ values are less than the required $N_{1,60}$ values) indicates liquefaction triggering is potentially extensive above the 4 to 6m depth, but is quite variable across the site. The open and closed Becker blow count data also indicates potential zones of liquefaction at larger depths in excess of 6m, extending down to the 15m depth. However, the data suggest that these deeper zones of liquefaction occur in sporadic lenses up to 3m thick and are laterally discontinuous across the site.

The occurrence of granular soil liquefaction above the 6 m depth is considered to be of greatest concern to seismic performance of James Park Elementary School insofar as this influences post-seismic settlements of building footings, as well as lateral ground displacement potential. The occurrence of localized deep-seated liquefaction (in likely discrete discontinuous layers) is judged to not be as serious for adequate school foundation performance if a surface layer of densified soil can be created to mask the effects of liquefaction at larger depths. Ishihara (1985) suggests that for relatively level ground sites subjected to peak ground surface accelerations of about 0.3 g (indicated from the previous site response analysis), limited surface expression of deep seated liquefaction will occur if the thickness of a non-liquefiable surface layer exceeds about 6 m. This is caused by bridging action which reduces the amount of settlement at the ground surface that causes damage to shallow foundations. Alternatively, it may be possible to design a structural solution wherein a foundation raft slab is created which can withstand post-liquefaction differential settlements under the building envelope. Various foundation remedial options are discussed further in Section 11.

8.0 POST-SEISMIC GROUND SETTLEMENTS

Using empirical correlations between Standard Penetration Test $N_{1,60}$ values (inferred from previous cpt, dcpt and Becker penetration test data) and CSR, estimates of post-seismic volumetric recompression of liquefied soils have been made based on the method by Wu (2003). CSR values from the total stress site response analysis were used, using CSR values at the upper end of the range of values plotted in Figure 10. No correction to the $N_{1,60}$ value for more silty soils (fines content greater than about 10%) was made in the evaluation, since the soil sampling indicated that fines contents were generally small below the 5 to 6m depth, and because fines contents above this depth were highly variable, including locations where fines contents were 10% or less.

Figures 4 to 6 indicate considerable spatial variability in $N_{1,60}$ values due mainly to differences in soil density and gravel content, as well as method of interpreting the $N_{1,60}$ value from the various penetration tests carried out. It was therefore decided to treat the $N_{1,60}$ values at a particular depth as a random probabilistic variable and to carry out an assessment of the influence of this variability on post-seismic settlement. From this, a better assessment of potential magnitudes of differential settlement under the building envelope could be made.

The soil profile was divided into 7 layers below the groundwater table down to the 20 m depth. The mean and standard deviation of $N_{1,60}$ data for a particular layer was estimated from Figures 4 to 6. The layer thicknesses and the mean and standard deviation $N_{1,60}$ values selected for the analyses are shown in Table 5. These correspond to values judged to be reasonable "lower bound" and "upper bound" values based on the scatter in the inferred $N_{1,60}$ values. A Monte Carlo simulation was then used to randomly select values of $N_{1,60}$ within a particular layer according to its defined statistical distribution. The random $N_{1,60}$ value for a particular layer was then used with the deterministic value of CSR to calculate a vertical strain potential for the layer using the method of Wu (2003). The strain potentials multiplied by the layer thicknesses were then used to compute the cumulative settlement down to the 20 m depth. The Monte Carlo simulation process was repeated for N trials (convergence of results occurred beyond about 100,000 trials – 1 million trials were used in the results presented herein) and a count (n_I) made of settlements computed within a certain settlement range (S_I). The frequency of occurrence f of settlements within a particular range was then computed as n_I / N. A cumulative frequency plot was then computed over the entire range of settlements, as shown in Figure 13.

Figure 13 suggests that a reasonable maximum differential settlement to be considered in seismic design of the school building due to "free field" shakedown settlements is about 300 to 350 mm. This is based on a computed upper bound settlement of 300 to 400 mm (95% or greater cumulative probability) and a computed lower bound settlement of about 0 to 50 mm (less than 5% cumulative probability). The computed median 50th percentile settlement is in the range of 120 to 210 mm. Neglecting settlements between the 5-8 m and from the 15-20 m depth, corresponding to locally dense layers, the above differential settlement corresponds to a maximum average vertical strain of 2.8% over the remaining layers. Based on the Wu (2003) correlations and average CSR values of 0.225 (see Figure 10), this average vertical strain value corresponds to an average $N_{1,60}$ of about 12 in these looser layers. In the writer's opinion, this is consistent with lower bound ranges of the data presented in Figures 4 to 6 and would contribute to upper bound estimates of post-liquefaction settlement. Where the soil layers considered are locally much denser, then the amount of post-liquefaction settlement would be close to zero, and would lead to the computed differential settlement using the Monte Carlo simulation.

In absence of ground improvement under the building envelope, it is recommended that the above amount of differential settlement be assumed to occur over a critical dimension of the building, for example, between columns or from one edge of a floor slab to its centre.

Table 5
Post-Liquefaction Settlement Parameters Used in Monte Carlo Simulation

Layer No.	Soil Type	Layer Thk. (m)	UB Mean $N_{1,60}$	UB SD $N_{1,60}$	LB Mean $N_{1,60}$	UB SD $N_{1,60}$
1	Silty Sand	1.5	25	20	30	25
2	Silty Sand	2.0	25	20	40	35
3	S&G	3	50	25	50	25
4	S&G	2	30	25	50	25
5	S&G	2	30	25	30	20
6	S&G	3	15	10	20	10
7	S&G	5	50	30	50	30

Notes: UB = upper bound (estimated)
LB = lower bound (estimated)
SD = standard deviation of inferred $N_{1,60}$ values about the mean
S&G = sand and gravel

The above differential settlement estimate excludes additional differential settlements that will occur under concentrated footing load. The magnitude of increased post-seismic settlement under footings is discussed in Section 10.

9.0 SITE RESPONSE ANALYSIS – EFFECTIVE STRESS ANALYSIS RESULTS

Site response analysis using effective stress approaches with consideration of pore pressure generation in some of the soil layers (reference Table 3) was carried out for the following reasons:

- To provide surface acceleration time histories to be used in analysis of seismic base shear carried out by UBC/TBG. It was expected that these motions would not provide as high a measure of base shear compared to the previous total stress analysis, since ground motions are attenuated through the soil column due to pore pressure generation.
- To indicate pore pressure generation potential within the soil column and zones potentially requiring ground improvement to mitigate liquefaction triggering.
- To indicate potential magnitudes of seismic lateral ground displacement versus depth in absence of ground improvement.

Site response analysis results using effective stress approaches are presented in the following figures:

Figure 14 – Pore pressure ratios versus depth.

Figure 15 – Lateral ground surface displacements versus time relative to seismic base motion displacements.

Examination of the above figures indicates:

- The occurrence of liquefaction in looser sand layers (PPR = 1.0) above the 5 to 6m depth and at larger depths within the soil profile where locally loose layers are present
- Post-seismic lateral displacements of up to 0.17 ft. (52 mm) at the ground surface, depending on the input earthquake record being considered. As noted earlier, the computed lateral displacements are for level ground conditions and would be increased if ground slope were to exist.

The effective stress analyses indicate that lateral ground displacements generated mainly above the 6 m depth are possible in absence of ground improvement. It is expected that these lateral displacements could occur differentially between building columns due to variations in soil conditions, as well as due to traveling ground wave effects where purely vertical shear wave transmission does not occur. Dissipation of pore pressures generated by seismic shaking will also result in post-earthquake consolidation (settlement) of liquefied sand layers, based on estimates presented previously.

10.0 POST-SEISMIC FOOTING BEARING CAPACITY

Discussions with Pomeroy indicate that spread footings are likely founded at the 600 to 900mm depth below grade. The near surface soil conditions appear highly variable across the site based on Figures 4 to 6, but at some locations soil liquefaction is clearly an issue at shallow depths below the footings under design levels of seismic loading. This could result in footing punching failure into the liquefied soils unless average bearing pressures are appropriately restricted.

In absence of soil ground improvement, it is recommended that vertical footing bearing pressures be limited to 35 kPa to preclude punching failure of the footings into the underlying liquefied soil and to provide a factor of safety of at least 1.2 against bearing failure. This analysis is based on a method proposed by Meyerhof (1974) and assumes:

- maximum footing dimensions of 2m
- a groundwater table depth of 1.0 m (i.e. a slight raise in levels recorded during previous geotechnical investigation)
- a post-liquefaction residual strength of loose, liquefied sand/silt deposits of 3 kPa

Where footing dimensions are larger than those estimated above, or where a raft slab foundation is used to improve the tolerance of the building envelope to differential settlement, it is recommended that the above recommended post-liquefaction bearing pressures be reviewed to check that post-seismic settlements of footings or raft slabs are within acceptable limits. It is noted that total footing settlements will be those due to the estimated ranges of “free field” settlements presented in Section 8, plus those settlements under concentrated footing load.

If ground improvement is carried out, for example, using compaction grouting (CG) under selected footings, then allowable bearing pressures will be significantly increased. The allowable footing bearing capacity where CG is used should be reviewed during final design of the seismic upgrade.

11.0 FOUNDATION IMPROVEMENT OPTIONS

If the previous estimated values of differential vertical and lateral ground displacement are judged to be excessive for acceptable levels of structural performance, then 2 foundation improvement options are judged to be viable.

The first would involve tying adjacent building columns and footings together using a series of grade beams which would be designed to accommodate the levels of differential settlement. This would result in an “egg crate” type of foundation construction, creating a stiffened raft slab.

Alternatively, compaction grouting is considered feasible, which can be carried out in a limited headroom environment. Compaction grouting would provide foundation underpinning to limit the potential for footing punching failure and post-seismic footing settlement.

Compaction grouting involves the injection under pressure of low slump, soil-cement mixtures down casings to create grout columns of higher stiffness relative to the surrounding soil. The pressure injection process will densify granular soil layers. The intent of CG is to mitigate soil liquefaction potential in granular soil layers and reduce their cyclic strain and pore pressure generation potential. It is intended that CG columns would be placed around the perimeter of a particular size footing, or along the edges of a strip footing. It is expected that CG columns of about 0.6m in average diameter would be required to cause sufficient densification of the looser sand layers to mitigate seismic liquefaction potential (injection volumes of 0.3 cu.m. per 0.3m length with average CG point spacings of 2.4 m). It is recommended that CG columns be constructed to the 6m depth to provide an upper layer of densified soil, which would mask the effects of deeper-seated soil liquefaction and ground settlement (where this occurs in looser soil layers at depth).

It is noted that for both the CG and JG methods, soil-cement columns can likely only be constructed up to about the 1m depth due to limited soil confining stresses at shallower levels. Careful monitoring of volumes and pressures used to create the grout columns, and preventing movements (heave or settlement) of existing surface footings will be required during application of the CG or JG process. Adequate control of construction waste is also required.

It is expected that an economic evaluation of the feasibility of either creation of an effective raft slab or use of compaction grouting will be undertaken by others. If one or both methods are selected as being feasible for the site, then it is recommended that:

- Additional detailed dynamic analysis of soil-cement column – ground interaction be undertaken supporting a particular footing, including the effect of densification of the surrounding ground within a particular grid of CG columns, surrounded by potentially non-densified liquefied ground.
- Field testing using Becker Density Testing be undertaken to demonstrate the densification effect of the method, both within a test section prior to production densification, and during production densification.
- Suitable "performance based" specifications be developed to permit bidding by qualified contractors

12.0 CLOSURE

We trust the above information is sufficient for your present requirements and have enjoyed working with you on this project.

This report has been prepared for the exclusive use of Pomeroy Consulting Engineers Ltd. (the addressee) and Port Coquitlam School District 45 for design of proposed seismic upgrade additions to James Park Elementary School. This report relates only to Pacific Geodynamics' performance of its limited scope of services. Pacific Geodynamics' is not responsible for any assumptions, extrapolations or conclusions made or drawn by the addressee from, or for any failure by the addressee to reasonably apply its own knowledge and expertise to the content of this communication. Pacific Geodynamics Inc. is not responsible for any use by, or reliance on, the content of this communication by any other parties.

Yours truly,

Pacific Geodynamics Inc.

Per:

W. Blair Gohl, Ph.D., P.Eng.

LIST OF REFERENCES

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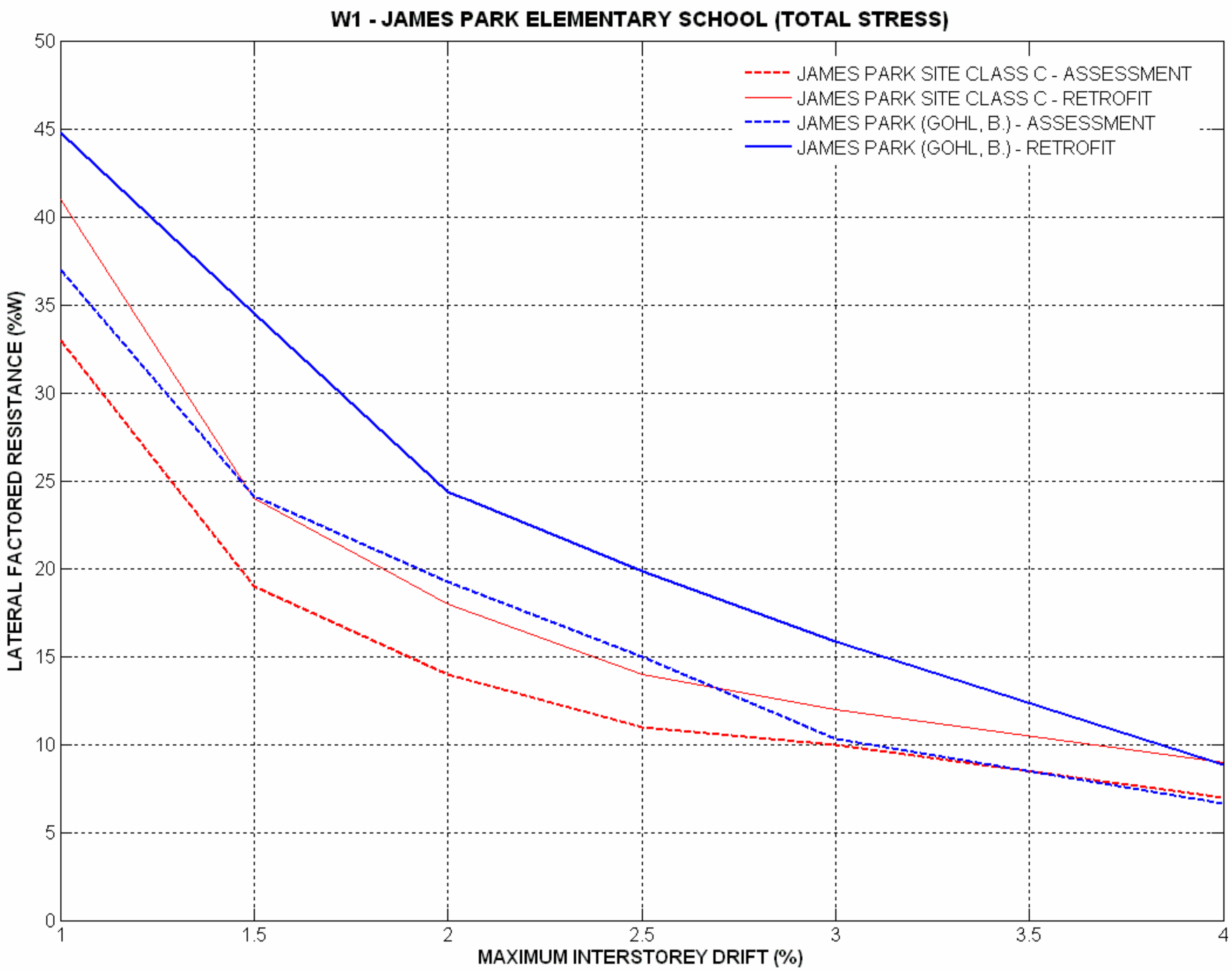
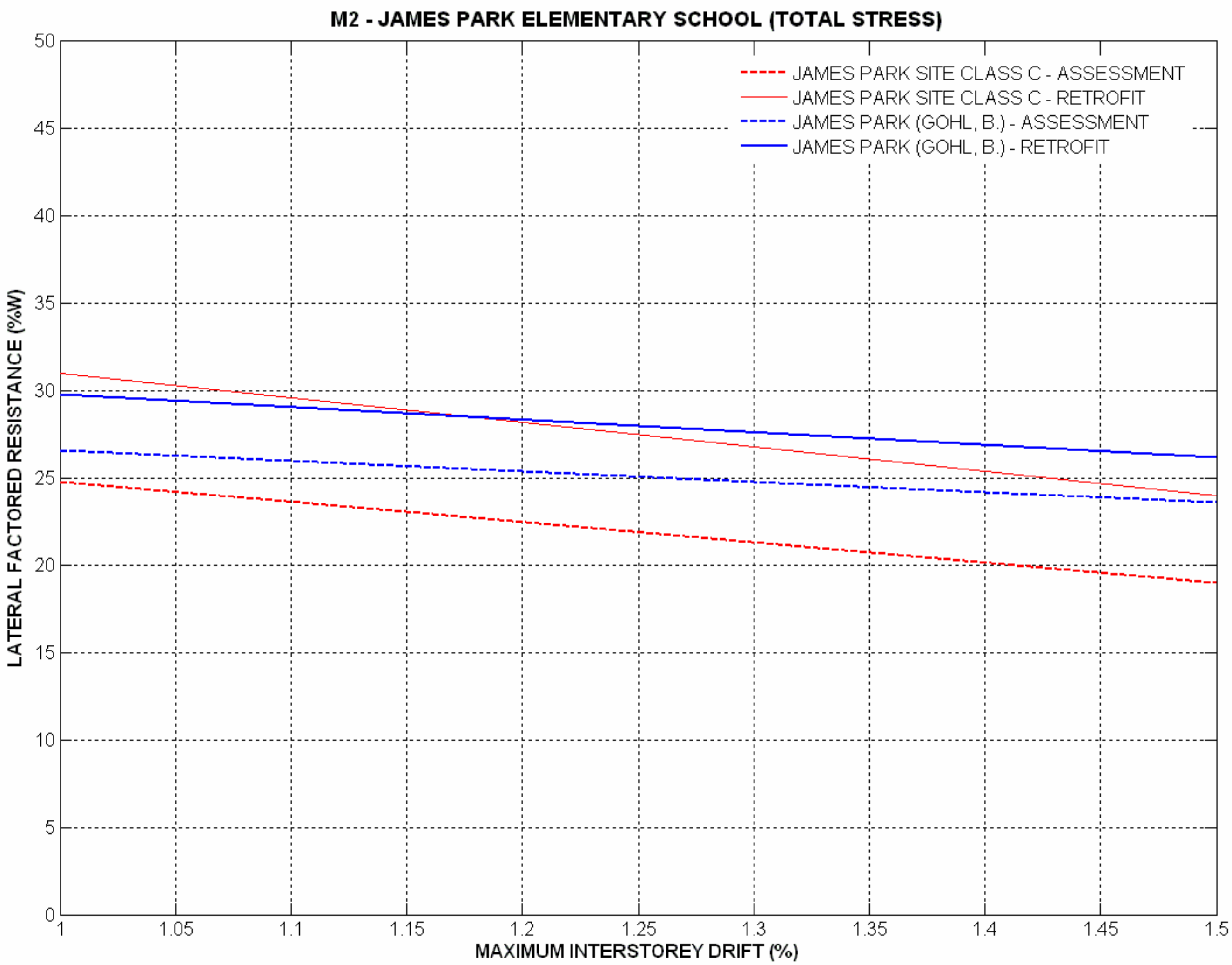
(2) <u>UBC Report (Structural Analysis)</u>

(a) Graph - lateral factored resistance versus drift for W-1 (total stress)


W1 - JAMES PARK ELEMENTARY SCHOOL (TOTAL STRESS)
JAMES PARK SITE CLASS C - ASSESSMENT
JAMES PARK SITE CLASS C - RETROFIT
JAMES PARK (GOHL, B.) - ASSESSMENT
JAMES PARK (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

(2) <u>UBC Report (Structural Analysis)</u>

(b) Graph - lateral factored resistance versus drift for M-2 (total stress)


M2 - JAMES PARK ELEMENTARY SCHOOL (TOTAL STRESS)
JAMES PARK SITE CLASS C - ASSESSMENT
JAMES PARK SITE CLASS C - RETROFIT
JAMES PARK (GOHL, B.) - ASSESSMENT
JAMES PARK (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX E

DETAILED RESULTS FOR LANGLEY FINE ARTS SCHOOL

CONTENTS

(4) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1 (lower bound)

(b) Graph - lateral factored resistance versus drift for W-2 (lower bound)

(c) Graph - lateral factored resistance versus drift for M-2 (lower bound)

(d) Graph - lateral factored resistance versus drift for W-1 (upper bound)

(e) Graph - lateral factored resistance versus drift for W-2 (upper bound)

(f) Graph - lateral factored resistance versus drift for M-2 (upper bound)

(1) <u>Trow Associates Inc. Report (SHAKE Analysis)</u>

(a) Report dated December 7, 2005

COPY

Trow

Since 1957

7025 Greenwood St.
Burnaby, BC
V5A 1X7

Tel: (604) 874-1245
Fax: (604) 874-2358

December 7, 2005

Reference: 051-01560

Sandwell Engineering Inc.
Suite 600 – 885 Dunsmuir Street
Vancouver, BC V5C 1N5

Via mail

Attention: Mr. Minoo Colah, P.Eng.

Geotechnical Seismic Assessment – Phase 2
Langley Fine Arts, Langley, B.C.

Dear Sir:

Introduction

Further to your authorization, Trow Associates Inc. has completed a geotechnical subsurface exploration program at the Langley Fine Arts from November 10 to 14, 2005. The purpose of this exploration is to gather subsoil information in order to provide geotechnical assessment pertaining to seismic site response based on National Building Code of Canada, NBCC (2005).

It is understood that the seismic assessment will be undertaken in accordance with the National Building Code of Canada (NBCC) 2005 edition. The geotechnical assessment has been conducted in two phases. Phase 1 of the assessment was completed using available subsoil data and the results are presented in our letter report dated October 3, 2005. This report presents the Phase 2 assessment, which included a subsurface exploration program and ground response analysis.

Information provided to us included a site plan showing the layout of the school.

Attached to this report are: a Test Hole Location Plan, a Test Hole Log and our Interpretation and Use of Study Report. Results of the ground response analysis are presented in the Appendix.

Site Description

Langley Fine Arts is located at the southeast corner of the intersection of Trattle Street and St. Andrews Street in the Village of Fort Langley. The surrounding topography is generally flat. The school borders on Trattle Street to the west, St. Andrews Street to the north, residential properties to the south and playgrounds and grass playfields to the east. The school consists of several inter connected one storey buildings, a theatre and a gymnasium.

The Geological Survey of Canada Map 1468A, Surfucial Geology, British Columbia (1976) indicates the sediments underlying the area are comprised Sumas Drift - outwash, ice-contact, and deltaic deposit, raised proglacial deltaic gravel and sand up to 40m thick.

Investigation and Results

A subsurface exploration program was carried out at the site from November 10 to 14, 2005 using a truck mounted mud rotary drill rig. The rig was equipped with Standard Penetration Testing (SPT) equipment and Dynamic Cone Penetration Testing (DCPT) equipment. A mud rotary hole was drilled to approximately 19.8m depth below the existing grade at the location shown in drawing 051-01561-01. SPTs were carried out at 1.5 m depth intervals in order to determine the in-situ density of the subsoils. Soil samples from the SPT split spoon sampler were taken to our laboratory for further tests and classification. The mud rotary drilling was terminated at 19.8m depth due to no circulation of drill mud. DCPTs were conducted from the bottom of the drill hole to a depth of 27m at refusal. Results of the field and laboratory work are shown in the attached drill hole log in drawing 051-01561-BH1.

Following completion of drilling, a standpipe piezometer was installed in the drill hole to monitor the ground water level. The bore hole was flushed with clean water prior to the installation of the standpipe piezometer in the bore hole. Details of the installation and measured ground water level are shown in the drill hole log.

The test holes were located, logged and sampled in the field by a representative from Trow.

Based on the review of subsoils encountered at the test hole location, we expect a generalized subsoil profile within the site can be summarized in order of increasing depth as follows:

Unit	Thickness	Description
A	0.9m	dark brown-rusty brown fine-coarse sandy SILT, some fine-coarse gravel (FILL)
B	12.8m	interlayered rusty brown to brown to grey fine-coarse SAND and occ.-some fine-coarse gravel -SPT results vary from 25-152 blows/0.3m -density varies medium dense to very dense
C	13.3m (to depth of investigation)	grey fine-coarse SAND, occ. fine-coarse gravel -soils below 19.8m depth was interpreted based on blow counts data from Dynamic Cone Penetration Testing -SPT blow counts vary from 28 to 46 blows/0.3m -density is medium dense.

Groundwater in the standpipe piezometer was at 13.6m below the existing grade on November 21, 2005. Note that groundwater level is expected to vary seasonally and with precipitation.

Upon completion, bore hole BH05-1 was grouted to the surface with a bentonite slurry tremied approximately 10 feet from the bottom of the bore hole to the surface.

Site Classification for Seismic Site Response

The design earthquake motions considered in NBCC 2005 has a 2% probability of exceedance in 50 years, or has a 2475 year return period. The NBCC 2005 provides Peak Ground Acceleration (PGA) and design response spectrum for structural design. The PGA and response spectrum given in NBCC 2005 are for soil conditions referred as "Firm Ground" near the surface. Shear wave velocity of this "Firm Ground" is in the range of 360 m/s to 760 m/s. Dense till-like or soft bedrock sites would classify as "Firm Ground" according to the NBCC 2005 guidelines.

The NBCC 2005 PGA at near-surface Firm Ground for Langley is 0.53g. The inferred earthquake magnitude for this event is M7.5. → NEW TASK FORCE REPORT (DRAFT) MAY RECOMMEND M7.0

Trow

The design earthquake motions would be altered or amplified at sites where the Firm Ground is deeper and as the motion propagates through the loose or soft soils. To account for the amplification in this type of ground, the NBCC 2005 recommends Foundation Factors F_a (for short period structures) and F_v (for long period structures) for structural analysis.

The NBCC 2005 Table 4.1.8.4.A, B and C provides guidelines for classification of sites ("Site Class"), the short period foundation factor F_a and the long period foundation factor F_v respectively. Site Class can be determined using "average" shear wave velocity, SPT N value or undrained shear strength in the top 30 m of soils.

Water saturated sands and gravel may liquefy during the design earthquake shaking. If any soil layer within the top 30 m of the ground is susceptible to liquefaction, then the site would be classified as "Site Class F".

Liquefaction susceptibility of the subsoils was initially assessed using the "Seed's simplified Procedure" (Youd et al, 2001) with the measured SPT data. The analysis, with a surface PGA of 0.53, implies potential liquefaction below 13.5 meters depth. Because of this liquefaction potential, the site would be classified as "Site Class F". Subsequently, a site specific ground response analysis was carried out as required by NBCC 2005. The results of the analysis are described below.

Ground Response Analysis

Ground response analysis was carried out using the 1-D computer program SHAKE91 (Idriss and Sun, 1992). The objectives of the ground response analyses are to develop site-specific design spectra for structural analyses and to obtain cyclic stress ratios for liquefaction assessment. The analysis was carried out using four earthquake records shown in drawing 1 in the Appendix. These are the input records to be applied at the near surface "Firm Ground". The program will model propagation of the seismic motion through the Firm Ground, which is below 30 m depth at this site, and propagate through the sands and gravels.

All four input motions (representing motions at near surface Firm Ground) have been modified such that they have a PGA of 0.53g and their response spectrum matches the NBCC 2005 values for Langley, B.C.

Firm Ground was assumed to be at 40 m depth below the existing grade at the school site. (A sensitivity analysis was carried out with Firm Ground depths of 40 m, 50 m and 75 m. The 40 m depth provided the conservative answers and therefore adopted for further analyses).

Drawing 2 in the Appendix presents the response spectra, obtained for 5% damping, from the four input motions. The thick solid lines represent response spectra at the ground surface. The thin lines with solid circles represent response spectra at the Firm Ground (from NBCC 2005). Significant amplification of the motion at periods longer than 0.5 seconds may be noted in drawing 2. The Foundation Factor at a given period is the ratio of the spectra (that at surface to that at Firm Ground). The recommended Foundation Factors are $F_a = 1.0$ and $F_v = 2.3$. If only short period response is of concern for these single to two storey buildings, then an equivalent "Site Class D" could be considered for periods not exceeding 0.3 seconds.

Liquefaction Assessment

Liquefaction susceptibility of the sub-soils was assessed using the results of the site specific ground response analysis and the procedures given in Youd et al (2001). Cyclic shear resistance of the soils against liquefaction (CRR) was calculated from the measured SPT N values. Results of the assessment are shown graphically as Factor of Safety (F.S.) against liquefaction in drawing 3 in the Appendix. Note that F.S. is defined as the ratio of the available resistance of the ground to the shear demand imposed by the earthquake.

The analysis indicates that the saturated sands between 13.5 m and 19.8 depth m are susceptible to liquefaction during the 1:2475 year earthquake. Soils between the existing grade and 13.55 m depth are not expected to liquefy.

Liquefaction analysis of the soils below 19.8m depth was not conducted. We are of the opinion that the DCPT data may not be unreliable especially due to the presence of gravel at that depth. However review of the DCPT data indicates that soil layer between 23.7 and 24.7m depth may be susceptible to liquefaction under the design earthquake shaking.

One of the consequences of liquefaction, which may affect this single to two storey buildings, is post-earthquake settlement. Bearing capacity of the spread and strip footings is not expected to be affected by soil liquefaction below 13.5m depth. The post-earthquake settlement due to liquefaction at such deep depth is expected to occur uniformly across the site and its magnitude was calculated using the procedures of Tokimatsu and Seed (1987). This procedure indicates ground settlement in the range of 100 mm to 150 mm at the surface following the 1:2475 year event. Improving the ground against liquefaction at depths more than 13.5m is very expensive and not economical.

If required, structural remediation such as tying the foundations so as to prevent collapse of building or against settlement may be considered.

Closure

We hope the information given in this report is satisfactory to your current requirement. If you have any questions please do not hesitate to contact the undersigned.

Yours truly,

Trow Associates Inc.

Mark Qian, P.Eng. Associate Geotechnical	M. Uthayakumar, P.Eng. Senior Associate Geotechnical

INTERPRETATION & USE OF STUDY AND REPORT

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering consulting practices in this area. No other warranty, expressed or implied, is made. Engineering studies and reports do not include environmental consulting unless specifically stated in the engineering report.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF THE REPORT

The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

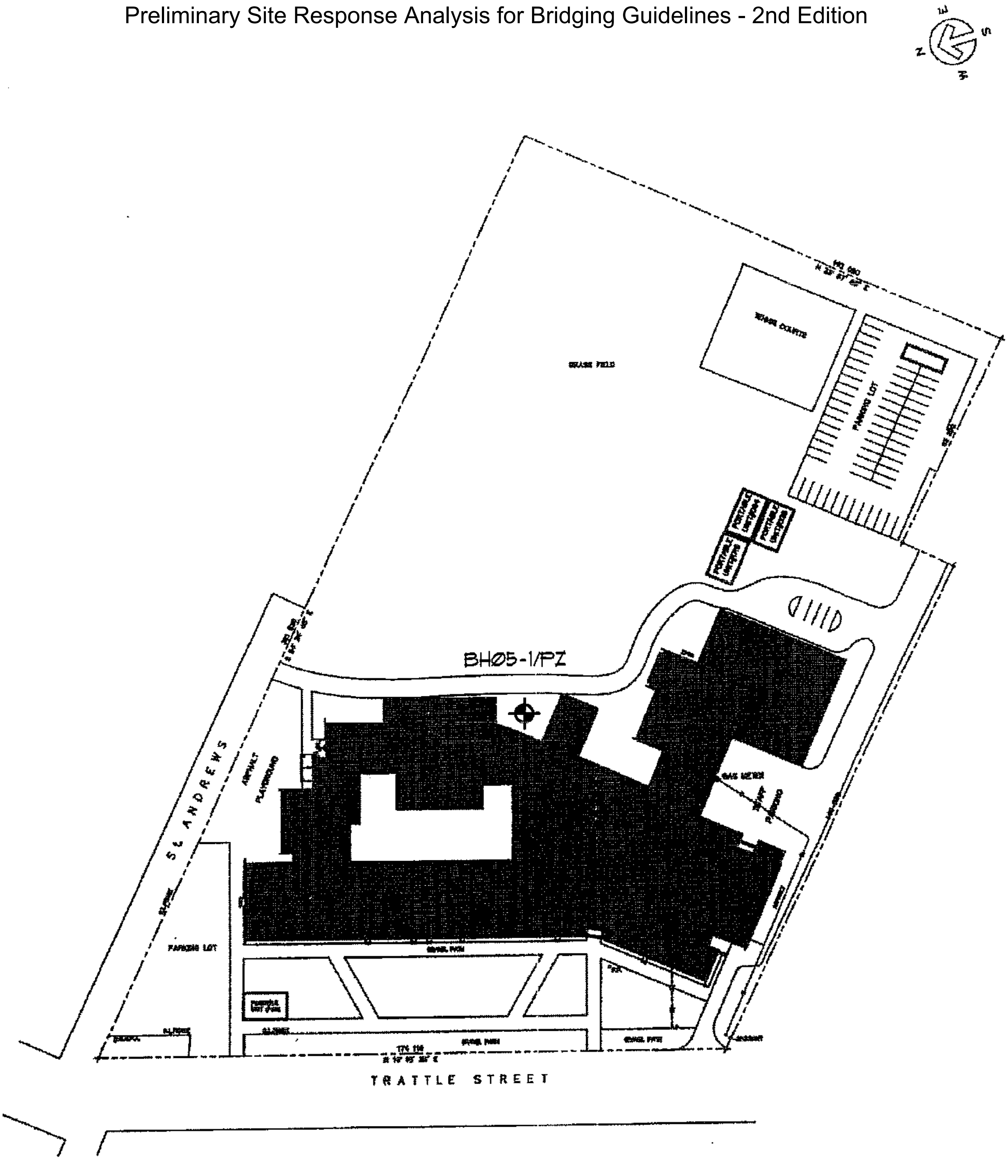
The information and opinions expressed in the Report, or any document forming the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorise only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorised use of the Report.

5. INTERPRETATION OF THE REPORT

a. Nature and Exactness of Descriptions: Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations, or building envelope descriptions, utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarising such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b. Reliance on Provided information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.

c. To avoid misunderstandings, Trow should be retained to work with the other design professionals to explain relevant engineering findings and to review their plans, drawings, and specifications relative to engineering issues pertaining to consulting services provided by Trow. Further, Trow should be retained to provide field reviews during the construction, consistent with building codes guidelines and generally accepted practices. Where applicable, the field services recommended for the project are the minimum necessary to ascertain that the Contractor's work is being carried out in general conformity with Trow's recommendations. Any reduction from the level of services normally recommended will result in Trow providing qualified opinions regarding adequacy of the work.



REFERENCE DRAWING
LANGLEY SCHOOL DISTRICT:
SITE PLAN DATED APR. 21/Ø5

LEGEND

BHØ5-1/PZ BOREHOLE/PIEZOMETER

Trow TROW ASSOCIATES INC.	CLIENT SANDWELL ENGINEERING INC.				TITLE: TESTHOLE LOCATION PLAN
	PROJECT LANGLEY FINE ARTS 9096 TRATTLE STREET, FORT LANGLEY, B.C.				
	PROJECT NO 051-01561	DR'N MG	DES'N GY	CHK MQ	DATE NOV. 25/05

SCALE: 1:1500 approx.

DWG: 051-01561-01

Nov 25, 2005 - 10:25am O:\2005\051-01561 MQ Langley Schools Assessment - Geotechnical\Drawings\051-01561 Testhole Plans.dwg

Borehole no. : BH05-1

Location : SEE LOCATION PLAN

Ground Surface Elevation :

Ground Water Elevation : SEE REMARKS

Equipment : TRUCK MOUNTED MUD ROTARY DRILL RIG, SPT - 140 lb. safety hammer, free falling 30 ins. (blows/6" penetration unless otherwise noted)

Method of Sampling: □ SPLIT SPOON SAMPLE ○ LUMP SAMPLE

depth, ft.	depth, m	blows per 6"	symbol	Description	sample no.	moisture content %	Remarks
0	0			grey-brown fine SAND, some silt, occasional fine to coarse gravel			
	1		○		S1	26	
5		2,16,22	□	grey-brown fine SAND, some silt, occasional fine to medium gravel, possible coarse gravel	SPT1		18" recovery
	2			-fine to coarse GRAVEL, trace to some fine sand			
				-fine to coarse SAND and GRAVEL			-smooth drilling, no recovery
10	3	11,12,12	□	grey fine SAND, occasional fine gravel	SPT2		
	4						
15		6,7,9	□	grey fine to coarse SAND, occasional fine gravel	SPT3		11" recovery
	5						Grout
				-fine to medium SAND, trace coarse sand			
20	6	7,8,9	□	grey fine to coarse SAND to fine SAND	SPT4		11" recovery
							PVC Standpipe Piezometer, slotted bottom 10'
	7			-fine to medium SAND and fine GRAVEL			
25		17,20,21	□	grey fine to coarse SAND, occasional fine to medium gravel	SPT5		8" recovery
	8			-fine to coarse SAND, some fine gravel			
30	9	20,35,46	□	grey fine to coarse SAND, occasional fine to medium gravel, possible coarse gravel	SPT6		12½" recovery
	10						

SANDWELL ENGINEERING INC.

TROW ASSOCIATES INC.

LANGLEY FINE ARTS
9096 TRATTLE STREET
FORT LANGLEY, B.C.

Borehole No. BH05-1

Logged by: GY

Date of Drilling: NOV. 10/05

Sheet: 1 of 3

Dwg No. 051-01561-BH05-1

Borehole no. : BHØ5-1

Location : SEE LOCATION PLAN

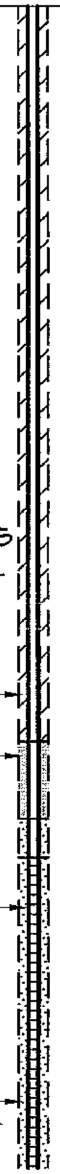
Ground Surface Elevation :

Ground Water Elevation : SEE REMARKS

Equipment : TRUCK MOUNTED MUD ROTARY DRILL RIG, SPT - 140 lb. safety hammer, free falling 3Ø ins. (blows/6" penetration unless otherwise noted)

Method of Sampling: □ SPLIT SPOON SAMPLE ○ LUMP SAMPLE

depth, ft.	depth, m	blows per 6"	symbol	Description	sample no.	moisture content %	Remarks
30				-grey fine to coarse SAND, some fine to coarse gravel			
	10			-grey fine coarse SAND and GRAVEL to fine to coarse GRAVEL, some sand			
35		35,52,51	□	grey fine to coarse gravelly SAND	SPT7		12½" recovery
	11			-fine to coarse SAND and GRAVEL			
40	12	2Ø,25,27	□	grey fine to coarse SAND, occasional fine to medium gravel, possible coarse gravel	SPT8		6¾" recovery -coarse gravel stuck in split spoon
	13			-grey fine to coarse SAND and fine to medium GRAVEL			GWL 44.7' Nov. 21, 2ØØ5
45		12,8,7	□	fine to coarse GRAVEL and SAND (?)	SPT9		poor to no recovery
	14			-fine to coarse SAND, occ. gravel -fine to coarse SAND & fine GRAVEL			Grout
50	15	6,7,3	□	brown fine to medium SAND	SPT1Ø		Bentonite seal
	16			-grey fine to coarse SAND, occ. gravel			PVC Standpipe Piezometer, slotted bottom 1Ø'
55	17	6,2Ø,17	□	grey-brown fine to medium SAND, occasional fine gravel -fine to coarse SAND & GRAVEL	SPT11		12½" recovery
	18						Pea gravel
60		27,11,23	□		SPT12		no recovery -coarse gravel stuck in split spoon
	19						



SANDWELL ENGINEERING INC.

TROW ASSOCIATES INC.

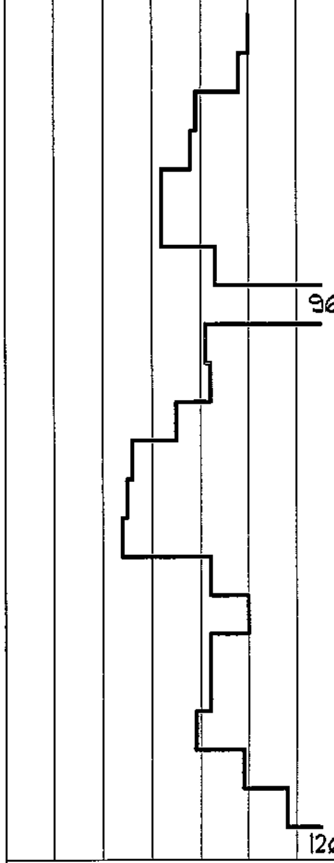
LANGLEY FINE ARTS 9Ø96 TRATTLE STREET FORT LANGLEY, B.C.	Borehole No. BHØ5-1	Logged by: GY	Date of Drilling: NOV. 1Ø/Ø5
		Sheet: 2 of 3	Dwg No. Ø51-Ø156[illegible] BHØ5-1

Borehole no. : BH05-1
Location : SEE LOCATION PLAN
Ground Surface Elevation :
Ground Water Elevation : SEE REMARKS

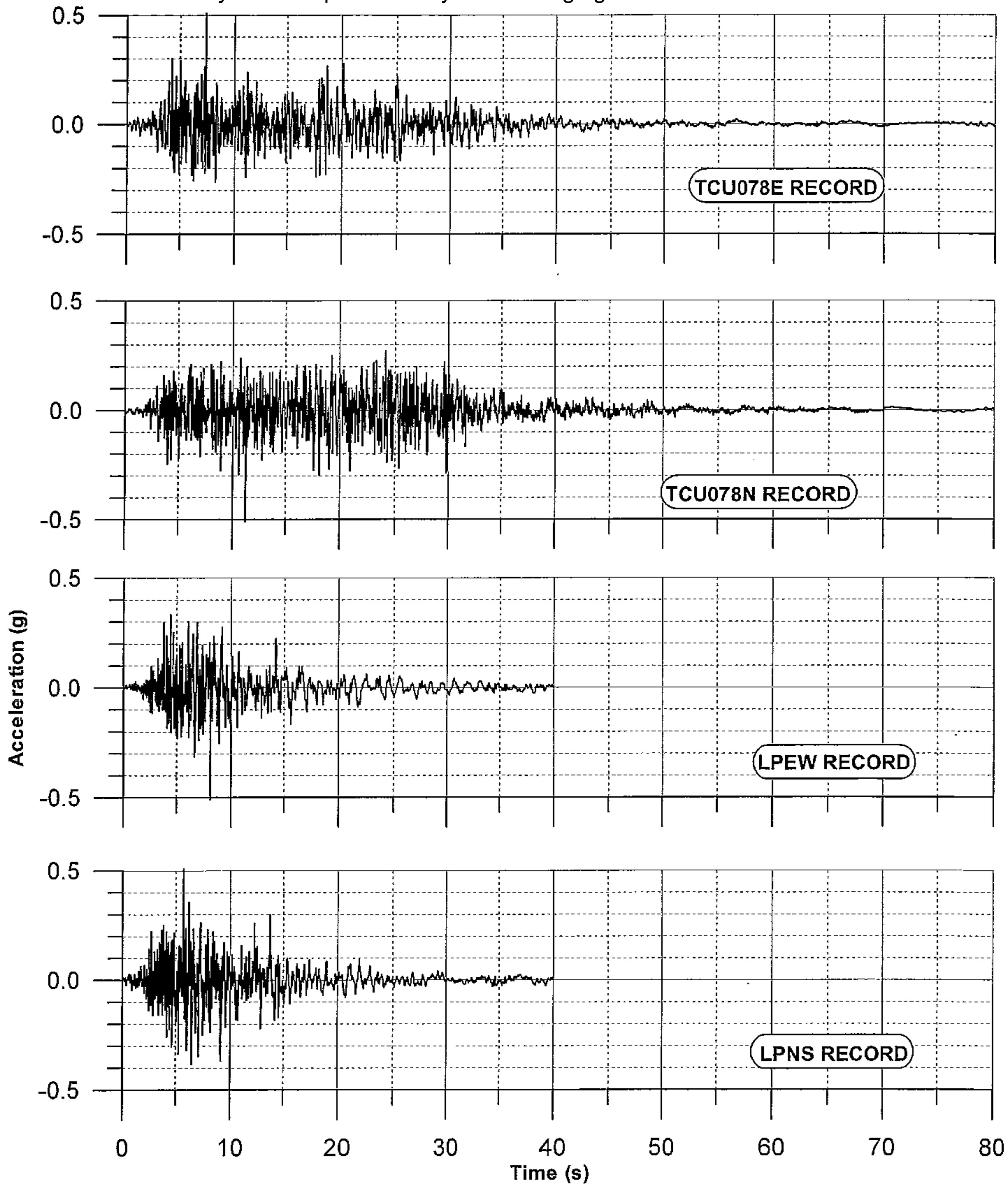
Equipment : TRUCK MOUNTED MUD ROTARY DRILL RIG, SPT - 140 lb. safety hammer, free falling 30 ins. (blows/6" penetration unless otherwise noted).
Dynamic Cone 2.4"ϕ x 7.25", blunt tip

Method of Sampling: □ SPLIT SPOON SAMPLE ○ LUMP SAMPLE

depth, ft.	depth, m	blows per 6"	symbol	Description	sample no.	moisture content %	Dynamic Cone Penetration Test P05-1 Blows/ft. 10 20 30 40 50 60	Remarks
60				possible fine to coarse SAND and GRAVEL			-no circulation, no recovery	
	19			-fine to medium GRAVEL			-no circulation	
				fine to medium GRAVEL, some fine to coarse sand				
65	20	9,6,5	□	End of hole @ 65'	SPT13		-poor to no recovery	
70	21, 22							
75	23						96	
80	24, 25							
85	26, 27						120/10"	
90	28						End of Penetration Test @ 88'-10"	

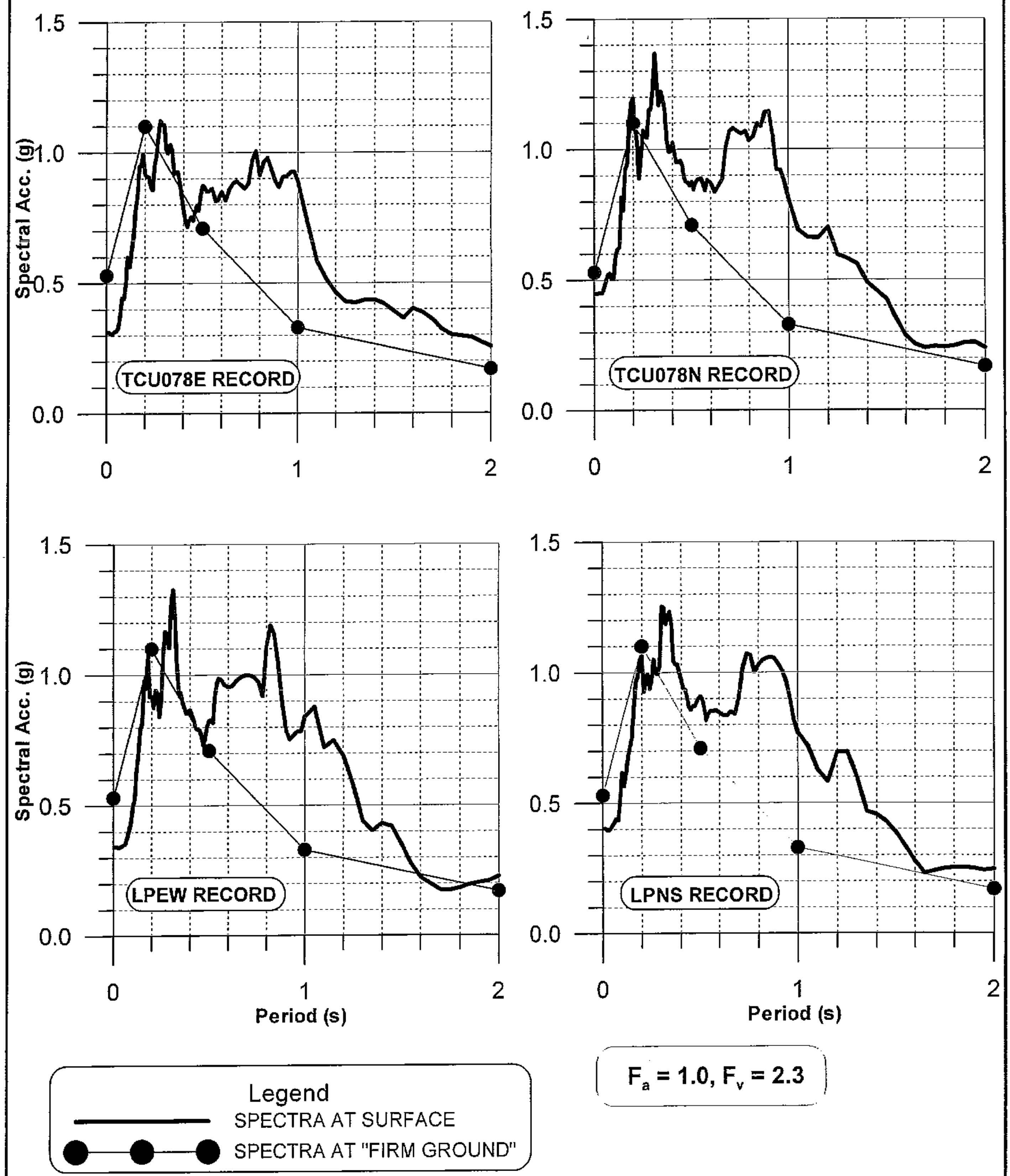


SANDWELL ENGINEERING INC.	TROW ASSOCIATES INC.		
LANGLEY FINE ARTS 9096 TRATTLE STREET FORT LANGLEY, B.C.	Borehole No. BH05-1	Logged by: GY Sheet: 3 of 3	Date of Drilling: NOV. 10/05 Dwg No. 051-01561-BH05-1



TROW ASSOCIATES INC. 7025 Greenwood Street Burnaby, B.C., V5A 1X7 Tel.: 604-874-1245 Fax.: 604-874-2358	CLIENT **SANDWELL ENGINEERING INC.** PROJECT **FINE ARTS SCHOOL** **LANGLEY, B.C.**	TITLE **SYNTHESIZED ACCELERATION RECORDS USED FOR GROUND RESPONSE ANALYSES. NBCC 2005, 1:2475 YEAR EVENT**

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
051-01561		**UMU**		**DEC., 2005**		**1**



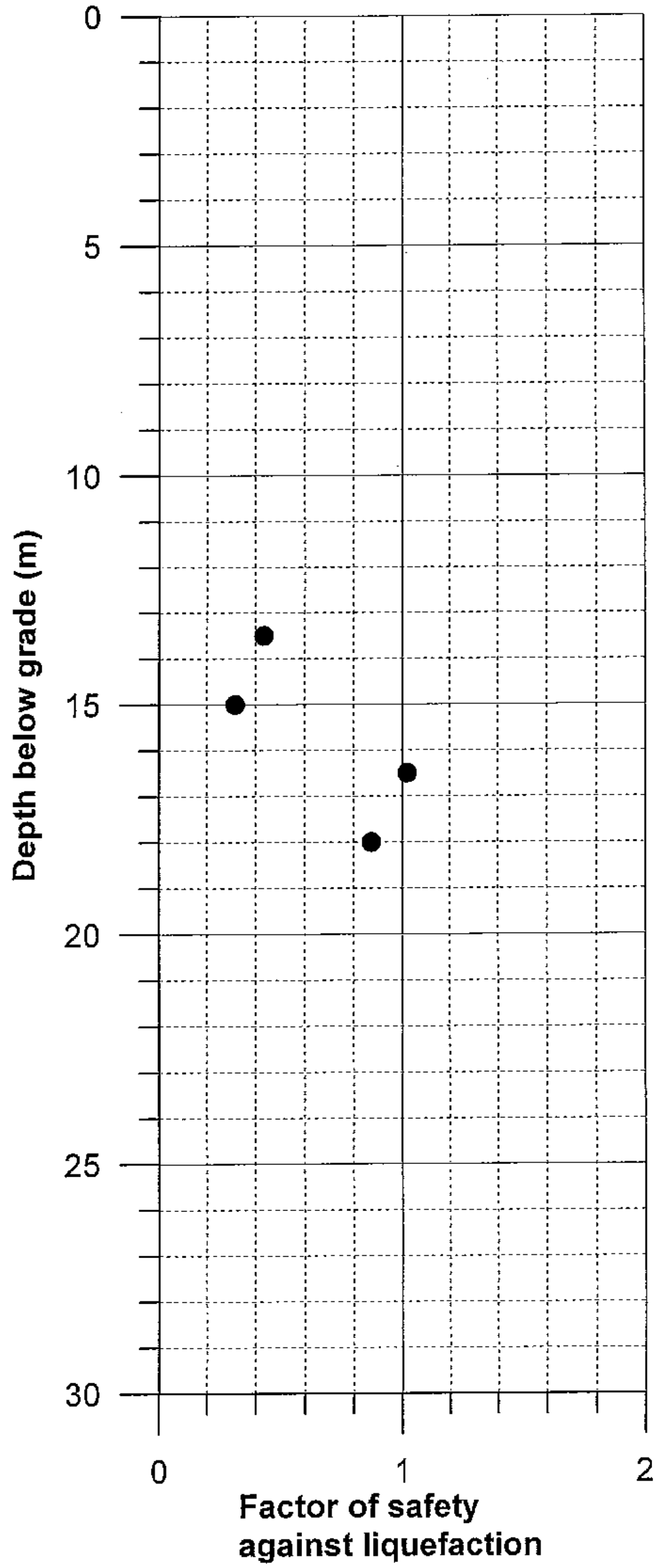
TROW ASSOCIATES INC.
7025 Greenwood Street
Burnaby, B.C., V5A 1X7
Tel.: 604-874-1245
Fax.: 604-874-2358

CLIENT
SANDWELL ENGINEERING INC.

PROJECT
FINE ARTS SCHOOL
LANGLEY, B.C.

TITLE
RESPONSE SPECTRA AT SURFACE, AND NBCC 2005 FIRM GROUND FOR 1:2475 YEAR EVENT, 5% DAMPING

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
051-01561		UMU		DEC., 2005		2



TROW ASSOCIATES INC.
7025 Greenwood Street
Burnaby, B.C., V5A 1X7
Tel.: 604-874-1245
Fax.: 604-874-2358

CLIENT
SANDWELL ENGINEERING INC.

PROJECT
FINE ARTS SCHOOL
LANGLEY, B.C.

TITLE
FACTOR OF SAFETY AGAINST LIQUEFACTION - NBCC 2005, 1:2475 YEAR EVENT.

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
051-01561		UMU		DEC., 2005		3

(1) Trow Associates Ltd. Report (SHAKE Analysis)

(b) Soil column properties

071-03092. LANGLEY FINE ARTS SCHOOL SEISMIC UPGRADE
05-Mar-07
SOIL DATA.

Unit	Thickness	Description
A	0.9m	dark brown-rusty brown fine-coarse sandy SILT, some fine-coarse gravel (FILL)
B	12.8m	interlayered rusty brown to brown to grey fine-coarse SAND and occ.-some fine-coarse gravel -SPT results vary from 25-152 blows/0.3m -density varies medium dense to very dense
C	13.3m (to depth of investigation)	grey fine-coarse SAND, occ. fine-coarse gravel -soils below 19.8m depth was interpreted based on blow counts data from Dynamic Cone Penetration Testing -SPT blow counts vary from 28 to 46 blows/0.3m -density is medium dense.

Soil layers consists of, mainly SANDS and fine Gravel. SPT N values were measured and the details are given in the attached borehole logs.

Shear wave velocity V(s) was calculated based on the following relationship:
G(max) = density * [V(s)]^2;
G(max) = 4500 * {[N1]_60}^0.33 * (sigma'_v)^0.5; Units of G(max) and Sigma'_v are in kPa.
{[N1]_60} = (N_60) * (100/sigma'_v)^.5
where N_60 is the measured N value, SPT blow counts per the last 300 mm of penetration, sigma'_v is the vertical initial effective stress
For depth graeter than 18 m, where no N value is available (only DCPT, SPT could not be performed, see the attached report),
V(s) was assumed to be proportional to the (depth)^0.25, with initial V(s) calculated from N values at 18 m depth.

soil layer top (ft)	soil layer bottom (ft)	thickness (ft)	total unit weight (psf)	V(s) (ft/s)	Friction angle (deg)	soil type	
0	3	3	110	500	27-29	FILL(sand&silt)	
3	5	2	120	655	36-38	S&G, silt	S&G : Sand and gravel
5	10	5	120	755	34-36	S&G	
10	15	5	120	755	34-36	S&G	
15	20	5	120	785	34-36	S&G	
20	23	3	120	950	34-36	SAND	
23	25	2	120	965	36-38	S&G	
25	30	5	120	1000	36-38	S&G	
30	40	10	120	1035	36-38	S&G	
40	45	5	120	855	32-34	SAND	
45	50	5	120	820	32-34	SAND	
50	55	5	120	900	32-34	S&G	
55	60	5	120	1035	34-36	S&G	
60	70	10	120	1050	34-36	S&G	
70	80	10	120	1100	34-36	S&G	
80	90	10	120	1130	34-36	S&G	
90	100	10	120	1180	34-36	S&G	
100	110	10	120	1195	34-36	S&G	
110	120	10	120	1215	34-36	S&G	
120	131	11	120	1245	34-36	S&G	
131			140	2500		"Firm Ground"	

(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound

(a) E-mail dated March 07, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: G.Taylor
Sent: Wednesday, March 07, 2007 3:21 PM
Subject: Re: scaling factors for input motions for Langley

Graham,

I just completed the analyses for the Langley Fine Arts School with the outcrop motions applied at the 40m depth using same scaling factors as used for Port Guichon. No deconvolution was considered but an energy absorbing bottom boundary with a Vs = 400 m/sec was used. The latter Vs of the half-space is more in line with the Vs cited by Trow at the 40 m depth. The only damping considered in the DESRA-2C model was that due to hysteretic damping in the soils - no additional Rayleigh damping was considered which might damp out some of the higher frequency acceleration response. Please have a look at the results before we decide whether deconvolution is necessary.

I have attached the following:

- acceleration time history files (time vs. acceleration in g's) computed at the 3 ft. depth for each input eq. record
- input soil properties versus depth based on use of same Vs, Gmax profile as Trow and using drained shear strength properties in the sands based largely on the phi values Trow recommended
- computed acceleration and velocity spectra at the 3 ft. depth (note the site period appears at around 0.8 seconds or so)
- computed effective cyclic shear stress ratios, maximum shear stresses, maximum shear strains, and peak ground accelerations versus depth
- computed hysteresis loops of shear stress-shear strain at 16m and 26m depths to indicate typical cyclic soil response

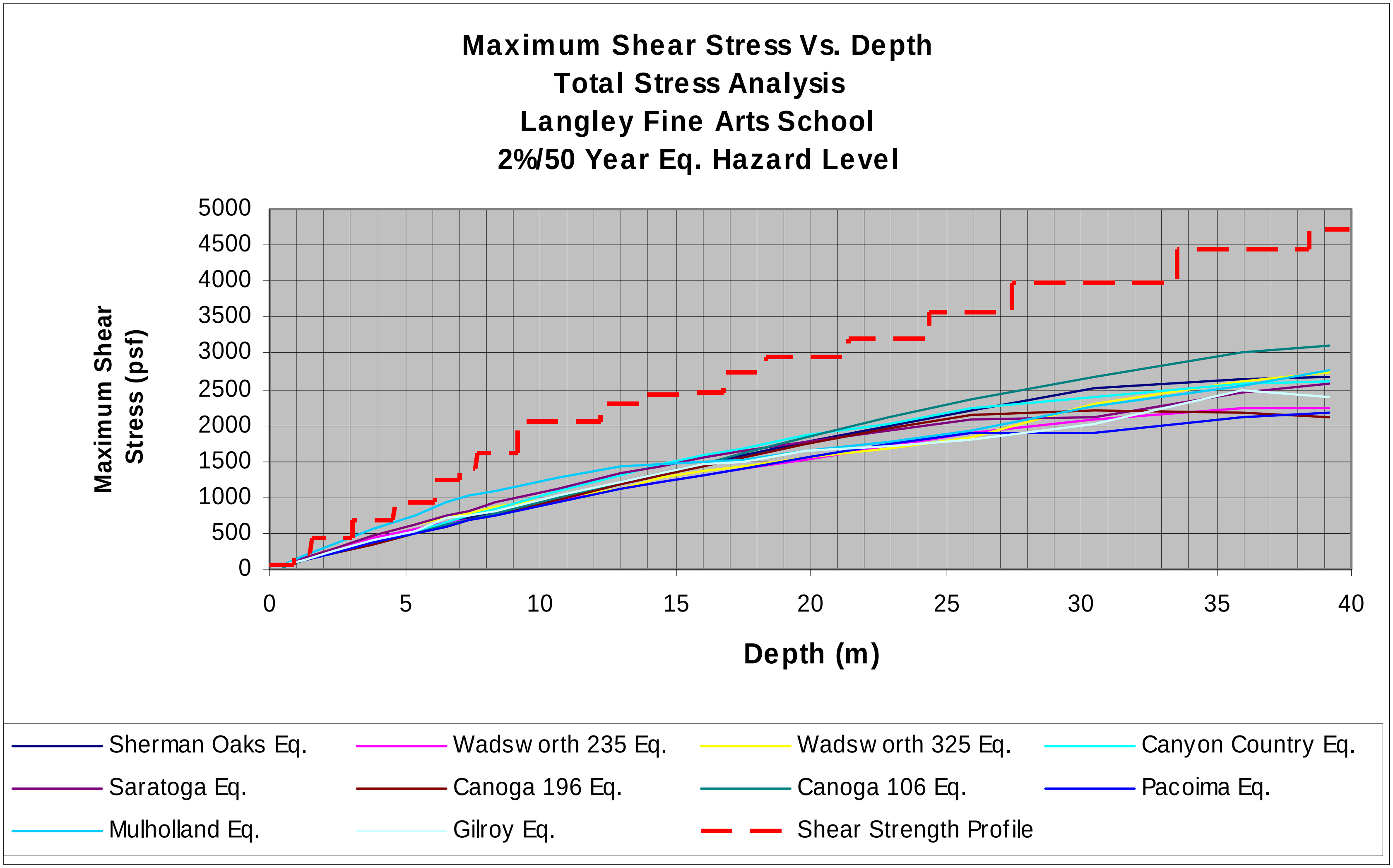
I would only forward these results to the UBC group at this stage until we are all collectively satisfied that no further iterations on soil properties or use of deconvolution is required. Note that I consider it possible that sand shear strength values in the lower saturated sands below the 14m depth could theoretically be higher if one has a strong dilative response (dense sands) so that pore water cavitation occurs. I have estimated the undrained shear strength of the sands with pore water cavitation as per the attached xl spreadsheet.

Regards,

Blair

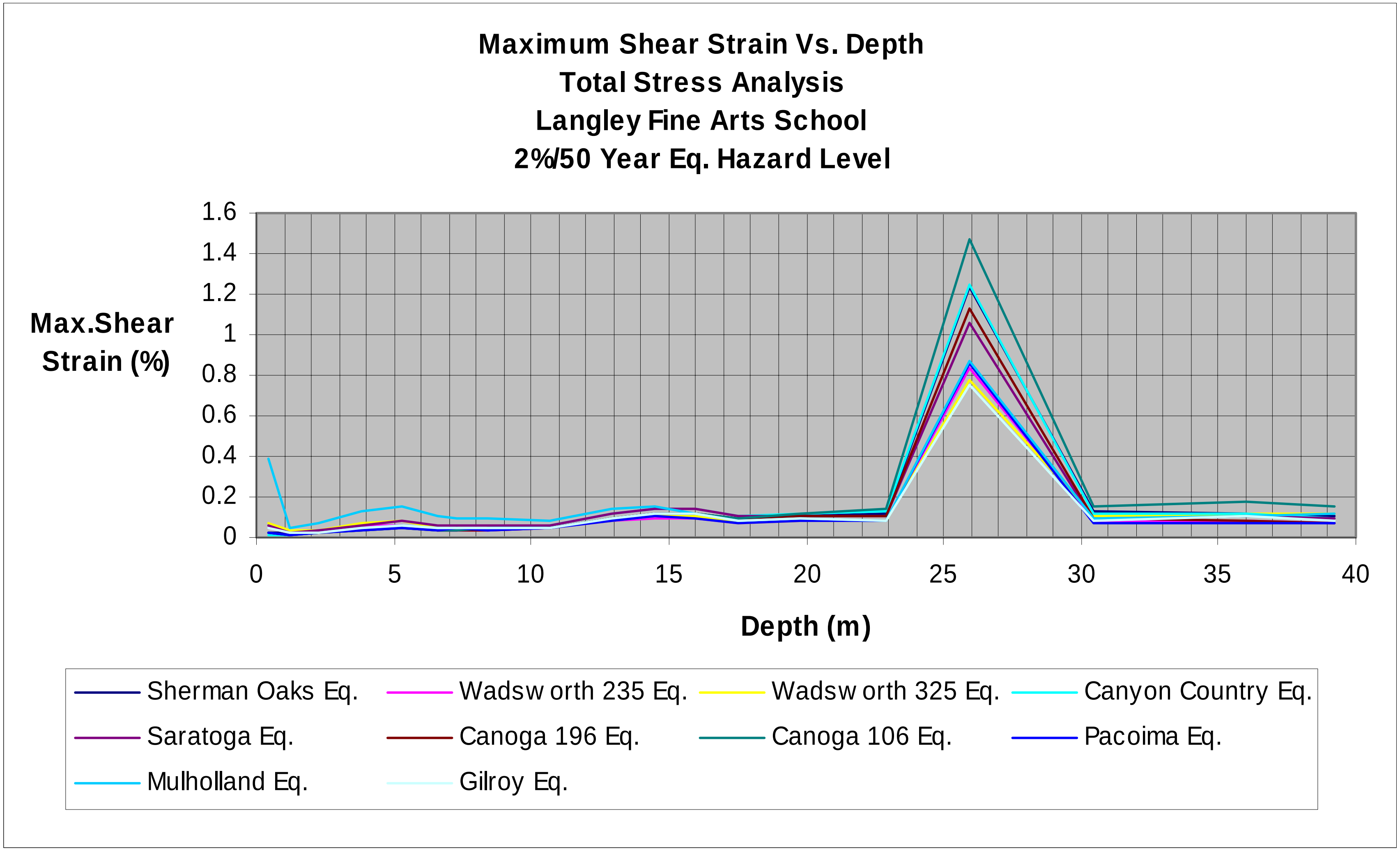
(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

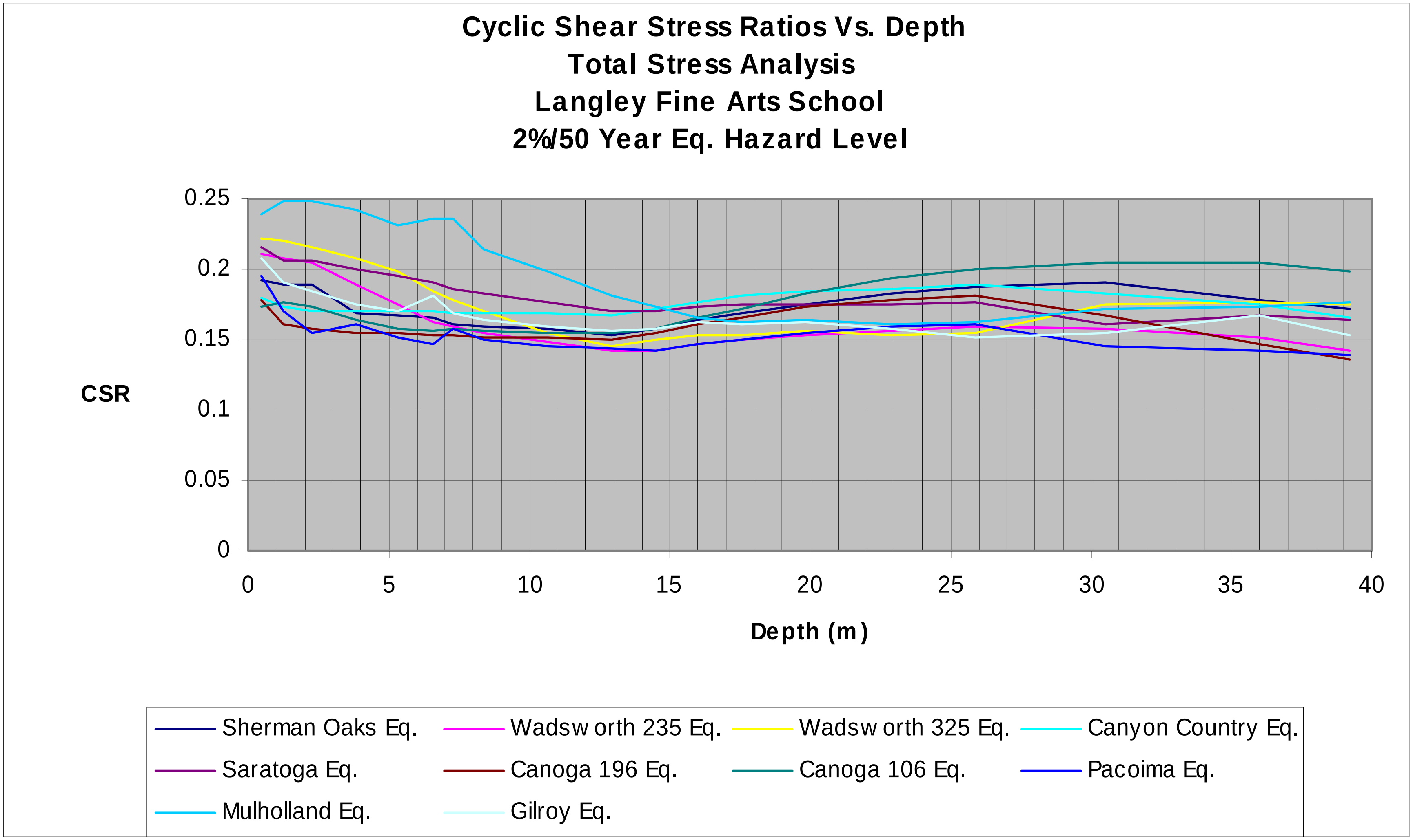
(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound</u>

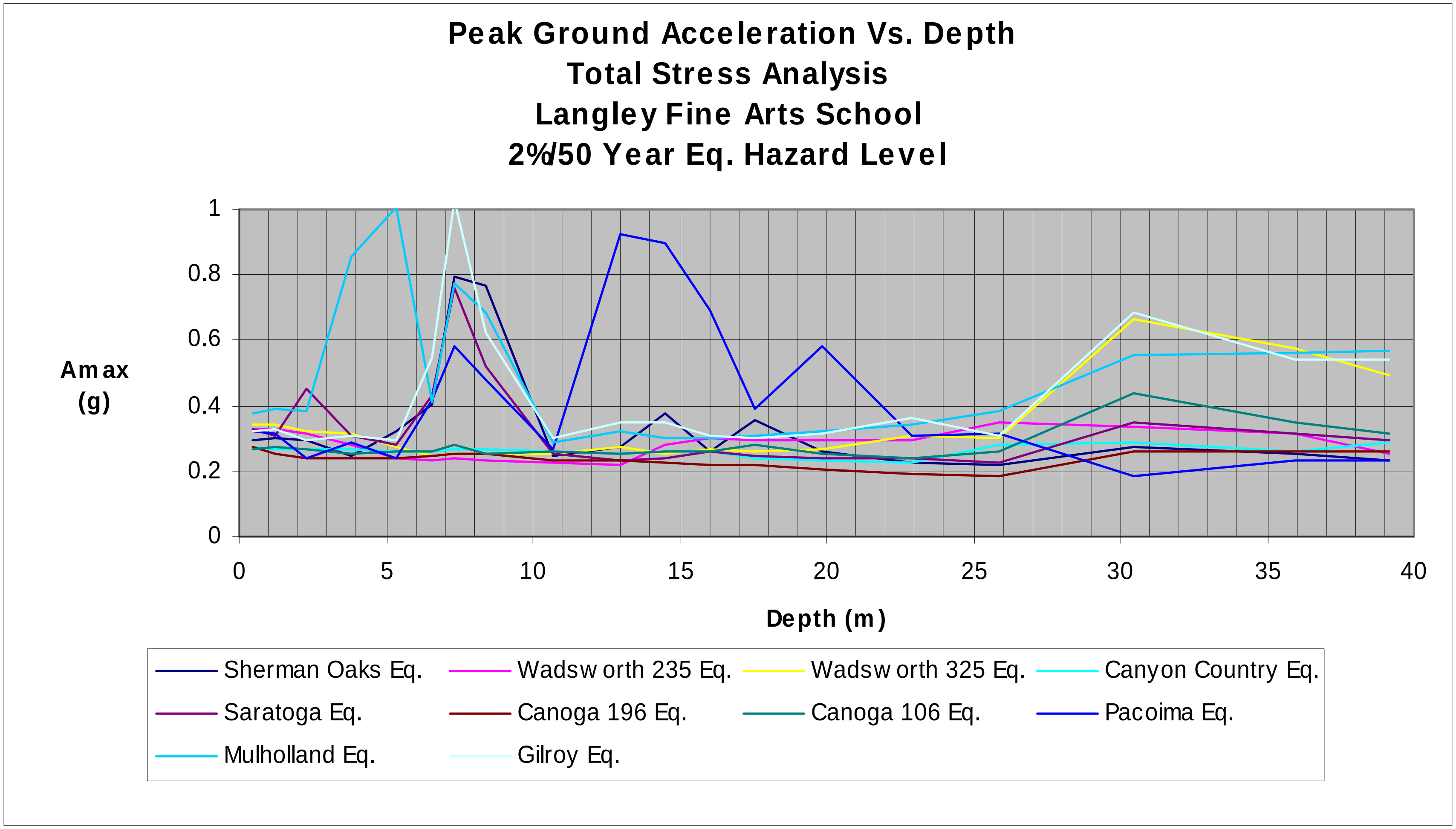
(d) Graph - cyclic shear stress ratio versus depth



Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
CSR
0.25
0.2
0.15
0.1
0.05
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

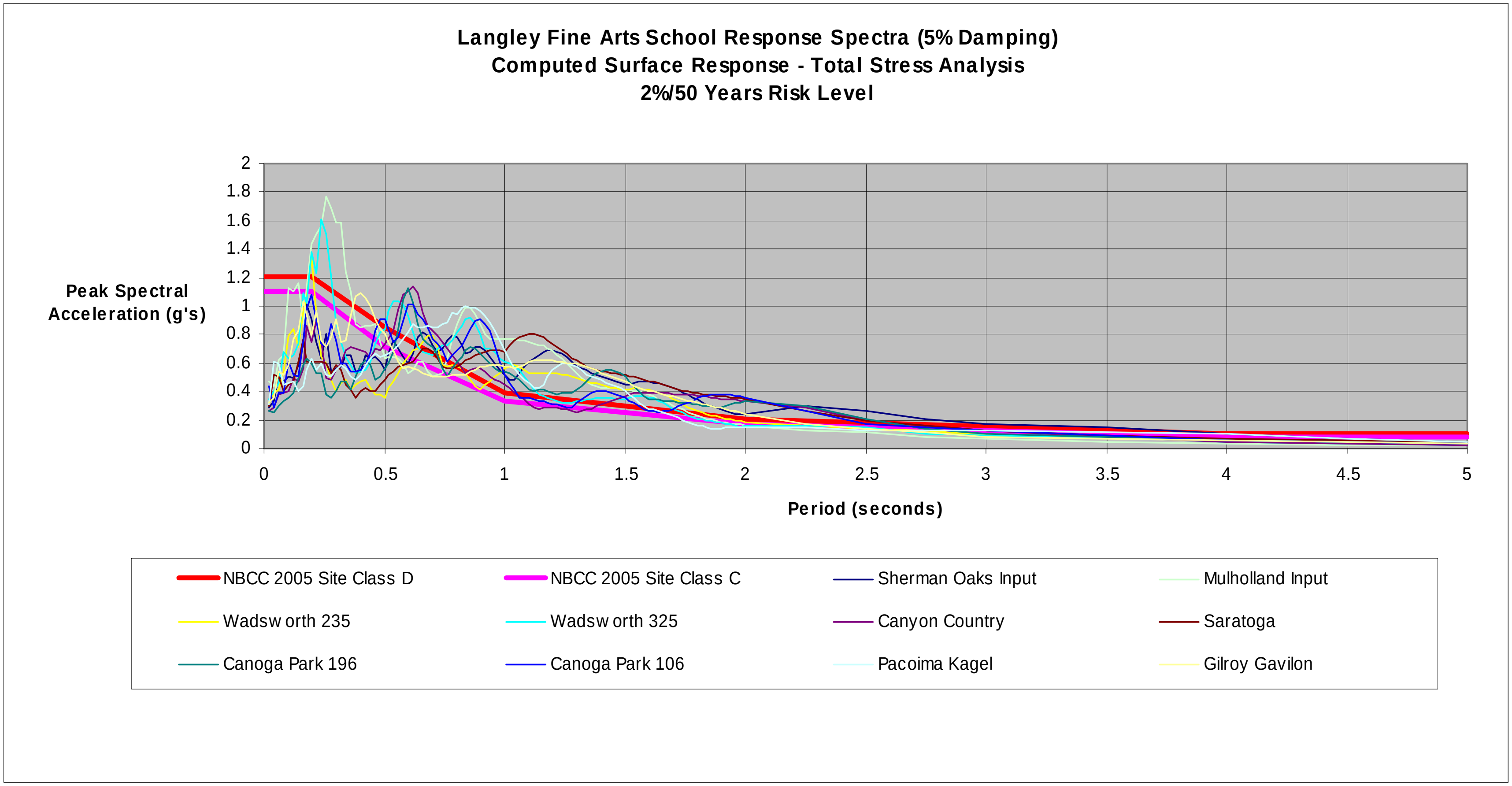
(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound</u>

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Amax (g)
1
0.8
0.6
0.4
0.2
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

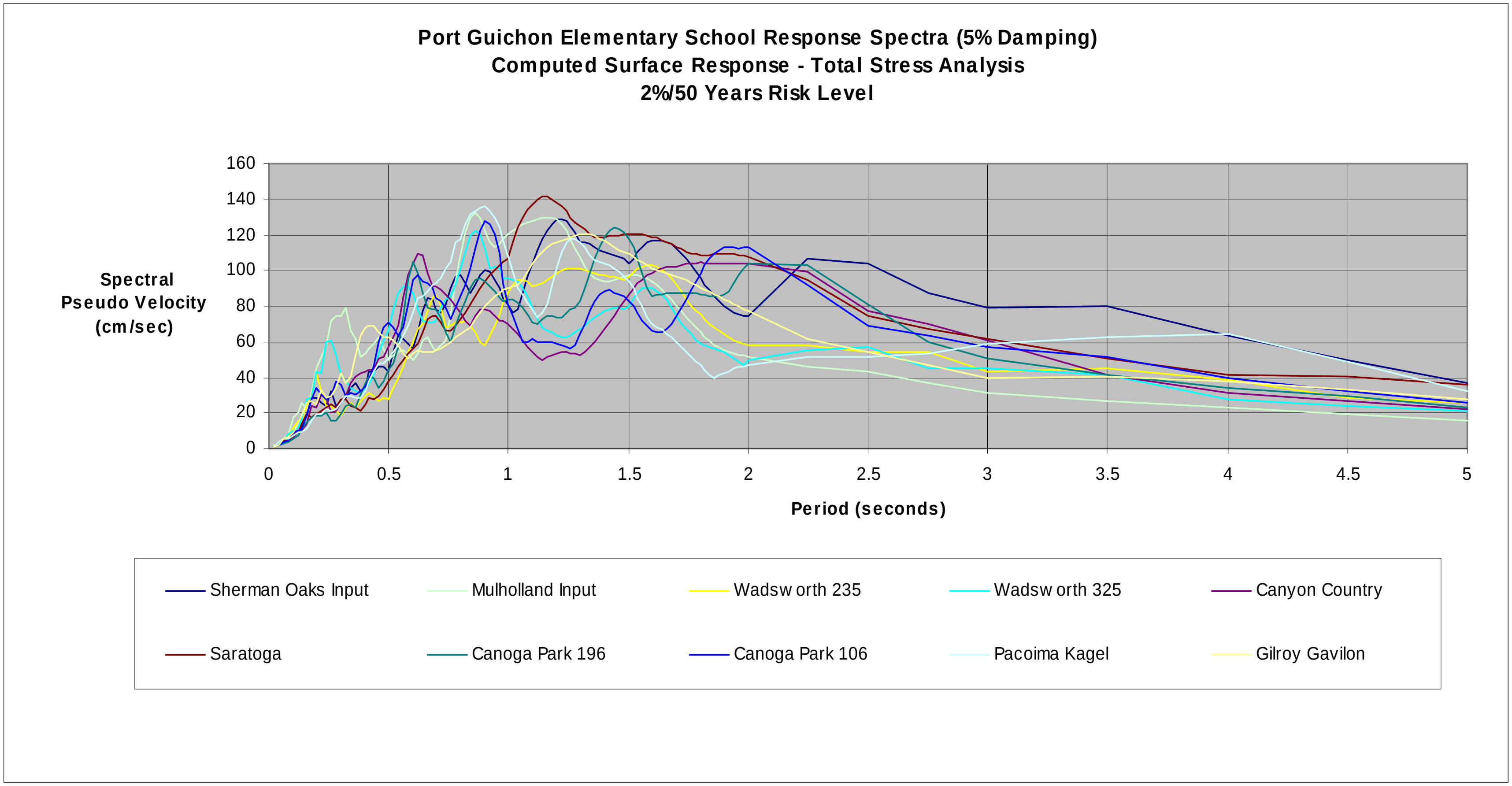
(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound

(f) Graph - surface acceleration response spectra


Langley Fine Arts School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound</u>

(g) Graph - surface velocity response spectra


Port Guichon Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Lower Bound

(h) DESRA soil properties

Nonlinear Site Response Analysis: Dynamic Soil Properties for Langley Fine Arts School

Layer No.	Soil Type	Thk. (ft)	Unit Weight (pcf)	Trow Vs (ft/sec)	Trow Gmax (psf)	Drained Sand shear strength (psf)	Undrained Shear Strength at PW Cavitation (psf)
1	granular fill	2.9848	110	500	854037.2671	62.41983	n/a
2	sand & gravel	2.0008	120	655	1598850.932	225.2539	n/a
3	sand & gravel	4.99872	120	755	2124316.77	436.4611	n/a
4	sand & gravel	4.99872	120	755	2124316.77	682.9742	n/a
5	sand & gravel	4.99872	120	785	2296490.683	935.4157	n/a
6	sand & gravel	2.9848	120	950	3363354.037	1249.282	n/a
7	sand & gravel	2.0008	120	965	3470403.727	1396.03	n/a
8	sand & gravel	4.99872	120	1000	3726708.075	1602.057	n/a
9	sand & gravel	10.004	120	1035	3992142.857	2043.652	n/a
10	sand	4.99872	120	855	2724316.77	2291.983	n/a
11	sand (saturated)	4.99872	120	820	2505838.509	2415.979	4352.966
12	sand & gravel (saturated)	4.99872	120	900	3018633.54	2465.343	4580.448
13	sand & gravel (saturated)	4.99872	120	1035	3992142.857	2748.043	4807.93
14	sand & gravel (saturated)	10.004	120	1050	4108695.652	2943.159	5149.301
15	sand & gravel (saturated)	10.004	120	1100	4509316.77	3203.371	5604.563
16	sand & gravel (saturated)	10.004	120	1130	4758633.54	3562.39	6059.825
17	sand & gravel (saturated)	20.008	120	1187	5250816.149	3963.842	6742.718
18	sand & gravel (saturated)	16.0064	120	1215	5501459.627	4445.585	7562.19
19	sand & gravel (saturated)	4.99872	120	1245	5776490.683	4726.558	8040.14

(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound

(a) E-mail dated March 08, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: graham taylor, tbg
Sent: Thursday, March 08, 2007 11:21 AM
Subject: Langley Fine Arts School - site response using "upper bound" soil strengths

Graham,

I reran the SRA using "upper bound" sand shear strengths with results as per the attached files. Here "upper bound" refers to the use of sand shear strengths below the water table (14m depth) based on assumed strong dilative sand response (typical of denser sands) where negative pore pressures approach -1 atmosphere (pore water cavitation) and undrained (zero volume change) sand response occurs during rapid cyclic shaking. As before, no additional Rayleigh-type damping was considered in the model - only internal hysteretic damping of the soil mass.

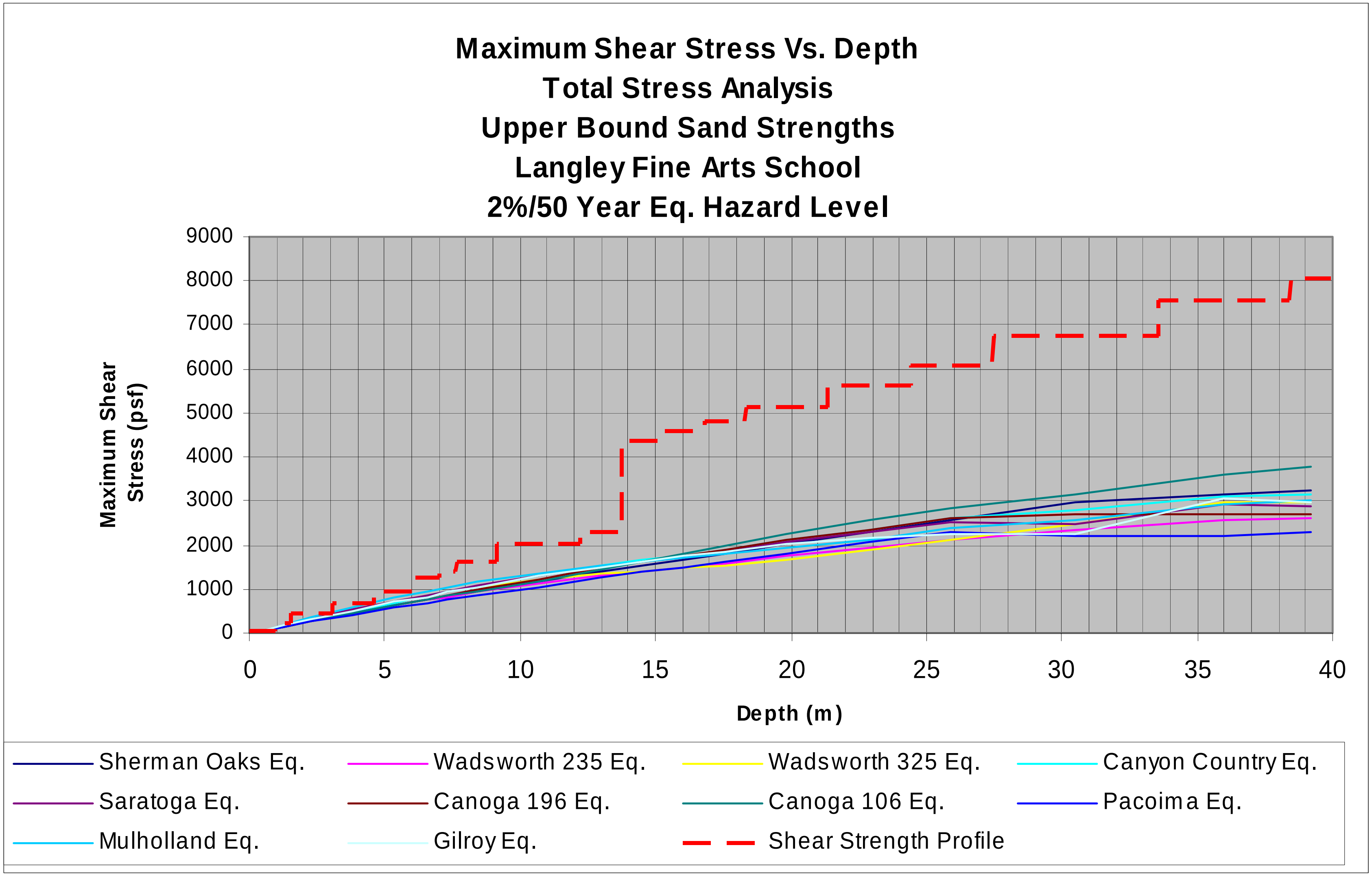
I would like to know how sensitive your structural response calculations are to the influence of soil strength.

Regards,

Blair

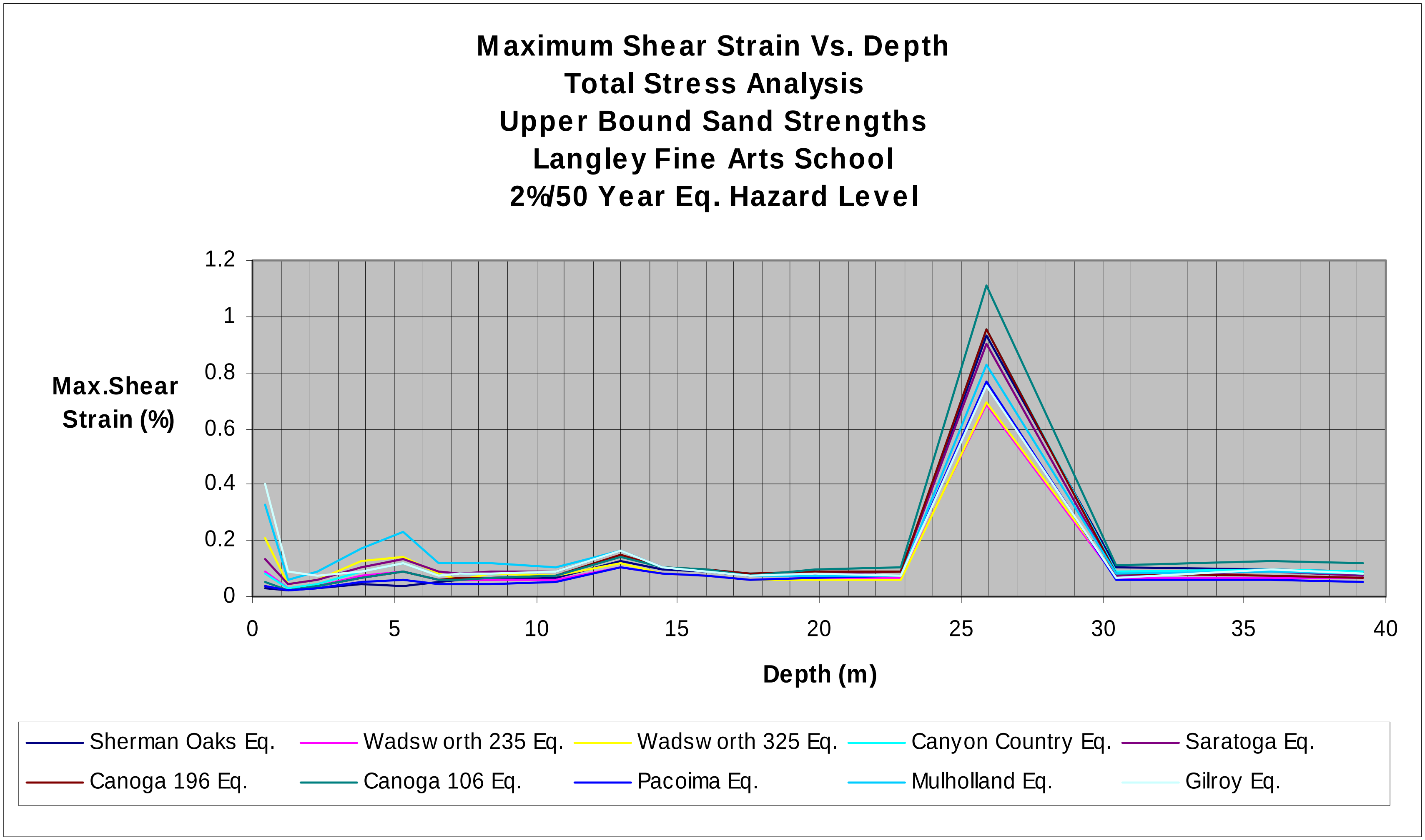
(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound</u>

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Upper Bound Sand Strengths
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

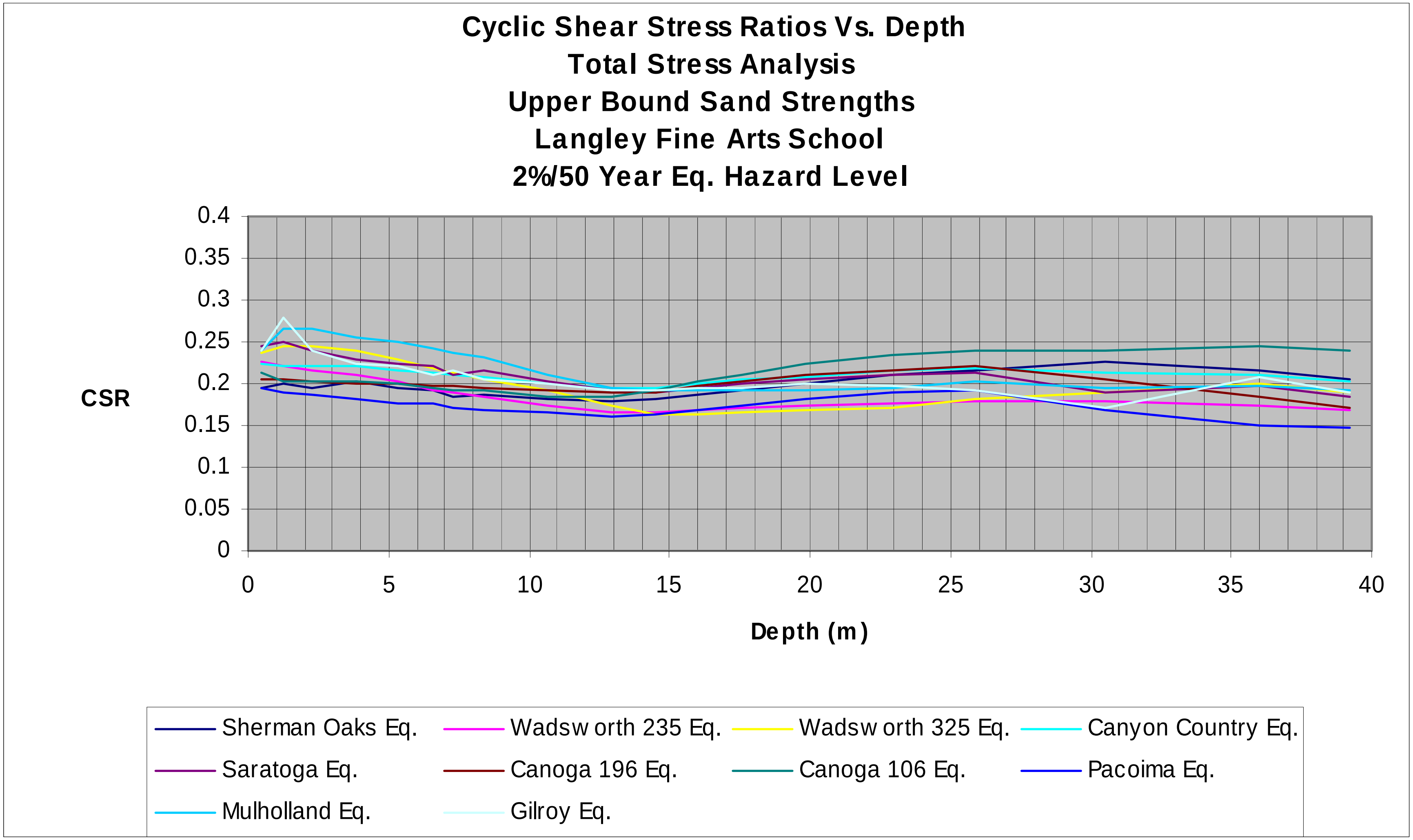
(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Upper Bound Sand Strengths
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
1.2
1
0.8
0.6
0.4
0.2
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

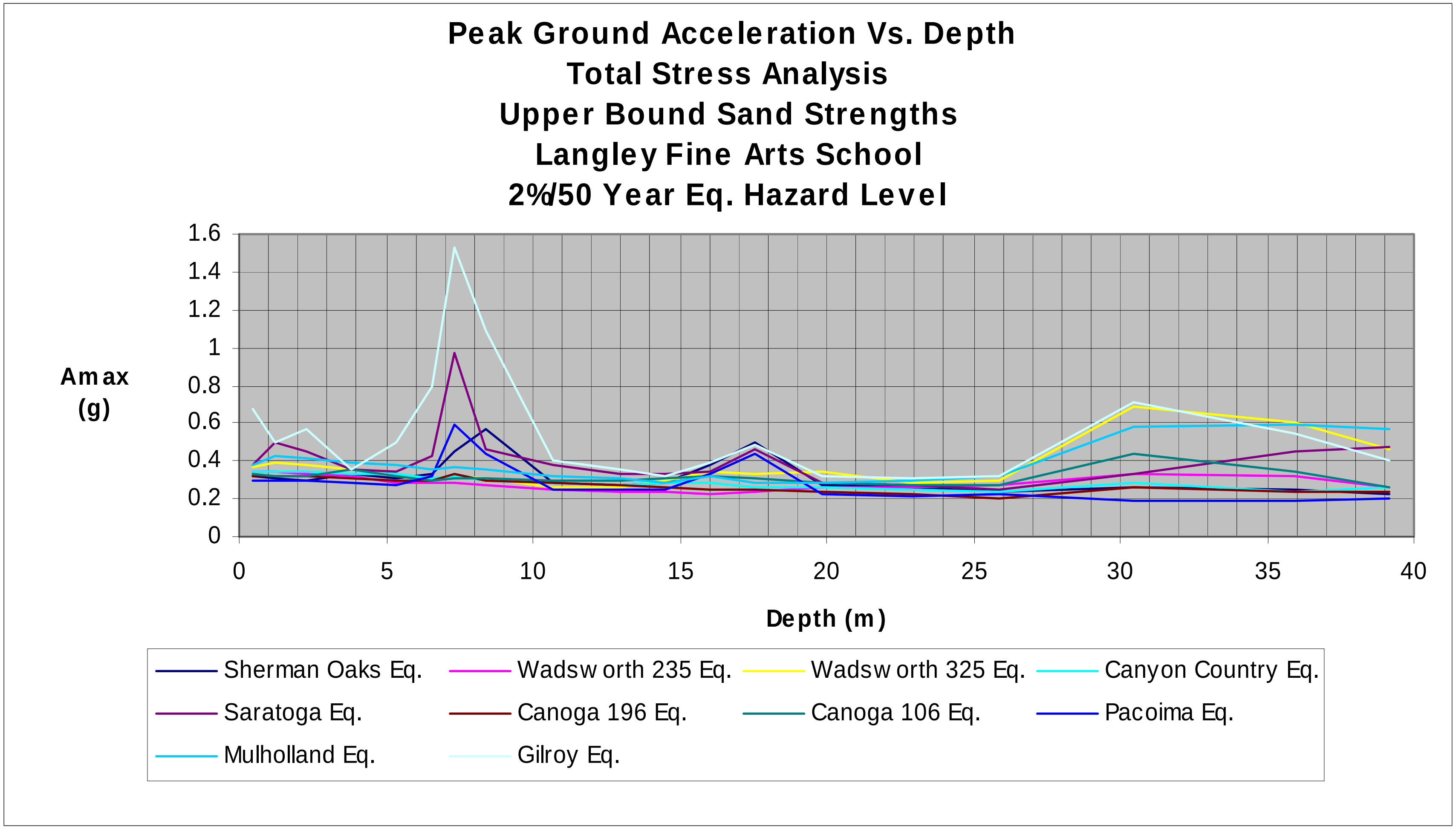
(2) Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Upper Bound Sand Strengths
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
CSR
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

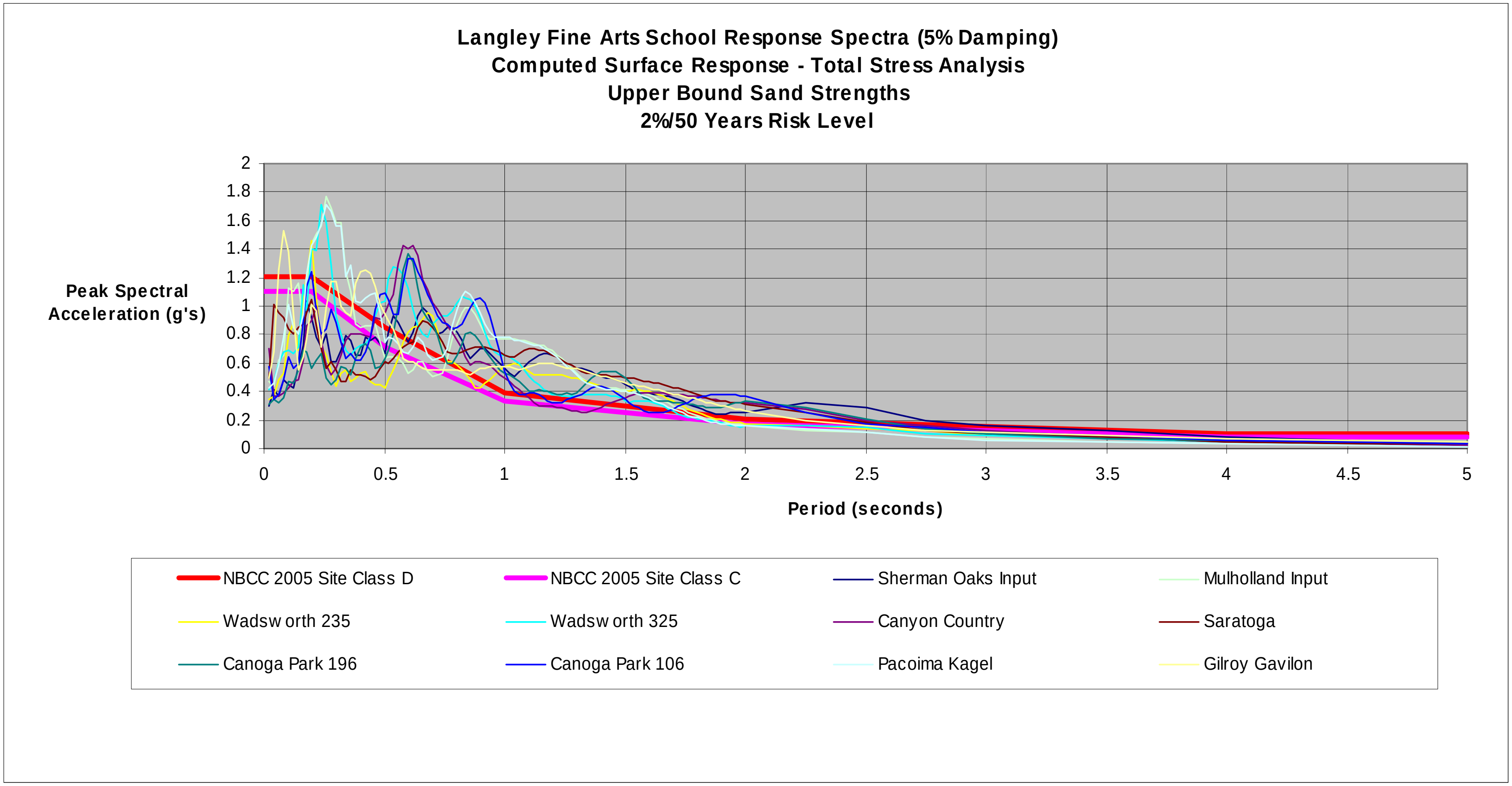
(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound</u>

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Upper Bound Sand Strengths
Langley Fine Arts School
2%/50 Year Eq. Hazard Level
Amax (g)
Depth (m)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
0
5
10
15
20
25
30
35
40
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound</u>

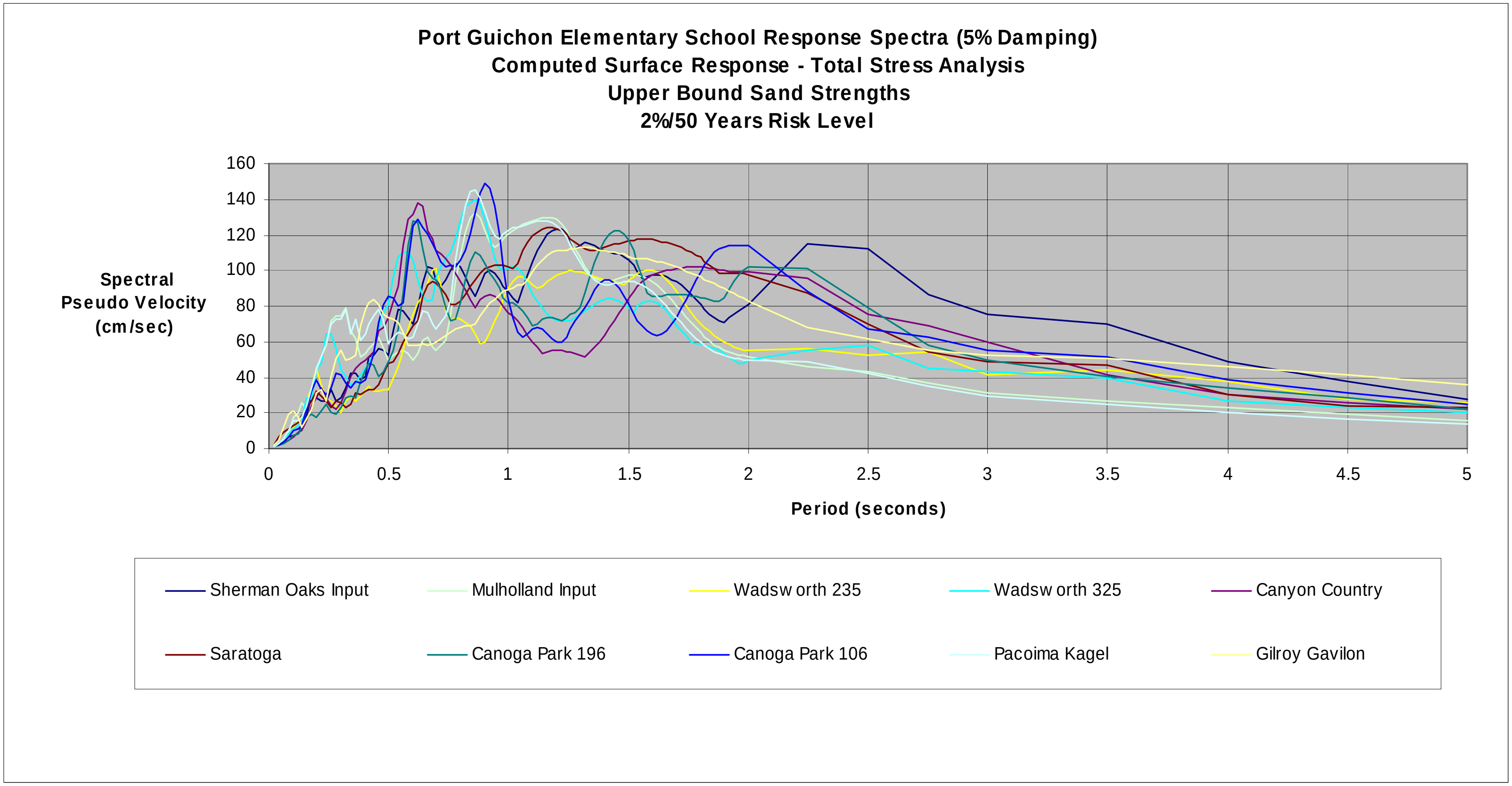
(f) Graph - surface acceleration response spectra



Langley Fine Arts School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
Upper Bound Sand Strengths
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

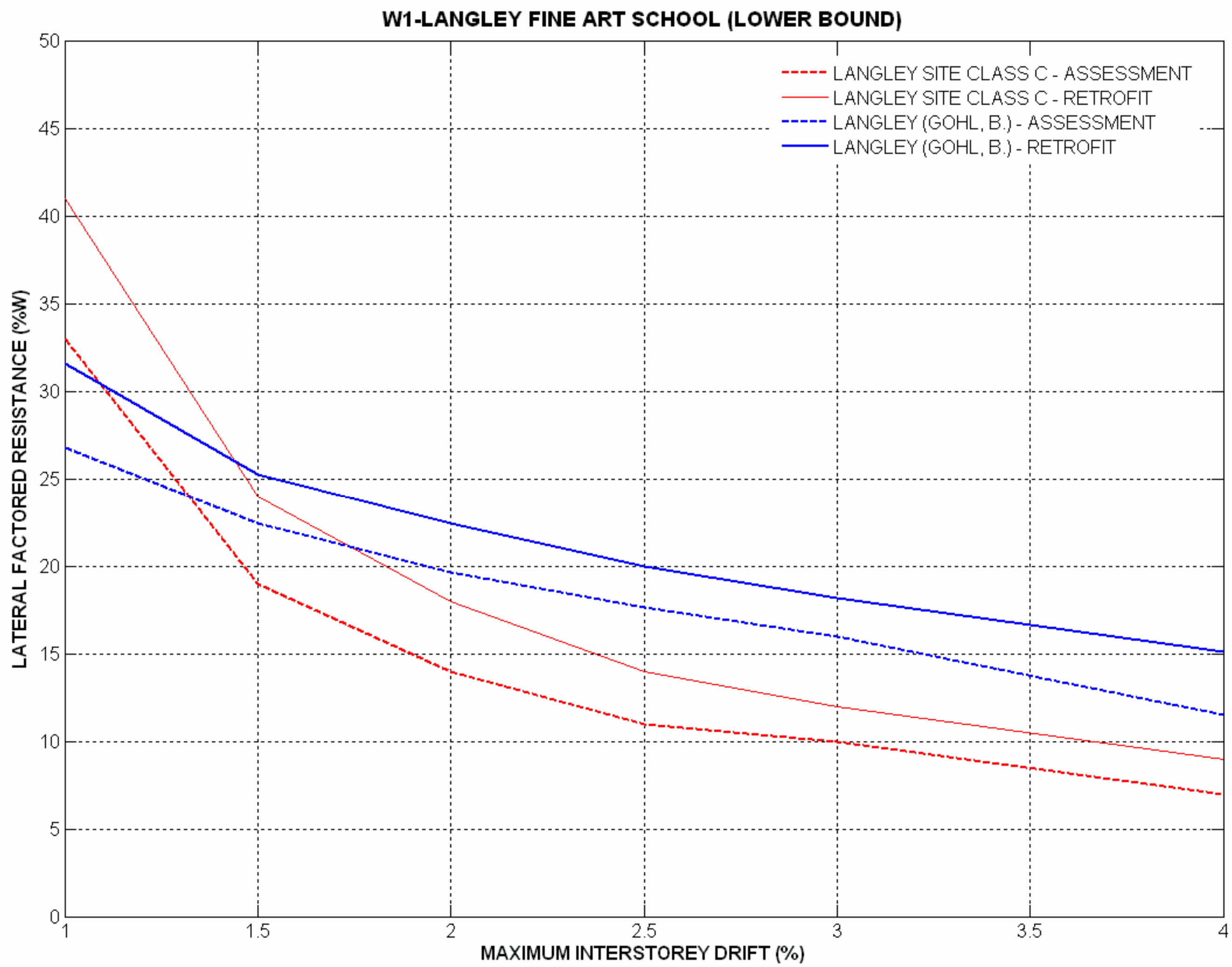
(2) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Upper Bound</u>

(g) Graph - surface velocity response spectra


Port Guichon Elementary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
Upper Bound Sand Strengths
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

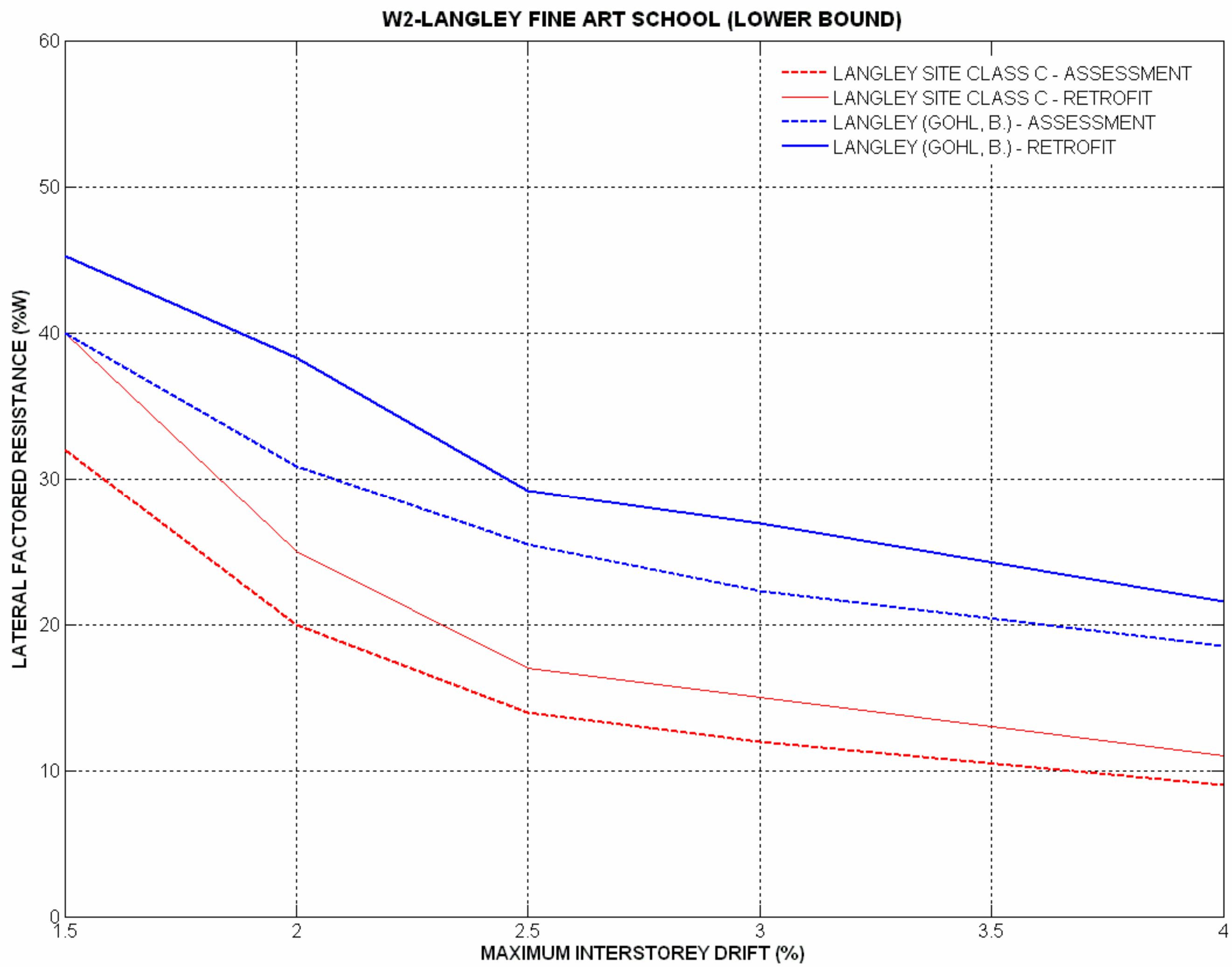
(4) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1 (Lower bound)


W1-LANGLEY FINE ART SCHOOL (LOWER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

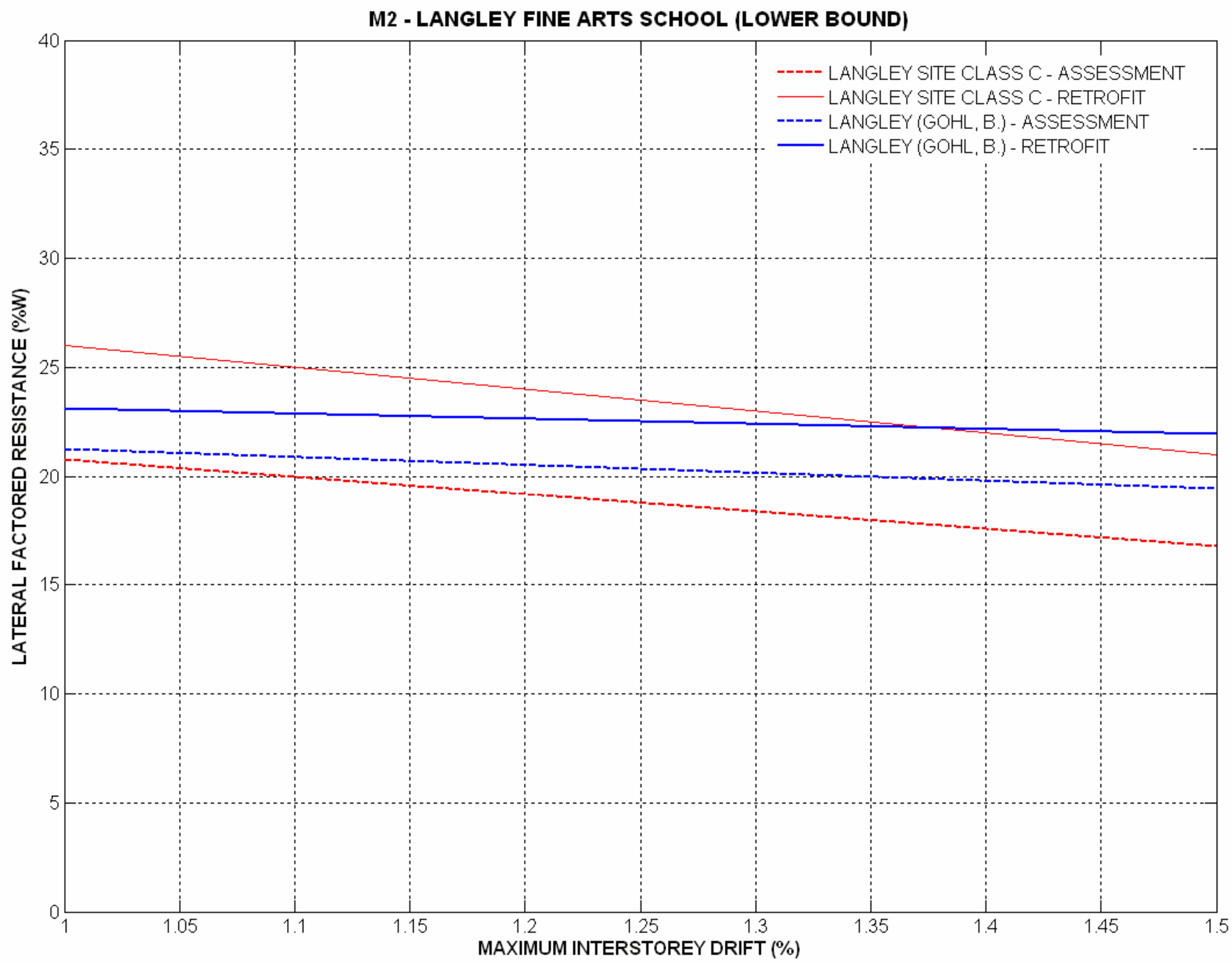
(4) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for W-2 (Lower bound)


W2-LANGLEY FINE ART SCHOOL (LOWER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1.5
2
2.5
3
3.5
4

(4) UBC Report (Structural Analysis)

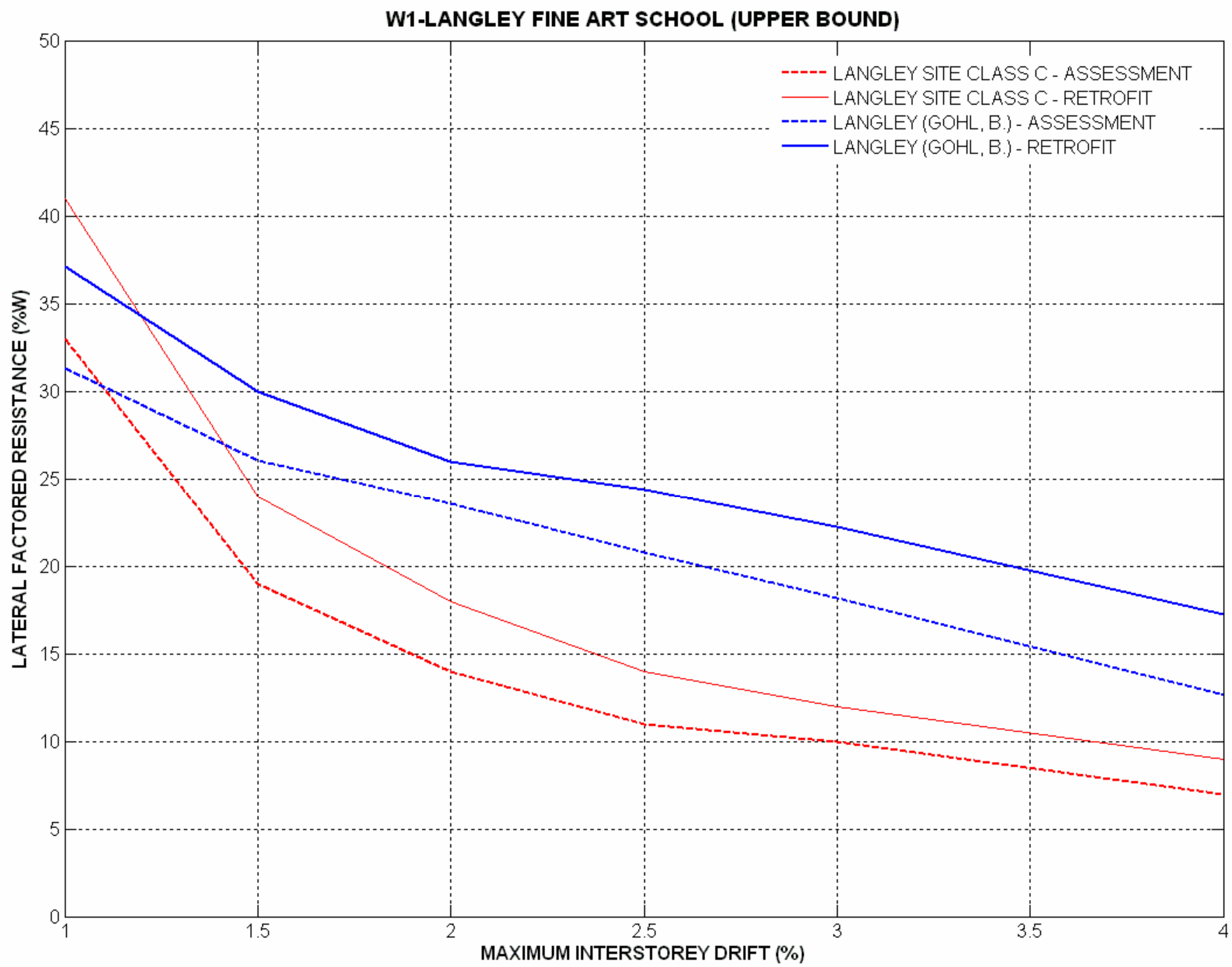
(c) Graph - lateral factored resistance versus drift for M-2 (Lower bound)



M2 - LANGLEY FINE ARTS SCHOOL (LOWER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

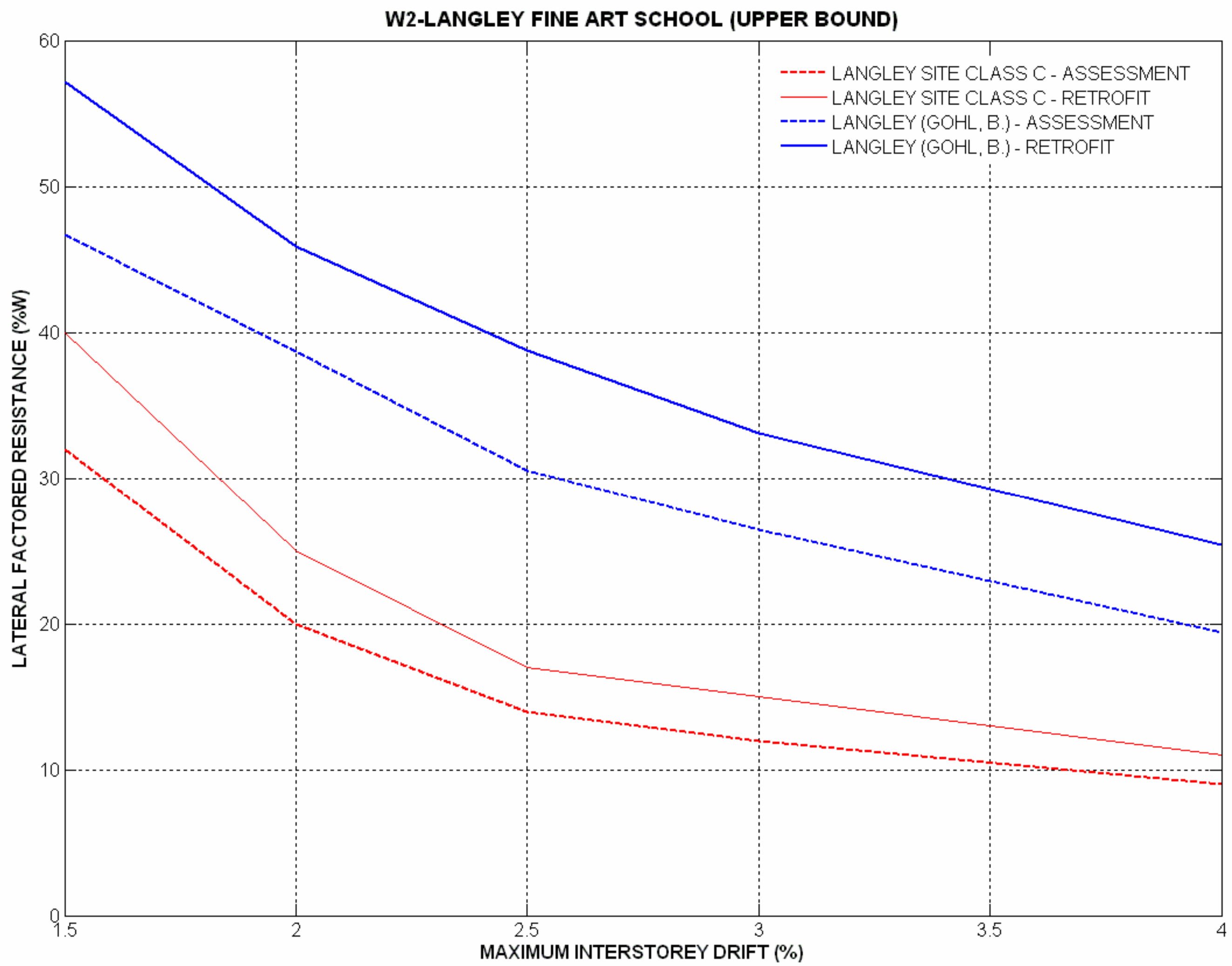
(4) UBC Report (Structural Analysis)

(d) Graph - lateral factored resistance versus drift for W-1 (Upper bound)


W1-LANGLEY FINE ART SCHOOL (UPPER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

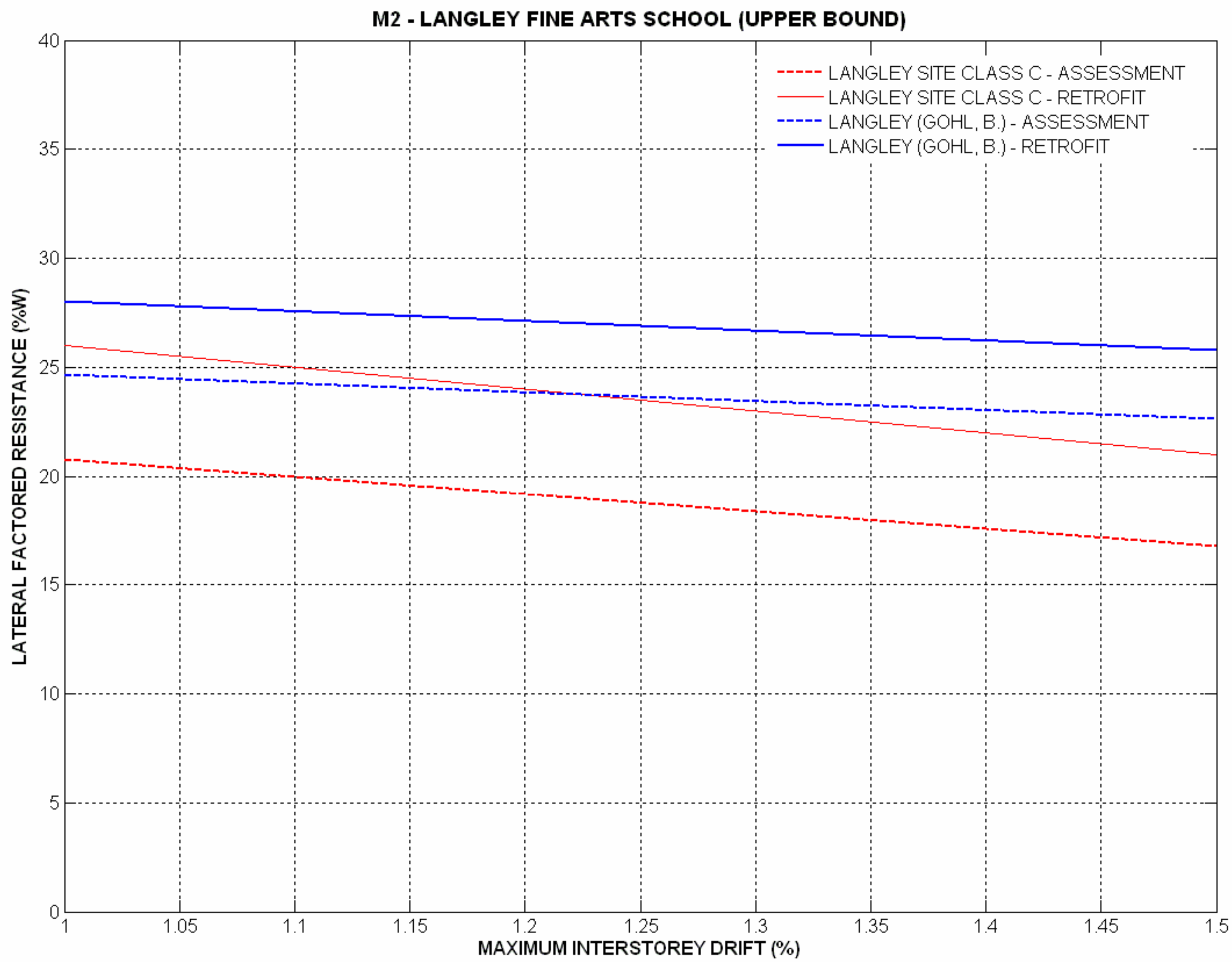
(4) UBC Report (Structural Analysis)

(e) Graph - lateral factored resistance versus drift for W-2 (Lower bound)


W2-LANGLEY FINE ART SCHOOL (UPPER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1.5
2
2.5
3
3.5
4

(4) UBC Report (Structural Analysis)

(f) Graph - lateral factored resistance versus drift for M-2 (Lower bound)



M2 - LANGLEY FINE ARTS SCHOOL (UPPER BOUND)
LANGLEY SITE CLASS C - ASSESSMENT
LANGLEY SITE CLASS C - RETROFIT
LANGLEY (GOHL, B.) - ASSESSMENT
LANGLEY (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX F

DETAILED RESULTS FOR
LINCOLN ELEMENTARY SCHOOL

CONTENTS

(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis)</u>

(a) Project draft report dated July 12, 2006

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Email: pgigohl@dccnet.com

July 12, 2006

Pomeroy Consulting Engineers Ltd.
Suite 400 – 6450 Roberts Street
Burnaby, B.C.
V5G 4E1

Attention: Mr. David Woo, P.Eng.

Re: Lincoln Elementary School, Port Coquitlam, B.C.
Draft Report on Geotechnical Aspects of Seismic Design

Dear Sirs,

This **draft** report summarizes geotechnical analyses and recommendations prepared by Pacific Geodynamics Inc. (PGI) pertaining to seismic retrofit design of Lincoln Elementary School. It is intended that these recommendations be used to facilitate seismic structural design of the school facilities to be carried out by Pomeroy Engineering Consultants Ltd. **The present report is submitted in draft form to permit comments to be received from all parties involved prior to the report being finalized.**

The following work tasks were carried out during the present study:

- Review of existing geotechnical information for the site provided by GeoPacific Consultants
- Assistance to GeoPacific in planning and conducting additional geotechnical site investigation, including seismic refraction surveys, Becker drill penetration testing, and downhole seismic testing
- Carrying out nonlinear, one dimensional site response analyses of earthquake wave propagation at the site to assess cyclic shear stresses versus depth, ground surface acceleration response, liquefaction triggering potential of granular soils and low plastic silts present at the site, and ground deformation (vertical and lateral) potential
- Simplified analysis of lateral deformation potential considering sloping ground conditions present at the northwest corner of the school site
- Review of potential methods of ground improvement for the foundation soils to mitigate soil liquefaction potential, reduce post-seismic ground deformations, and provide foundation underpinning.

<u>EXECUTIVE SUMMARY</u>

Geotechnical engineering analysis of the earthquake response of the Lincoln Elementary School site, Port Coquitlam, B.C. has been carried out by Pacific Geodynamics Inc., working in conjunction with Pomeroy Consulting Engineers Ltd. and GeoPacific Consultants. Discussions with the University of B.C. Dept. of Civil Engineering (UBC) and Transit Bridge Group (TBG) were also held, who provided “firm ground” seismic input motions used in the site response analysis. Seismic input motions representative of a 2% probability of exceedance in 50 years were considered, consistent with seismic design provisions of the 2005 National Building Code of Canada.

Available geotechnical drill hole and geophysical data for the site indicate that the upper 15m or so of the soil profile contains locally loose granular sands and non-plastic silts which are potentially liquefiable under design levels of seismic shaking. The upper soil profile also contains sensitive, organic silt and clayey silt layers which, while not expected to fully liquefy during a strong earthquake, will generate internal excess water pressures (over and above static ground water pressures). This pore pressure generation combined with the sensitivity of these clayey silt materials to lose shear strength if strained beyond a certain limit will weaken these materials during seismic shaking. The occurrence of liquefaction in looser granular soil layers and cyclic strain softening in the more plastic silt/clayey silt deposits will result in rather large post-seismic settlements. These will adversely impact the school building since building footings are expected to settle differentially. Analysis indicates potential ground settlements of up to 500 mm or so and differential vertical settlements of up to 400 mm in absence of ground improvement. Differential settlements will occur because of soil variability across the site.

Under level ground conditions (excluding the northwest corner of the school where a drainage ditch exists in close proximity to the building), cyclic softening of the soils above the 15 m depth could result in post-seismic lateral ground displacements of up to 0.9 m (depending on the input ground motion). Lateral displacements will increase from approximately zero at the 15 m depth (near the surface of a dense sand and gravel layer) to a maximum value near the soil surface. Non-level ground conditions at the northwest corner of the school building could increase lateral ground movements significantly. Calculations have indicated that lateral ground displacement at the soil surface could increase by up to 1.4 m (depending on ground surface velocities prior to soil liquefaction and soil strength properties) relative to level ground conditions. Since the magnitude of lateral ground movement could also vary over the site, it is recommended that building footings be tied together horizontally to minimize the effects of differential lateral ground displacement.

While seismic pore pressure generation is also predicted at larger depths (greater than 20 m) in some of the granular soil layers present, the effects of consolidation of these materials caused by pore pressure dissipation is not expected to significantly impact the school. This is due to a capping layer of dense sand and gravel between about the 15 and 20 m depth which will mask the effects of deep seated settlement. The use of near surface ground improvement (if carried out) would also reduce the effects of deep seated, post-seismic settlement.

Seismic wave propagation analyses carried out to model the effects of earthquake shaking at the site assumed vertical shear wave propagation. Seismic input motions were applied at the 40 m depth which were judged to be representative of “firm ground” consistent with the provisions of the 2005 NBCC. Energy absorbing boundaries at the base of the model were considered. The available drill hole and geophysics data were used to construct a soil layer model (with estimates of soil shear strength, stiffness and pore pressure generation properties). The input motions were then propagated upwards through the overburden soils. Computed ground surface response (acceleration time histories) were provided to UBC/TBG who processed these using a structural frame model representative of the school structure. This analysis was carried out to provide Pomeroy with seismic base shear levels to be used in structural retrofit design of the school building.

In view of the potential for significant vertical and lateral ground movements under the building envelope due to the presence of the problematic soils above the 15 m depth, ground improvement is recommended to stabilize these soils. Two methods of ground improvement are judged to be viable, depending on relative cost considerations: compaction grouting (CG) and jet grouting (JG). Details of each method are provided in the main body of the report. The compaction grouting technique is expected to mitigate the liquefaction susceptibility of looser granular soil layers through a densification process, as well as reinforcing cohesive soil layers by creating soil-grout columns that are expanded out into the soil medium. The jet grouting approach creates enlarged soil-cement columns (nominally 800 mm diameter) that will replace the surrounding soils using high pressure jetting through a rotating cutting head. The use of CG or JG methods is expected to underpin building footings and reduce post-seismic settlements of footings. If economic analysis indicates that either or both methods are feasible from a cost point of view, further analysis of the dynamic interaction between the vibrating ground and the soil-cement columns is recommended. Field testing is also recommended to demonstrate the performance of either method.

SITE DESCRIPTION

The school site is located in Port Coquitlam at 1019 Fernwood Avenue at the north end of Fernwood Avenue, north of Lincoln Avenue. The site is generally flat lying except at the northwest corner where a drainage ditch exists 7m to the west. The base of the ditch lies approximately 1.2m below the asphalt pavement surrounding the school. The presence of this ditch creates the potential for increased lateral ground deformations during seismic shaking, which would affect the northwest corner of the school structure.

ADDITIONAL FIELD INVESTIGATION

In addition to the previous auger hole and electronic cone penetration testing carried out at the site under the supervision of GeoPacific Consultants, the following additional geotechnical site investigation was carried out during the present study:

- 2 seismic refraction lines located along an east-west line north of the school, and a north-south line located west of the school. This work was carried out by Frontier Geosciences Inc. and their report is presented in Appendix A. The seismic refraction data were used to infer seismic shear wave velocities versus depth along each seismic line. The latter constitute key data required for analysis of earthquake wave propagation up through the overburden soils under design levels of earthquake shaking.
- Becker drilling involving the advance of a 168 mm OD open end, double walled casing using a percussive air hammer. During casing advance, air was forced down the annulus of the casing resulting in soil samples being blown up the inside of the casing for collection by a field technician supplied by GeoPacific and logging of the soil strata encountered. It was found that the open end Becker casing was able to be driven through dense sand and gravel strata encountered at the site to maximum depths of 37.5 m. At some locations, casing advance was halted at the surface of a dense gravel layer located at the 10 to 15m depth. The presence of this dense gravel layer made drilling to larger depths difficult. Four open end Becker holes (BH-1 and BH-3 to BH-5) were advanced to depths varying between 11.0 m and 37.5 m.
- Becker Penetration Testing at the BH-2 location involving the advance of the above casing but fitted with a closed end drill bit. Energy measurements of the air hammer used to drive the casing were made by attaching strain gauges and accelerometers to the top of the casing. Strain gauge/accelerometer output recorded during selected impacts of the air hammer were recorded using a high speed data acquisition system, and this data further processed to compute the input energy applied to the top of the casing. The input energy of the Becker hammer is important in order to correct Becker blow counts (number of blows required of the hammer to advance the casing 0.3m) to a standard hammer energy efficiency (3.3 kN-m) for typical Becker drills. From this corrected blow count, correlations between corrected Becker blow count (Nbc) and Standard Penetration Test blow count (N_{60}) were made. The latter is commonly used within geotechnical engineering practise to infer relative density and liquefaction resistance of granular soils (sands, gravels and non-plastic silts). The energy measurements indicated that a reasonable average energy of 3.0 to 3.6 kJoules was applied to the casing based on measurements recorded at the 9.15 m depth (within a sensitive firm clayey silt layer) and 14.15 m depth (within a dense sandy gravel layer). The closed end Becker casing was advanced to the 16m depth before the dense gravel conditions encountered prevented further advance. Beyond this point an open end casing was driven to a maximum of the 19m depth and soil samples obtained.

- A downhole seismic survey carried out at the BH-5 location by Frontier Geosciences. This required grouting in a PVC casing after completion of Becker drilling. Geophones were then be placed down the casing and used to detect shear wave and compressive wave arrivals created by impacting a shear beam at the ground surface. Details of the testing methodology are provided in the Frontier Geosciences report presented in Appendix A. The casing was placed to a maximum depth of 29.5 m due to the presence of artesian water pressures below this depth which precluded further casing advance.

The Becker drilling and soils logging was carried out under the full time supervision of GeoPacific Consultants Ltd. Pacific Geodynamics was on site during advance of BH-2 and carried out the Becker hammer energy measurements. Drill hole logs prepared by GeoPacific are presented in Appendix B.

Moisture content and Atterberg Limit measurements were made on selected cohesive (silt, clayey silt and organic silt) soil samples which, in turn, were used to infer cyclic pore pressure generation characteristics during seismic shaking, as well as post-earthquake volume change potential.

GENERAL SOIL AND GROUNDWATER CONDITIONS

We have reviewed an earlier geotechnical report prepared by GeoPacific Consultants (2005) for the site as well as the results of more recent drilling and seismic surveys in order to determine a reasonable average soil stratigraphy to be used in analysis of earthquake wave propagation at the site. The earlier 2005 GeoPacific drill hole data is presented in Appendix C, including auger hole and electronic cone penetration test logs.

Available surficial geology maps for the area (reference Geological Survey of Canada Map 1484A) indicate the site is underlain by post-glacial lowland and mountain stream deltaic, channel fill and overbank sediments up to 15m thick. The latter overlie glaciated Capilano Sediments consisting of raised deltaic and channel fill sand to cobbly gravels up to 15m thick deposited by glacial meltwater streams. These sediments were deposited in the Pleistocene epoch during the last glaciation. The deltaic and channel fill materials are commonly underlain by silt/clay materials.

The above surficial geology is generally consistent with soil conditions encountered at the site. Based on the drilling and electronic cone penetration testing carried out at the site and review of available moisture content and Atterberg Limit data, the following average soil profile exists under the school building:

0 – 1.2 m	Dense, sand or sand and gravel (Fill)
1.2 – 3.5 m	Firm, over-consolidated, sensitive, clayey Silt, low to intermediate plasticity (post-glacial deposit)
3.5 – 4.6 m	Compact to loose, sand or gravelly sand
4.6 – 8.8 m	Firm to stiff, over-consolidated, sensitive, clayey Silt, intermediate plasticity (post-glacial deposit)
8.8 – 10.5 m	Loose to compact, sand or sandy gravel (post-glacial)
10.5 – 14.3 m	Firm to stiff, lightly over-consolidated, sensitive, clayey Silt, intermediate plasticity (post-glacial deposit)
14.3 – 19.8 m	Dense, sandy Gravel (glacial Capilano Sediment)
19.8 – 26.5 m	Compact to dense, Sand with sand and gravel layers (glacial Capilano Sediment)
26.5 – 29.5 m	Very stiff to hard, Silt, heavily over-consolidated, low plasticity (glacial Capilano Sediment)
29.5 – 37.5 m	Compact to dense, Sand, some gravel (glacial Capilano Sediment)

The groundwater table was encountered at the 1.2 to 3m depth during the various field investigations. Artesian pressures were also encountered below the 29.5m depth with water heads rising to about 3m above the existing ground surface.

The above average soil profile will vary somewhat across the site as may be seen from examination of the various borehole/cone penetration test logs presented in Appendix B and C. At some locations, organic silts, amorphous peats and loose sands were found at shallow depths. This soil variability impacts building footing bearing capacity, particularly under seismic loading conditions.

From a seismic design point of view, the soils above about the 15m depth are considered problematic and subject to seismically induced softening or, for the granular non-plastic soils, complete liquefaction during seismic shaking. The granular soils below about the 20m depth may also have locally looser zones which are potentially liquefiable. However, the relatively large depth of these looser zones means that the effects of deep-seated liquefaction will be masked by the near surface soils, especially where ground improvement is carried out. Thus, deep seated liquefaction should not be of concern to the school structure for reasons discussed in more detail subsequently.

Moisture contents measured in the clayey silts located above the 15m depth are shown plotted in Figure 1 for comparison with Atterberg Limit (plastic limit, liquid limit) values on selected silt samples. Moisture contents are typically in excess of the liquid limit of the material, indicating that the materials are subject to significant strength loss when disturbed beyond a particular strain threshold. This strain softening behaviour is termed “sensitivity”. Previous auger drill sampling of these materials have indicated a very soft to soft consistency due to the fact that these materials were disturbed during auger advance.

The available electronic cone penetration data indicate peak undrained strengths of 25 kPa or greater in the clayey silts (in absence of disturbance), as shown plotted in Figure 2. The peak undrained strengths have also been used to estimate the over-consolidation ratio of the materials (OCR = maximum past vertical effective stress divided by existing vertical effective stress). OCR versus depth is shown plotted in Figure 3 and indicates that above about the 8 m depth, the clayey silt materials are over-consolidated (OCR > 1) and approximately normally consolidated below this depth. Over-consolidation is likely due to the effects of groundwater table fluctuation or sub-aerial weathering at the site. It is understood that no site preloading was carried out prior to school construction. Over-consolidation, in general, increases the peak undrained strengths of the materials and their resistance to water pressure build-up and soil stiffness/strength reduction during earthquake shaking.

Atterberg Limit results for selected cohesive soil samples above the 15m depth are shown plotted in Figure 4 in a chart of liquid limit versus plasticity index to indicate susceptibility to liquefaction for soils having high fines contents. The majority of samples plot in a zone where complete liquefaction is not considered likely. One of the samples plots in a zone (Zone B) where liquefaction is considered possible but further cyclic soils testing is required to confirm this. Based on cyclic test data obtained from BC Hydro for lower plasticity silts present in the Fraser Delta as well as data presented in the engineering literature for Fraser Delta silts (Wijewickreme and Sanin, 2004), it is judged that complete liquefaction of these materials is unlikely under design levels of earthquake shaking. Cyclic pore pressure build-up is likely to occur in these materials, discussed later in this report, which will result in post-seismic consolidation (settlement) of these materials.

The electronic cpt and Becker blow count information (closed end and open end) have been used to infer equivalent Standard Penetration Test N_{60} values corrected to a vertical effective pressure of 1 atmosphere (termed an $N_{1,60}$ value) for the various granular soil layers. The $N_{1,60}$ values have been used to estimate relative densities and cyclic liquefaction resistance during earthquake loading following methods used in standard geotechnical engineering practise. The N_{60} values have been estimated using the following procedures:

- Assuming a ratio of cone tip resistance (in units of bars where 1 bar = 100 kPa) to N_{60} of 5 in predominantly clean sand layers. The adopted ratio is consistent with geotechnical engineering practise for relatively clean, fine to medium grained sands
- Assuming a corrected, closed end Becker blow count Nbc (for an average input hammer energy of 3.3 kJoules considered representative of the Becker hammer used at the site) equal to 1.5 times N_{60} based on previous correlations between Becker hammer blow counts (corrected to a standard energy level of 3.3 kN-m) and Standard Penetration N_{60} values carried out in the Fraser River Delta (Sy and Campanella, 1994)
- Assuming an average ratio between a closed end Becker blow count and an open end Becker blow count of 1.16 to 1.5 based on correlations established at the site below a depth of about 8 m. This ratio reflects the influence of soil stresses on the tip of a closed end casing which increase driving resistance and blow counts relative to an open end casing. This ratio will vary depending on the relative contributions of external casing friction and casing tip resistance, which is site specific and depth dependent.
- Using the relationship $N_{1,60} = C_N N_{60}$ where C_N is a stress level correction factor. At shallow depths, C_N should not exceed a value of 2.

Inferred $N_{1,60}$ values versus depth in the granular soil layers are plotted in Figures 5a and 5b using ratios of closed to open Becker blow counts of 1.5 and 1.16, respectively. Values inferred using cpt and closed end Becker measurements are also shown. The plots indicate very loose to loose granular deposits between the 7 and 10m depths ($N_{1,60} < 10$) and generally compact ($N_{1,60}$ = 10 to 30) to dense ($N_{1,60} > 30$) deposits above the 7m depth. Locally loose, near surface sands were encountered at the AH-1/CPT-1 location. There is an intermediate “marker layer” of dense sandy gravel at depths ranging between 10 m and 20 m based on the available drill hole data. At larger depths, the granular strata appear to have a generally compact density with locally loose and dense layers based on the available open casing Becker data from BH-5.

Shear wave velocities (Vs) versus depth derived from seismic refraction and downhole seismic methods are shown plotted in Figure 6. Seismic refraction Vs profiles developed by Frontier Geophysics in the vicinity of AH-2 at the north-west corner of the school are compared with downhole Vs measurements in the same general area (BH-5). The Vs versus depth trend-line from the seismic refraction survey is in reasonable agreement with the downhole seismic measurements down to the 24m depth. Below this depth, the downhole seismic velocities exceed 500 m/sec whereas the seismic refraction survey indicates velocities in the range of 300 to 390 m/sec velocities from the 30 to 40 m depth. The Vs versus depth profile used for seismic wave propagation analysis at the site (discussed later) is also indicated in the figure based on the average soil layer profile adopted.

The downhole seismic velocity between the 22 and 24 m depth indicates an average velocity of 218 m/sec in the granular materials present at this depth. Liquefaction triggering under design earthquake motions is indicated as being likely for these lower velocity materials (described later in the section on liquefaction triggering). This supports indications provided by inferred $N_{1,60}$ values developed from open casing Becker drilling that localized liquefaction in the deeper granular soil deposits (Capilano Sediments) may occur.

“FIRM GROUND” SEISMIC INPUT MOTIONS

It is necessary to define input earthquake motions at “firm ground” level in order to carry out analysis of earthquake wave propagation for a particular site. These input motions will depend on seismic risk levels being considered for design. In the case of Lincoln Elementary School, a seismic risk level having a 2% probability of being exceeded in 50 years has been adopted, consistent with the provisions of the 2005 National Building Code of Canada (NBCC).

The Geologic Survey of Canada (2003) report defining seismic ground motion parameters to be considered throughout Canada for the above seismic risk level states that “firm ground” is defined by materials having shear wave velocities in the range of 360 to 750 m/sec. Thus input earthquake motions selected for the study were placed at the 40 m depth where the soil materials had shear wave velocities in excess of 500 m/sec based on the downhole seismic profiling. The 2005 NBCC defines these firm ground conditions as "Site Class C" soil conditions, representative of very dense soil, or soft rock.

The earthquake input motions (specified as horizontal accelerations versus time, termed an accelerogram) selected for seismic wave propagation analysis were supplied by the University of British Columbia Dept. of Civil Engineering (UBC) and the Transit Bridge Group (TBG) who are actively engaged in seismic research pertaining to seismic design of school structures. The input motions were recorded at the ground surface during previous earthquakes at a variety of sites in California on soil conditions considered representative of Site Class C soils. The input firm ground motions were scaled from the original accelerograms so that after scaling their peak spectral velocity (PSV) averaged over the 0.5 to 1.5 second period range matched a target PSV (= 55.4 cm/sec) specified by the Geologic Survey of Canada (2003) for the Port Coquitlam area. The input accelerograms adopted for the present study and the scaling factors applied to the original accelerograms are presented in Table 1. The peak firm ground acceleration (PGA) and average peak spectral velocity (PSV) over the 0.5 to 1.5 second period range <u>prior to scaling</u> for each input motion are also presented in the table. Elastic response spectra computed for 5% structural damping after scaling of each accelerogram are shown in Figure 7.

Table 1
Input Firm Ground Motions

INPUT ACCELEROGRAM	SCALE FACTOR	PSV (cm/sec)	PGA (g's)
(1) Sherman Oaks – 105E	1.25	44.1	0.214
(2) Wadsworth - 235E	1.14	48.4	0.303
(3) Wadsworth - 325E	1.32	42.1	0.389
(4) Canyon Country – 0E	0.83	67.0	0.396
(5) Saratoga - 0E	0.8	69.2	0.504
(6) Canoga Park - 196E	0.71	77.8	0.300
(7) Canoga Park - 106E	1.06	52.3	0.350
(8) Pacoima Kagel – 90E	0.84	49.2	0.301
(9) 12520 Mulholland Dr. - 35E	1.13	39.7	0.588
(10) Gilroy Gavilon College - 67E	1.4	39.7	0.356

The above earthquakes have been recorded during earthquakes with magnitudes in the range of 6.5 to 7.5 and are considered representative of earthquake magnitudes likely to affect the school site for the seismic risk levels being considered.

SITE RESPONSE ANALYSIS – GENERAL METHODOLOGIES

Earthquake wave propagation at the site was assumed due predominantly to vertically propagating shear waves for purposes of assessing soil liquefaction and deformation potential and estimating horizontal inertial forces (base shear) acting on the school superstructure. This assumption is consistent with seismic design practice in the Vancouver Lower Mainland. The one dimensional analysis program DESRA2C developed by Lee and Finn (1978) was used for this purpose. The program models a column of soil elements subject to seismic base excitation. The nonlinear, cyclic shear stress-shear strain response of individual soil elements at a particular depth are modeled, including the effects of pore water pressure generation. Level ground conditions are assumed, i.e. without the effects of imbalanced shear stress acting within a soil element due to sloping ground conditions or the effects of local stresses caused by building footings. A level ground condition is judged to be a reasonable assumption over most of the school site, except in the northwest corner of the school where a local drainage ditch exists. The effects of non-level ground conditions on seismic lateral ground deformations are discussed in a later section of this report.

Where pore pressure generation is not considered and a constant over time, soil stress-strain "backbone" curve is modeled, this is termed a "total stress" analysis. Use of a total stress analysis leads to maximum prediction of shear stresses, accelerations and inertial forces transmitted to the school structure through the soil profile. Where softening of the soil stress-strain backbone curve occurs over time due to pore pressure generation, this is termed an "effective stress" analysis (since soil shear strength depends on total stress less pore pressure, termed effective stress). Use of an "effective stress" analysis results in lower transmission of shear stresses and accelerations through the soil profile, and lower transmitted inertial forces to the school structure. However, since greater softening of the foundation soils occurs relative to the total stress analysis, larger foundation soil deformations (vertical and lateral) develop. This leads to greater potential racking of the school superstructure unless ground improvement is carried out to mitigate these softening effects.

An average soil layering profile was considered in the analysis, discussed previously. Based on the measured shear wave velocity profiles (Vs), the small strain shear stiffness of a particular soil element (Gmax) was computed as Gmax = Δ Vs^2 where Δ is the total mass density of the soil at a particular depth. The maximum shear strength of a soil element was computed using the following general procedures:

- For cohesive soil units (organic silts, silts and clays), the static simple shear strength was estimated based on available cpt data, and then increased by a factor of 20% to account for increases in strength due to strain rate (rapid seismic loading) effects. It is noted that the use of higher strengths generally leads to maximum prediction of transmission of shear stresses and accelerations.
- For loose to compact granular soils ($N_{1,60}$ < 20), the shear strength on the horizontal plane was computed based on the assumption of fully drained strengths (using estimated peak friction angles, lateral stress K_0 coefficients, and vertical effective stresses). These soils were judged would liquefy under design levels of seismic shaking and therefore use of drained strengths was considered to provide a conservative (higher) level of strength, leading to prediction of maximum shear stress/acceleration transmission.
- For compact to dense granular soils ($N_{1,60}$ > 20), shear strengths were computed based on the assumption that these materials would not liquefy under design levels of seismic shaking and would dilate during seismic shear. This dilation generates negative pore water pressure. At peak shear strain levels, it was assumed that a limiting pore pressure equal to -1 atmosphere would develop and these negative pore pressures were assumed would contribute to the shear strength of the material based on effective stress principles. Thus, the shear strength of these denser materials was based on initial vertical effective stresses, lateral stress K_0 coefficients, and the estimated constant volume friction angle of these soils (assuming during rapid cyclic loading, no volume change occurs).

A summary of soil properties used in total stress, site response analysis is presented in Table 2. These soil properties are considered to represent reasonable "upper bound" (highest) soil resistances and therefore to lead to highest prediction of stresses and accelerations transmitted through the soil profile.

A summary of soil properties used in effective stress (with pore pressure generation), site response analysis is presented in Table 3. These soil properties are considered to represent reasonable "lower bound" soil resistances and to lead to highest prediction of lateral soil deformations, pore water pressures, and post-seismic settlements. It is noted that computed lateral ground deformations using DESRA2C assume a completely level ground topography, with no imbalanced shear stresses acting in the ground caused by soil slopes or local effects of building footing load. The effects of non-level ground will cause increased lateral ground deformation, and this is discussed in a later section.

Pore pressure generation constants used in the DESRA2C analysis were calibrated for each soil layer to achieve a specified degree of positive pore pressure normalized with respect to the effective vertical stress at the depth being considered (termed a pore pressure ratio, PPR). A PPR value of 1.0 implies that complete soil liquefaction has occurred. Ten effective cycles of shaking at a specified cyclic shear stress level divided by the vertical effective stress (or cyclic stress ratio, CSR) were used in each calibration analysis. The number of effective cycles of shaking is considered representative of the number of equivalent cycles of shaking for a magnitude 7 earthquake. For granular soil layers, the critical CSR value to cause complete liquefaction (PPR = 1.0) was based on correlations between $N_{1,60}$ and liquefaction triggering considering magnitude 7 earthquakes (Seed et al, 2003). For cohesive soil layers, the engineering literature (Wijewickreme and Sanin, 2004; Andersen et al, 1988; Tan and Vucetic, 1989) as well as previous cyclic simple shear testing carried out by BC Hydro on low to moderate plasticity silts from the Fraser River Delta were consulted to estimate PPR as a function of CSR (or cyclic shear stress divided by the simple shear undrained strength of the soil), plasticity index, and degree of over-consolidation.

Table 2 - Site Response Analysis (Upper Bound Soil Properties)

Soil Type	Layer Thk. (m)	Vs (m/sec)	Total Unit Weight (kN/cu.m.)	Gmax (kPa)	Inferred $N_{1,60}$	Shear Strength (kPa)
Dense, Sand & Gravel (Fill)	1.2	140	18.6	3.72e4	30	4.7
Sensitive, organic Silt	2.3	90	15.7	1.30e4	n/a	24
Loose Sand	1.2	160	18.5	4.86e4	10	20.5
Sensitive, Silt	4.2	95	15.7	1.44e4	n/a	24
Loose to compact, Sand or sandy Gravel	1.7	190	18.5	6.86e4	8	38.8
Stiff, clayey Silt	3.8	127	18.5	3.06e4	n/a	40
Dense, sandy Gravel	5.5	224	20.6	10.54e4	40	294*
Compact to dense, Sand with gravel layers	6.7	275	19.6	15.13e4	20	288*
Hard, Silt	3.0	300	19.6	18.01e4	n/a	200
Compact to dense, Sand, some gravel	10.5	516	20.6	55.91e4	20	364*

*Based on undrained strength for dense materials with pore water pressure at -1 atm at cavitation.

**Assume groundwater table at 1.2m depth below existing ground surface.

It is important to note that the seismic input motions specified by the UBC research group and the Geologic Survey of Canada are considered to be representative of motions occurring at the ground surface on a firm ground “outcrop”. Since firm ground representative of Site Class C conditions occurs at relatively large depth (40 metres), then some accounting for seismic wave energy dissipation into deeper materials below the 40 m depth must be made. Application of an interior seismic excitation combined with consideration of an energy absorbing bottom boundary reduces the effective seismic energy transmitted to the overlying soil layers. An energy absorbing bottom boundary was used in all DESRA2C analyses presented herein based on the theory presented by Lee and Finn. The energy absorption characteristics of the lower boundary was based on an average shear wave velocity of 550 m/sec, consistent with the average velocity implied for Site Class C soils and the local geophysical testing.

Table 3 - Site Response Analysis (Lower Bound Soil Properties)

Layer No.	Soil Type	Layer Thk. (m)	Vs (m/sec)	Total Unit Weight (kN/cu.m.)	Gmax (kPa)	$N_{1,60}$	Shear Strength (kPa)	Pore Pressure Generation
1	Compact, Sand & Gravel (Fill)	1.2	140	18.6	3.72e4	30	4.7	No
2	Sensitive, organic Silt	2.3	90	15.7	1.30e4	n/a	24	No***
3	Loose Sand	1.2	160	18.5	4.86e4	10	20.5	Yes (PPR = 1.0 for CSR = 0.23)
4	Sensitive, Silt	4.2	95	15.7	1.44e4	n/a	24	Yes (PPR = 0.6 for CSR = 0.20)
5	Loose to compact, Sand or sandy Gravel	1.7	190	18.5	6.86e4	8	38.8	Yes (PPR = 1.0 for CSR = 0.12)
6	Stiff, clayey Silt	3.8	127	18.5	3.06e4	n/a	40	Yes (PPR = 0.5 for CSR = 0.20)
7	Dense, sandy Gravel	5.5	224	20.6	10.54e4	40	294*	No
8	Compact to dense, Sand with gravel layers	6.7	275	19.6	15.13e4	20	95	Yes (PPR = 1.0 for CSR = 0.28)
9	Hard, Silt	3.0	300	19.6	18.01e4	n/a	200	No
10	Dense, Sand, some gravel	10.5	516	20.6	55.91e4	20	364*	No

*Based on undrained strength for dense materials with pore water pressure at -1 atm at cavitation.
**Assume groundwater table at 1.2m depth below existing ground surface.
***Assumed to have higher over-consolidation and low cyclic shear stress/undrained strength ratio so that limited cyclic pore pressure build-up expected.

SITE RESPONSE ANALYSIS – TOTAL STRESS ANALYSIS RESULTS

Site response analysis results using total stress approaches are presented in the following figures:

Figure 8 – Peak ground surface acceleration versus depth.

Figure 9 – Cyclic stress ratios (CSR) versus depth. The CSR at a particular depth is computed as 0.65 times the peak cyclic shear stress, divided by the vertical effective overburden stress, consistent with geotechnical engineering practice.

Figure 10 – Peak horizontal shear strains versus depth.

Figure 11 – Elastic response spectra (peak spectral acceleration versus structural building period for 5% structural damping). The latter were obtained from computed horizontal accelerations at the 1.2 m depth (approximate underside of building footing level) using the theory derived from a single degree of freedom oscillator. The computed spectra are compared with generic spectra provided in the 2005 NBCC for Site Class C and Site Class E soils.

The figures present analysis results for all 10 seismic input motions.

Computed acceleration time histories at the 1.2 m depth have been provided to the UBC/TBG research group for further input into a structural model used to compute seismic base shears transmitted to buildings representative of those at Lincoln Elementary School. We understand that this seismic base shear information will be provided separately to Pomeroy.

Ratios of the peak ground acceleration near the bottom of the soil column model (39.6m depth) used in DESRA2C to the peak input base acceleration are presented in Table 4 for each seismic input motion. The ratios are in the range of 0.58 to 0.79, showing the effect of the energy absorbing bottom boundary used in the model.

Table 4
DESRA2C Analysis Results (Total Stress Analysis)
Ratios of Peak Accel. Near Base of Model (Amax) to Peak Input Base Accel. (PGA)

INPUT ACCELEROGRAM	PGA (g’s)	Amax (g’s)	Amax/PGA
(1) Sherman Oaks – 105E	0.267	0.202	0.76
(2) Wadsworth - 235E	0.380	0.302	0.79
(3) Wadsworth - 325E	0.511	0.323	0.63
(4) Canyon Country – 0E	0.340	0.235	0.69
(5) Saratoga - 0E	0.402	0.288	0.70
(6) Canoga Park - 196E	0.308	0.201	0.65
(7) Canoga Park - 106E	0.377	0.247	0.65
(8) Pacoima Kagel – 90E	0.253	0.164	0.65
(9) 12520 Mulholland Dr. - 35E	0.674	0.423	0.63
(10) Gilroy Gavilon College - 67E	0.499	0.290	0.58

Examination of the above figures leads to the following observations:

- Amplification of ground accelerations from the base of the soil column through the overlying denser materials to about the 15 m depth
- De-amplification of ground accelerations above the 15 m depth due to the limited shear strength of the near surface soils
- Peak ground surface accelerations in the range of 0.2 to 0.29 g

- Peak CSR's in the range of 0.15 to 0.25 above the 15 m depth in the softer/looser near surface soils. The latter are used to estimate liquefaction triggering potential for the granular soil layers based on correlations between CSR and $N_{1,60}$ (Seed et al, 2003).
- Highest cyclic shear strain potential and lateral ground displacement potential above the 15 m depth in the near surface soils. The total stress analysis indicates high shear strain potential in the sensitive silt and clayey silt layers between the 4.6 - 8.8 m and 10.5 – 14.3 m depths, respectively. The looser sand layers also have high shear strain potential but this is not indicated by the total stress analysis since pore pressure generation (leading to potential liquefaction) is not considered in this analysis.
- Computed response spectra (on average) show broad agreement with the 2005 NBCC Site Class E design spectrum. Higher and lower spectral response is indicated for some of the input earthquake records relative to the Site Class E spectrum over a range of structural periods. The ongoing work by the UBC/TBG research group, however, indicates that consideration of peak spectral velocity over the period range of most interest (0.5 to 1.5 seconds) provides a better measure of seismic base shear transmitted to school building structures. Their analysis of base shears to be considered in design using computed near surface accelerograms is considered to supercede base shears computed using acceleration response spectra. The latter are provided strictly for comparison with 2005 NBCC design spectra. **(this section may require rewording or perhaps it should be entirely deleted, at the discretion of Pomeroy and UBC/TBG)**

SOIL LIQUEFACTION TRIGGERING

Based on the computed cyclic stress ratios in the range of 0.15 to 0.25 above the 15 m depth, localized loose to compact sand or sand and gravel layers with $N_{1,60}$ values less than about 23 could undergo seismic liquefaction. This is based on liquefaction triggering curves for clean sands (fines contents less than 5%) provided by Seed et al (2003) based on a design earthquake magnitude 7.0 and maximum CSR's of 0.25.

Figure 5 indicates that locally loose sand and gravel layers are present above the 15 m depth, as well as below about the 20 m depth. Higher CSR's are computed below the 20 m depth (see Figure 9) so liquefaction triggering may also be possible at larger depths.

Available shear wave velocity data (stress normalized shear wave velocities less than about 200 m/sec) also support the possibility of liquefaction triggering for sand and gravel layers above the 15 m depth, and locally within generally compact sand and gravel layers at larger depths.

The occurrence of granular soil liquefaction above the 15 m depth is considered to be of greatest concern to seismic performance of Lincoln Elementary School insofar as this influences post-seismic settlements of building footings, as well as lateral ground displacement potential. The occurrence of localized deep-seated liquefaction (in likely discrete discontinuous layers) is judged to not be as serious for adequate school foundation performance since the presence of the capping layer of dense sand and gravel from about the 15 to 20 m depth along with the effect of ground improvement (if carried out) above the 15 m depth will tend to minimize the effects of liquefaction at larger depths. This is caused by bridging action which reduces the amount of settlement at the ground surface that causes damage to shallow foundations. Ishihara (1985) suggests that for relatively level ground sites subjected to peak ground surface accelerations of about 0.3 g (indicated from the previous site response analysis), limited surface expression of deep seated liquefaction will occur if the thickness of a non-liquefiable surface layer exceeds about 6 m.

The sensitive organic silt and clayey silt layers above the 15 m depth having moderate plasticity (based on available Atterberg Limit test data) are not considered susceptible to complete liquefaction where cyclic pore pressures equal the vertical effective stress in the soil element. This was discussed in an earlier section based on measured plasticity of selected samples and available cyclic simple shear test data (from BC Hydro) on low plasticity silts from the Fraser River Delta. Some cyclic pore pressure generation in these materials is judged to be likely.

Where non-plastic silts exist above the 15 m depth (typically grading from sands to sandy silts based on available cpt data) and behave as granular soils, these are also considered potentially liquefiable.

SITE RESPONSE ANALYSIS – EFFECTIVE STRESS ANALYSIS RESULTS

Site response analysis using effective stress approaches with consideration of pore pressure generation in some of the soil layers (reference Table 3) was carried out for the following reasons:

- To provide surface acceleration time histories to be used in analysis of seismic base shear carried out by UBC/TBG. It was expected that these motions would not provide as high a measure of base shear compared to the previous total stress analysis, since ground motions are attenuated through the soil column due to pore pressure generation.
- To indicate pore pressure generation potential within the soil column and zones potentially requiring ground improvement to mitigate liquefaction triggering (within granular soil layers) or significant softening (within cohesive soil layers).
- To indicate potential magnitudes of seismic lateral ground displacement versus depth in absence of ground improvement.

Site response analysis results using effective stress approaches are presented in the following figures:

Figure 12 – Pore pressure ratios versus depth.

Figure 13 – Lateral ground surface displacements versus time relative to seismic base motion displacements.

Figure 14 – Lateral ground surface velocities (absolute) versus time

Examination of the above figures indicates :

- The occurrence of liquefaction in looser sand layers (PPR = 1.0) and cyclic pore pressure ratios of up to 0.6 in cohesive layers (organic silt and clayey silt) above the 15 m depth
- Partial pore pressure generation (PPR's < 0.6) in compact sand/gravel layers below the 20 m depth
- Post-seismic lateral displacements of up to 3 ft. (0.9 m), depending on the input earthquake record being considered. This lateral displacement would vary from approximately zero at the 15 m depth (surface of the dense sand and gravel marker layer) to a maximum value at the ground surface, corresponding to the occurrence of significant cyclic shear strains within the liquefiable sand layers and sensitive organic silt or clayey silt layers. As noted earlier, the computed lateral displacements are for level ground conditions and would be increased where ground slope exists.
- Peak ground surface velocities of up to 0.5 m/sec prior to the onset of soil liquefaction in the near surface sand layers. After liquefaction, ground velocities reduce dramatically since shear wave transmission is largely prevented.

The effective stress analyses indicate that significant lateral ground displacements above the 15 m depth are possible in absence of ground improvement. Dissipation of pore pressures generated by seismic shaking will also result in post-earthquake consolidation (settlement) of liquefied sand layers and cohesive layers. Liquefaction of the sensitive organic silt and clayey silt layers is not expected but partial pore pressure build-up (PPR's less than about 0.6) is judged will occur. The potential magnitudes of post-seismic settlement due to dissipation of pore water pressures in the sands and silts above the 15m depth are commented on in a later section.

LATERAL GROUND DISPLACEMENTS UNDER NON-LEVEL GROUND CONDITIONS

Estimates of lateral ground displacements at the northwest corner of Lincoln Elementary School have been made taking into account sloping ground conditions due to the presence of a nearby drainage ditch. The analysis is based on an approach suggested by Byrne (1990) which assumes a non-liquefied surface layer (sand and gravel fill and over-consolidated silt crust) translating on top of a liquefied sand layer just after soil liquefaction has been triggered. The assumptions used in the analysis are:

- Drainage ditch approximately 1.2 m deep with 2.5H:1V ditch side slopes, with the top of the ditch starting some 8 m away from the northwest corner of the school, giving an average ground slope from the school to the base of the ditch of 6.5E
- Initial ground surface velocity just prior to sand layer liquefaction of 0.5 m/sec (reference Figure 14)
- Post-liquefaction residual strength of 10 kPa in the liquefied sand layer of 10 kPa occurring at shear strains of 10 to 30%
- Thickness of non-liquefied soil crust of 3.5 m
- Thickness of liquefied sand layer of 1.1 m

The non-level ground analysis indicates that additional lateral ground displacements of up to 1.4 m could occur at the soil surface over and above those computed for level ground conditions. Computed displacements are strongly dependent on initial velocities assumed upon initiation of liquefaction.

While the above analysis is considered somewhat simplified and could be refined using dynamic finite element methods, it indicates the high potential for large lateral ground displacements following a major earthquake where non-level ground conditions exist around the school. In absence of ground improvement, it is judged that the northwest corner of the school would be heavily damaged caused by large differential lateral movements relative to areas further away from the drainage ditch.

POST-SEISMIC GROUND SETTLEMENTS

Post-seismic ground settlements above the 15 m depth due to reconsolidation of soils following pore pressure dissipation have been computed using the following methodologies:

- Within moderately plastic organic silts or clayey silts, one dimensional rebound consolidation theory was used (Yasuhara and Andersen, 1989) to compute consolidation settlements based on pore pressure ratios (PPR's) developed following seismic loading and rebound compression coefficients (Cr), increased by a factor of 1.5 to account for cyclic disturbance effects. Rebound compression coefficients in the range of 0.12 to 0.24 were used. PPR's of up to 0.6 and average values of 0.35 to 0.4 were considered in the analysis based on the results of previous, effective stress site response analysis. Initial void ratios in the materials were estimated from available moisture content data at various depths.
- Within loose to compact, granular soils post-liquefaction vertical strain potentials were estimated using correlations between vertical strain, $N_{1,60}$ and cyclic shear stress ratio (CSR). An average CSR value of 0.225 was considered in the analyses.

Results of the analysis using the average soil profile used in the site response analysis indicate lower (LB) and upper bound (UB) settlements of 80 and 440 mm, respectively, with a best estimate (BE) of 260 mm (or an average vertical strain of 2.0% of the maximum layer thickness of

13.1 m). The results of calculations for various soil profiles at different locations around the school site are summarized in Table 4. Comparison of upper and lower bound settlement estimates provides an indication of maximum differential settlements considered possible due to variations in soil stratigraphy, cyclic pore pressure generation characteristics, and post-cyclic compressibility characteristics. The table indicates that significant differential settlements between footings are possible (up to 400 mm) in absence of ground improvement.

Table 4 – Calculated Post-Seismic Settlements

Soil Profile Location	LB Settlement (mm)	BE Settlement (mm)	UB Settlement (mm)
Average profile	80	260	440
BH1, BH5 (NW corner)	150	270	380
AH2, CPT2 (NW corner)	165	260	350
AH1, CPT1 (SW corner)	180	280	360
AH3, CPT3 (NE corner)	130	320	530

POST-SEISMIC FOOTING BEARING CAPACITY

The near surface soil conditions appear highly variable across the site. For example, auger hole AH-1 and cone penetration test hole CPT-1 near the southwest corner of the school indicate compact to loose sands grading to silts at depth in the upper 2.3m of the soil profile. These are judged prone to seismic liquefaction and could result in school footing bearing failure, depending on the depth and width of the footing relative to the zone of liquefaction below the water table. In other areas, dense granular fill soils are underlain by cohesive deposits (typically over-consolidated silt or organic silt) or amorphous peat (reference AH-3). The latter deposits would not be expected to liquefy during strong seismic shaking but their undrained shear strength is limited, consequently limiting footing bearing pressures that may be applied.

For seismic design purposes, it is recommended that vertical footing bearing pressures be limited to 75 kPa where non-liquefiable, moderately plastic organic silts or clayey silts exist close to the underside of the building footings. The earlier GeoPacific Consultants (2005) report indicates that existing building footings were designed using allowable bearing pressures of about 72 kPa. In other areas, the presence of locally liquefiable sands or non-plastic silts close to the underside of building footings suggests that punching failure of the footings is possible under the above design pressures (depending on a number of factors including footing width, depth and post-liquefaction residual strength of the materials). This combined with the significant potential for post-seismic lateral and vertical ground displacements under the school building envelope dictates the requirement that some form of ground improvement be carried out under and around footings, discussed subsequently.

GROUND IMPROVEMENT OPTIONS

Ground improvement options which are considered to be feasible to provide foundation underpinning to limit the potential for footing punching failure and post-seismic footing settlement include compaction grouting and jet grouting. Based on discussions with qualified contractors, both techniques can work in limited headroom environments as exist at Lincoln Elementary School. The use of structural remediation options, including tying footings together to minimize relative lateral movements, will also need to be considered.

Compaction grouting involves the injection under pressure of low slump, soil-cement mixtures down casings to create grout columns of higher stiffness relative to the surrounding soil. The pressure injection process will densify granular soil layers and reinforce cohesive soil units. The intent of CG is to mitigate soil liquefaction potential in granular soil layers (where present) and by reinforcing cohesive layers, reduce their cyclic strain and pore pressure generation potential. It is intended that CG columns would be placed around the perimeter of a particular size footing, or along the edges of a strip footing. Pacific Geodynamics has provided preliminary design guidelines for the CG process in an earlier e-mail to Pomeroy Consultants dated June 15, 2006. A copy of this e-mail is provided in Appendix D which indicates expected number of compaction grout points to be placed around various sized footings proposed for use by Pomeroy. It is expected that CG columns of about 0.6m in average diameter would be required to cause sufficient densification of the looser sand layers to mitigate seismic liquefaction potential (injection volumes of 0.3 cu.m. per 0.3m length with average CG point spacings of 2.4 m). Discussions with qualified contractors indicate that typical costs of each CG point to maximum installed depths of 15 m are in the range of $3000 to $5000. It is expected that each JG column would be placed down to the top of the dense sand and gravel marker layer at a nominal depth of 14 to 15 m. This depth will vary across the site.

Jet grouting involves creating large diameter (0.8 to 1.2 m) soil-cement columns but cutting the soil with a horizontal grout jet that rotates and creates a soil-cement column. Each JG column will be placed around the perimeter of a particular footing. Approximately half the diameter of the column will extend under the edge of a footing. The JG installation process is not expected to densify looser granular soil layers. Rather, the installation of a JG column will reinforce the soil mass, which may limit cyclic shear stresses transmitted to looser soil layers under design levels of earthquake shaking and thereby provide increased liquefaction resistance. The JG column will also provide footing underpinning to limit post-seismic settlements. Discussions with qualified personnel who have carried out JG projects indicates that columns with a nominal diameter of 0.8 m should be expected in softer silt soils at an average cost per column of about $500 per metre length of column. It is expected that each JG column would be placed down to the top of the dense sand and gravel marker layer at a nominal depth of 14 to 15 m. This depth will vary across the site. A preliminary design memo has been provided by Pacific Geodynamics to Pomeroy (reference e-mail dated June 15, 2006) to indicate expected number of JG columns to be placed around various sized footings. A copy of this e-mail is provided in Appendix D.

It is noted that for both the CG and JG methods, soil-cement columns can likely only be constructed up to about the 1m depth due to limited soil confining stresses at shallower levels. It will be necessary to ensure that all footings are underlain by properly compacted, structural fill to bear on top of the CG or JG column.

Careful monitoring of volumes and pressures used to create the grout columns, and preventing movements (heave or settlement) of existing surface footings will be required during application of the CG or JG process. Adequate control of construction waste is also required.

It is expected that an economic evaluation of the feasibility of either ground improvement method will be undertaken by others. If one or both methods are selected as being feasible for the site, then it is recommended that:

- Additional detailed dynamic analysis of soil-cement column – ground interaction be undertaken supporting a particular footing. This would be intended to look primarily at the influence of seismic lateral ground movement on CG/JG column cracking and their ability to carry axial load following a major earthquake. It is expected that since the CG method causes granular soil densification, soil liquefaction will be mitigated, and lateral ground movements correspondingly reduced. The use of CG has a longer history compared to JG in mitigating seismic liquefaction potential and therefore we have less concern about the use of the CG method for this application. The JG method on the other hand does not cause soil densification and therefore JG columns will be subjected to potentially larger lateral ground movements. The larger diameter of the JG columns will provide resistance to these movements but some cracking and permanent lateral displacement of the columns is anticipated.

- A test section be carried out at the school site to demonstrate the efficiency of either process, diameter of columns achieved, and level of densification achieved (for CG). It would also be desirable to construct a large footing and to carry out the CG or JG process around the footing to simulate actual production conditions. It would also be possible to cyclically load the footing after ground improvement using a large shaker device available from the University of B.C Civil Engineering Department. The shaker would transmit vibrations into the underlying soil mass and generate pore water pressures. These would gradually dissipate, resulting in soil settlement. The test would demonstrate whether the CG or JG columns would provide adequate footing support and limit footing settlements following pore pressure dissipation.

- Suitable “performance based” specifications be developed for either technique to permit bidding by qualified contractors

CLOSURE

We trust the above information is sufficient for your present requirements and have enjoyed working with you on this project.

This report has been prepared for the exclusive use of Pomeroy Consulting Engineers Ltd. (the addressee) and Port Coquitlam School District 45 for design of proposed seismic upgrade additions to Lincoln Elementary School. This report relates only to Pacific Geodynamics’ performance of its limited scope of services. Pacific Geodynamics’ is not responsible for any assumptions, extrapolations or conclusions made or drawn by the addressee from, or for any failure by the addressee to reasonably apply its own knowledge and expertise to the content of this communication. Pacific Geodynamics Inc. is not responsible for any use by, or reliance on, the content of this communication by any other parties.

Yours truly,

Pacific Geodynamics Inc.

Per:

W. Blair Gohl, Ph.D., P.Eng.

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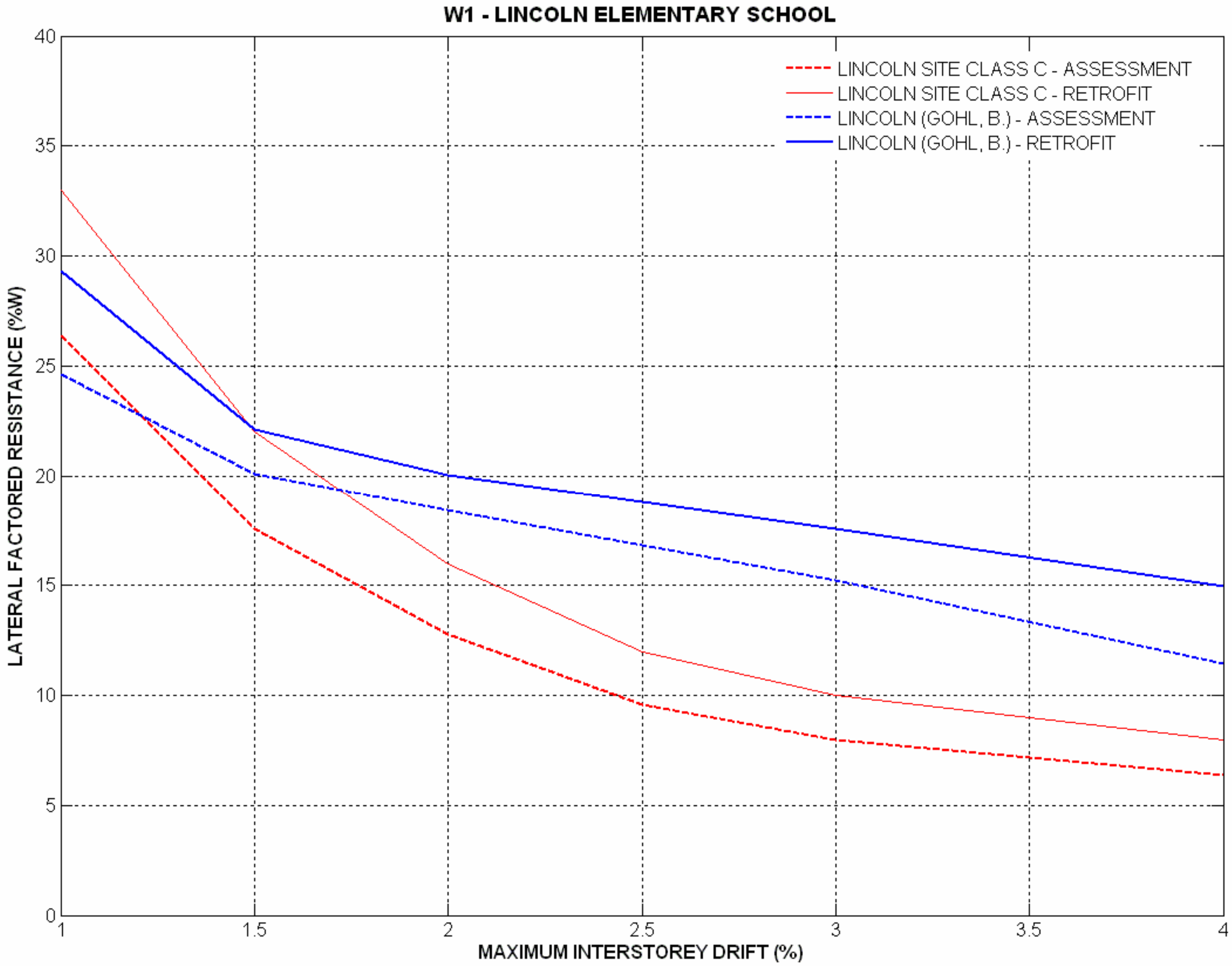
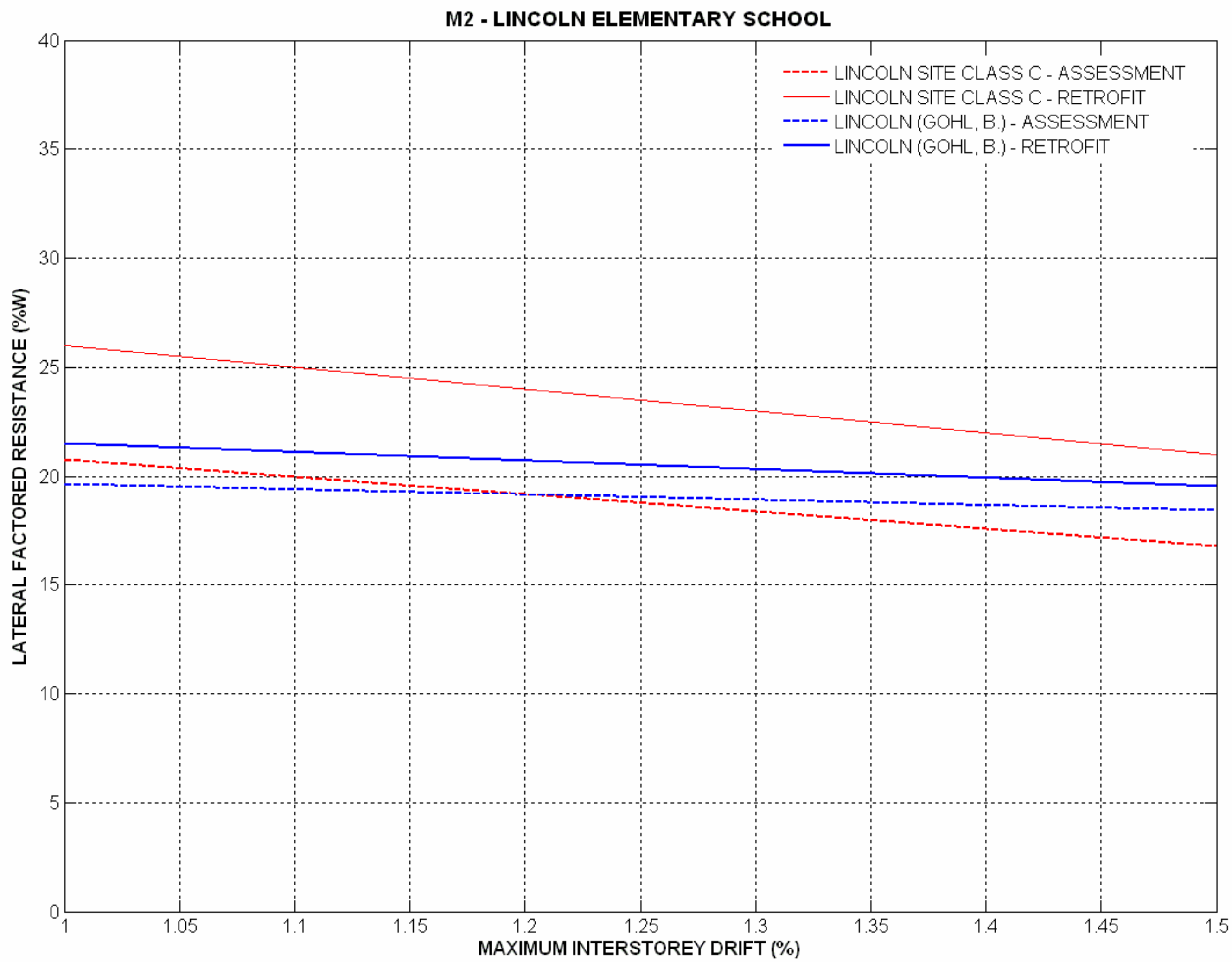
(2) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1 (total stress)


W1 - LINCOLN ELEMENTARY SCHOOL
LINCOLN SITE CLASS C - ASSESSMENT
LINCOLN SITE CLASS C - RETROFIT
LINCOLN (GOHL, B.) - ASSESSMENT
LINCOLN (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.5
2
2.5
3
3.5
4

(2) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for M-2 (total stress)



M2 - LINCOLN ELEMENTARY SCHOOL
LINCOLN SITE CLASS C - ASSESSMENT
LINCOLN SITE CLASS C - RETROFIT
LINCOLN (GOHL, B.) - ASSESSMENT
LINCOLN (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX G

DETAILED RESULTS FOR MARGARET JENKINS ELEMENTARY SCHOOL

CONTENTS

(1) MEG Consulting Ltd. Report (DESRA Analysis)

(a) E-mail dated March 23, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: gwt@tbgsc.bc.ca
Sent: Friday, March 23, 2007 3:43 PM
Subject: Site Response Analysis for Margaret Jenkins School

Graham,

Based on the estimated soil properties for Margaret Jenkins School that Pat and Chris sent me, I have constructed a DESRA-2C model considering 1m of compact sand, 3m of stiff clay, over 6m of firm clay, underlain by bedrock with an estimated Vs of 1500 m/sec (based on estimates by Pat for weathered gneissic bedrock). The soil profile selected Pat defines as representative of the "Soil Class C-D" boundary. Since the depth to bedrock is thinner in this one location at the school site (other areas across the site have apparently thicker clay layers), it was reasonable to assume that one would have more amplification of ground motions in the 0.5 to 1 second period range of most interest (I think) to structural response. The Vs, soil density and undrained strength properties of the clay are as provided by Pat and Chris based on available borehole data at the east end of the site. I have not attempted to refine this information given the limited site specific geotechnical data for the site.

It may be worthwhile setting up a similar model for deeper depths to clay but I have not had the time to do this yet. This model would indicate dominant ground response greater than about 1 second I would estimate.

Using the 10 input Site Class C ground motions applied at the top of till level (10m depth) scaled to represent Zone 5 for Victoria, I have propagated the motions up from firm ground level. An energy absorbing bottom boundary was considered with a Vs of 1500 m/sec (appropriate for rock) and a density of 23.6 kN/cu.m. No additional Rayleigh-type damping was considered, over and above internal hysteretic damping of the soil.

The computed peak ground accelerations versus depth are provided in the attached Excel spreadsheet ("CSR&Amax") which indicates de-amplification through the clay profile mainly because of the lower clay shear strengths in the lower 6m of the profile. Cyclic shear strains are relatively high in this layer, and extremely high in the near surface layer of sand which has been assigned a very low shear strength based on a small friction angle of 30 degrees (i.e. cyclic shear stresses equal to the shear strength of the material, leading to large strains).

From the computed acceleration time histories at the 1m depth, I have computed elastic response spectra (spectral acceleration and velocity) as per the attached spreadsheet. Significant amplification is indicated between about the 0.5 and 1 second period range. I have also attached the computed acceleration time histories.

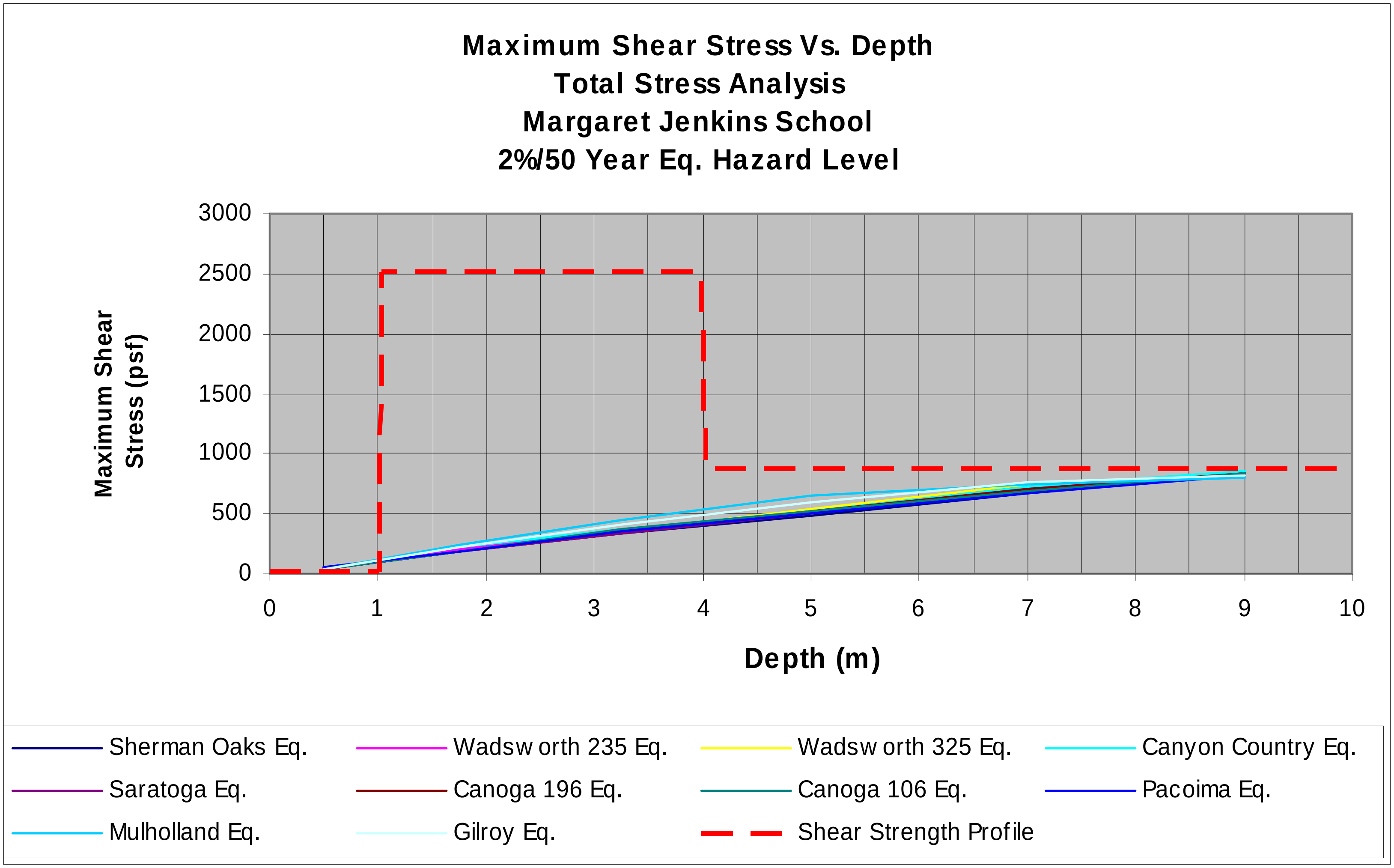
As noted earlier, it would be necessary to confirm local geotechnical soil conditions prior to proceeding with final seismic retrofit design of Margaret Jenkins School since these properties are critical to confirming the computed site response whether this level of site amplification is real. Also please advise whether you want me to consider a larger depth to clay elsewhere at the site? For now, I will proceed with completion of the site response analysis at Willows School which I should be able to complete by noon next Monday.

Regards,

Blair Gohl, Ph.D., P.Eng.
Principal
MEG Consulting Ltd.

(1) MEG Consulting Ltd. Report (DESRA Analysis)

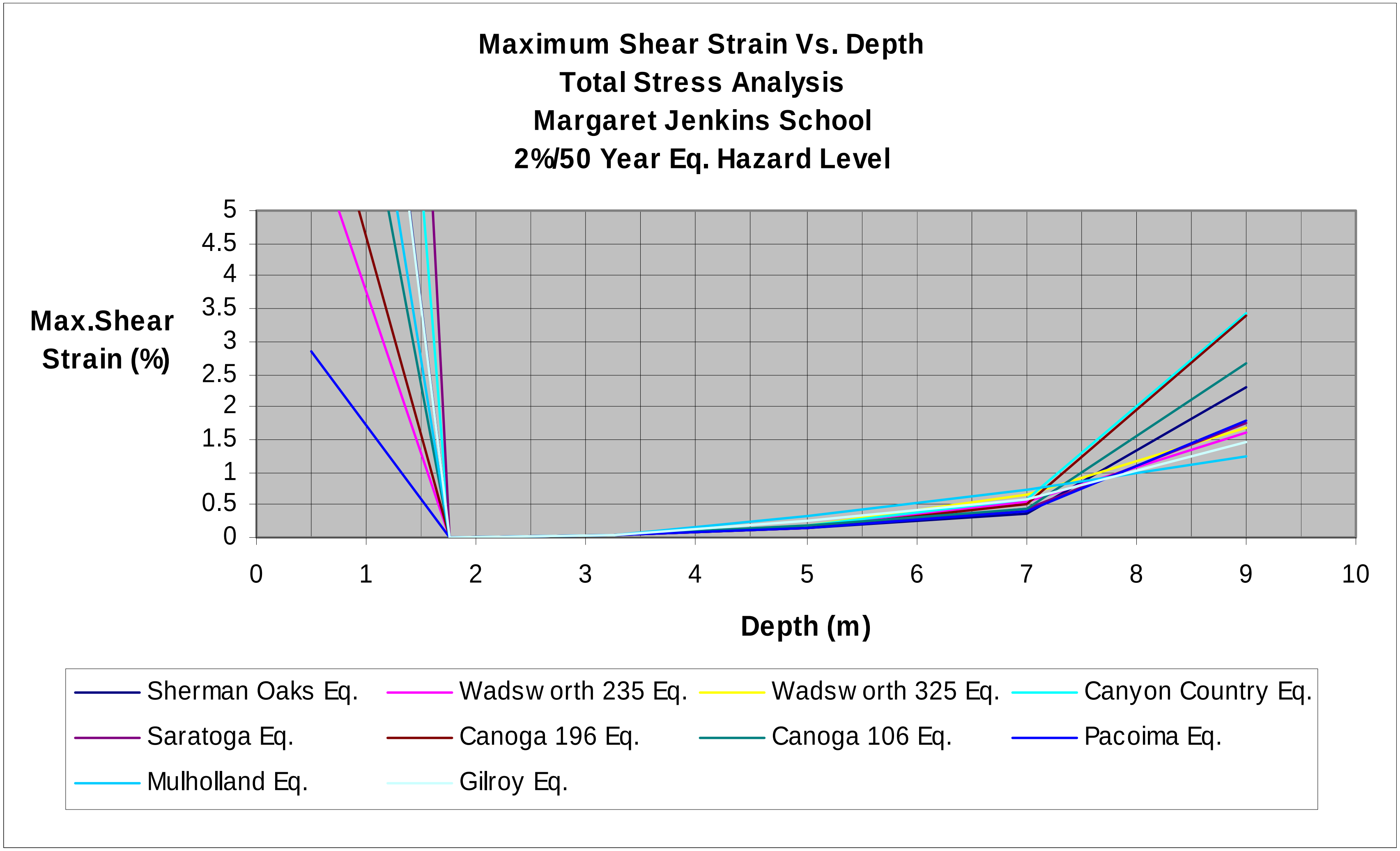
(b) Graph - maximum shear stress versus depth



Maximum Shear Stress Vs. Depth
Total Stress Analysis
Margaret Jenkins School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
3000
2500
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
9
10
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

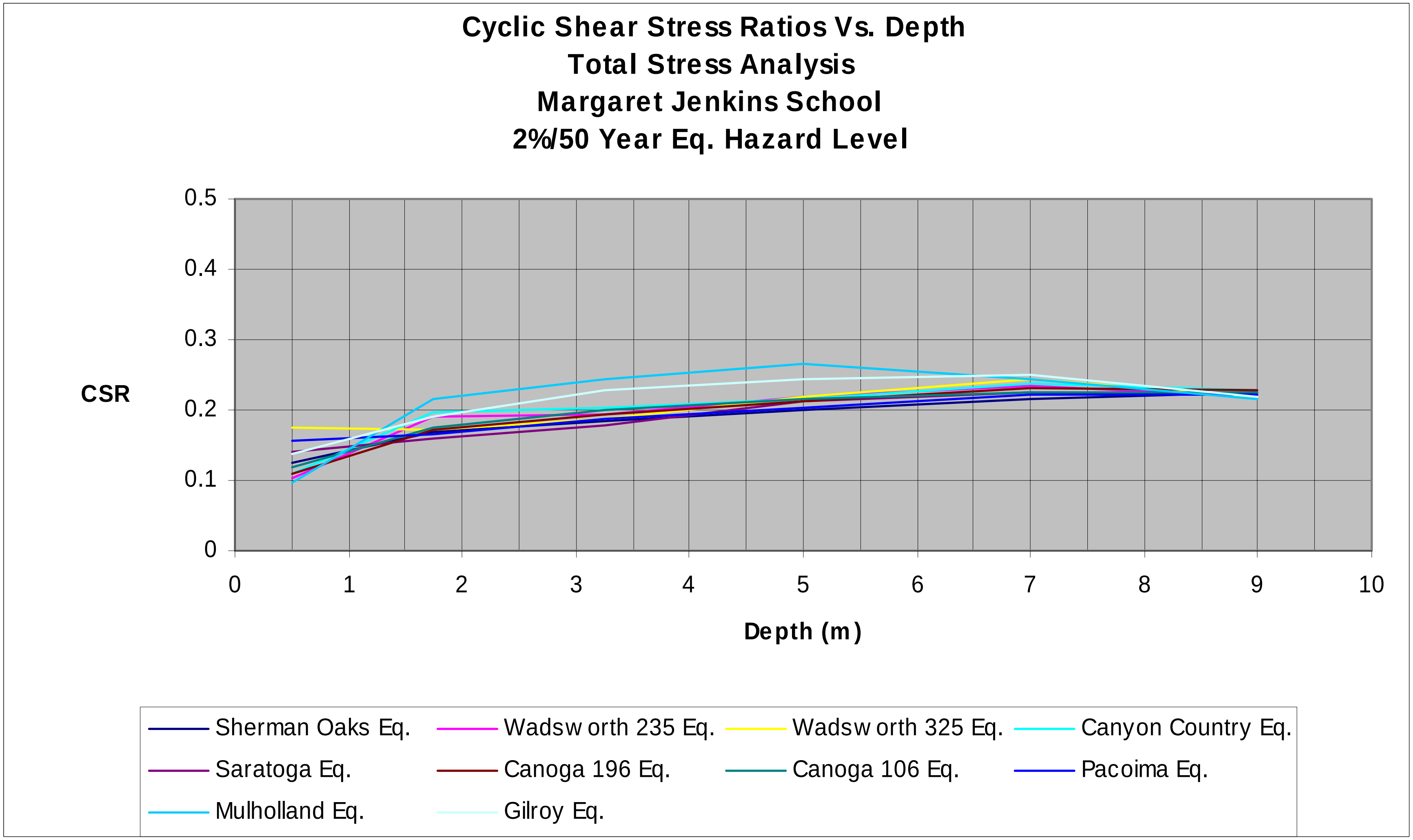
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Margaret Jenkins School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
1
2
3
4
5
6
7
8
9
10
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

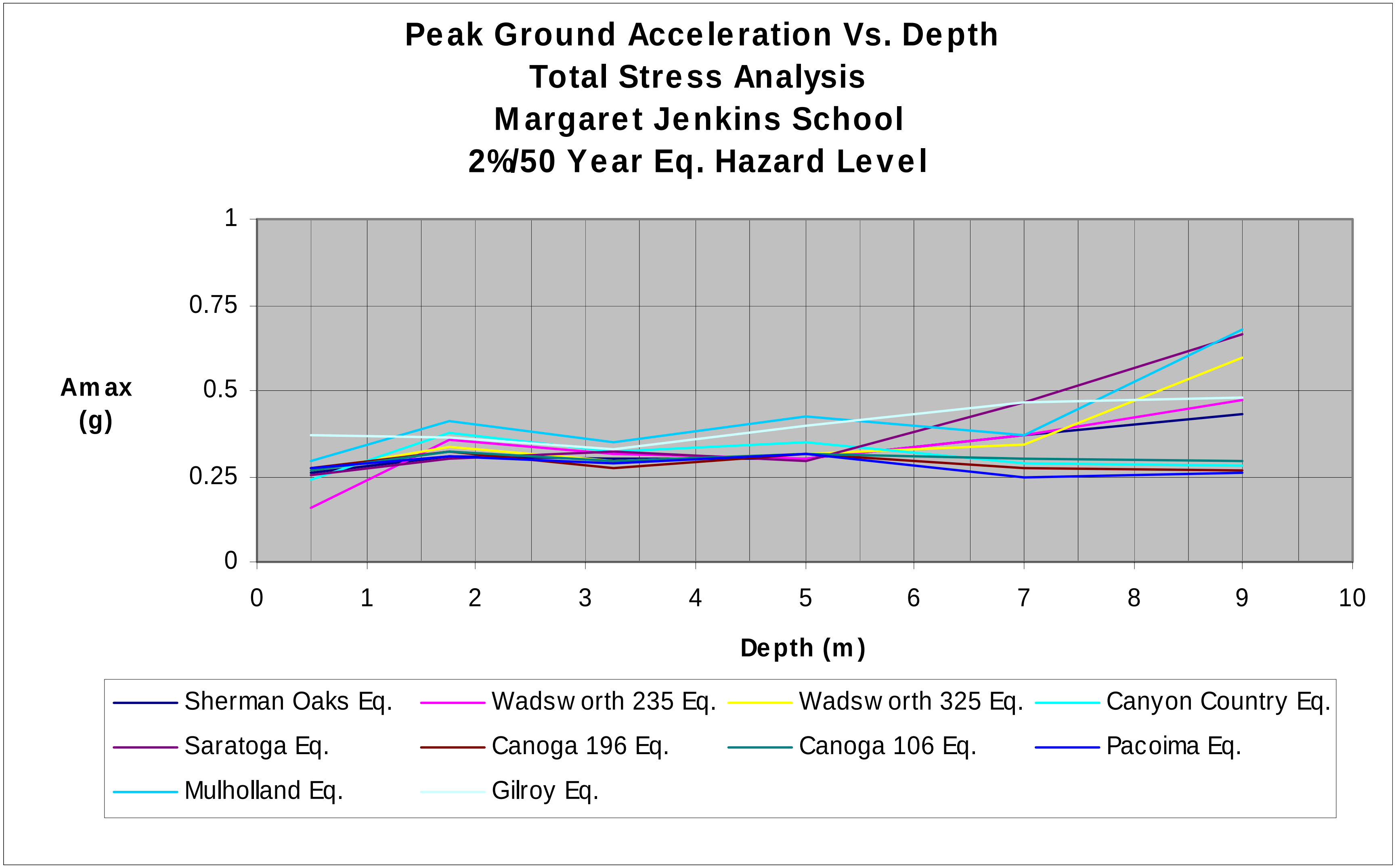
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Margaret Jenkins School
2%/50 Year Eq. Hazard Level
CSR
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
5
6
7
8
9
10
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

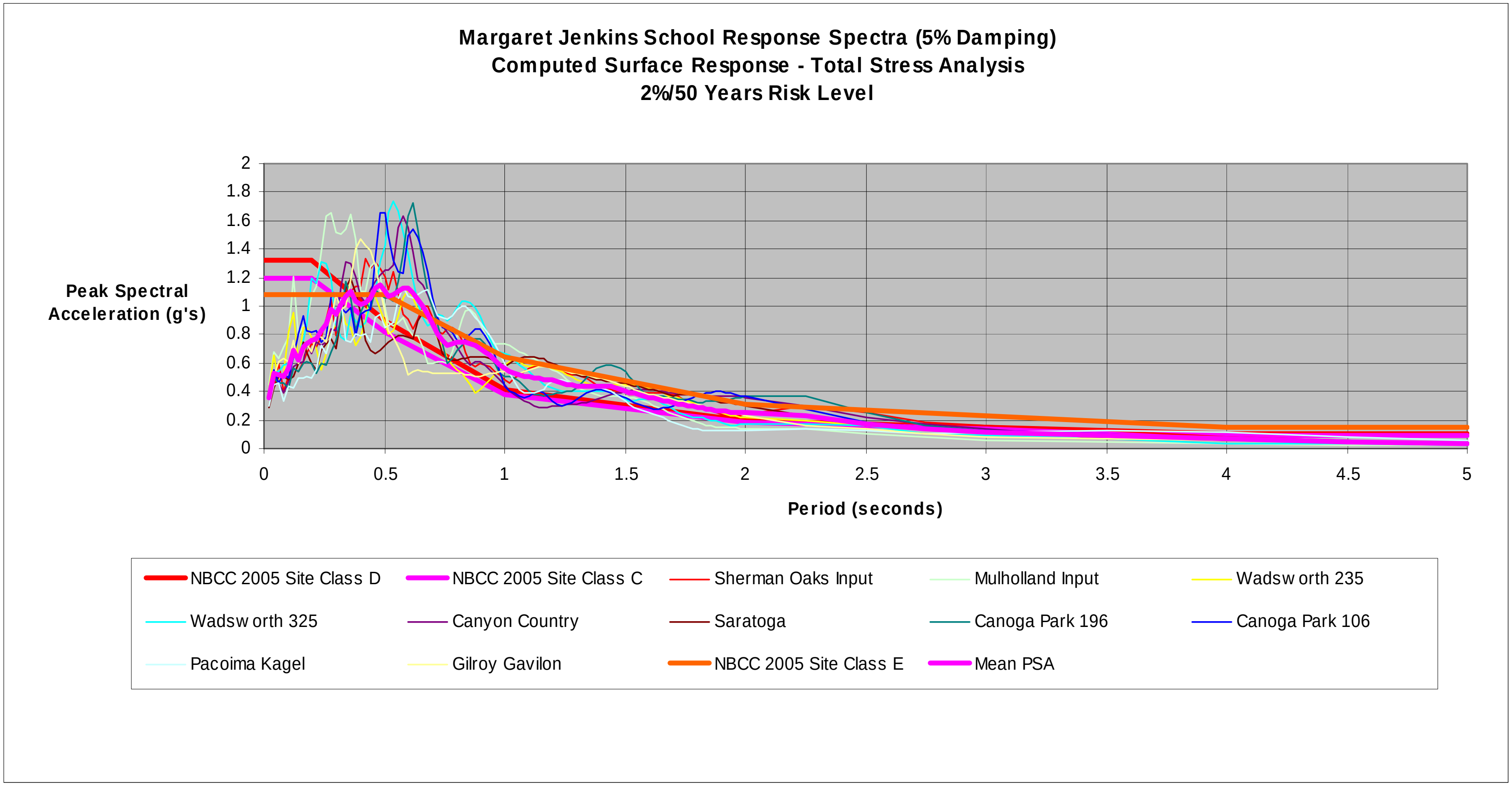
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Margaret Jenkins School
2%/50 Year Eq. Hazard Level
Amax (g)
1
0.75
0.5
0.25
0
0
1
2
3
4
5
6
7
8
9
10
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

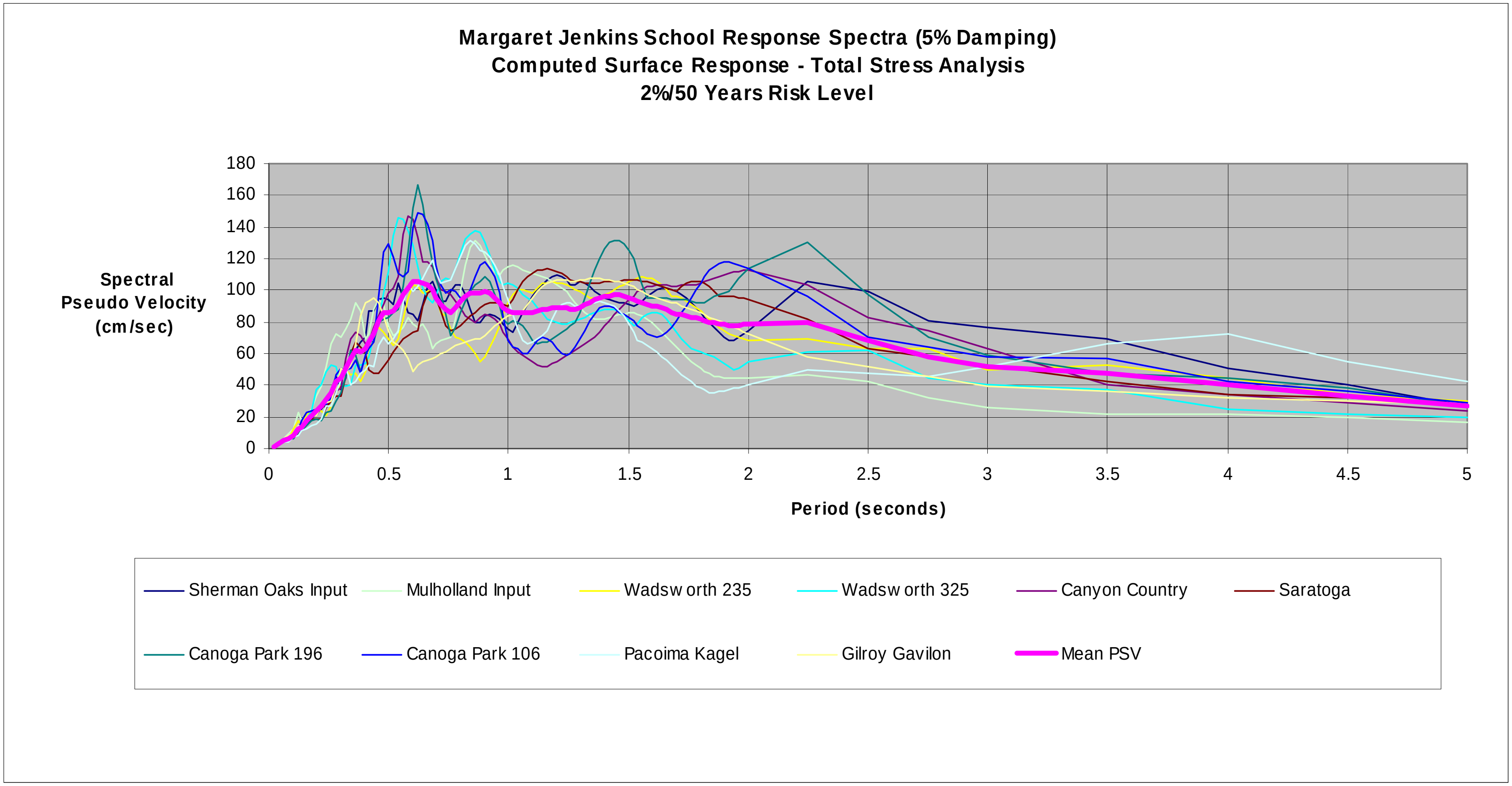
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(f) Graph - surface acceleration response spectra


Margaret Jenkins School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
NBCC 2005 Site Class E
Mean PSA

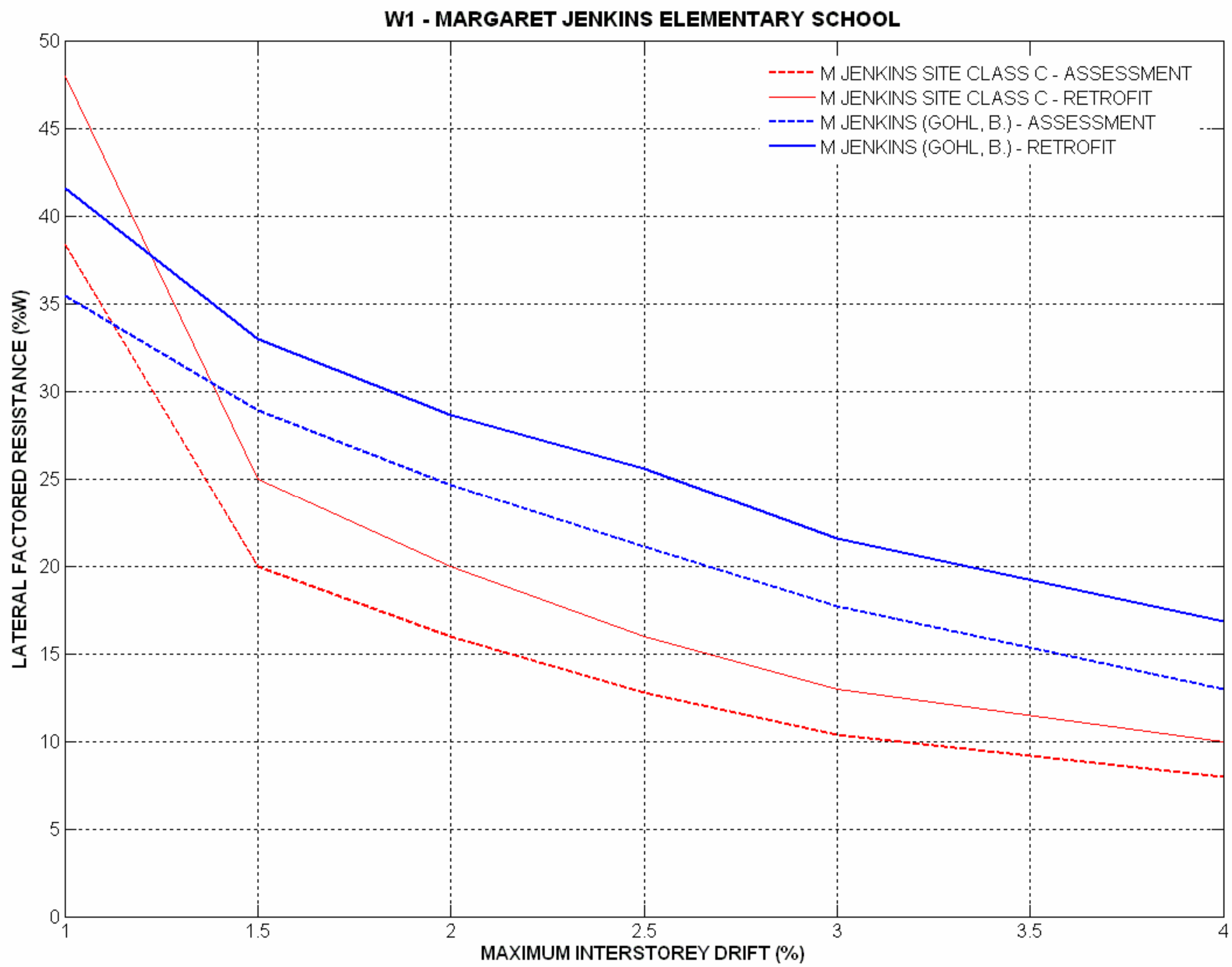
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(g) Graph - surface velocity response spectra


Margaret Jenkins School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
180
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSV

(2) UBC Report (Structural Analysis)

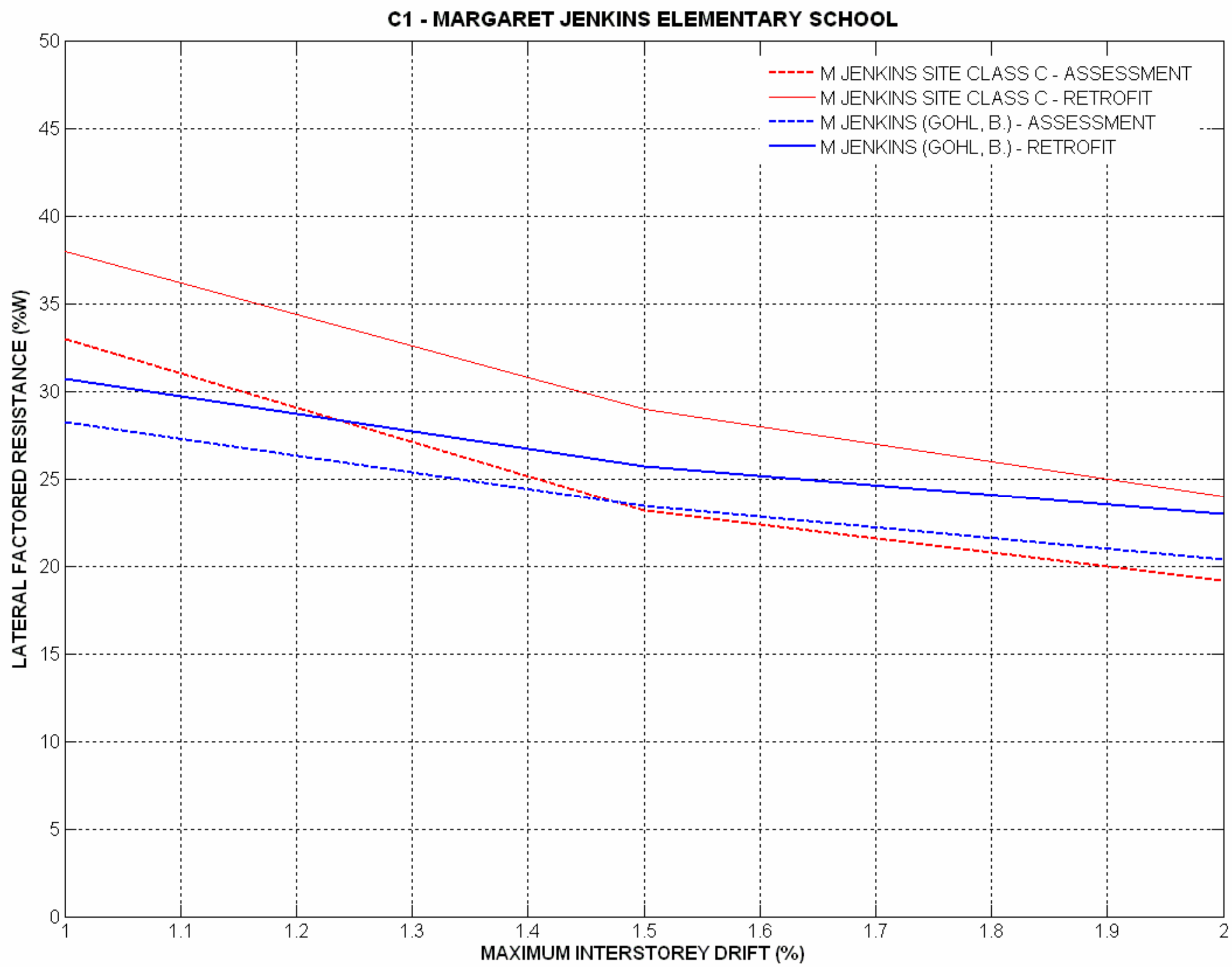
(a) Graph - lateral factored resistance versus drift for W-1



W1 - MARGARET JENKINS ELEMENTARY SCHOOL
M JENKINS SITE CLASS C - ASSESSMENT
M JENKINS SITE CLASS C - RETROFIT
M JENKINS (GOHL, B.) - ASSESSMENT
M JENKINS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

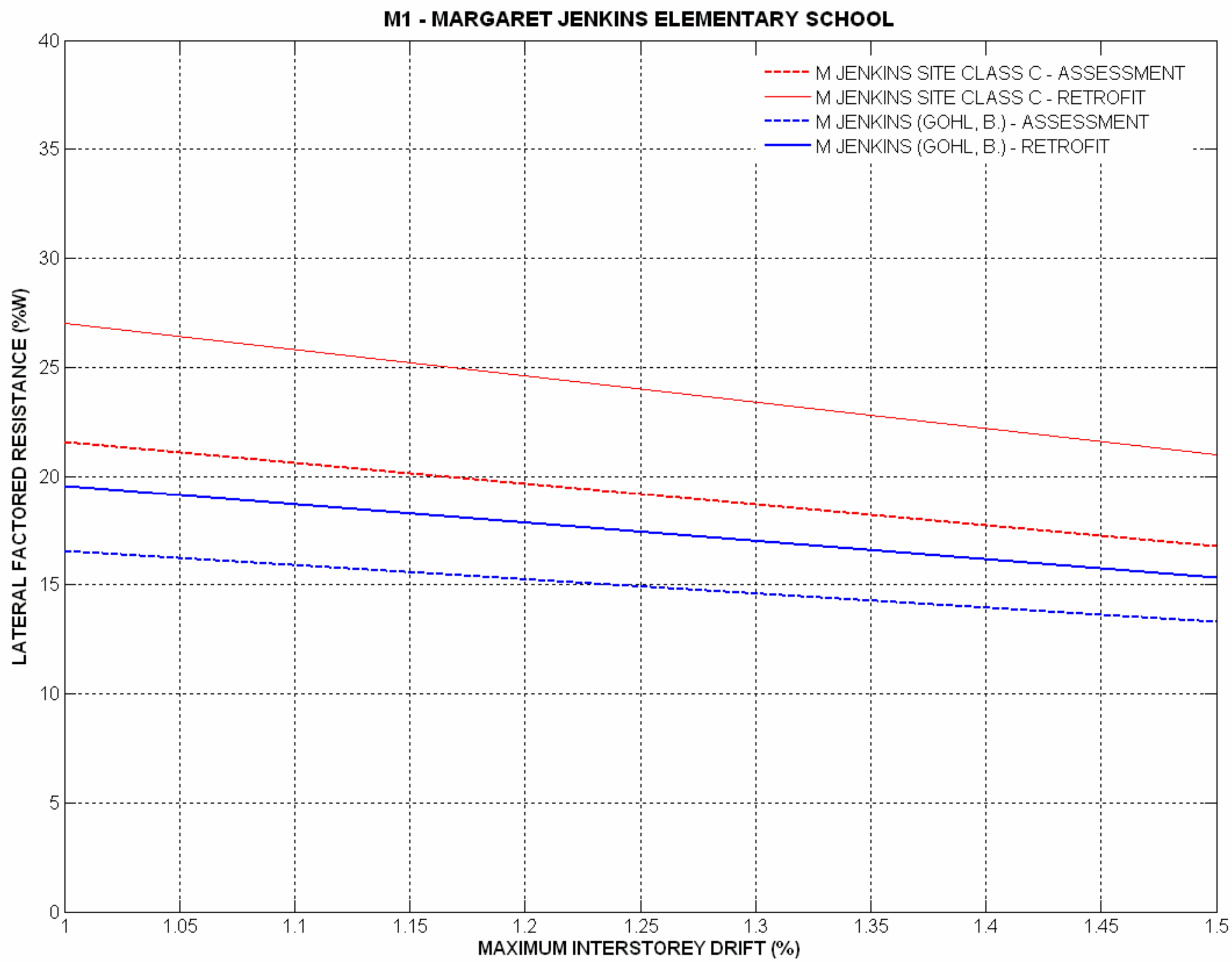
(2) <u>UBC Report (Structural Analysis)</u>

(b) Graph - lateral factored resistance versus drift for C-1


C1 - MARGARET JENKINS ELEMENTARY SCHOOL
M JENKINS SITE CLASS C - ASSESSMENT
M JENKINS SITE CLASS C - RETROFIT
M JENKINS (GOHL, B.) - ASSESSMENT
M JENKINS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2

(2) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for M-1


M1 - MARGARET JENKINS ELEMENTARY SCHOOL
M JENKINS SITE CLASS C - ASSESSMENT
M JENKINS SITE CLASS C - RETROFIT
M JENKINS (GOHL, B.) - ASSESSMENT
M JENKINS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX H

DETAILED RESULTS FOR MOUNT DOUGLAS SECONDARY SCHOOL

CONTENTS

(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column

(a) E-mail dated March 16, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: graham taylor, tbg
Sent: Friday, March 16, 2007 4:21 PM
Subject: Mount Douglas School - Initial Site Response Analysis

Graham,

Using Pat and Chris' suggested soil profile for Mount Douglas with "firm ground" considered to be at the 25.5m depth with Vs > 650 m/sec, I constructed a soil layer model for use in DESRA-2C with dynamic soil properties as per the attached Excel spread sheet. An energy absorbing bottom boundary with a Vs of 750 m/sec was assumed. Undrained strengths were assumed in the upper 5.5m of very stiff clay and a dynamic strain rate factor of 1.1 was applied to the static undrained strengths estimated by Chris. Drained strength properties based on friction angles in the sand units provided by Chris were assumed using an at rest earth pressure coefficient of 0.7. No additional damping was considered in the DESRA-2C model over and above that due to internal hysteretic damping within each soil layer.

The 10 Site Class C input ground motions were applied with scaling factors for Victoria as per Table C.4-2 of the Bridging Guidelines.

Computed maximum ground accelerations versus depth for each input motion are presented in the Excel spread sheet entitled "CSR & Amax". De-amplification of peak ground accelerations was computed at the base of the soil column due to the energy absorbing bottom boundary. Above the bottom of the soil column, amplification of peak ground accelerations was computed throughout the various soil layers.

Computed spectra at the ground surface are shown plotted in the attached Excel spread sheet, indicating significant short period amplification of the input firm ground motions below a period of 1 second. Due to the high strength of the soils at the site relative to the cyclic shear stresses applied in the upper regions of the soil profile above the 6m depth (see Excel plot "CSR & Amax"), little hysteretic damping would be expected in this region. Therefore, high frequency (short period) amplification was computed for some of the input records. The latter could be artificially controlled through specification of additional Rayleigh type damping which would likely lead to better agreement with peak spectral accelerations dictated by the 2005 NBCC for Site Class C and D soils (see attached Excel plot). Based on the limited soil model damping, acceleration time histories at the ground surface are also attached.

Since the Mount Douglas site indicates high stiffness "Site Class C" soils below the 9.5m depth based on the information provided by Pat Monahan (i.e. Vs > 420 m/sec) I propose to set up another soil layer model with firm ground specified at the 9.5m depth, and examine the sensitivity of computed surface response to this depth factor. For now, no additional Rayleigh damping will be considered which could be used to control high frequency amplification. Admittedly, this is a numerical artifact and some judgment would need to be applied to consider its use in order to obtain better agreement with "code design spectra".

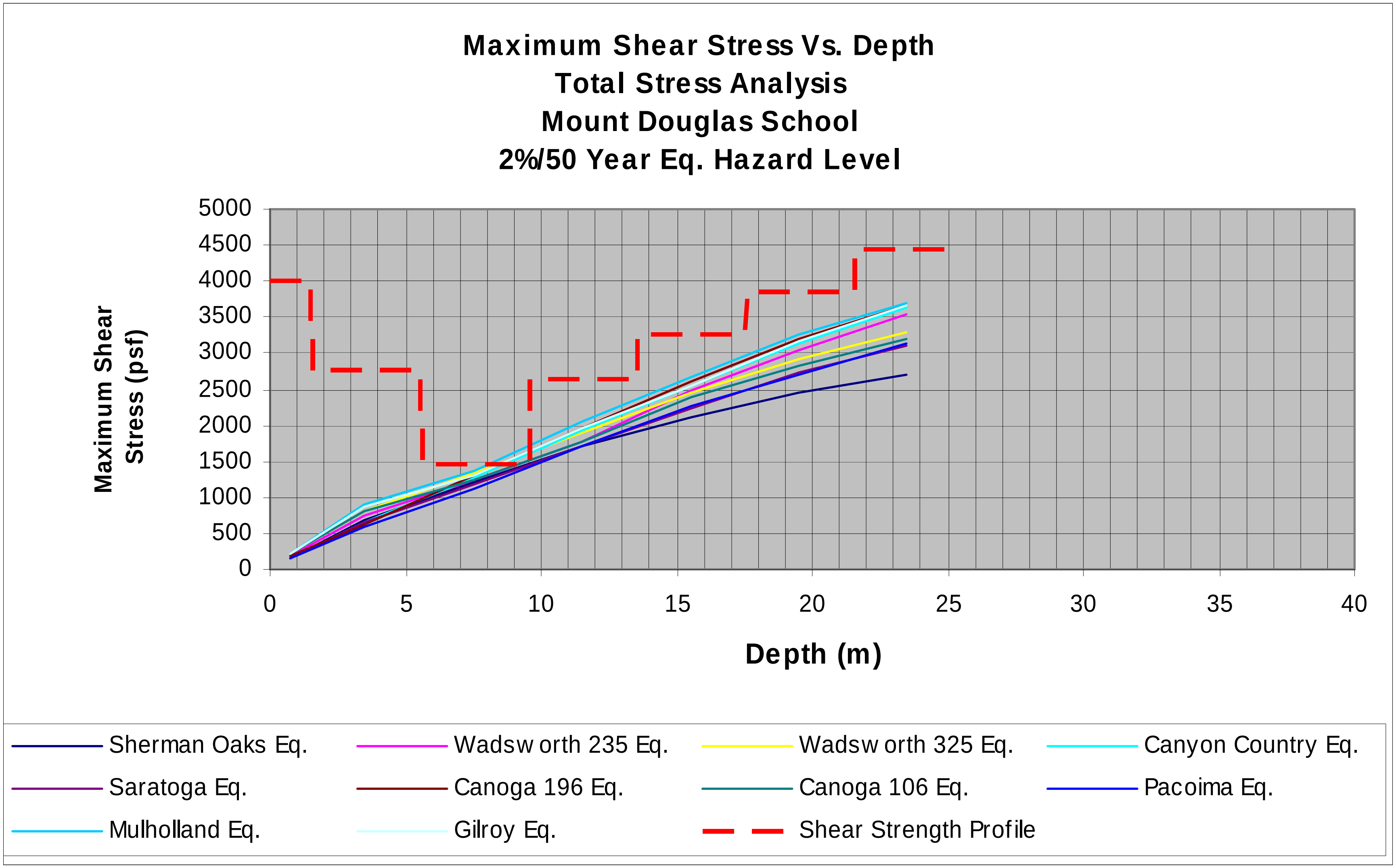
In the interim, would you please run your structural response model with the above computed surface acceleration time histories and let me know whether structural response is considered to be overly severe. We can discuss this issue further on Monday when I will be at MEG's office.

Regards,

Blair Gohl

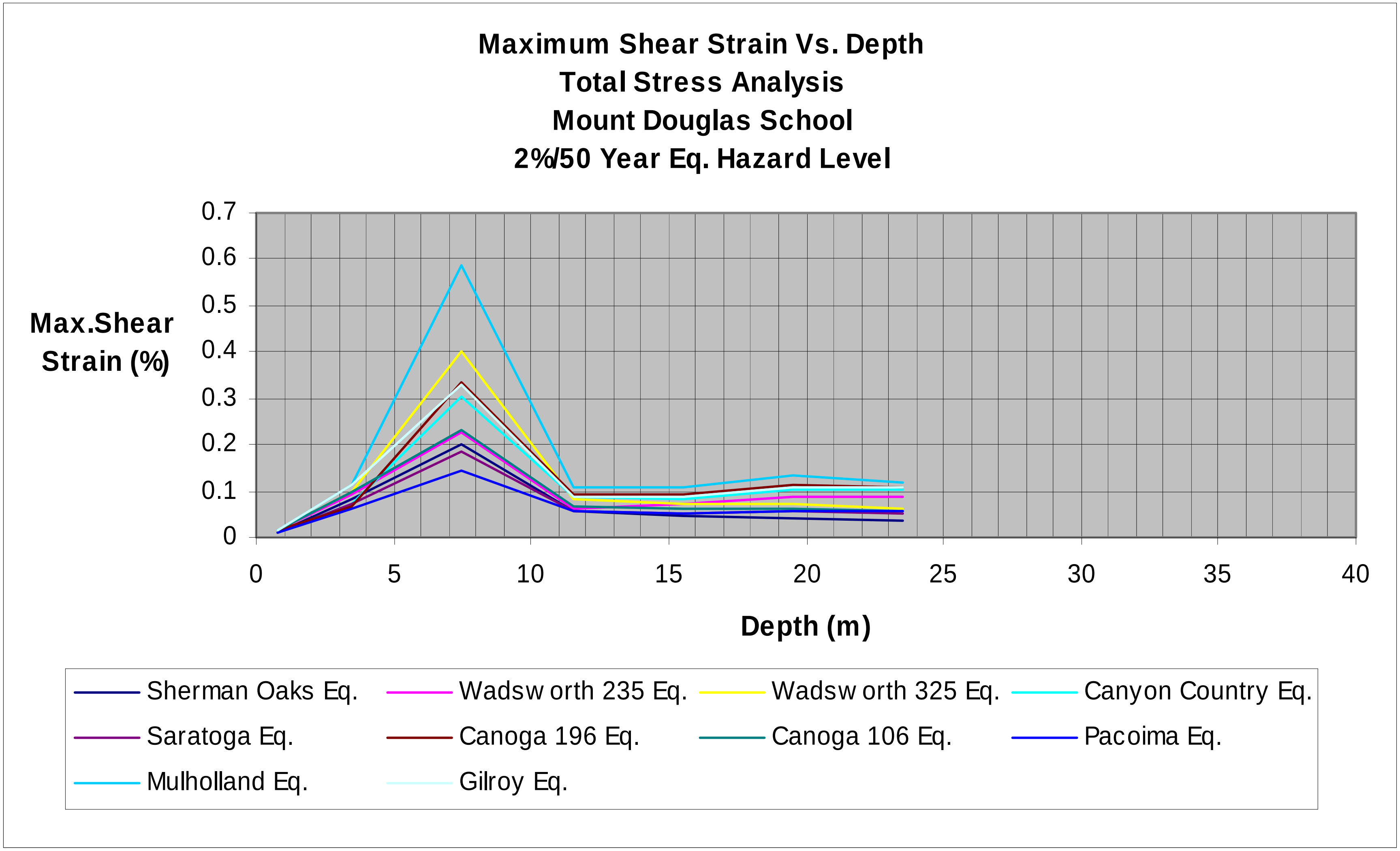
(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column</u>

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Mount Douglas School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

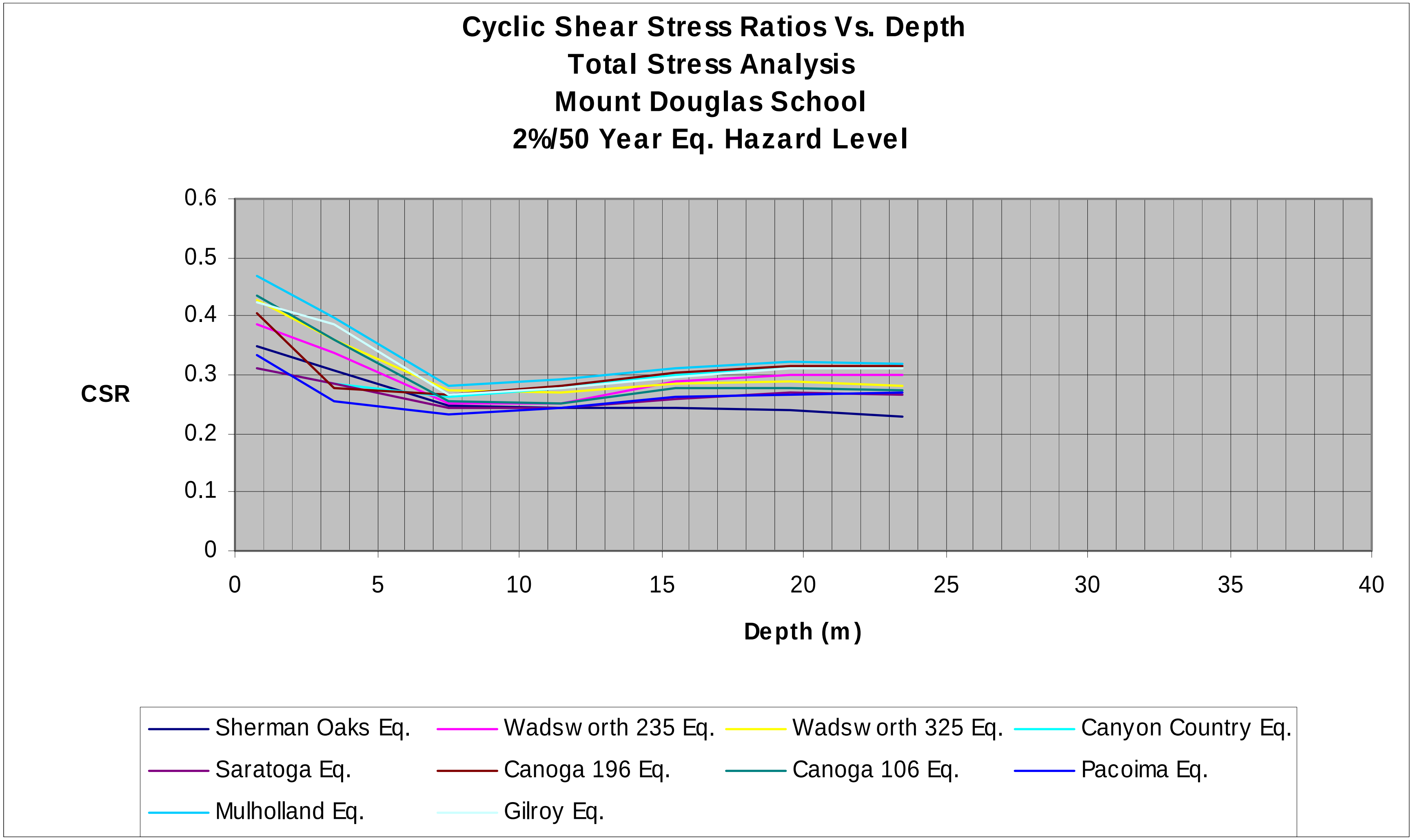
(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Mount Douglas School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

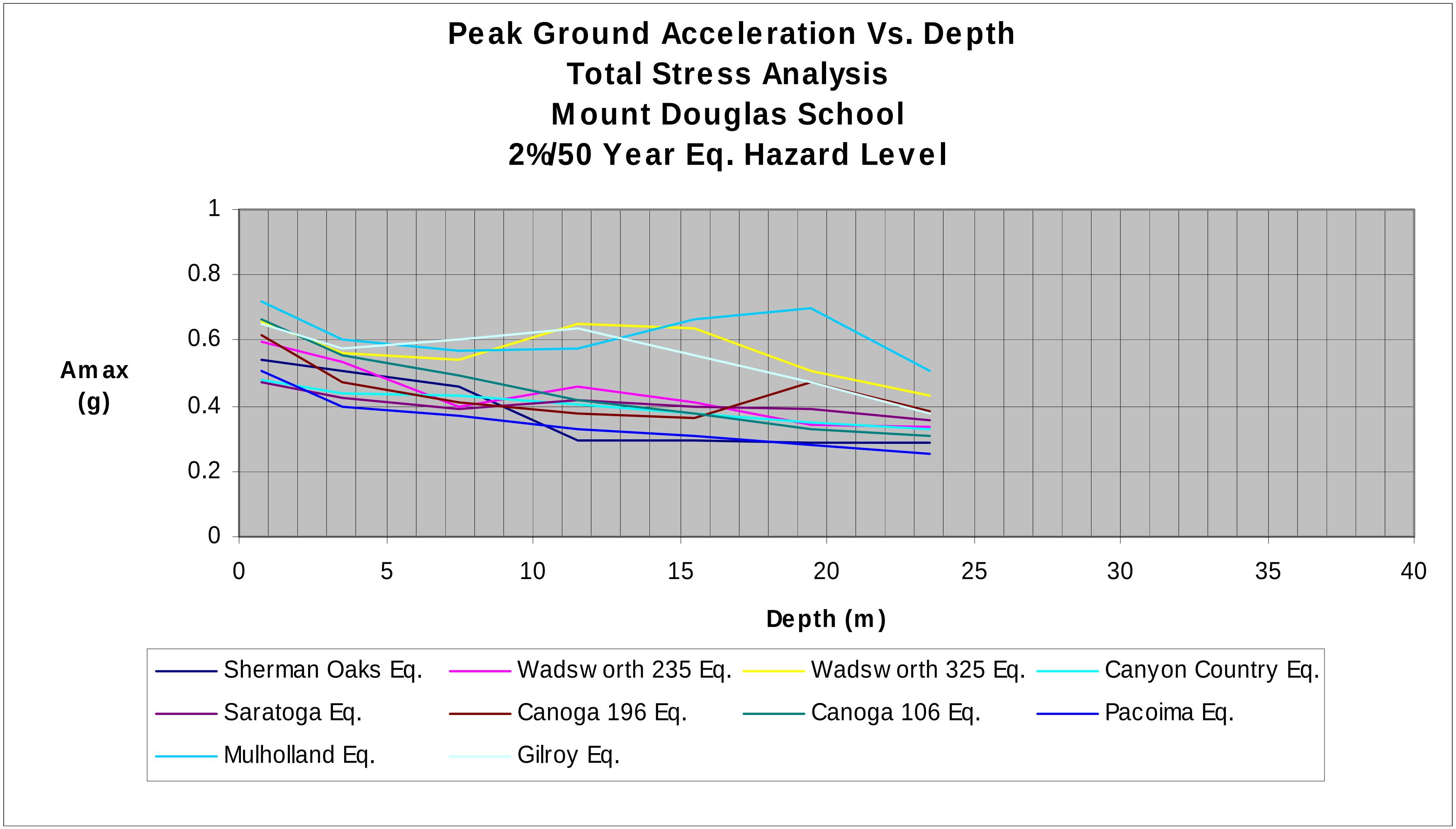
(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column</u>

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Mount Douglas School
2%/50 Year Eq. Hazard Level
CSR
0.6
0.5
0.4
0.3
0.2
0.1
0
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

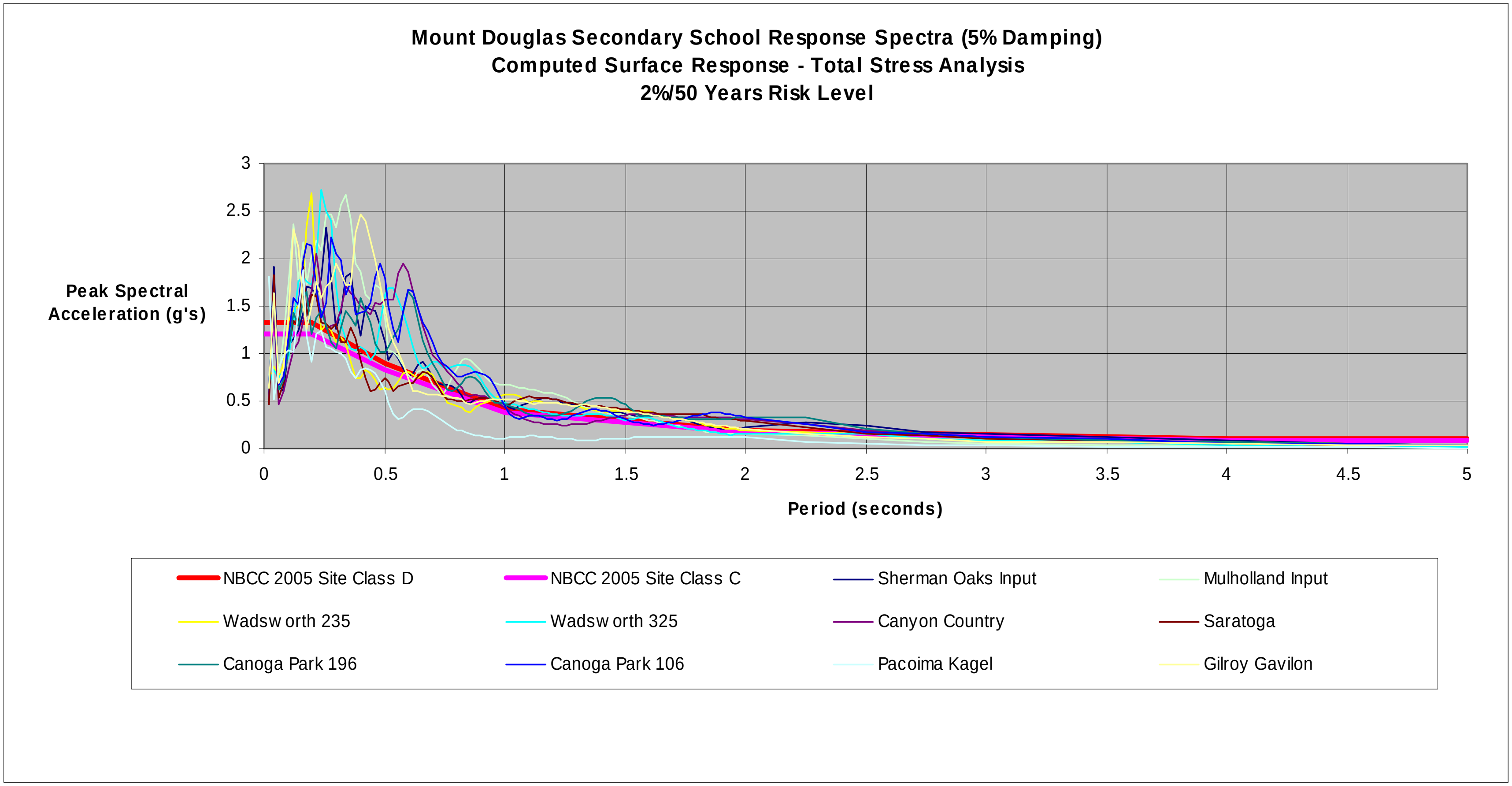
(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Mount Douglas School
2%/50 Year Eq. Hazard Level
Amax (g)
0
0.2
0.4
0.6
0.8
1
0
5
10
15
20
25
30
35
40
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

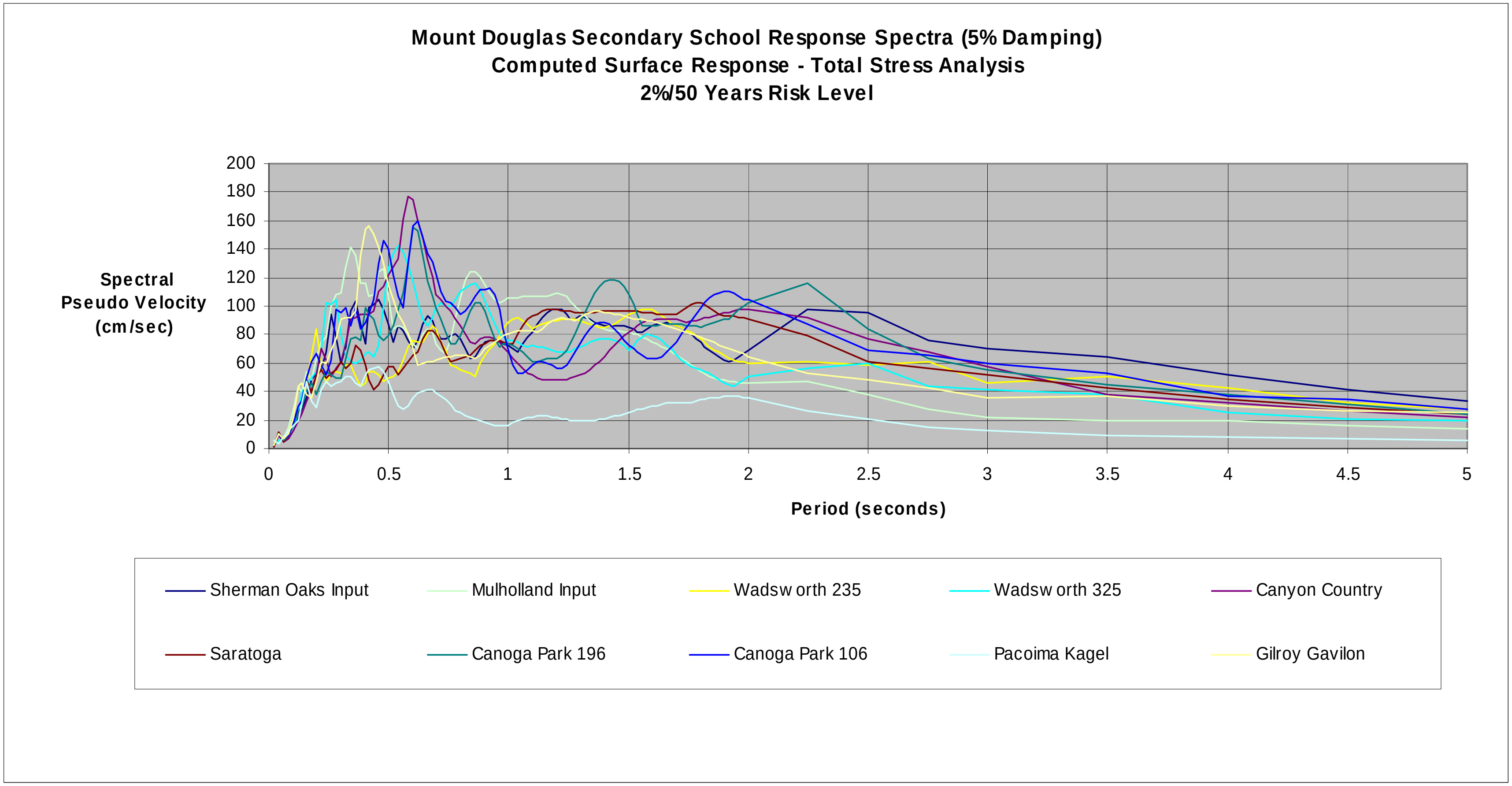
(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column</u>

(f) Graph - surface acceleration response spectra


Mount Douglas Secondary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
3
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column

(g) Graph - surface velocity response spectra


Mount Douglas Secondary School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
0
20
40
60
80
100
120
140
160
180
200
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

(1) <u>Pacific Geodynamics Inc. Report (DESRA Analysis) - Deeper Soil Column</u>

(h) DESRA soil properties

Nonlinear Site Response Analysis: Dynamic Soil Properties for Mt. Douglas School

Layer No.	Soil Type	Thk. (ft)	Unit Weight (pcf)	Vs (ft/sec)	Gmax (psf)	Drained Sand shear strength (psf)	Undrained Shear Strength (psf)
1	brown v. stiff clay	1.968	127.2171	606.8	1457615.189		4021.325
2	brown v. stiff clay	2.952	127.2171	606.8	1457615.189		4021.325
3	brn-grey v. stiff clay	13.12	127.2171	541.2	1159490.826		2757.48
4	compact Capilano sand	13.12	133.578	902	3381848.242	1472.984	
5	v. dense Quadra sand	13.12	139.9388	1377.6	8264007.339	2655.16	
6	v. dense Quadra sand	13.12	139.9388	1804	14171554.54	3248.005	
7	v. dense Quadra sand	13.12	139.9388	1968	16865321.1	3840.851	
8	v. dense Quadra sand	13.12	139.9388	2132	19793328.24	4433.696	

(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Shallower Soil Column

(a) E-mail dated March 18, 2007 - analysis assumptions

----- Original Message -----
From: Blair Gohl
To: graham taylor, tbg
Sent: Sunday, March 18, 2007 12:01 PM
Subject: Mount Douglas School - Site Response Analysis #2

Graham,

I ran the DESRA-2C site response analysis for Mount Douglas assuming 9.5m depth to "firm ground" using the same soil properties over this depth range sent previously and assuming an elastic half-space velocity of 500 m/sec. You will recall that the previous SRA considered a 25.5m depth to "firm ground" and an elastic half space velocity of 750 m/sec. Again, no additional soil damping was considered over and above internal hysteretic damping of the soil layers.

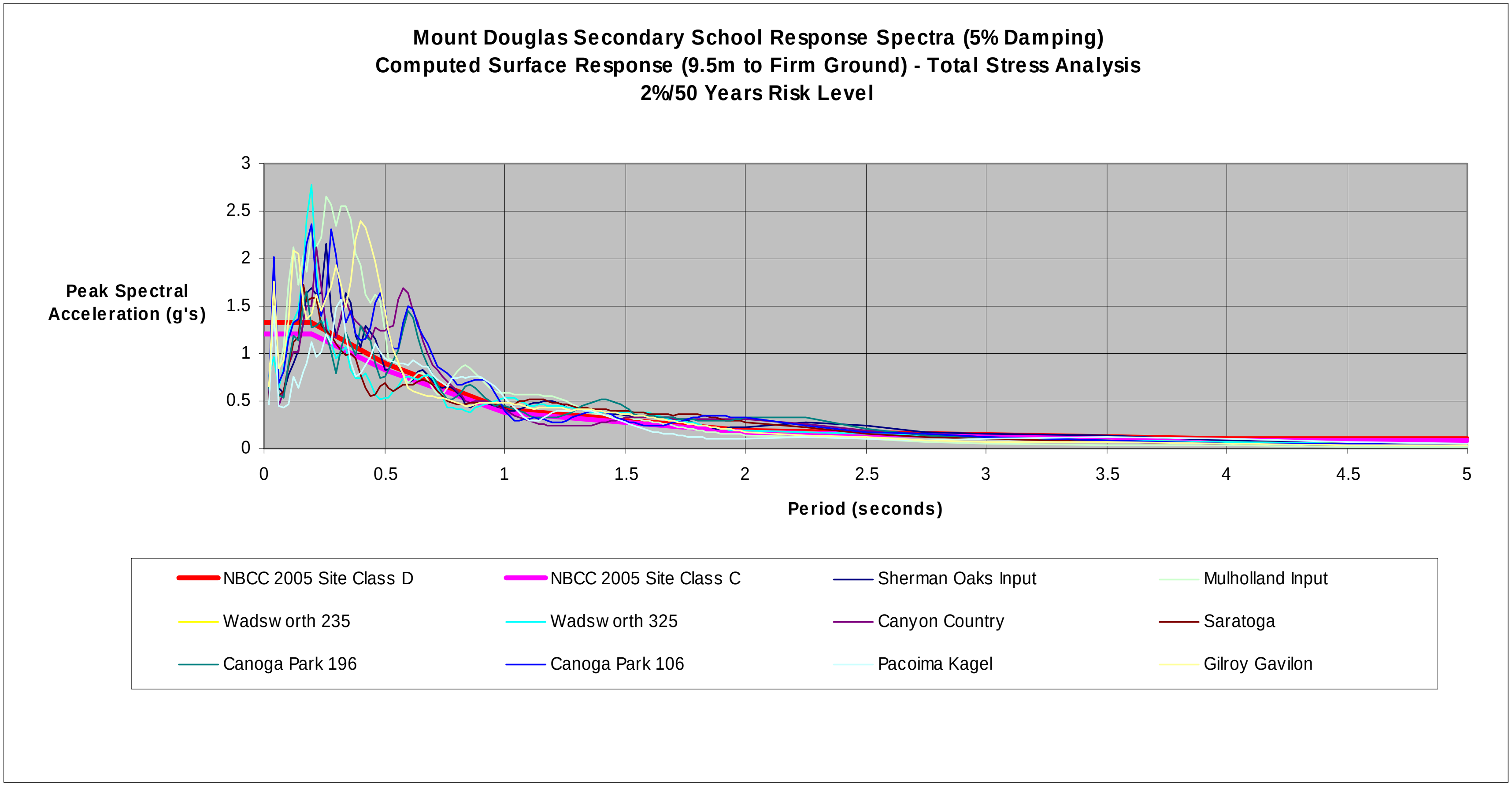
I have attached the computed surface acceleration time histories and elastic response spectra. From comparison of the average surface response spectrum for both depth cases, there does not appear to be much difference in response. Please advise whether you think it worth while to reduce the short period spectral response using additional Rayleigh-type (stiffness proportional) damping. As noted earlier, this would be an artificial high frequency control to better match the 2005 NBCC Site Class C and D spectra in the short period range.

Regards,

Blair Gohl

(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Shallower Soil Column

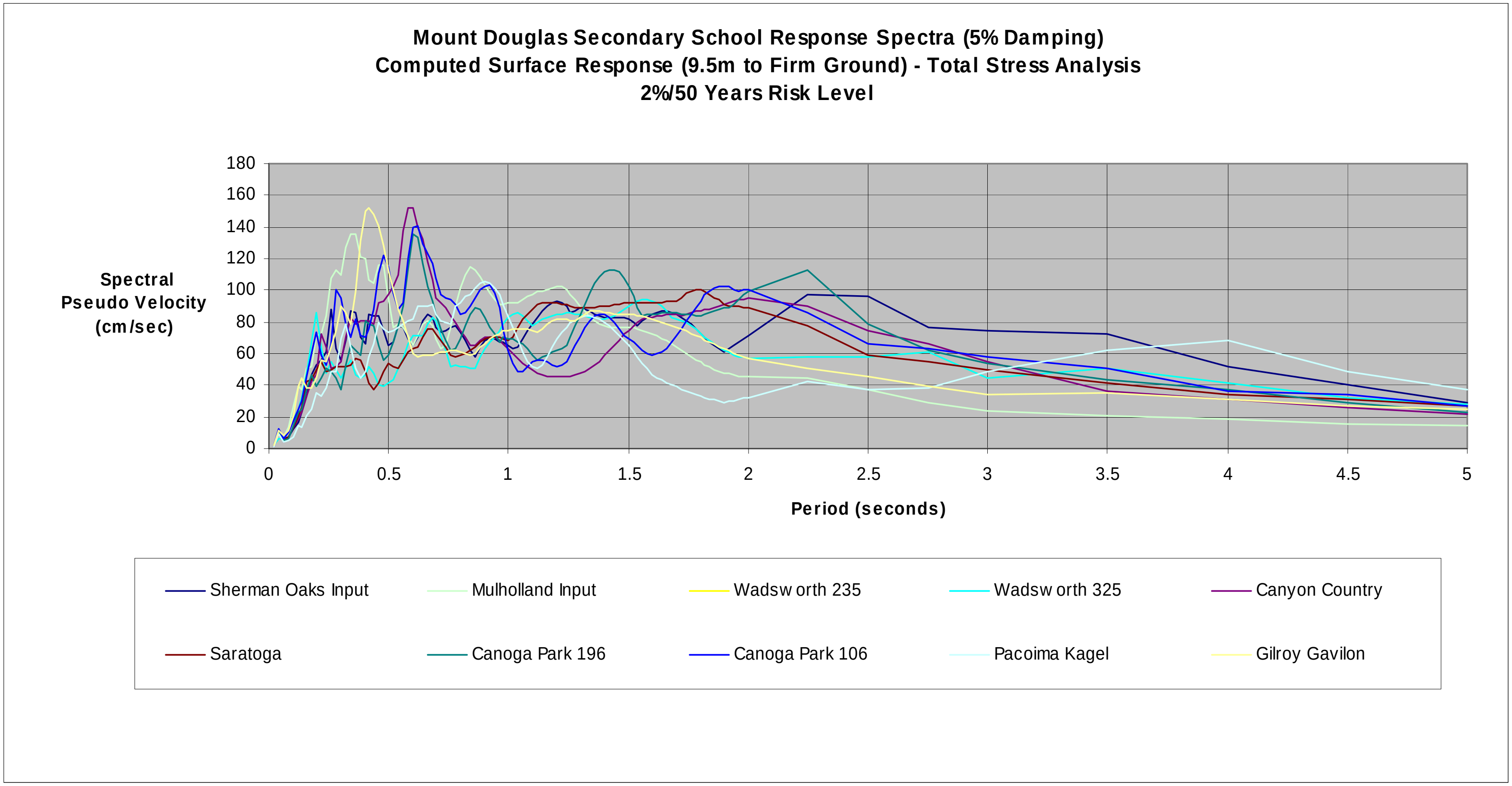
(b) Graph - surface acceleration response spectra



Mount Douglas Secondary School Response Spectra (5% Damping)
Computed Surface Response (9.5m to Firm Ground) - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
3
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

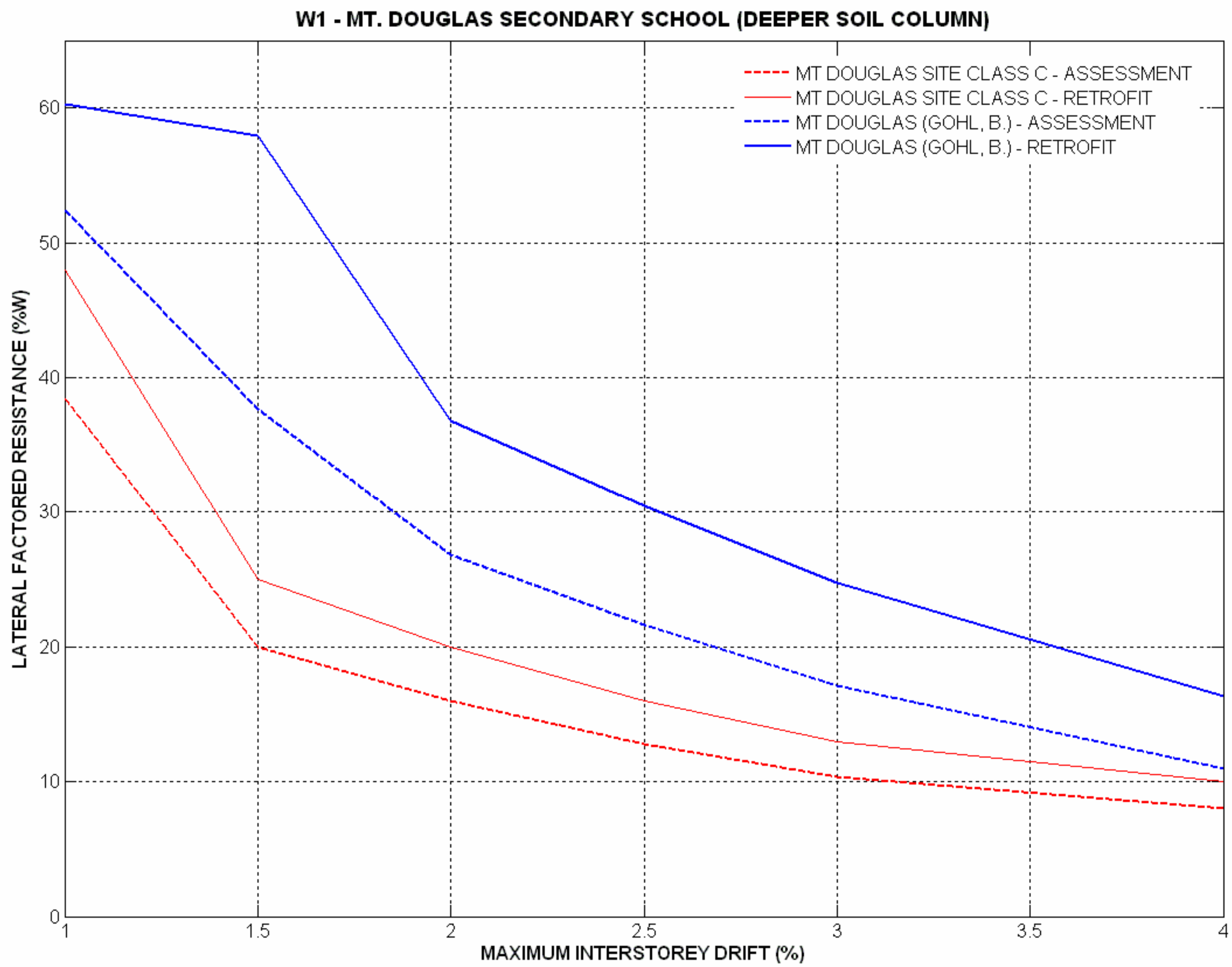
(1) Pacific Geodynamics Inc. Report (DESRA Analysis) - Shallower Soil Column

(c) Graph - surface velocity response spectra


Mount Douglas Secondary School Response Spectra (5% Damping)
Computed Surface Response (9.5m to Firm Ground) - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
180
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

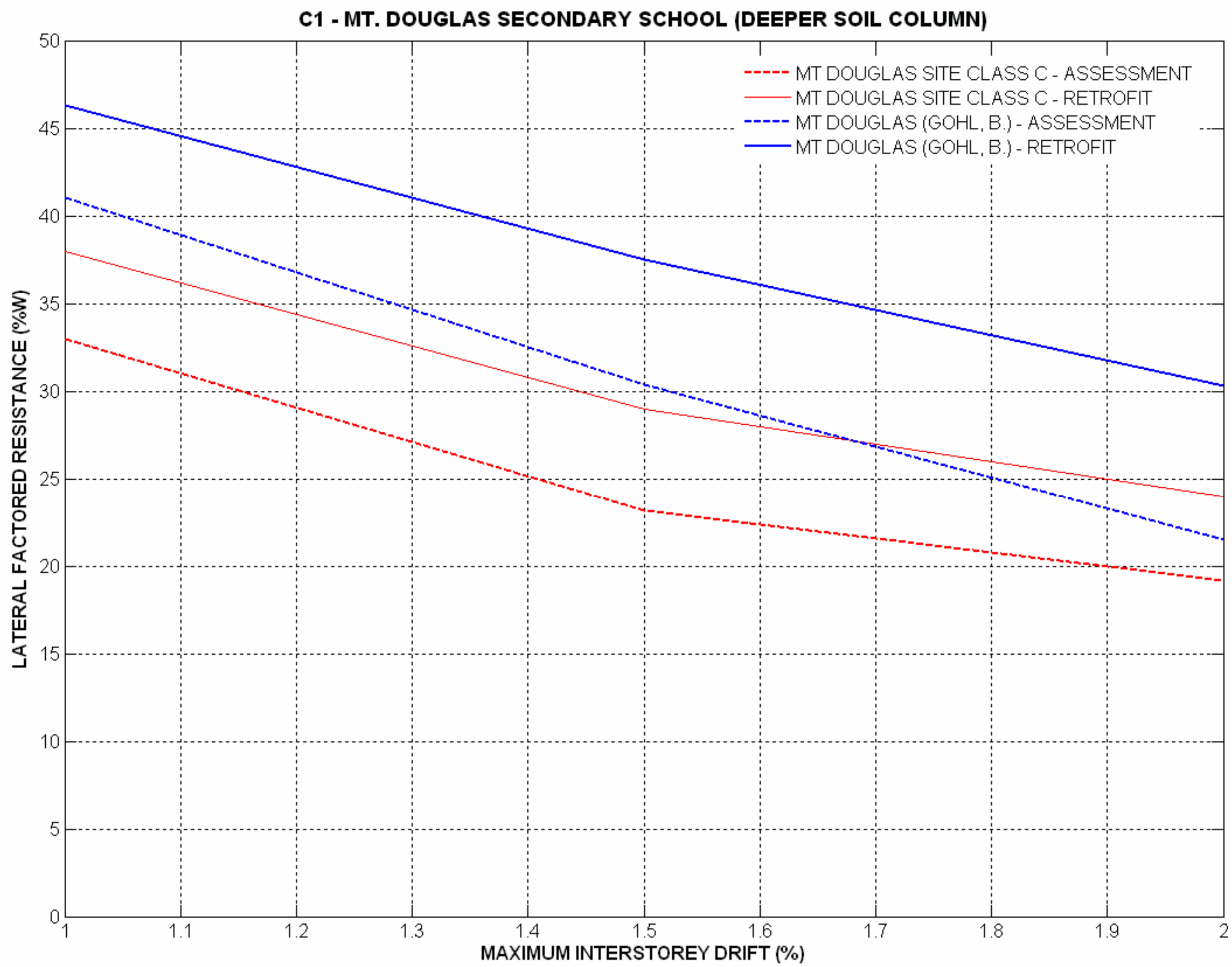
(3) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1 (Deeper soil column)


W1 - MT. DOUGLAS SECONDARY SCHOOL (DEEPER SOIL COLUMN)
MT DOUGLAS SITE CLASS C - ASSESSMENT
MT DOUGLAS SITE CLASS C - RETROFIT
MT DOUGLAS (GOHL, B.) - ASSESSMENT
MT DOUGLAS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1
1.5
2
2.5
3
3.5
4

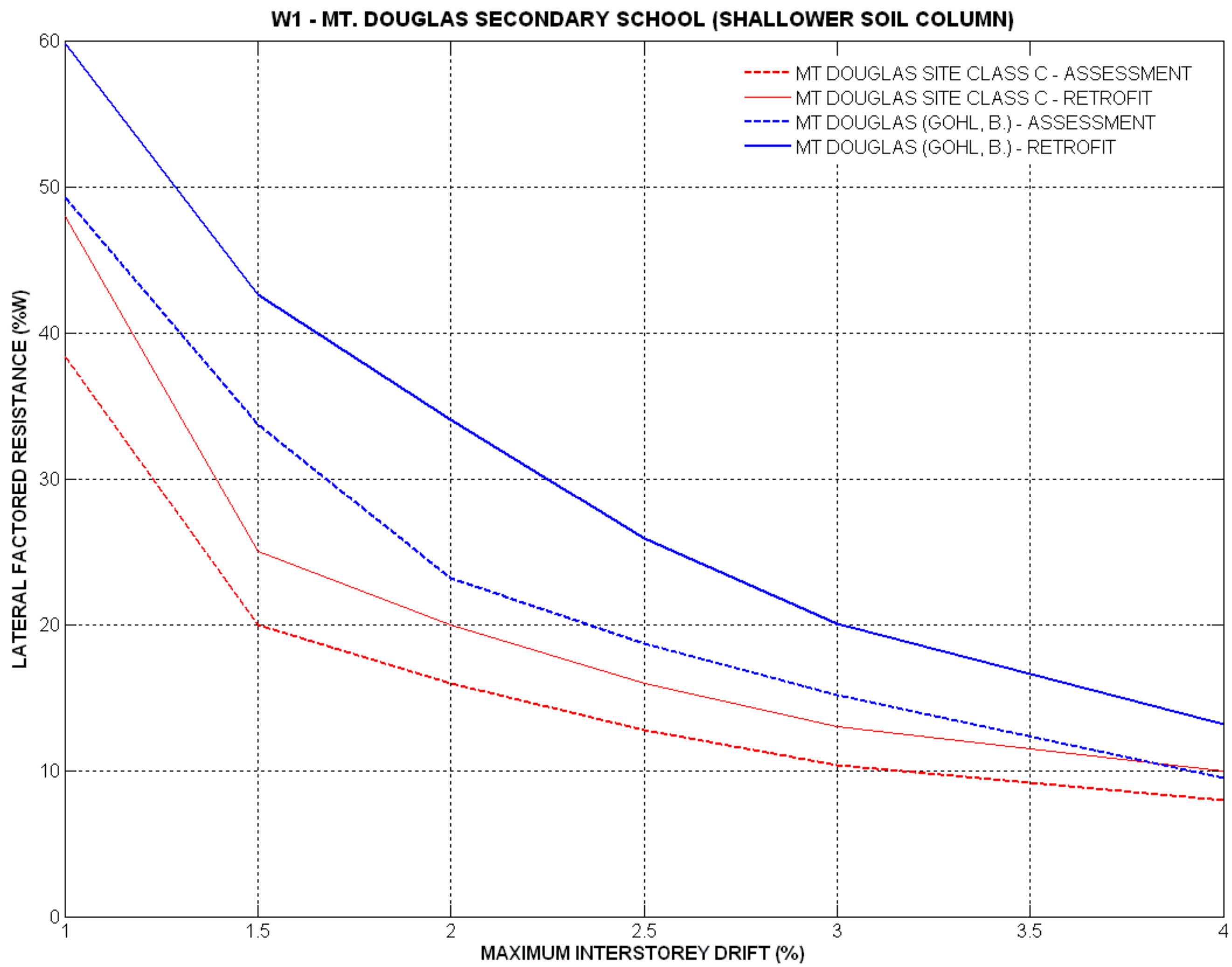
(3) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for C-1 (Deeper soil column)


C1 - MT. DOUGLAS SECONDARY SCHOOL (DEEPER SOIL COLUMN)
MT DOUGLAS SITE CLASS C - ASSESSMENT
MT DOUGLAS SITE CLASS C - RETROFIT
MT DOUGLAS (GOHL, B.) - ASSESSMENT
MT DOUGLAS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2

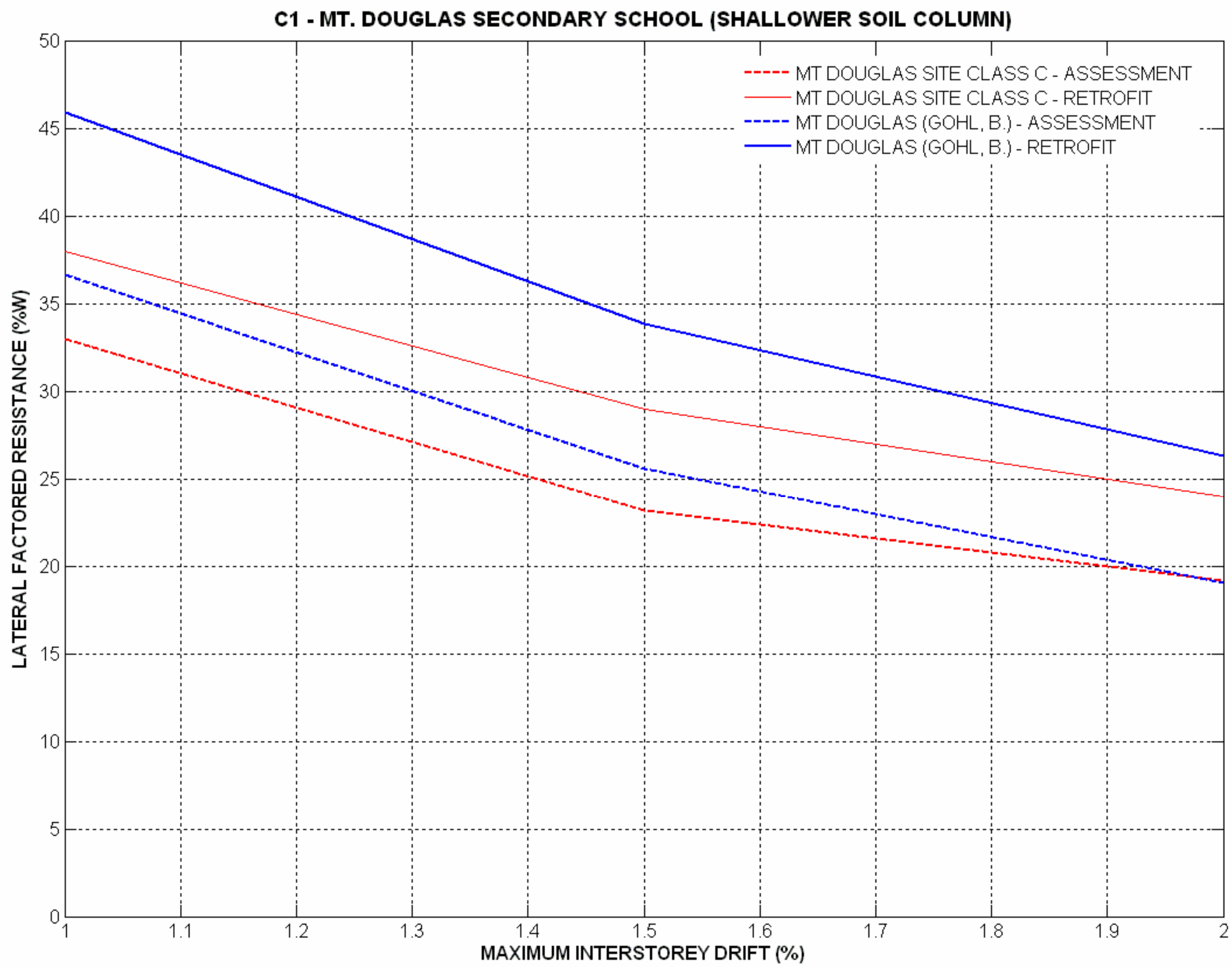
(3) <u>UBC Report (Structural Analysis)</u>

(c) Graph - lateral factored resistance versus drift for W-1 (Shallower soil column)


W1 - MT. DOUGLAS SECONDARY SCHOOL (SHALLOWER SOIL COLUMN)
MT DOUGLAS SITE CLASS C - ASSESSMENT
MT DOUGLAS SITE CLASS C - RETROFIT
MT DOUGLAS (GOHL, B.) - ASSESSMENT
MT DOUGLAS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1
1.5
2
2.5
3
3.5
4

(3) UBC Report (Structural Analysis)

(d) Graph - lateral factored resistance versus drift for C-1 (Shallower soil column)


C1 - MT. DOUGLAS SECONDARY SCHOOL (SHALLOWER SOIL COLUMN)
MT DOUGLAS SITE CLASS C - ASSESSMENT
MT DOUGLAS SITE CLASS C - RETROFIT
MT DOUGLAS (GOHL, B.) - ASSESSMENT
MT DOUGLAS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2

STUDY OF THE GROUND RESPONSE ANALYSIS IN PORT GUICHON USING PRO-SHAKE PROGRAM

By Dr. Liam Finn and Juan C. Carvajal
February 20 / 2007

This study was carried out to analyze in detail the site response ground motions determined by The Throw Company at Port Guichon. After using these records to develop response tables for the W1 LDRS, some questions arose about the site response procedure recommended in the 2nd edition of the Bridging Guidelines using an Equivalent Non-Linear 1-D Shear Wave propagation model. The main concerns were:

- The presence of base-line problems in some Displacement-Time histories.
- High Pseudo Acceleration values in the inelastic-response spectra.

The site response analysis was done using the Pro-Shake computer program and assuming the same soil conditions that The Throw Company used.

A. GROUND RESPONSE ANALYSIS

Acceleration, Displacement and Shear Strain time histories were analyzed to check potential errors in the evaluation of the equivalent shear moduli.

In the analyses, some ground motions showed base-line problems in the Displacement-time histories. However, this didn't happen in the Acceleration and Shear Strain time histories. Pro-Shake evaluates the ground response based on the latter ones. None of the Acceleration and Shear Strain time histories showed base-line problems. This allowed concluding that no errors in the evaluation of the equivalent shear moduli occurred in the ground response analysis. The base-line problems may be due to some base-line problems observed in the input ground motions and to the integration procedure of the acceleration in the frequency domain inherent in Pro-Shake.

B. SENSITIVITY ANALISYS OF GROUND RESPONSE

The following assumptions were made about the layers where no field data was available:

- *" The shear wave velocity of the firm ground was assumed as Vs = 760 m/s in accordance with the recommendations given in the NBCC 2005 ".*
- *" The shear wave velocity profile between 30 m and 150 m is assumed proportional to (depth)$^{0.33}$ ".*

According to the NBCC 2005, the Site class C for Seismic Site Response has a Shear Wave Velocity Vs that varies from 360 to 760 m/s. Based on these limits, 5 scenarios were considered to analyze the influence of the Vs of firm ground and the depth of the soil deposit in the ground response.

- Depth = 150 m and Vs of Till = 760 m/s — Depth and Vs assumed by The Trow Company.
- Depth = 150 m and Vs of Till = 500 m/s — Vs reported by Finn and Ventura.
- Depth = 150 m and Vs of Till = 400 m/s — The lower limit was assumed as 400 m/s since the lowest layer of the soil deposit had a Vs=395 m/s.
- Depth = 110 m and Vs of firm ground = 371 m/s — Shear modulus G of the firm ground doesn't degrade with shaking.
- Depth = 110 m and Vs of firm ground = 371 m/s — Shear modulus G and Damping ξ of the firm ground

degrade according to Vucetic and Dobry's model.

The sensitivity analysis concluded that the average of pseudo velocity response spectra between 0.5 and 1.5 seconds varies from 1.4 to 1.9 times the mean of pseudo velocity response spectra of firm ground. Based on this, it was recommended that a Vs = 500 m/s were assumed as the shear wave velocity of the Glacial Till (firm ground).

A. GROUND RESPONSE ANALYSIS

Figures 1 to 10 plot the Acceleration time histories at 0.6 m depth in the soil deposit for each input ground motion in the firm ground. The output ground motions didn't show any base-line problems.

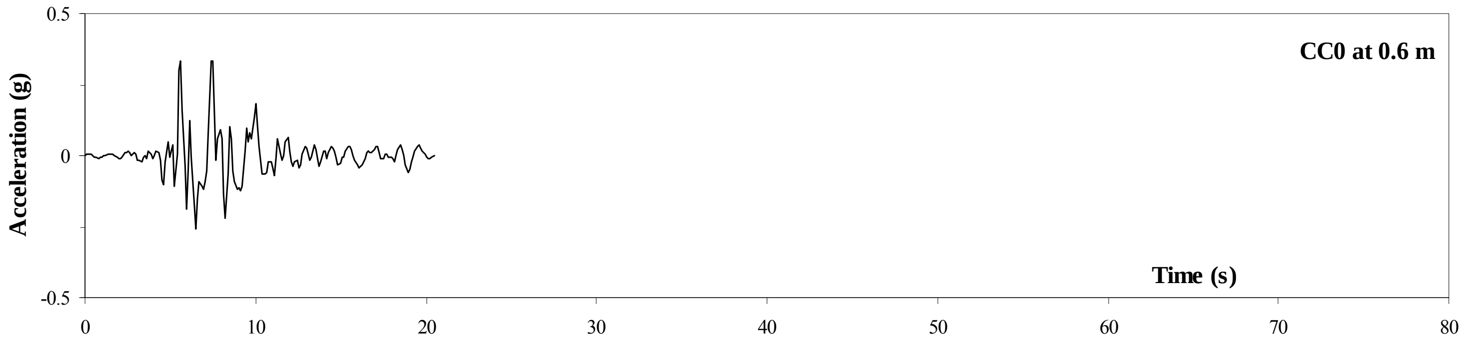


Figure 1. Output acceleration time history in the soil deposit for the CCO ground motion

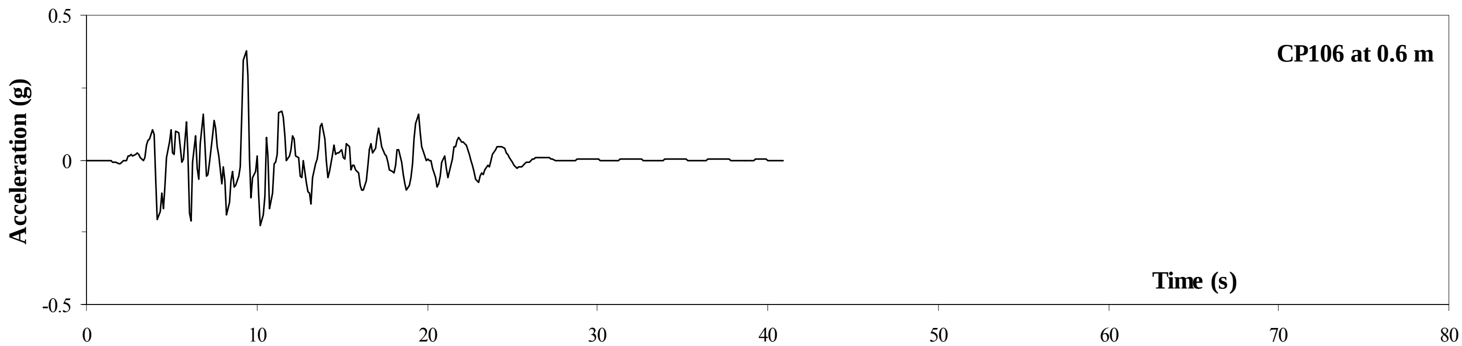


Figure 2. Output acceleration time history in the soil deposit for the CP106 ground motion

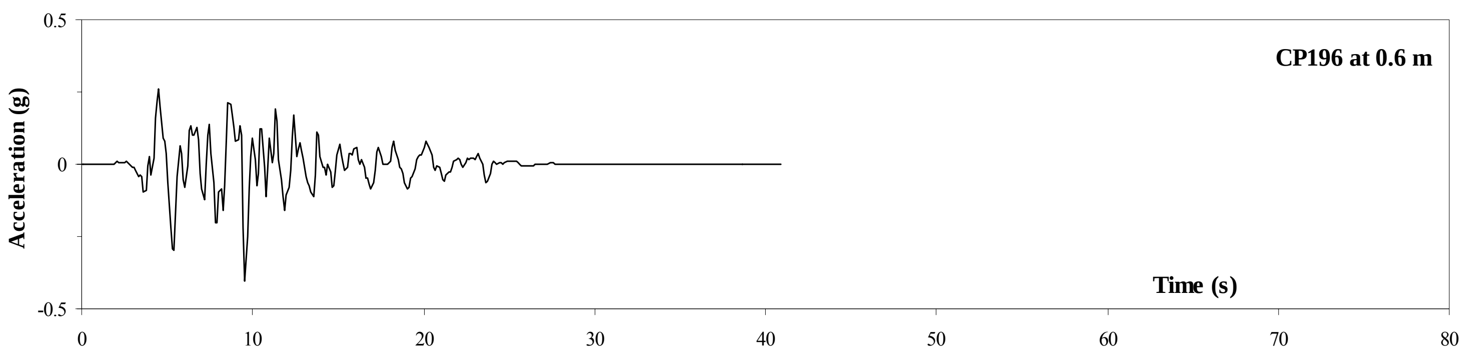


Figure 3. Output acceleration time history in the soil deposit for the CP196 ground motion

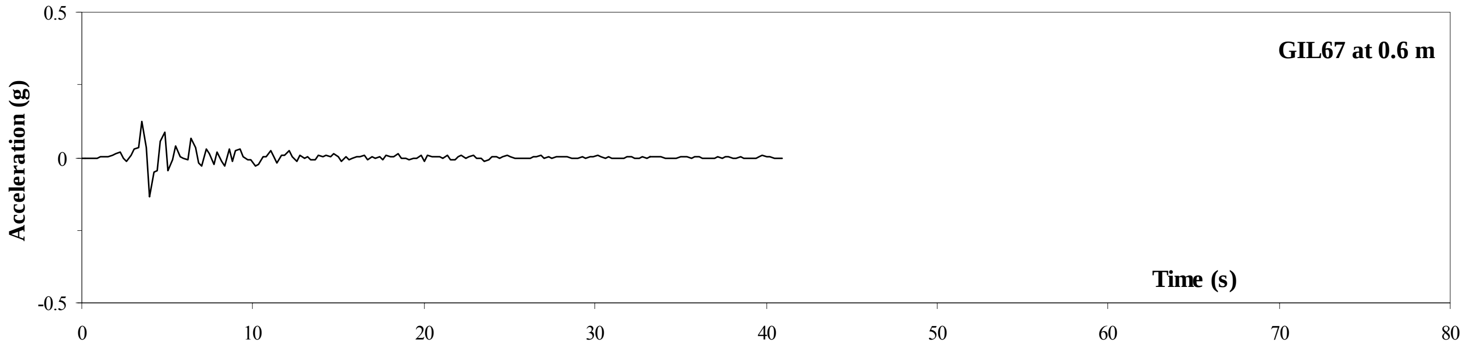


Figure 4. Output acceleration time history in the soil deposit for the GIL67 ground motion

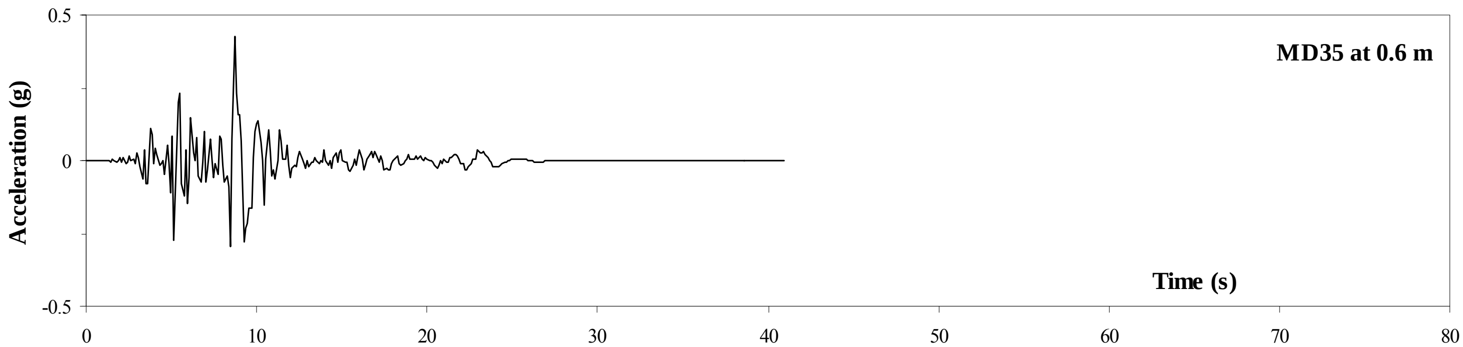


Figure 5. Output acceleration time history in the soil deposit for the MD35 ground motion

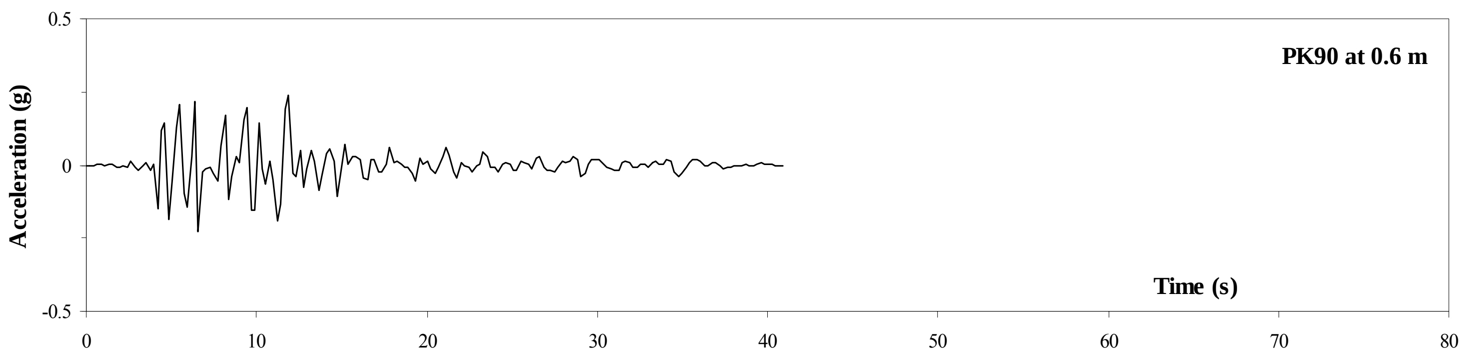


Figure 6. Output acceleration time history in the soil deposit for the PK90 ground motion

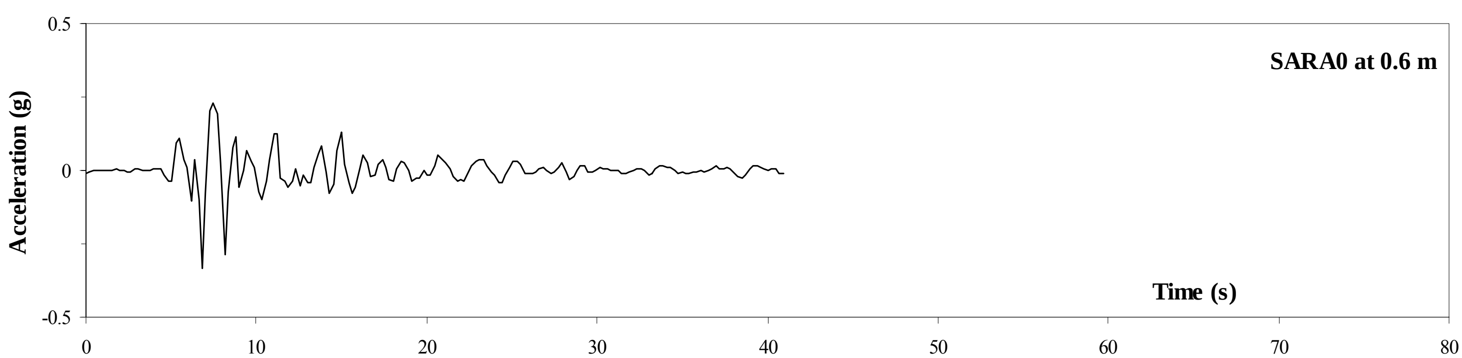


Figure 7. Output acceleration time history in the soil deposit for the SARA0 ground motion

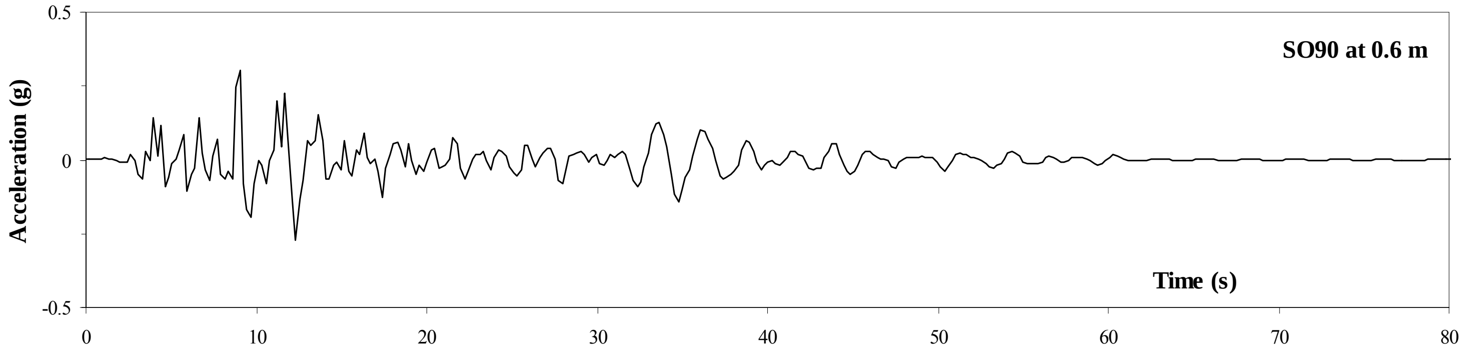


Figure 8. Output acceleration time history in the soil deposit for the SO90 ground motion

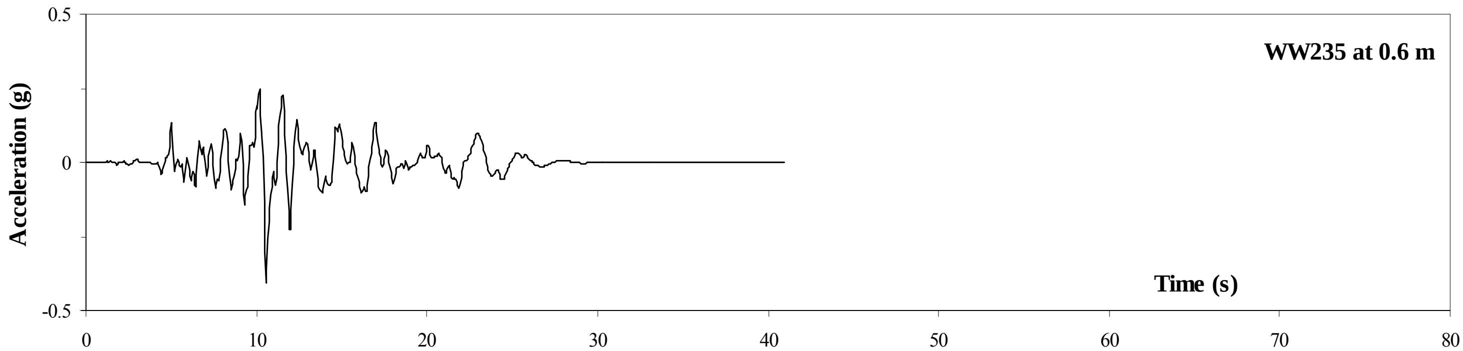


Figure 9. Output acceleration time history in the soil deposit for the WW235 ground motion

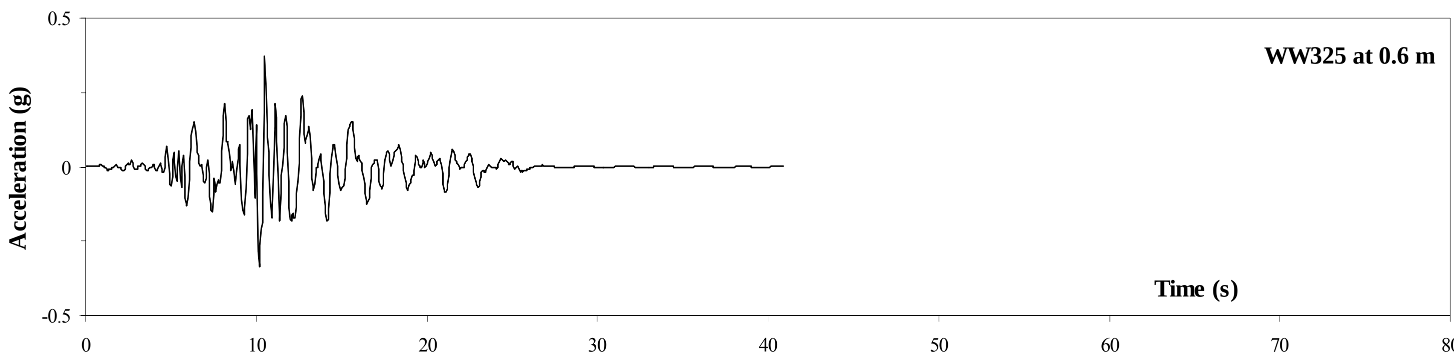


Figure 10. Output acceleration time history in the soil deposit for the WW325 ground motion

Figures 11 to 20 plot the Velocity time histories at 0.6 m depth in the soil deposit for each input ground motion in the firm ground. Figure 20 showed slightly base-line problems.

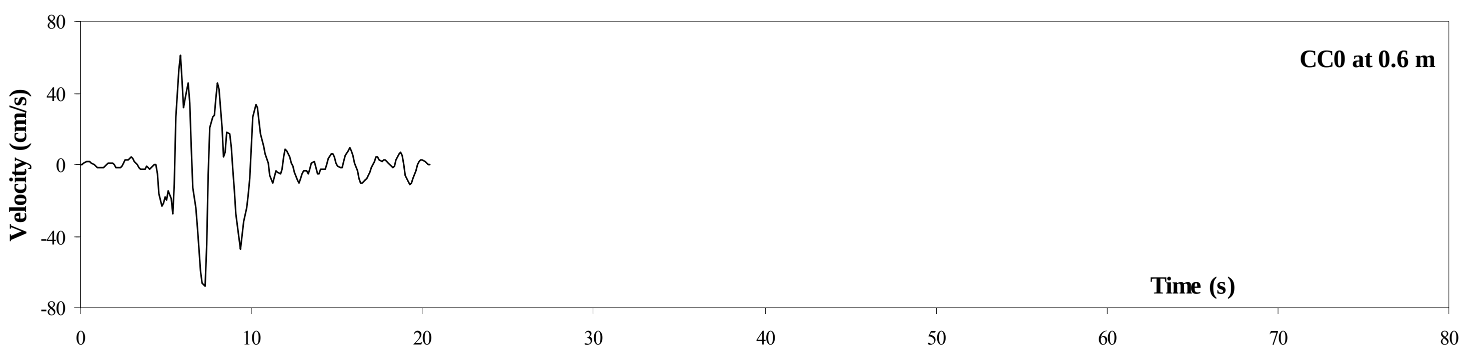


Figure 11. Output velocity time history in the soil deposit for the CCO ground motion

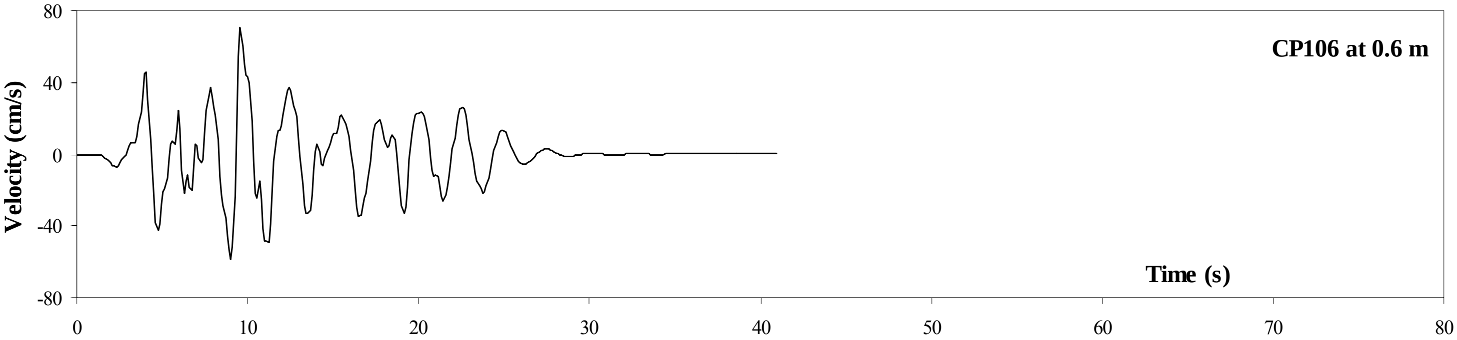


Figure 12. Output velocity time history in the soil deposit for the CP106 ground motion

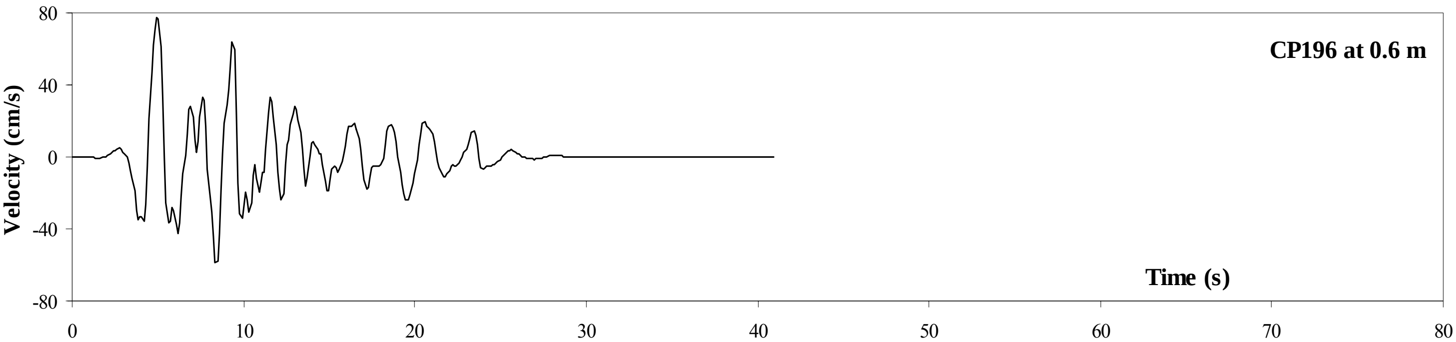


Figure 13. Output velocity time history in the soil deposit for the CP196 ground motion

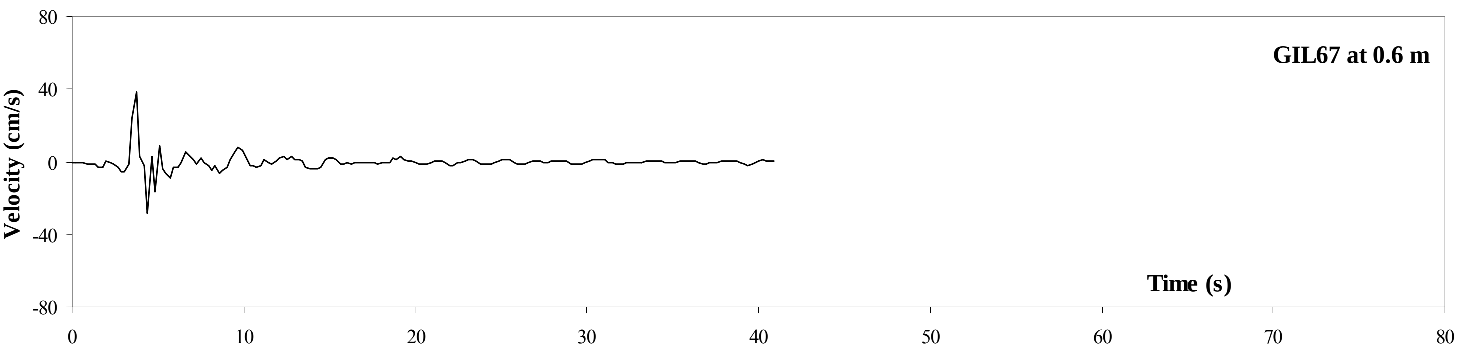


Figure 14. Output velocity time history in the soil deposit for the GIL67 ground motion

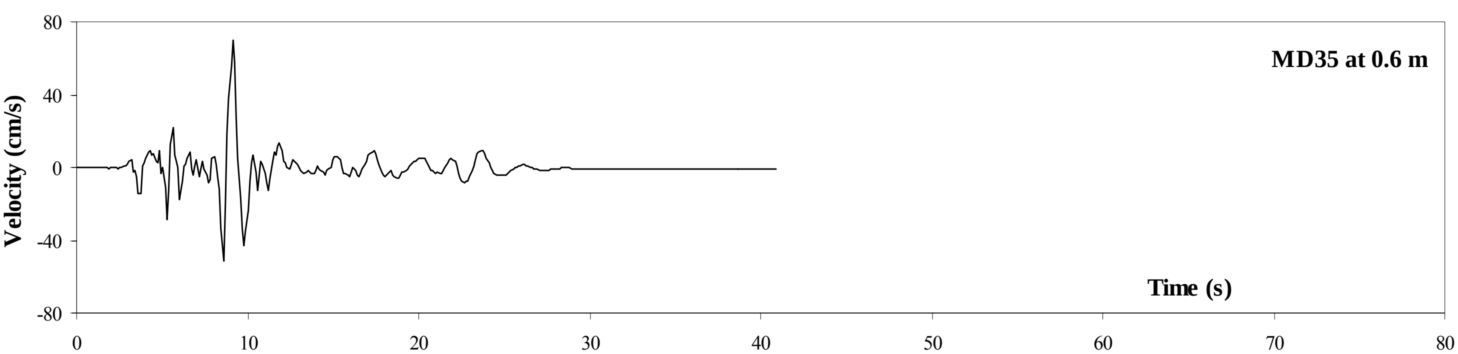


Figure 15. Output velocity time history in the soil deposit for the MD35 ground motion

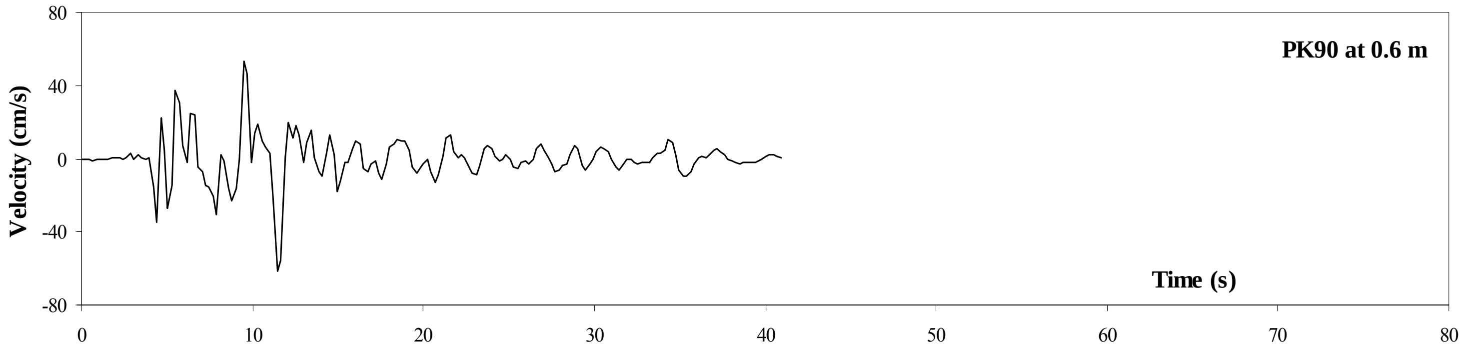


Figure 16. Output velocity time history in the soil deposit for the PK90 ground motion

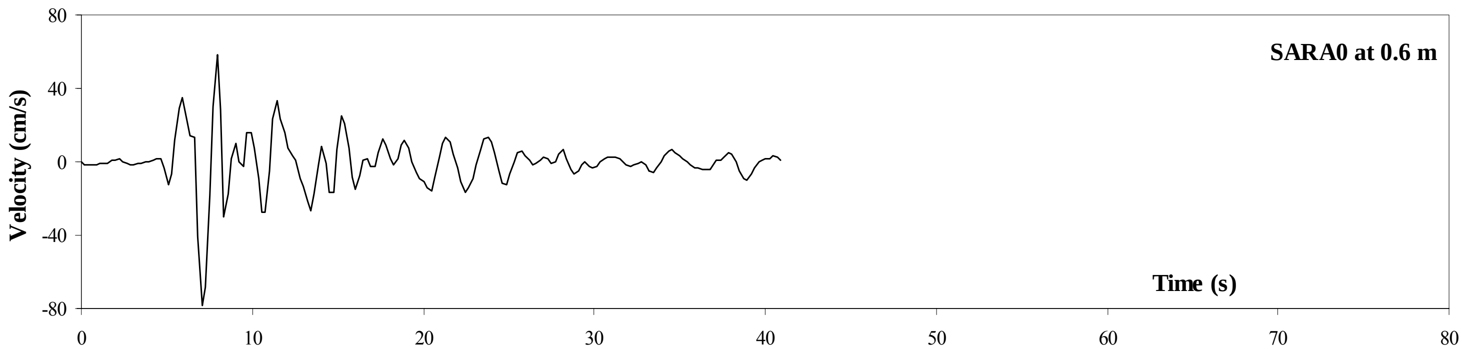


Figure 17. Output velocity time history in the soil deposit for the SARA0 ground motion

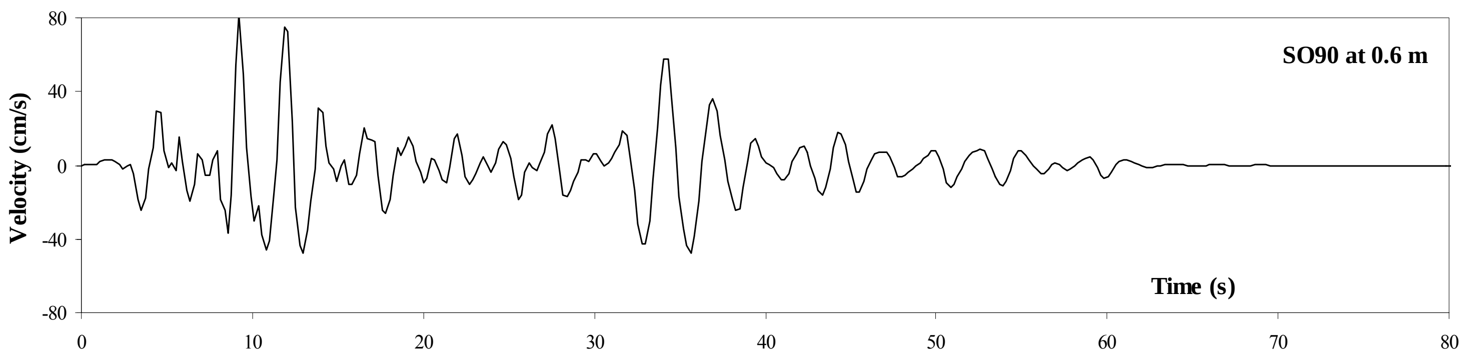


Figure 18. Output velocity time history in the soil deposit for the SO90 ground motion

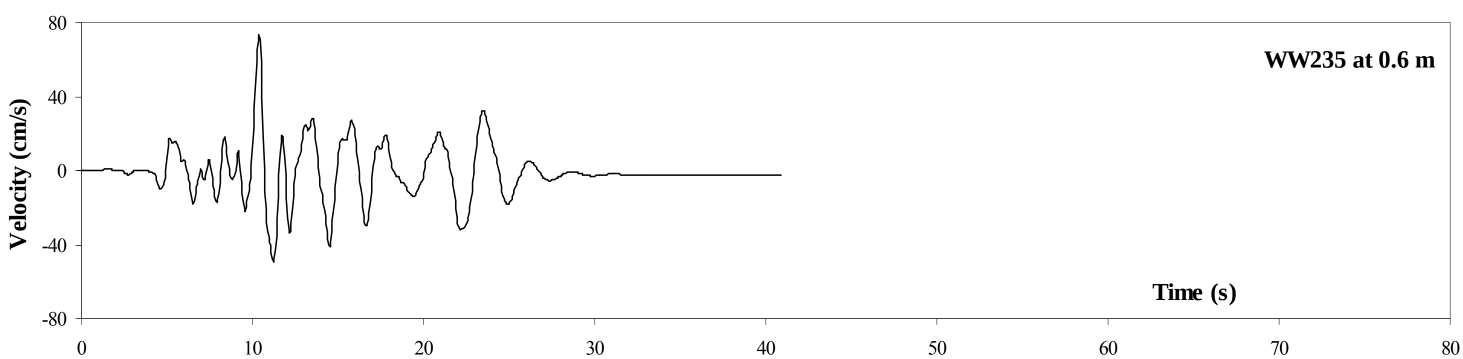


Figure 19. Output acceleration time history in the soil deposit for the WW235 ground motion

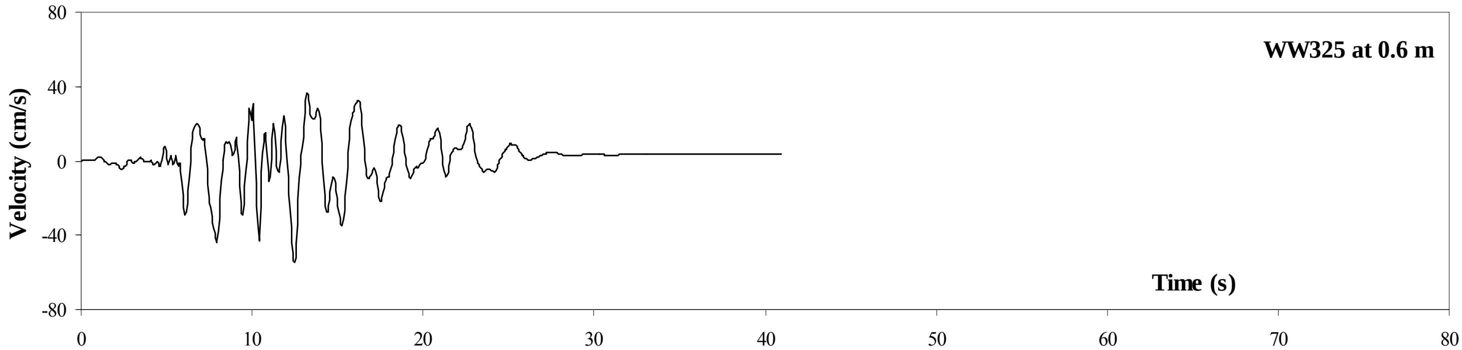


Figure 20. Output velocity time history in the soil deposit for the WW325 ground motion

Figures 21 to 30 plot the Displacement time histories at 0.6 m depth in the soil deposit for each input ground motion in the firm ground. Figures 21, 25, 26, 27, 29 and 30 showed base-line problems.

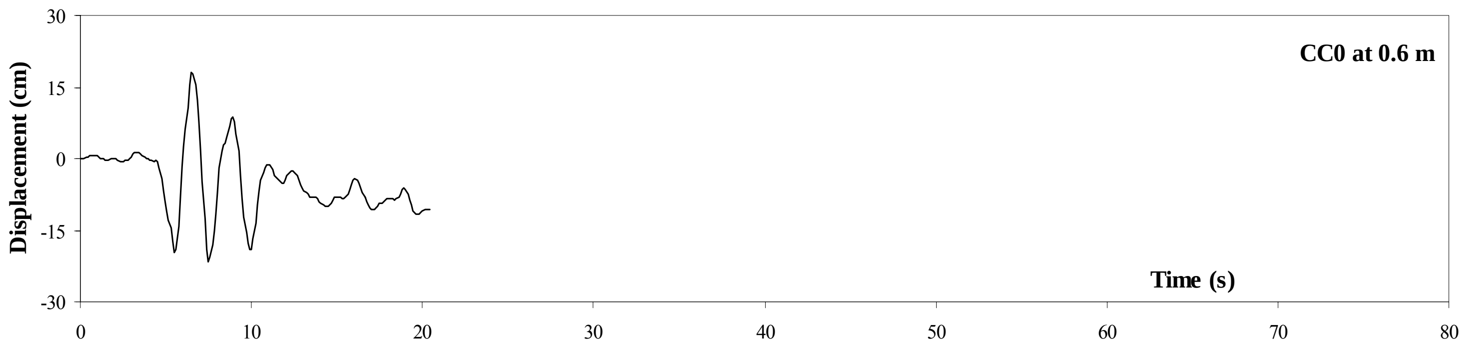


Figure 21. Output displacement time history in the soil deposit for the CCO ground motion

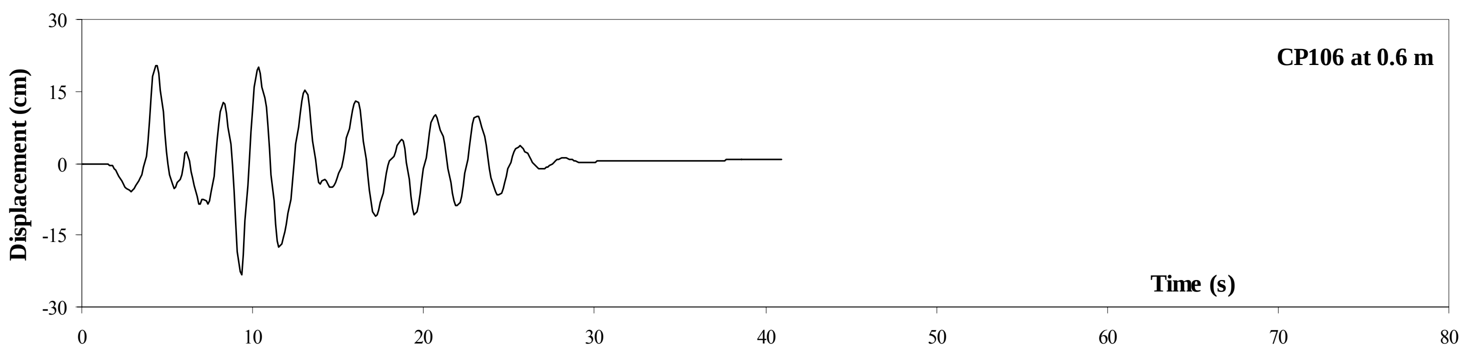


Figure 22. Output displacement time history in the soil deposit for the CP106 ground motion

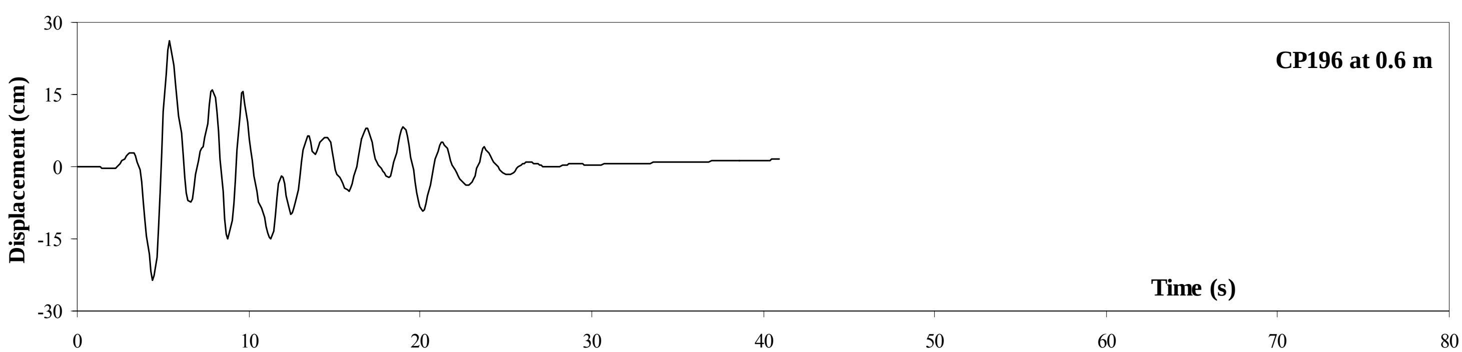


Figure 23. Output displacement time history in the soil deposit for the CP196 ground motion

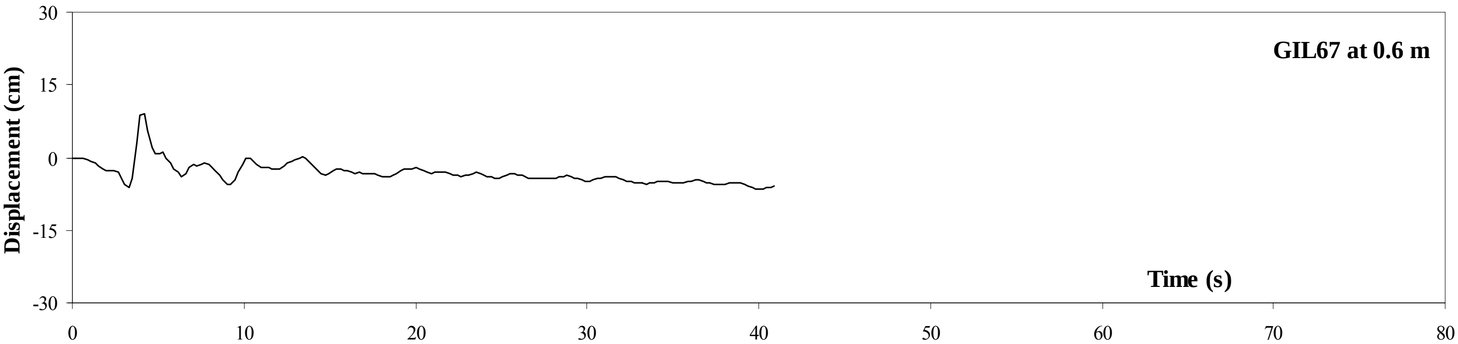


Figure 24. Output displacement time history in the soil deposit for the GIL67 ground motion

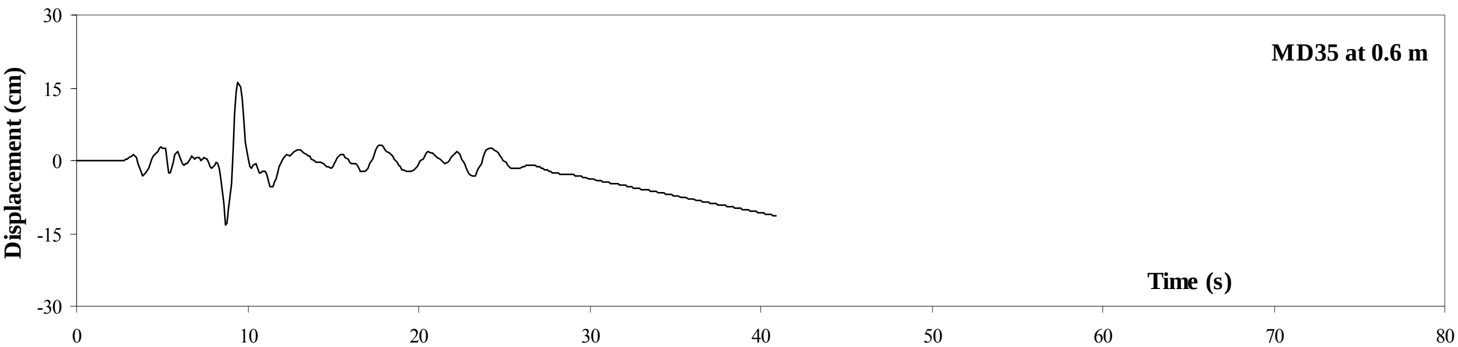


Figure 25. Output displacement time history in the soil deposit for the MD35 ground motion

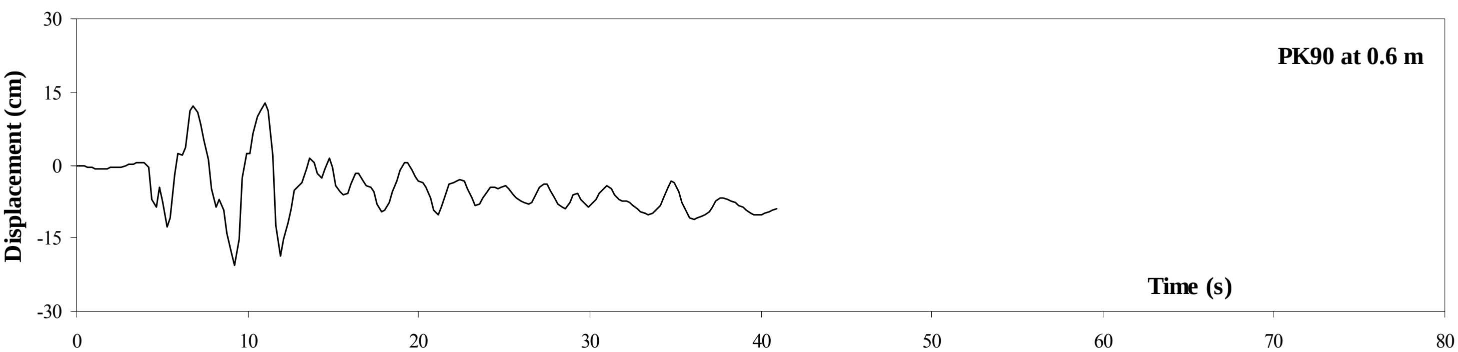


Figure 26. Output displacement time history in the soil deposit for the PK90 ground motion

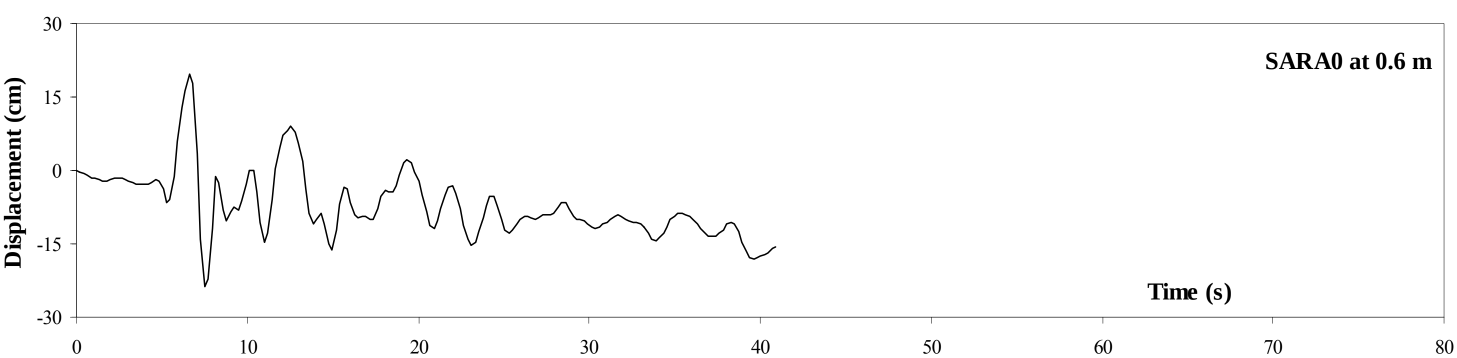


Figure 27. Output displacement time history in the soil deposit for the SARA0 ground motion

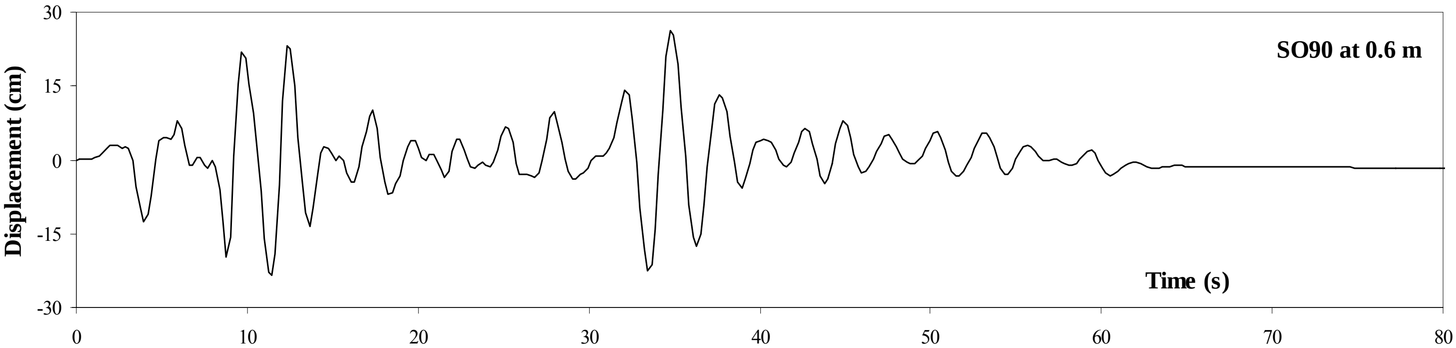


Figure 28. Output displacement time history in the soil deposit for the SO90 ground motion

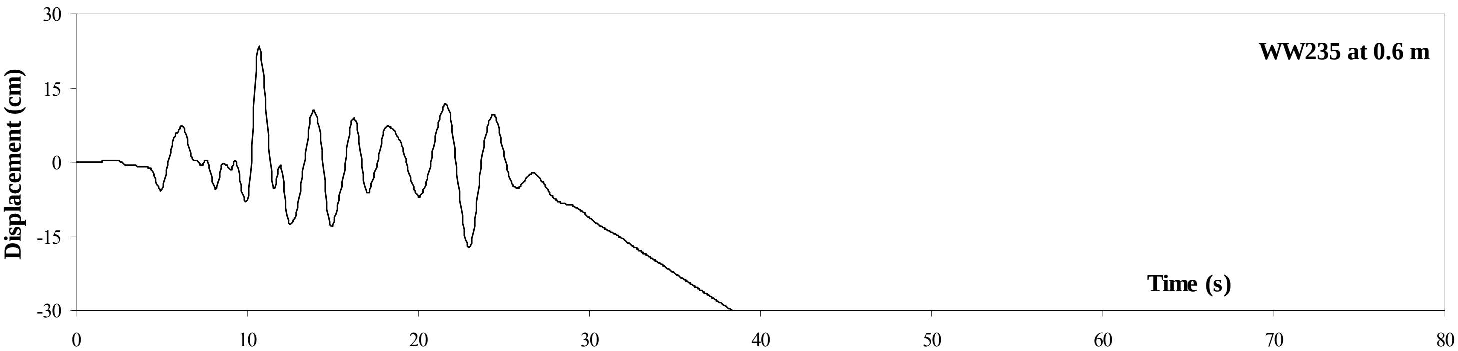


Figure 29. Output displacement time history in the soil deposit for the WW235 ground motion

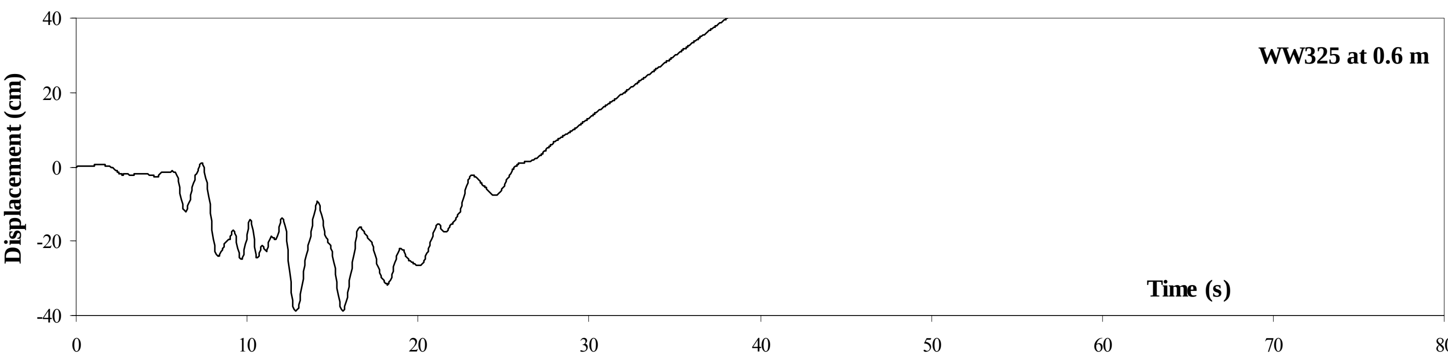


Figure 30. Output displacement time history in the soil deposit for the WW325 ground motion

Figures 31 to 35 plot the Fourier spectra of the output Acceleration time histories at 0.6 m depth in the soil deposit.

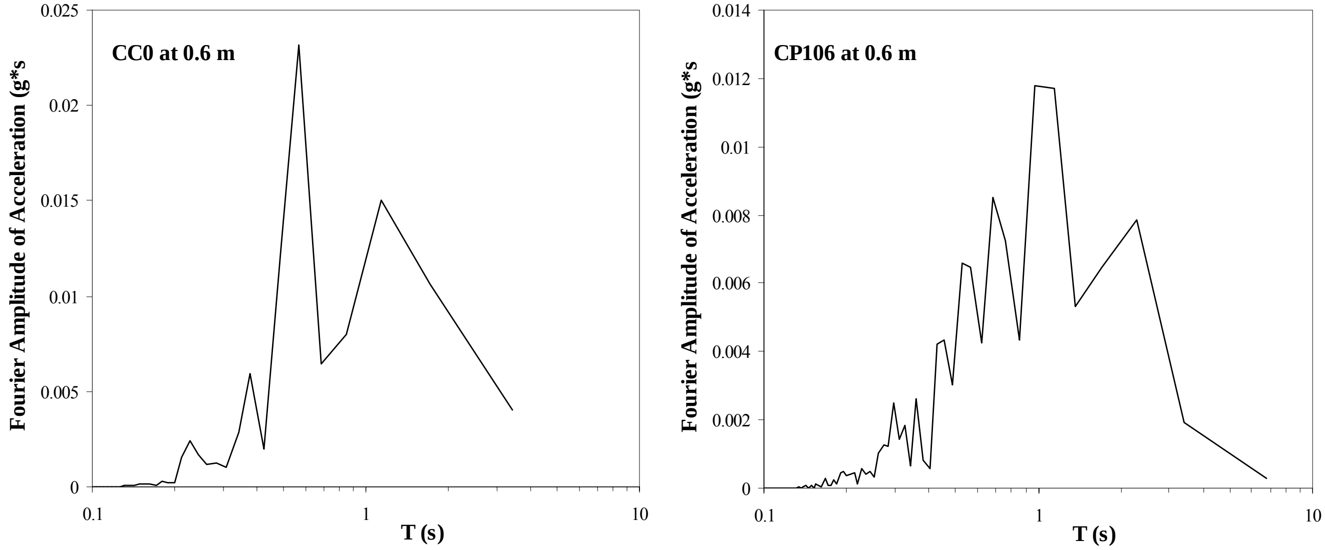


Figure 31. Output acceleration Fourier spectra in the soil deposit for the CC0 and CP106 ground motions

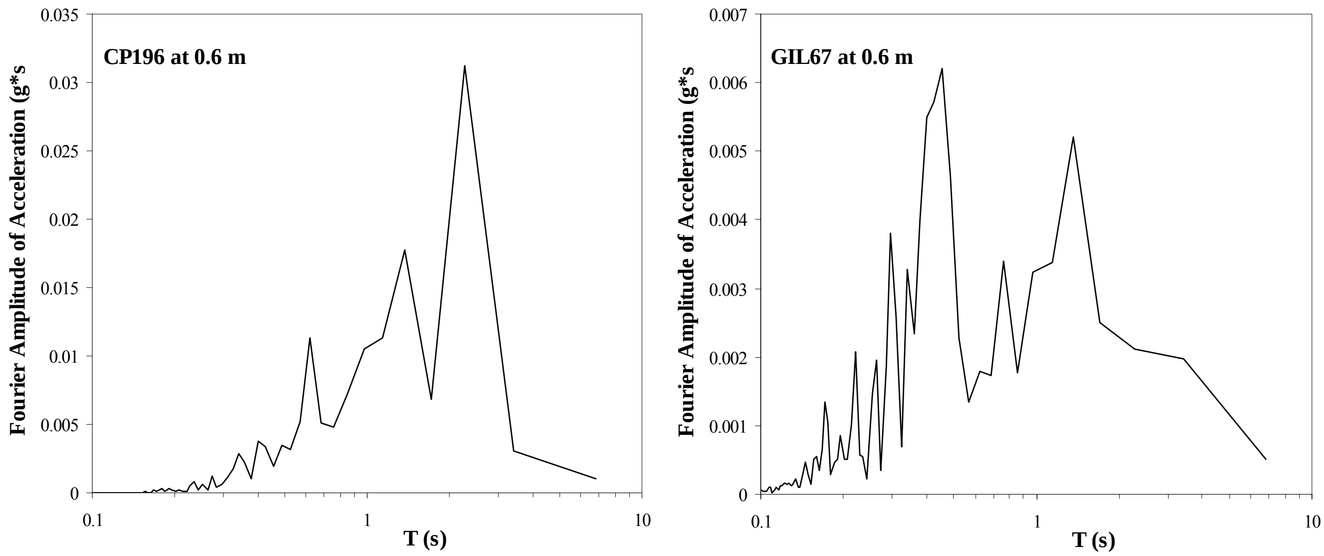


Figure 32. Output acceleration Fourier spectra in the soil deposit for the CP196 and GIL67 ground motions

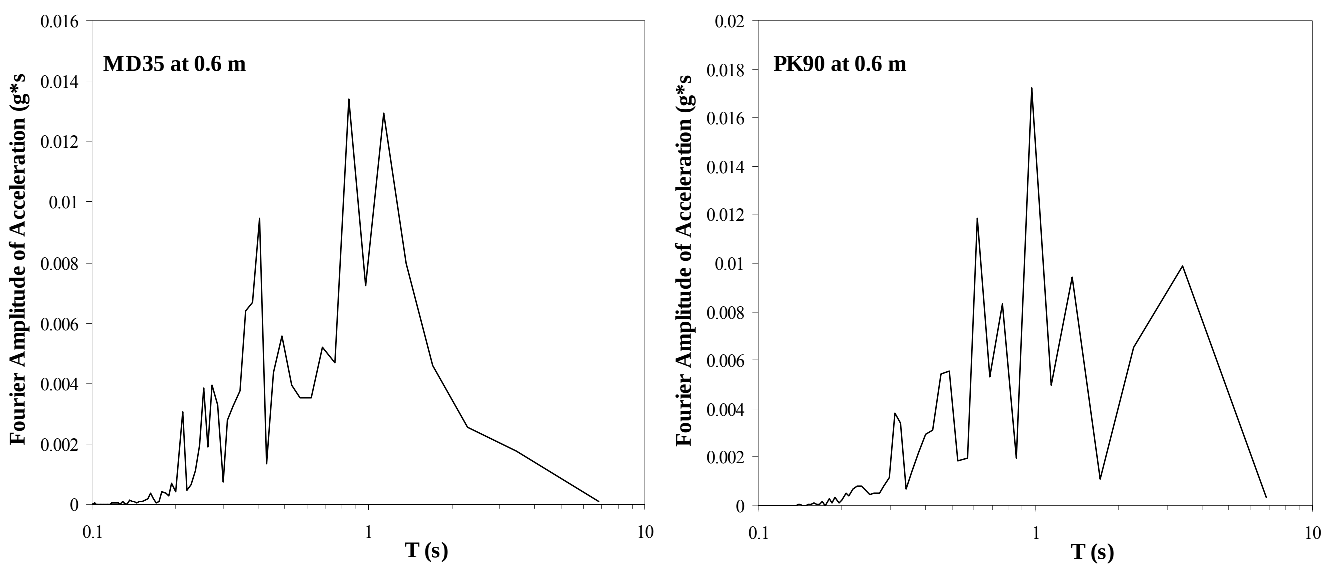


Figure 33. Output acceleration Fourier spectra in the soil deposit for the MD35 and PK90 ground motions

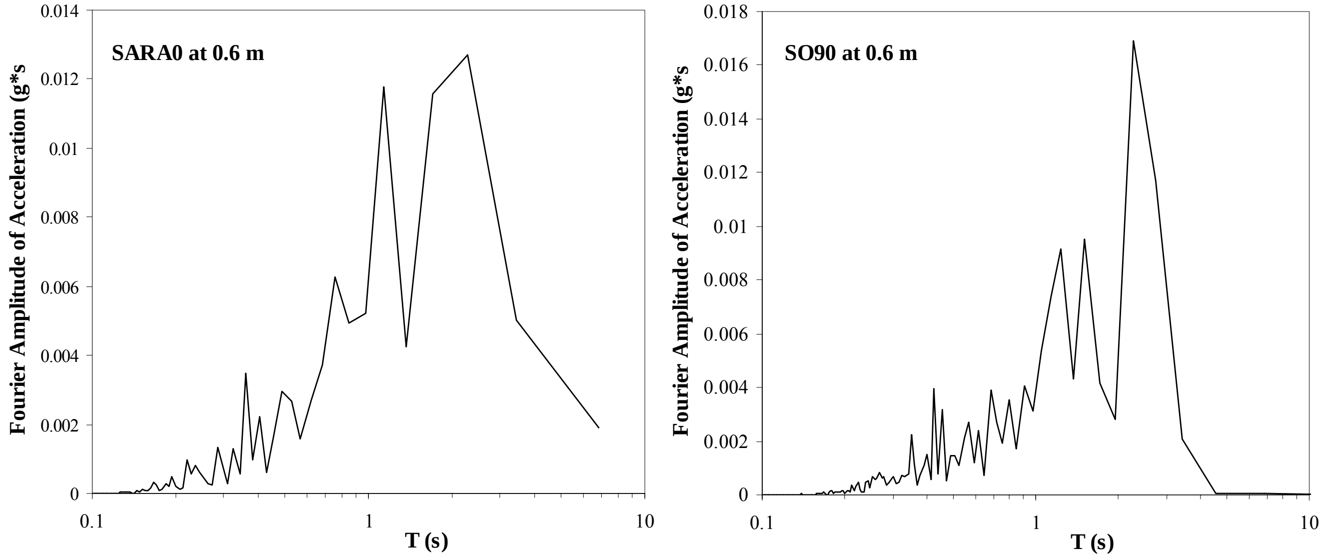


Figure 34. Output acceleration Fourier spectra in the soil deposit for the SARA0 and SO90 ground motions

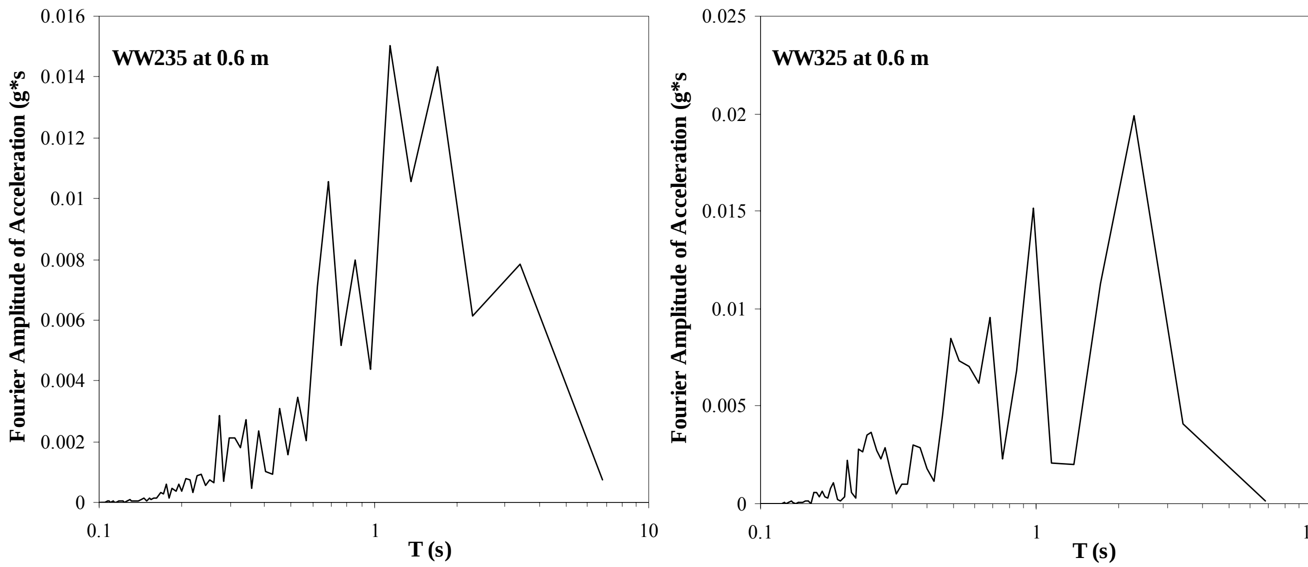


Figure 35. Output acceleration Fourier spectra in the soil deposit for the WW235 and WW325 ground motions

Figures 36 to 40 plot the Fourier spectra of the output Velocity time histories at 0.6 m depth in the soil deposit.

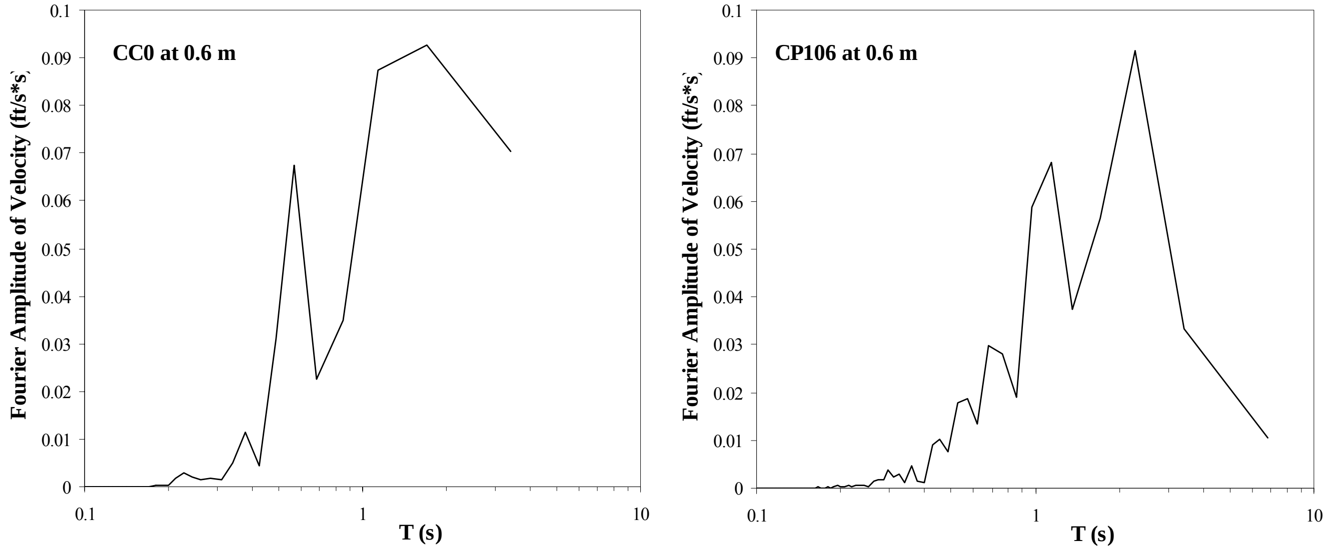


Figure 36. Output velocity Fourier spectra in the soil deposit for the CC0 and CP106 ground motions

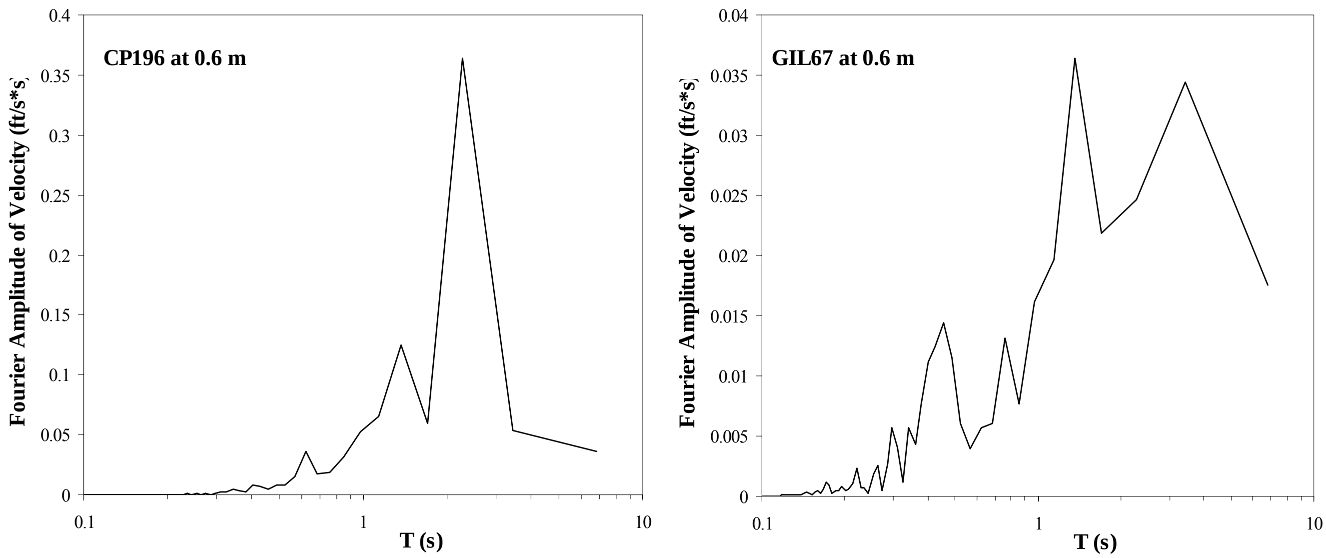


Figure 37. Output velocity Fourier spectra in the soil deposit for the CP196 and GIL67 ground motions

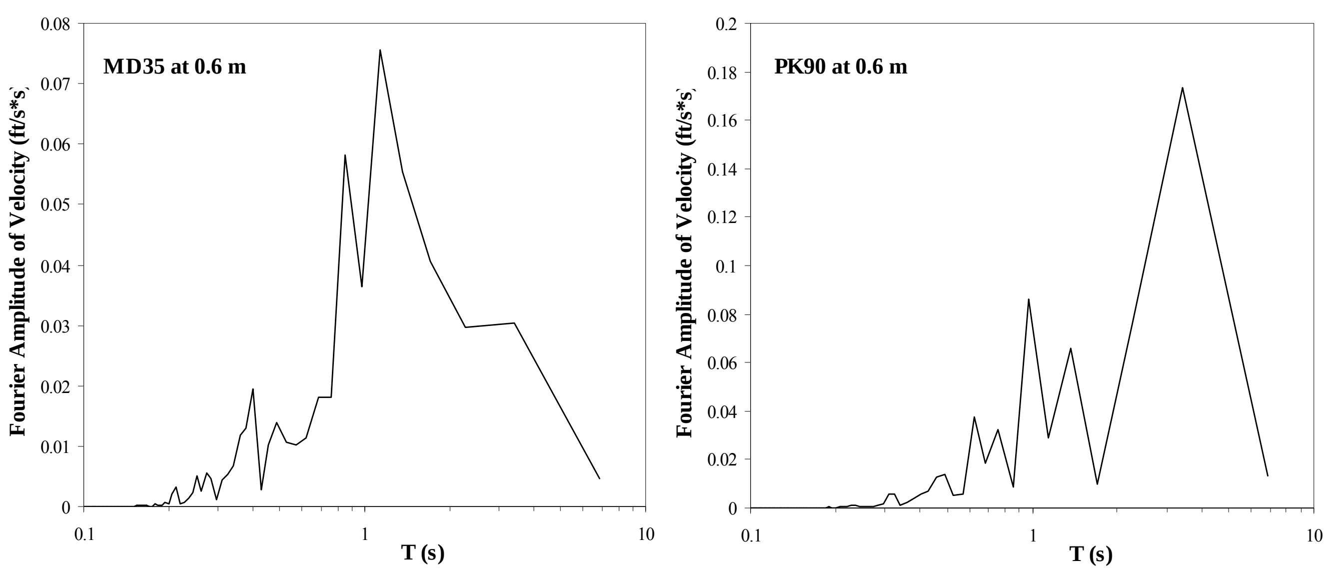


Figure 38. Output velocity Fourier spectra in the soil deposit for the MD35 and PK90 ground motions

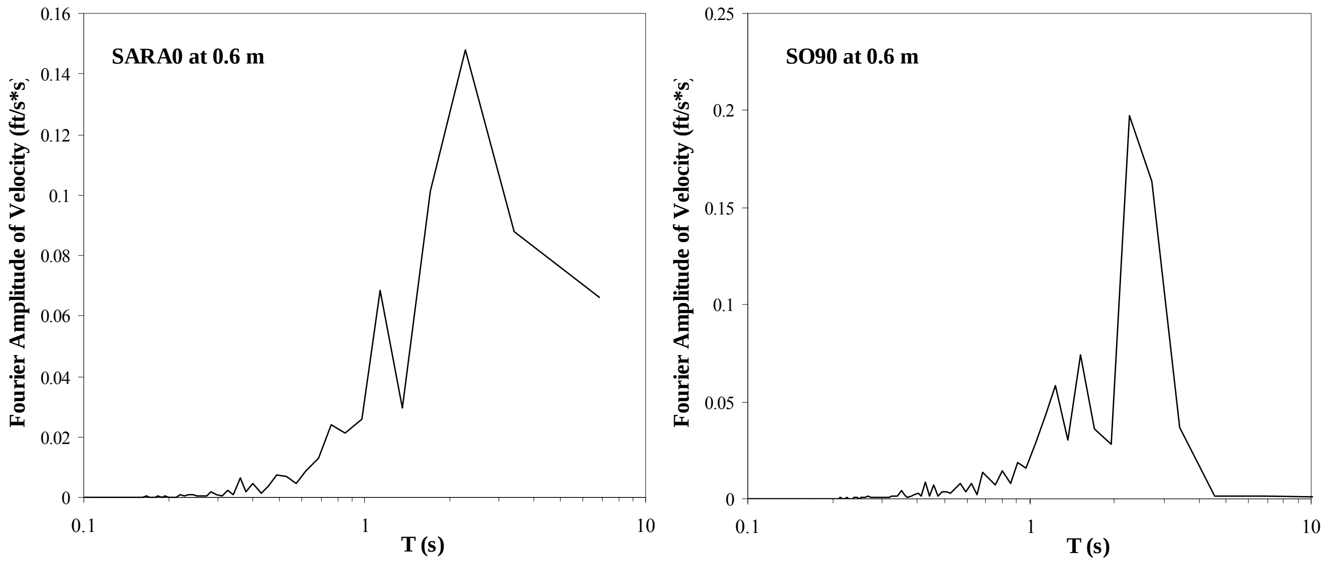


Figure 39. Output velocity Fourier spectra in the soil deposit for the SARA0 and SO90 ground motions

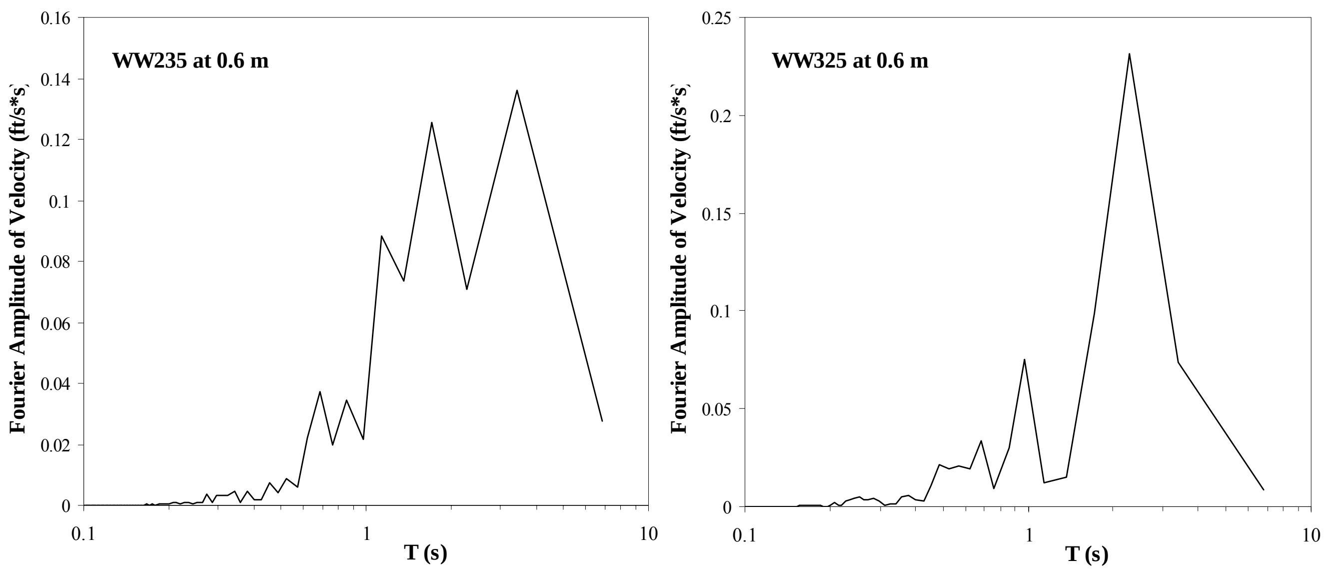


Figure 40. Output velocity Fourier spectra in the soil deposit for the WW235 and WW325 ground motions

Figures 41 to 45 plot the Fourier spectra of the output Displacement time histories at 0.6 m depth in the soil deposit.

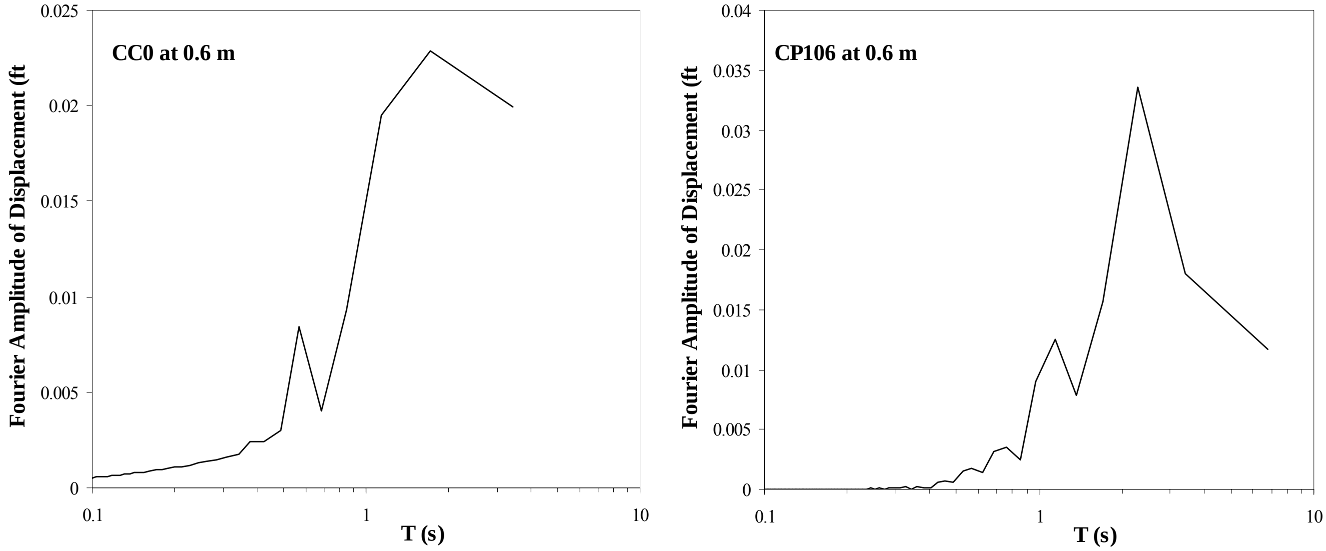


Figure 41. Output displacement Fourier spectra in the soil deposit for the CC0 and CP106 ground motions

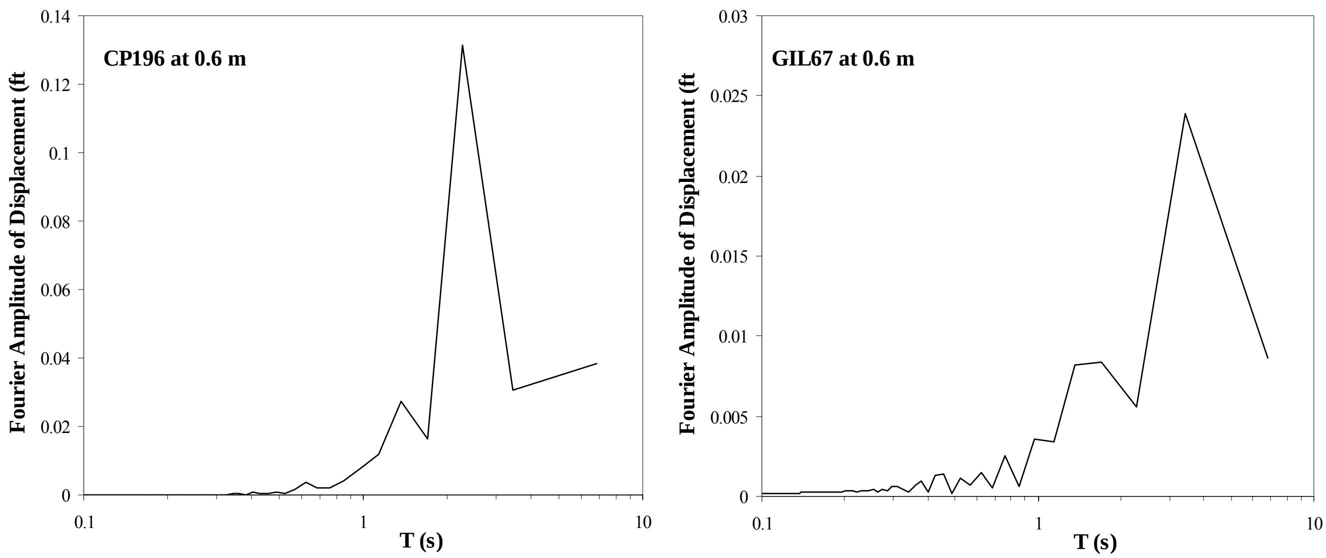


Figure 42. Output displacement Fourier spectra in the soil deposit for the CP196 and GIL67 ground motions

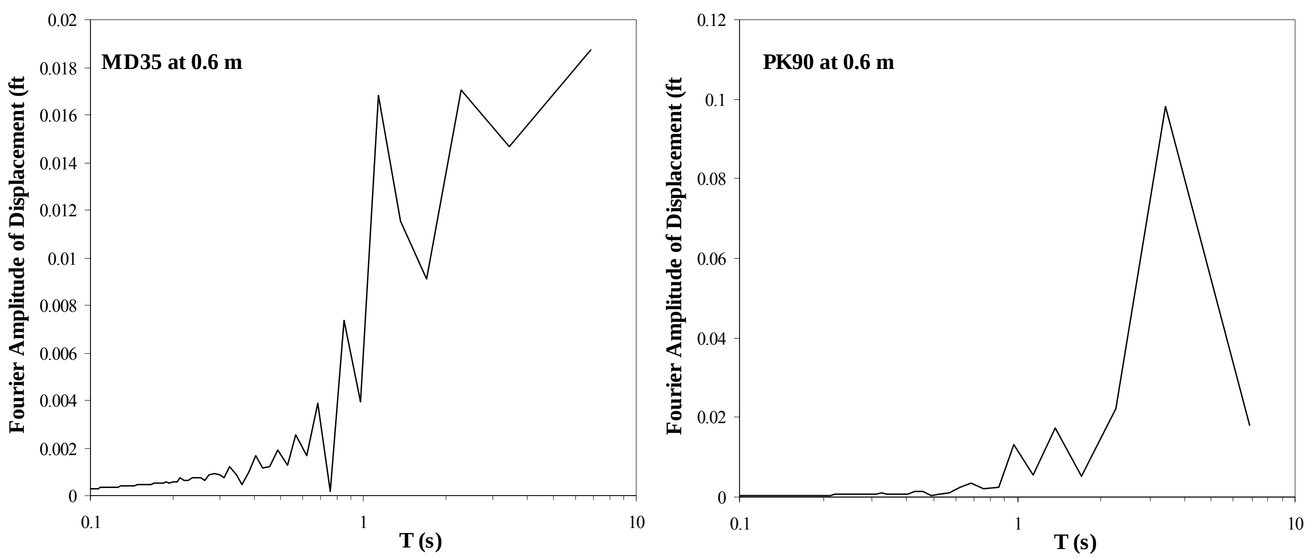


Figure 43. Output displacement Fourier spectra in the soil deposit for the MD35 and PK90 ground motions

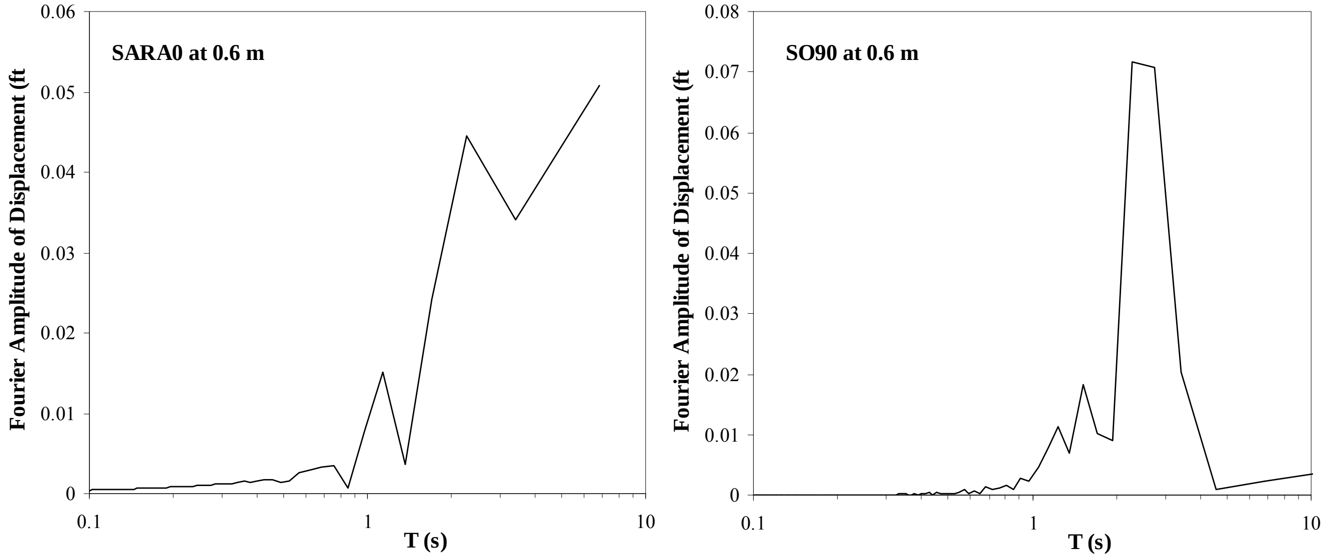


Figure 44. Output displacement Fourier spectra in the soil deposit for the SARA0 and SO90 ground motions

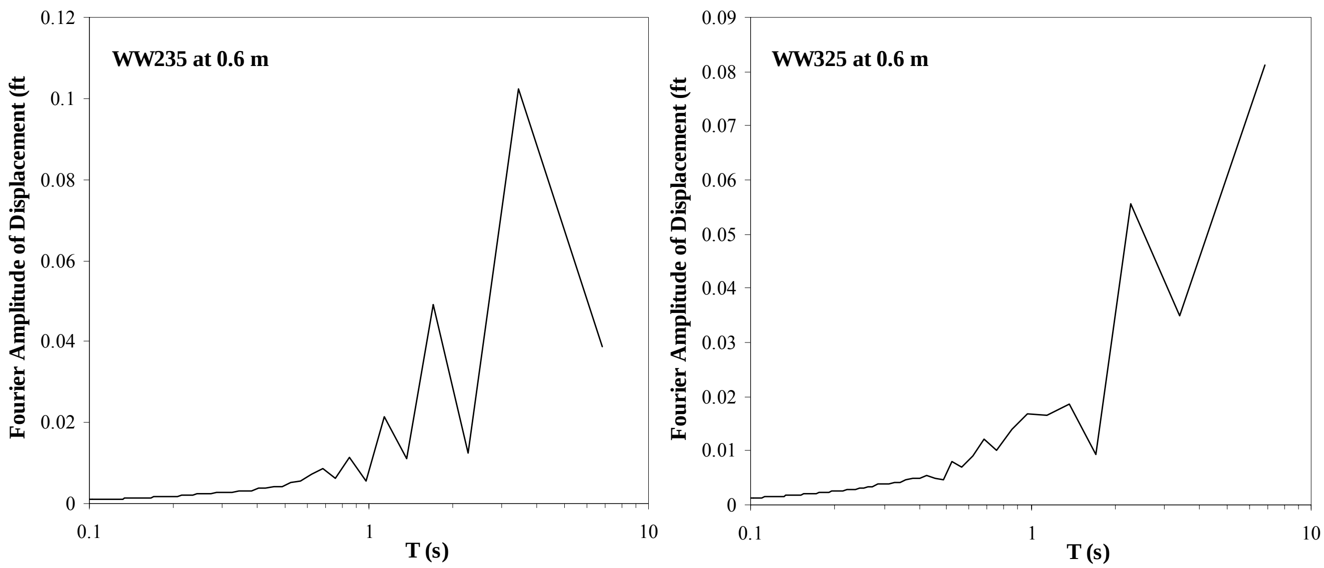


Figure 45. Output displacement Fourier spectra in the soil deposit for the WW235 and WW325 ground motions

Figures 46 to 55 plot the Shear Strain time histories at 0.6 m depth in the soil deposit for each input ground motion in the firm ground. The output ground motions didn't show any base-line problems.

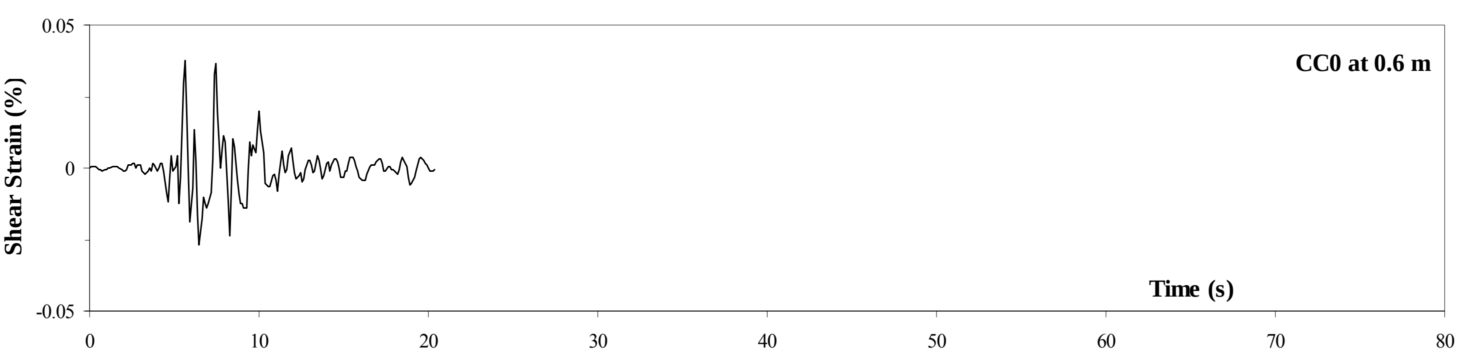


Figure 46. Output Shear Strain time history in the soil deposit for the CCO ground motion

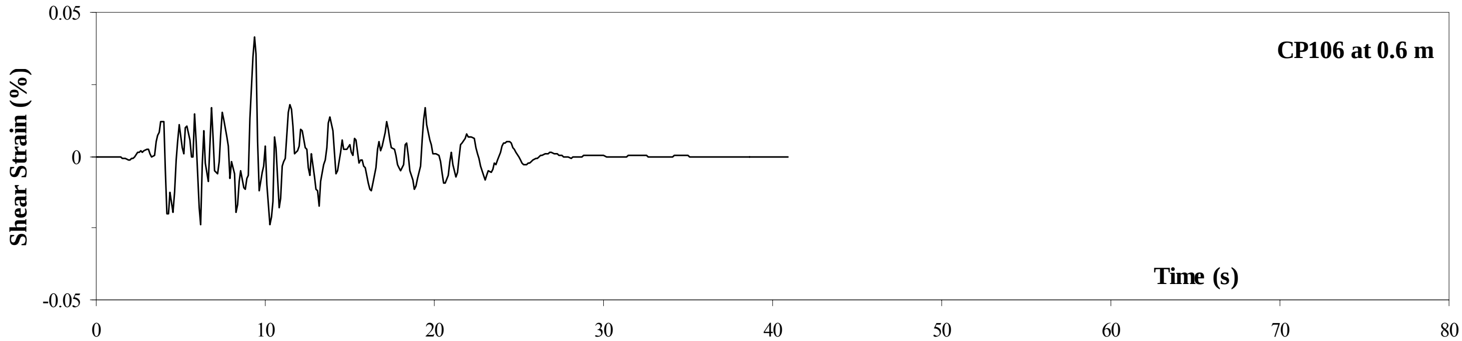


Figure 47. Output Shear Strain time history in the soil deposit for the CP106 ground motion

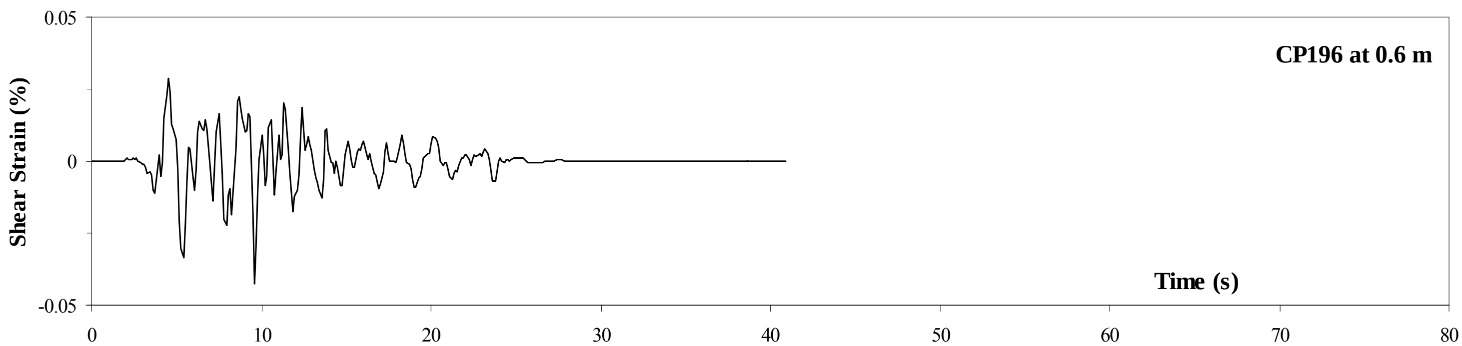


Figure 48. Output Shear Strain time history in the soil deposit for the CP196 ground motion

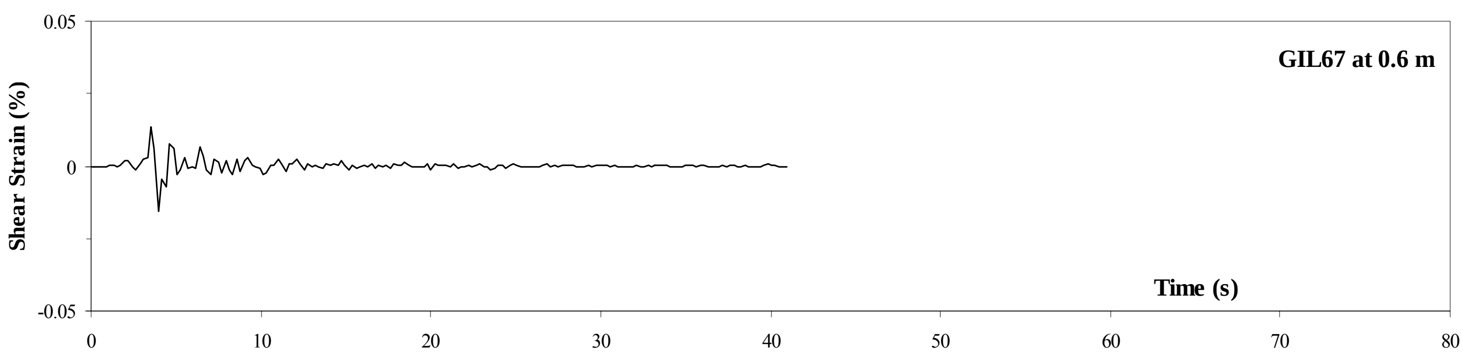


Figure 49. Output Shear Strain time history in the soil deposit for the GIL67 ground motion

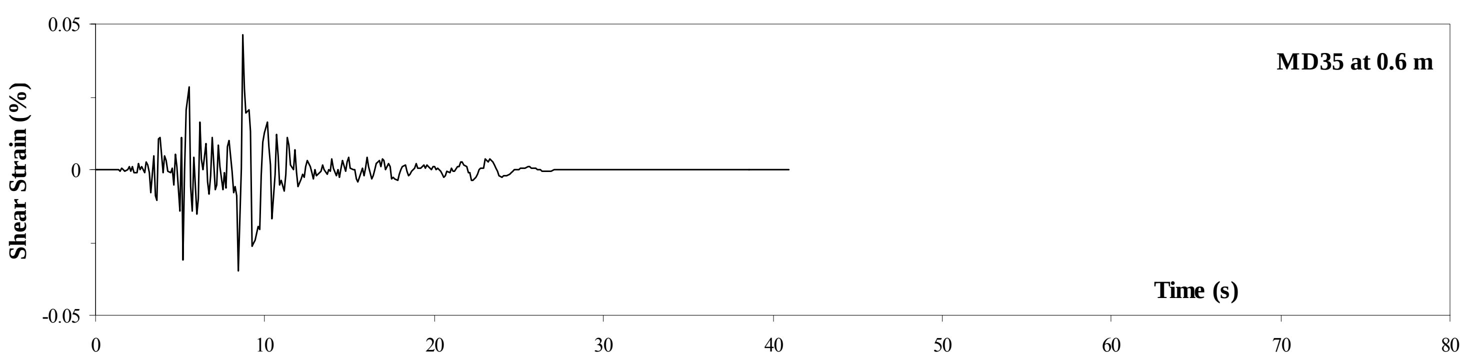


Figure 50. Output Shear Strain time history in the soil deposit for the MD35 ground motion

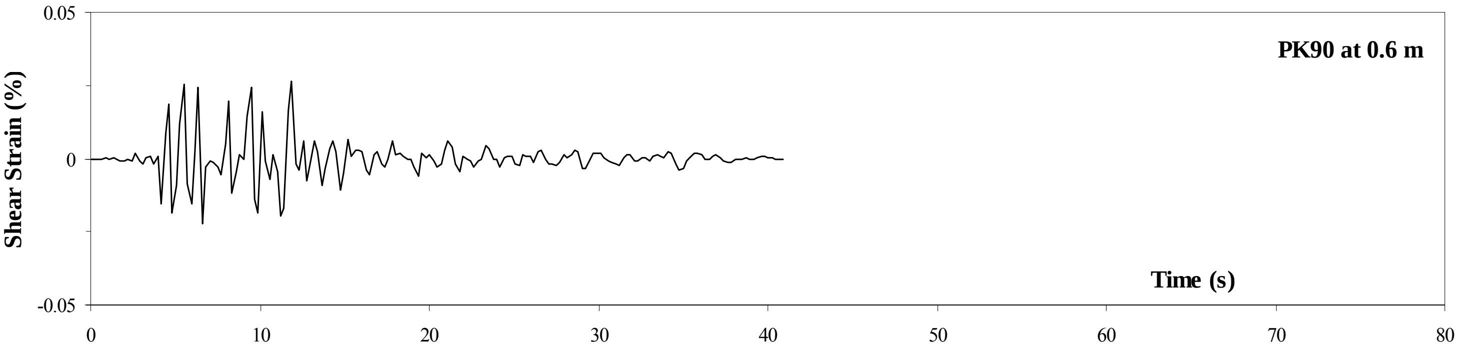


Figure 51. Output Shear Strain time history in the soil deposit for the PK90 ground motion

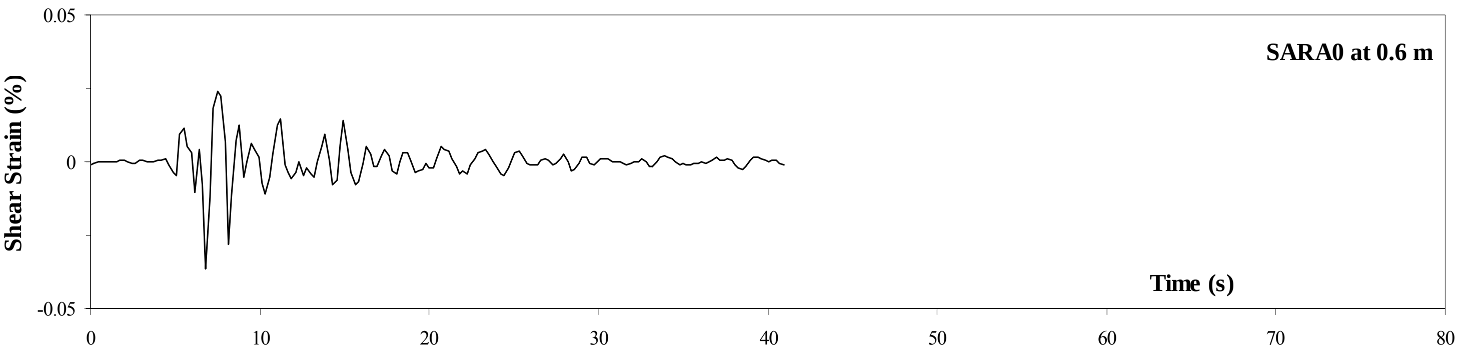


Figure 52. Output Shear Strain time history in the soil deposit for the SARA0 ground motion

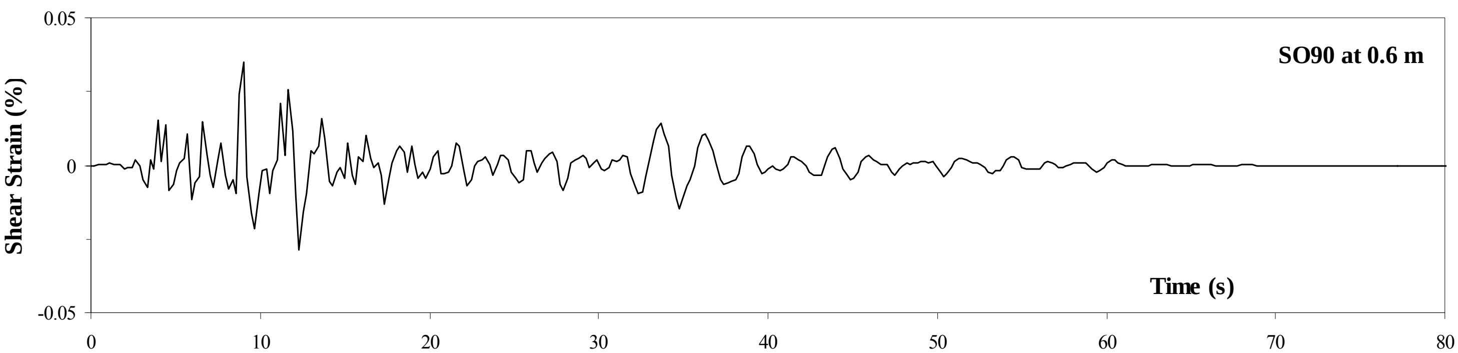


Figure 53. Output Shear Strain time history in the soil deposit for the SO90 ground motion

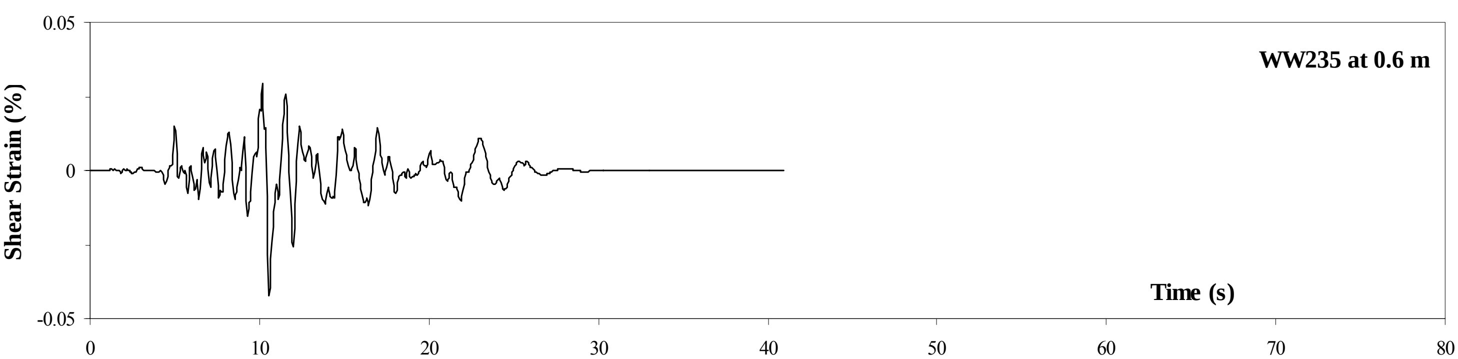


Figure 54. Output Shear Strain time history in the soil deposit for the WW235 ground motion

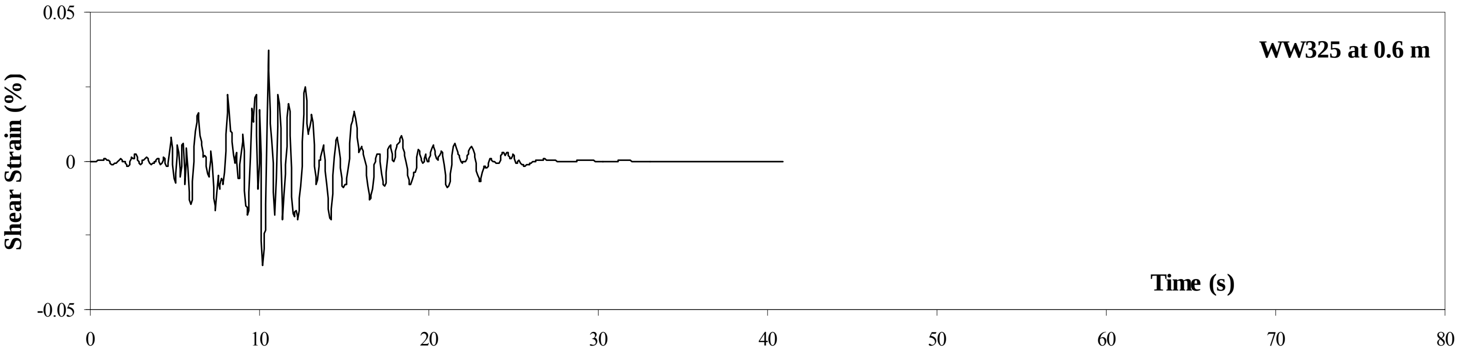


Figure 55. Output Shear Strain time history in the soil deposit for the WW325 ground motion

Figures 56 to 65 plot the Shear Stress time histories at 0.6 m depth in the soil deposit for each input ground motion in the firm ground. The output ground motions didn't show any base-line problems.

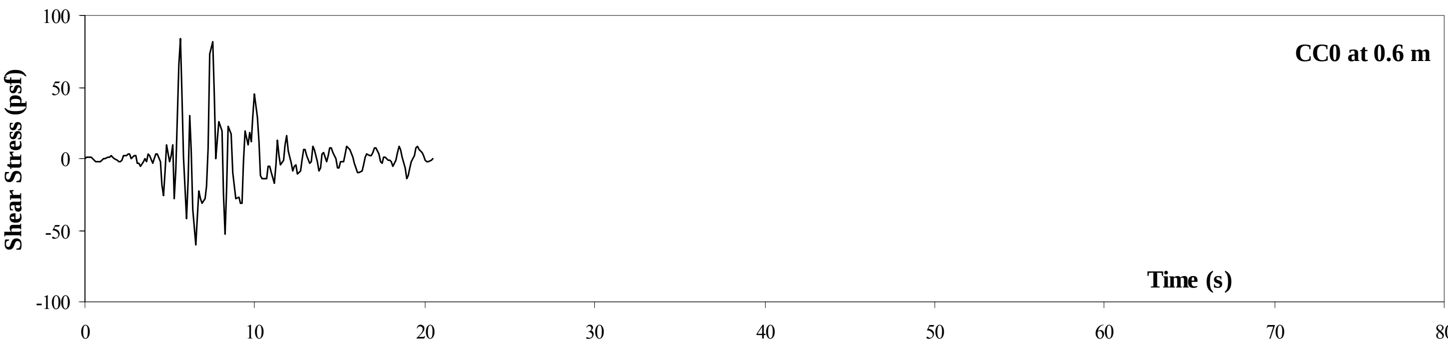


Figure 56. Output Shear Stress time history in the soil deposit for the CCO ground motion

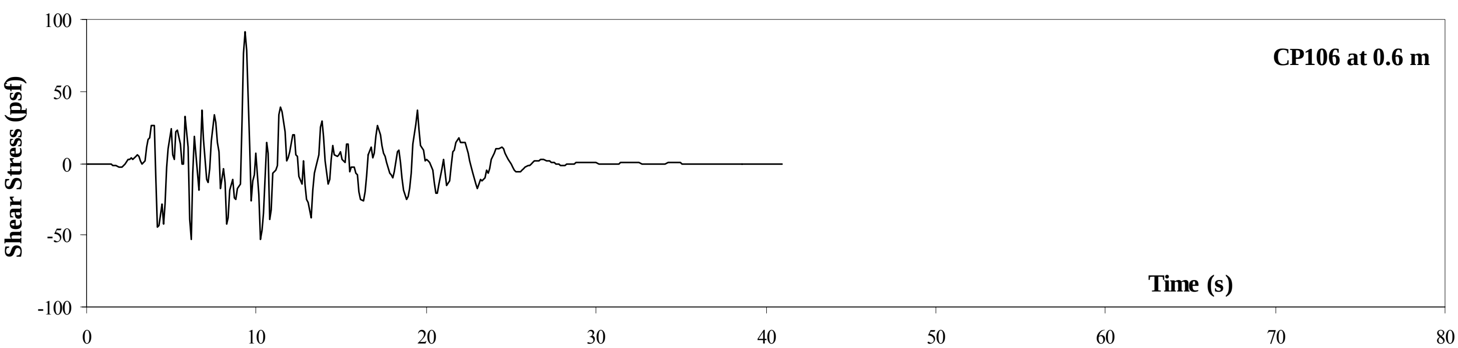


Figure 57. Output Shear Stress time history in the soil deposit for the CP106 ground motion

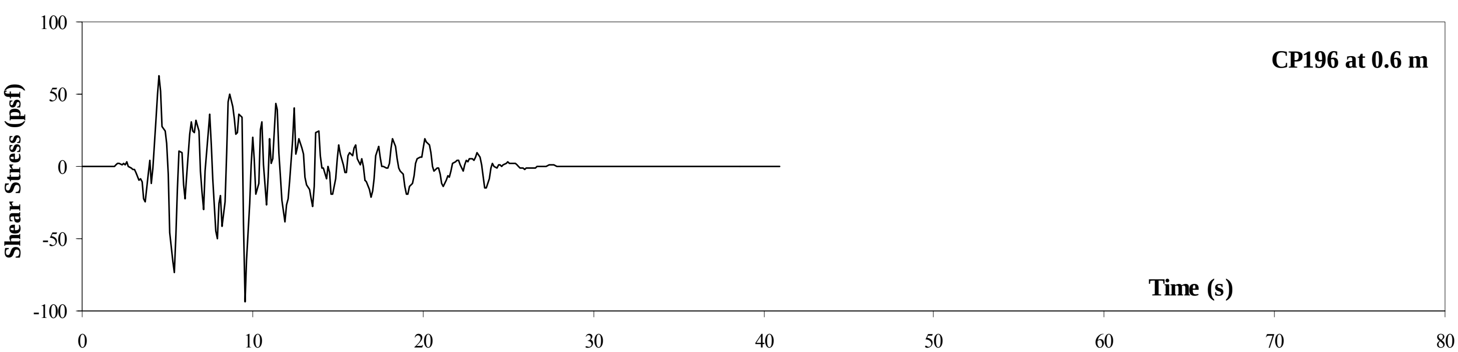


Figure 58. Output Shear Stress time history in the soil deposit for the CP196 ground motion

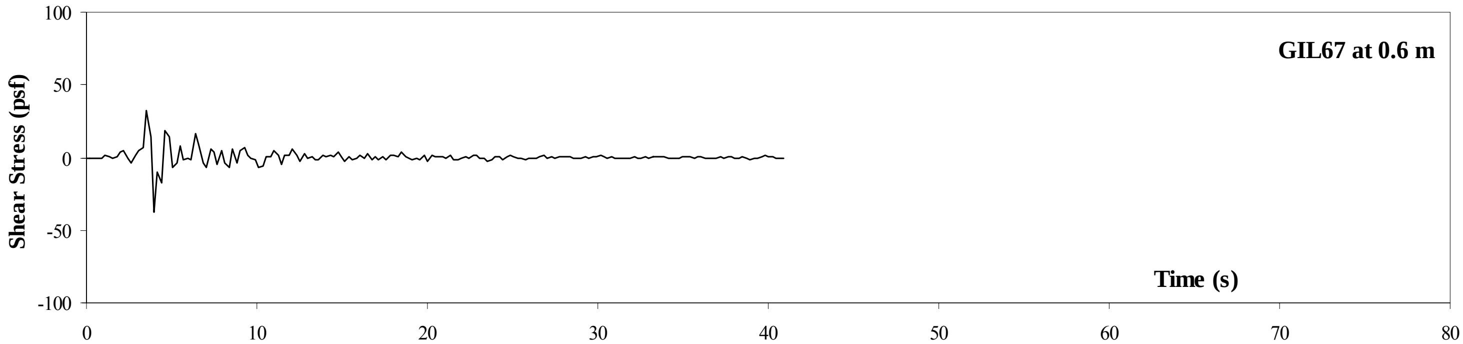


Figure 59. Output Shear Stress time history in the soil deposit for the GIL67 ground motion

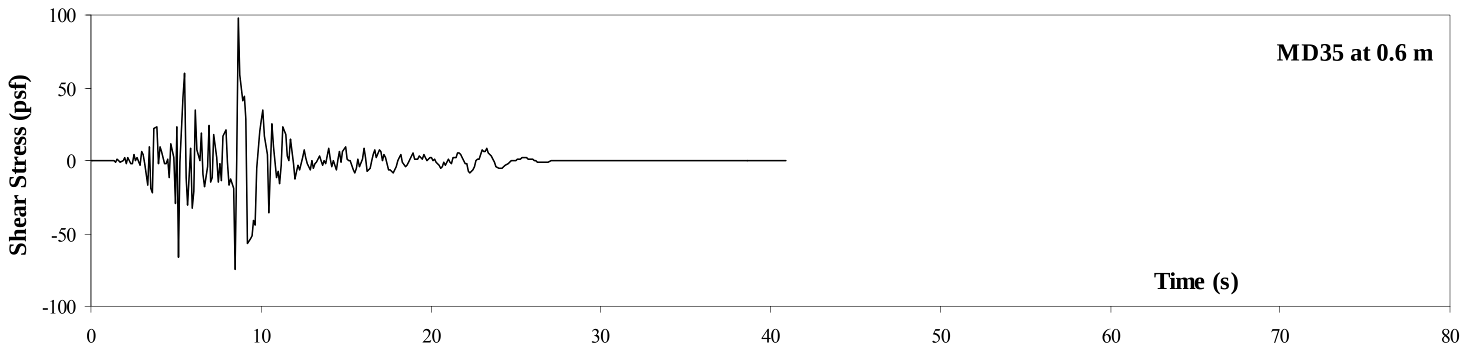


Figure 60. Output Shear Stress time history in the soil deposit for the MD35 ground motion

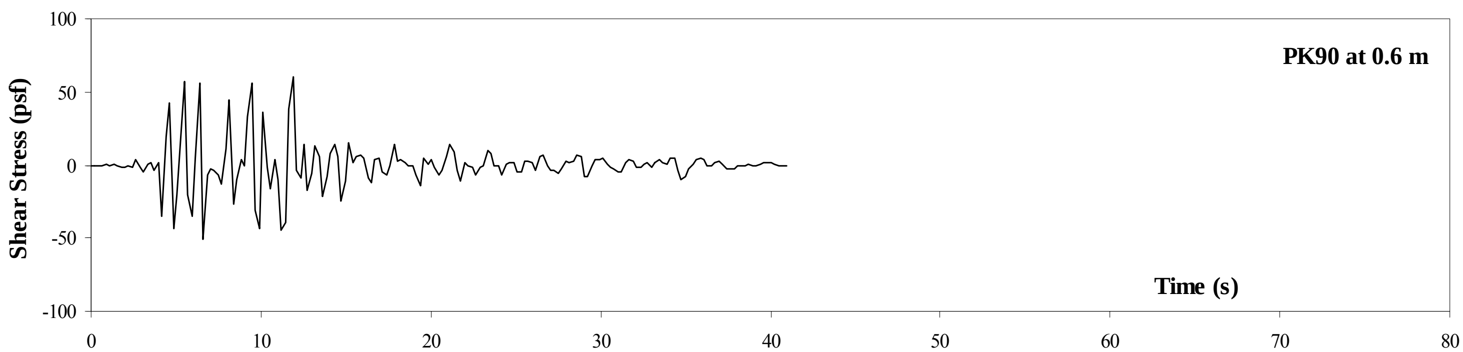


Figure 61. Output Shear Stress time history in the soil deposit for the PK90 ground motion

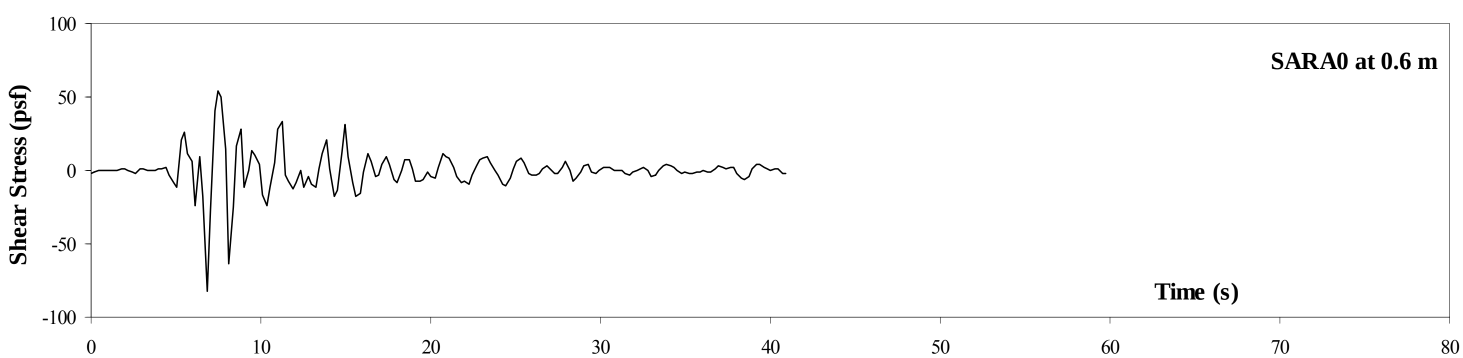


Figure 62. Output Shear Stress time history in the soil deposit for the SARA0 ground motion

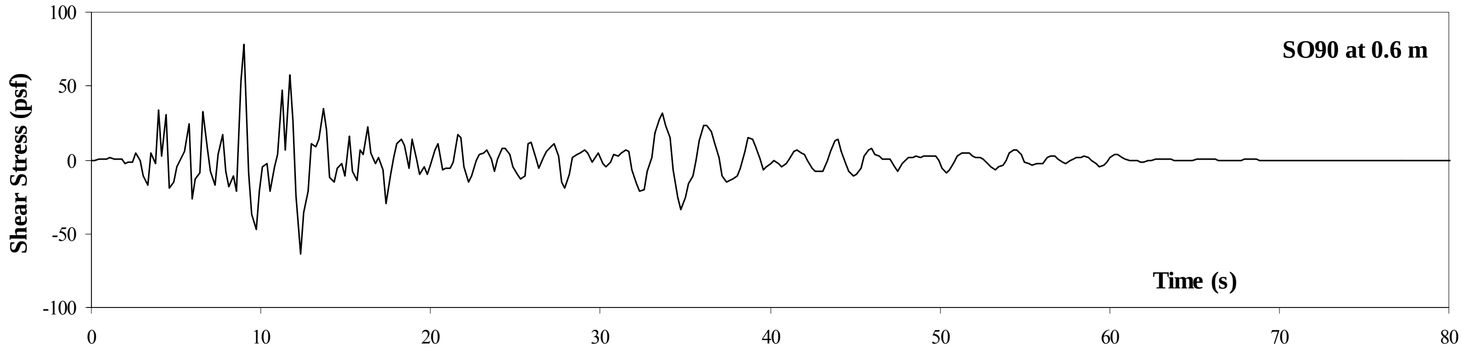


Figure 63. Output Shear Stress time history in the soil deposit for the SO90 ground motion

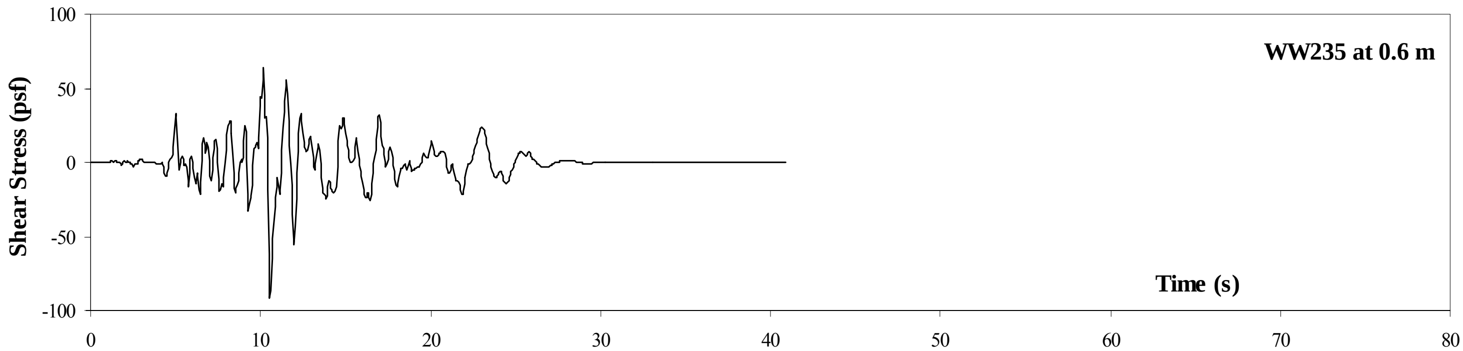


Figure 64. Output Shear Stress time history in the soil deposit for the WW235 ground motion

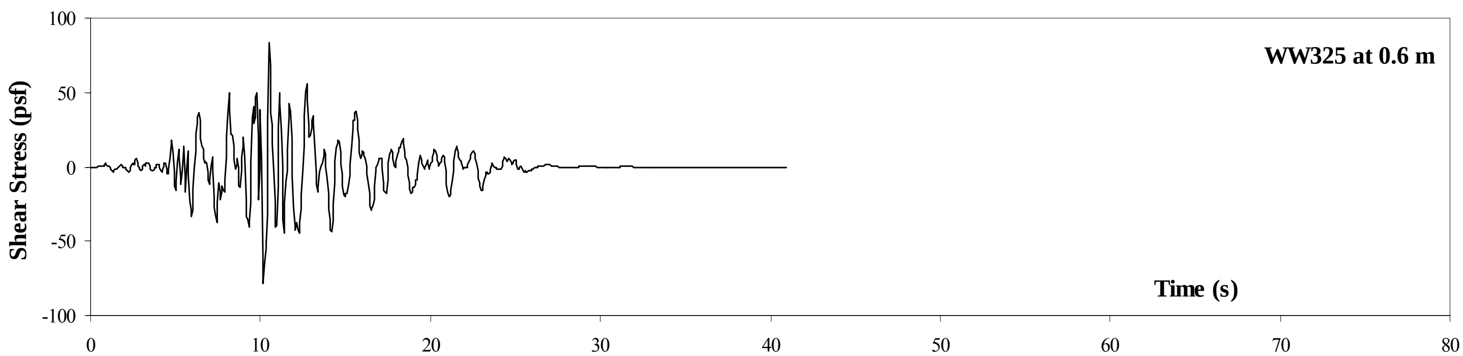


Figure 65. Output Shear Stress time history in the soil deposit for the WW325 ground motion

Figures 66 to 68 plot the Pseudo Acceleration Elastic Response Spectra ($\xi = 5\%$) at 0.6 m depth in the soil deposit.

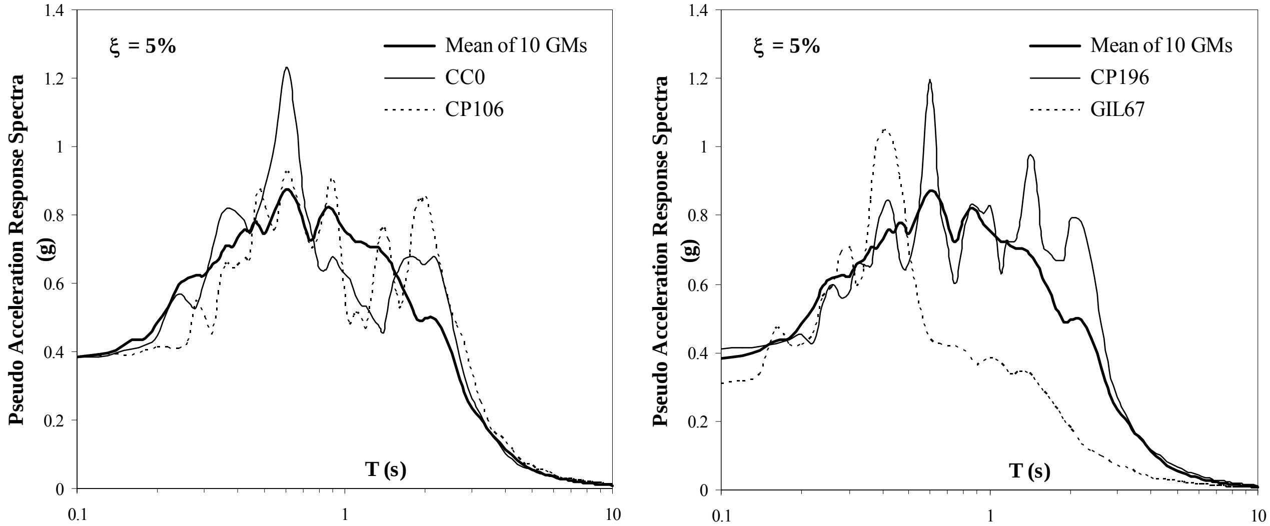


Figure 66. Pseudo acceleration response spectra in the soil deposit for the CC0, CP106, CP196 and GIL67 ground motion

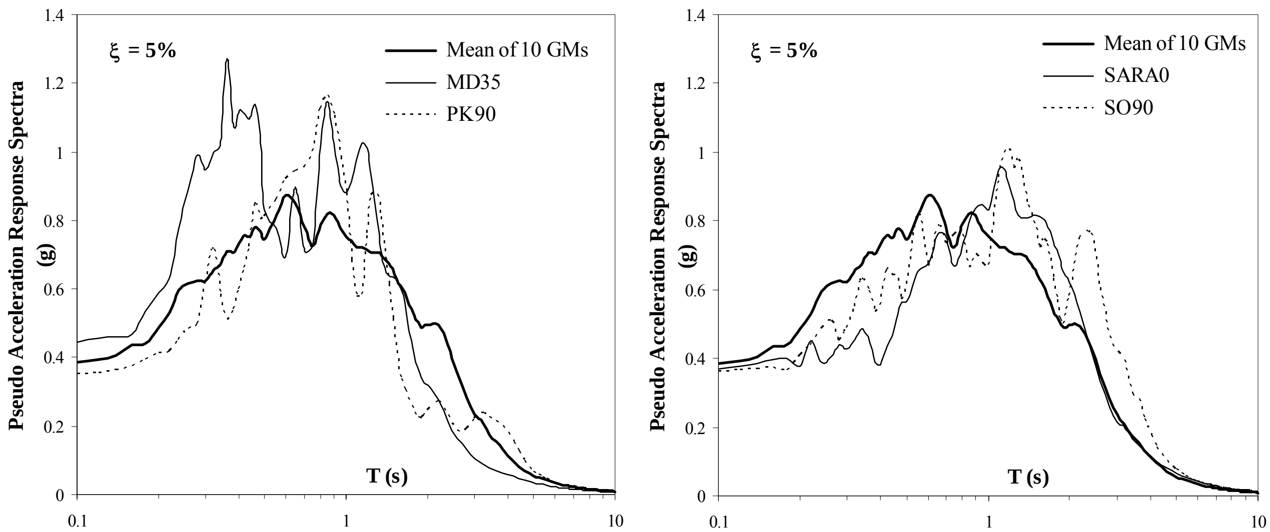


Figure 67. Pseudo acceleration response spectra in the soil deposit for the MD35, PK90, SARA0 and SO90 ground motion

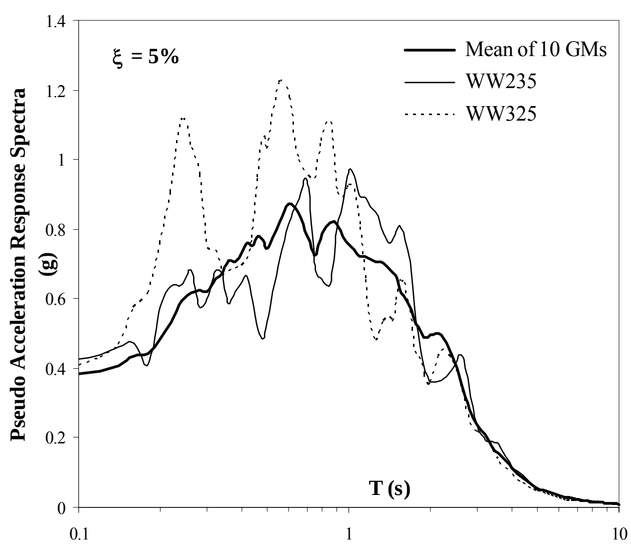


Figure 68. Pseudo acceleration response spectra in the soil deposit for the WW235 and WW325 ground motion

Figures 69 to 71 plot the Pseudo Velocity Elastic Response Spectra (ξ = 5%) at 0.6 m depth in the soil deposit.

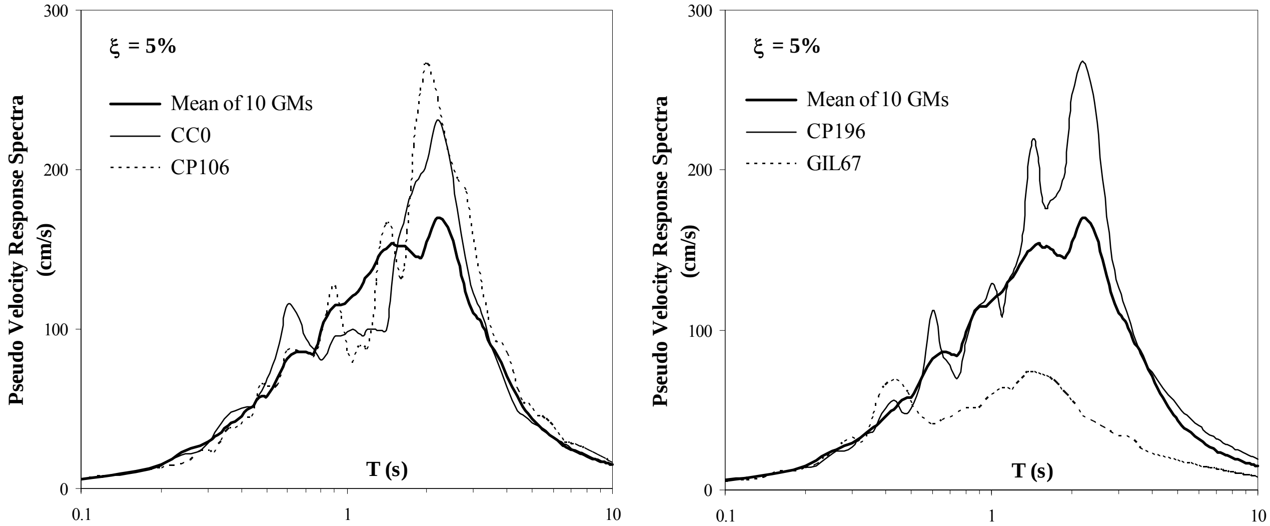


Figure 69. Pseudo velocity response spectra in the soil deposit for the CC0, CP106, CP196 and GIL67 ground motion

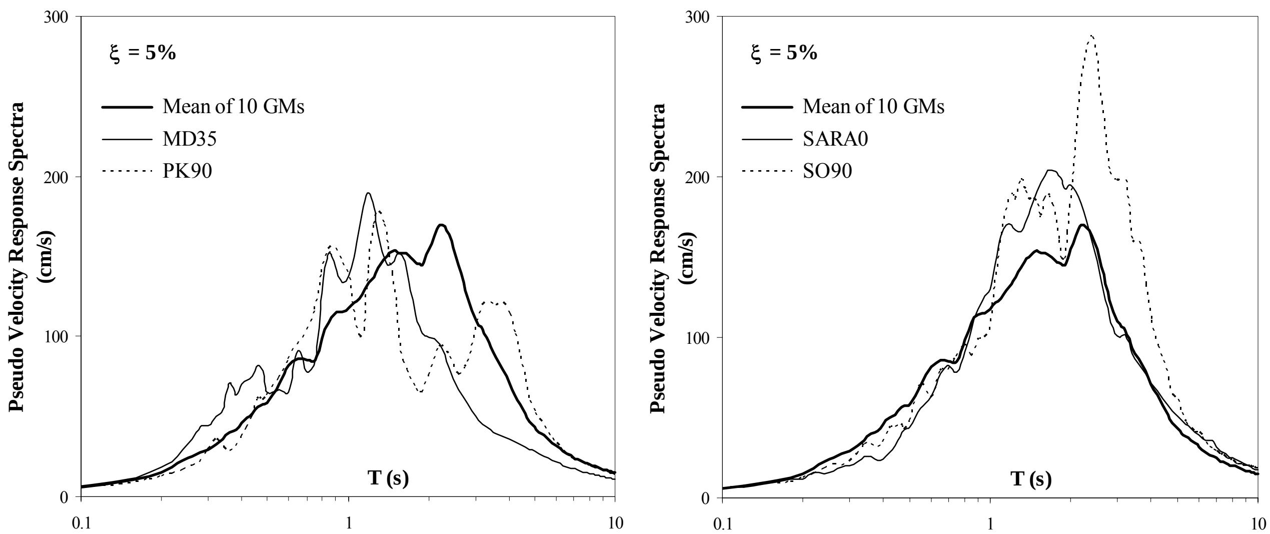


Figure 70. Pseudo velocity response spectra in the soil deposit for the MD35, PK90, SARA0 and SO90 ground motion

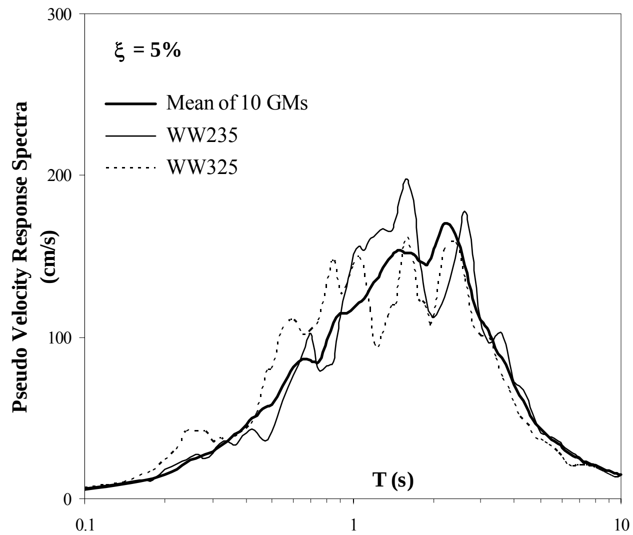


Figure 71. Pseudo velocity response spectra in the soil deposit for the WW235 and WW325 ground motion

Figures 72 to 74 plot the Pseudo Displacement Elastic Response Spectra (ξ = 5%) at 0.6 m depth in the soil deposit.

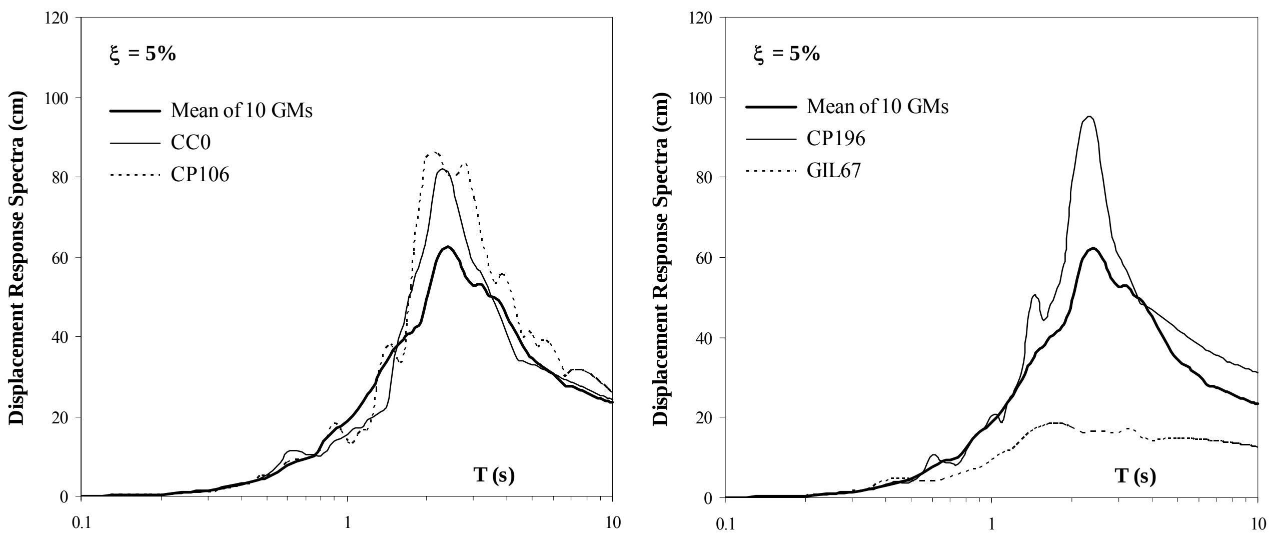


Figure 72. Displacement response spectra in the soil deposit for the CC0, CP106, CP196 and GIL67 ground motion

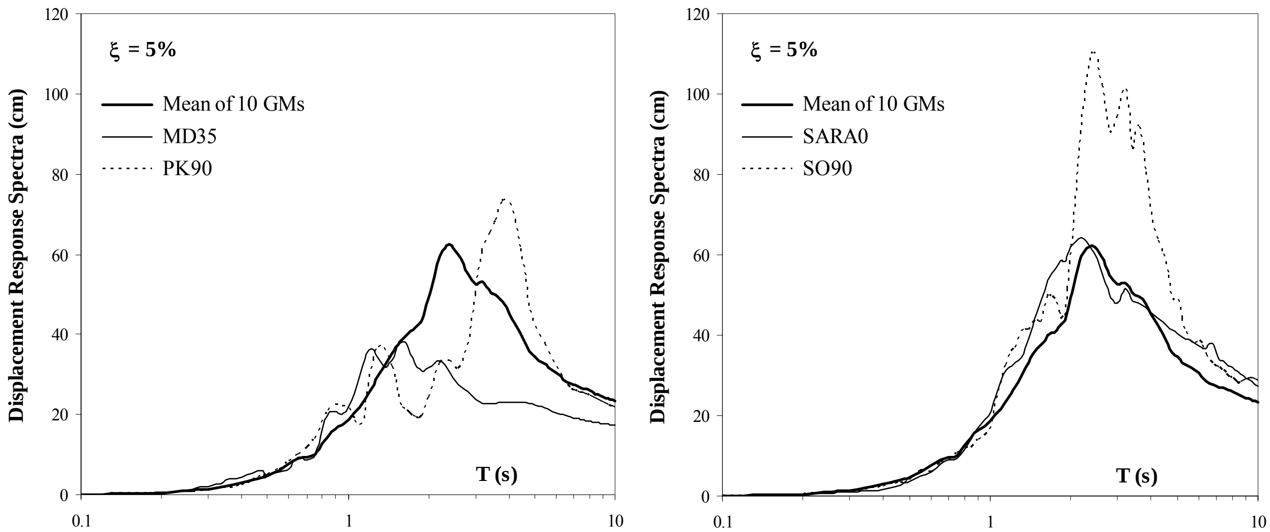


Figure 73. Displacement response spectra in the soil deposit for the MD35, PK90, SARA0 and SO90 ground motion

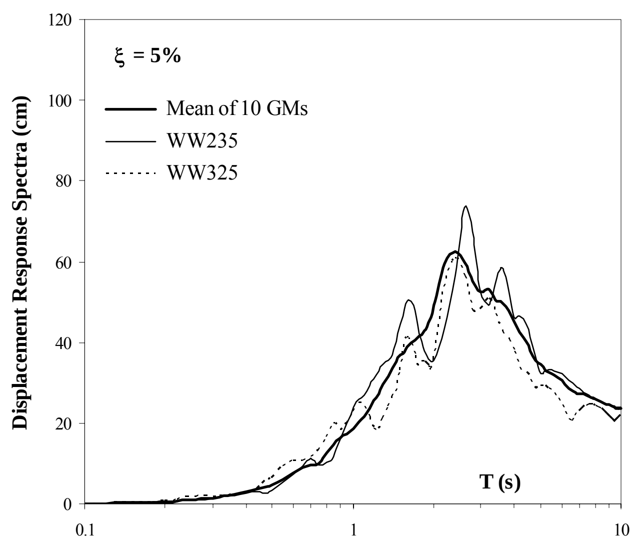


Figure 74. Displacement response spectra in the soil deposit for the WW235 and WW325 ground motion

Figure 75 plots Peak Acceleration profile in the soil deposit. These plots show that there is a slightly amplification of the peak acceleration between the bedrock to the free surface.

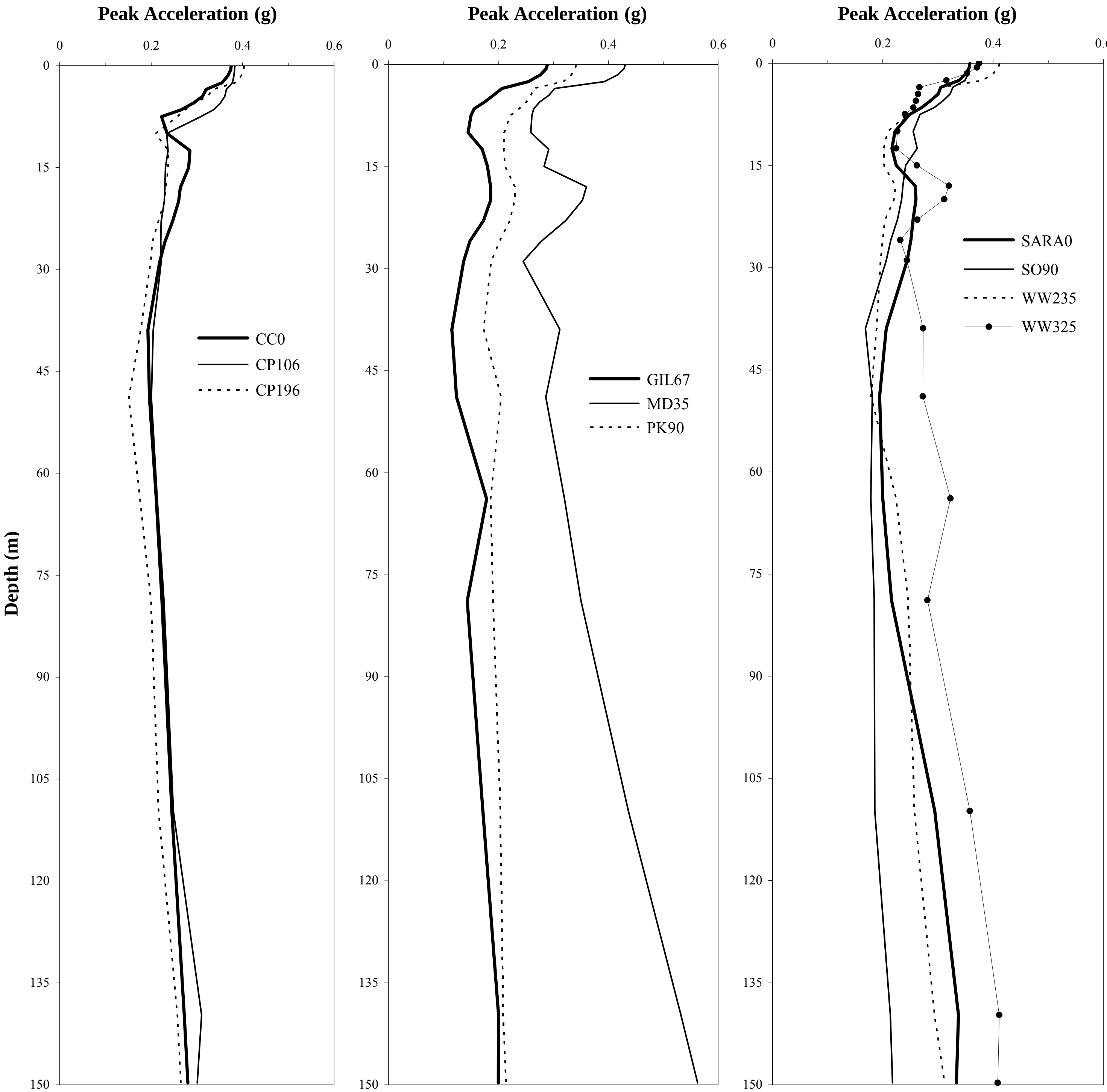


Figure 75. Peak Acceleration profile

Figure 76 plots Peak Velocity profile in the soil deposit. These plots show that the amplification of the peak velocity between the bedrock to the free surface is 100%, approximately.

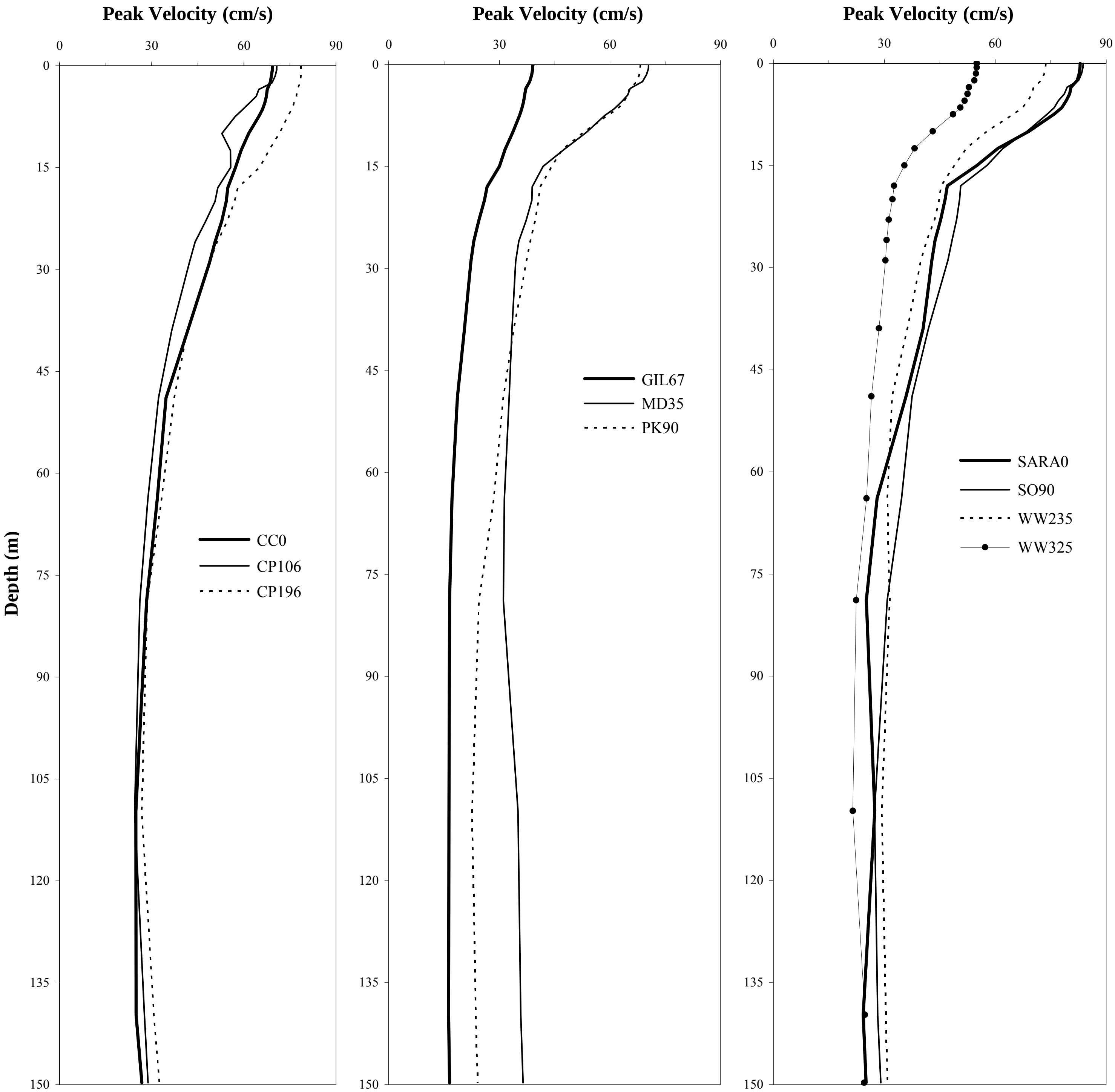


Figure 76. Peak Velocity profile

Figure 77 plots Peak Displacement profile in the soil deposit. These plots show a slightly amplification of the peak displacement between the bedrock to the free surface. Some profiles show no variation of the displacement along the depth due to base line problems.

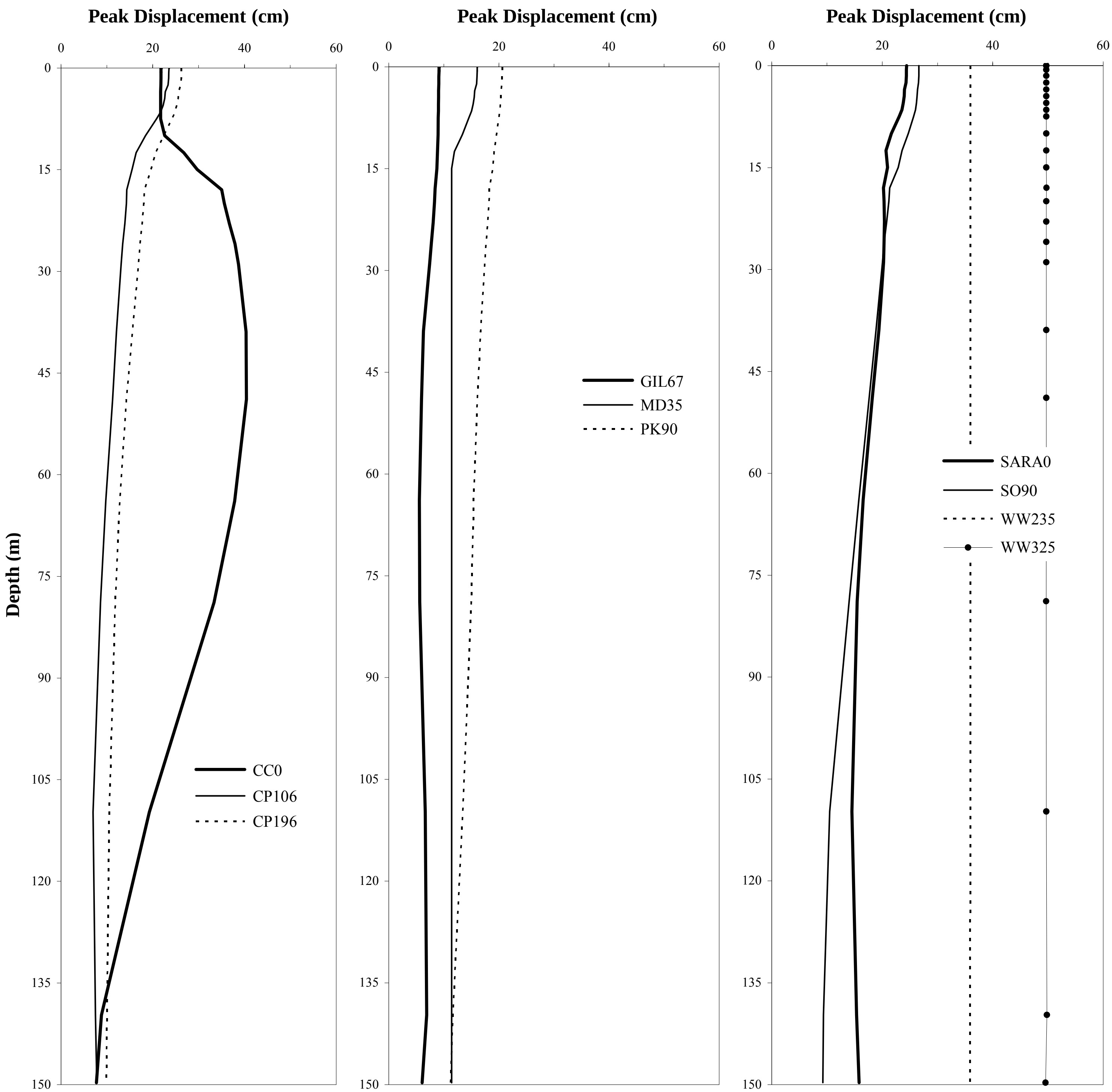


Figure 77. Peak Displacement profile

Figure 78 plots Peak Shear Strain profile in the soil deposit. These plots show that most of the deformation in the soil profile occurred between 5 and 17 mt.

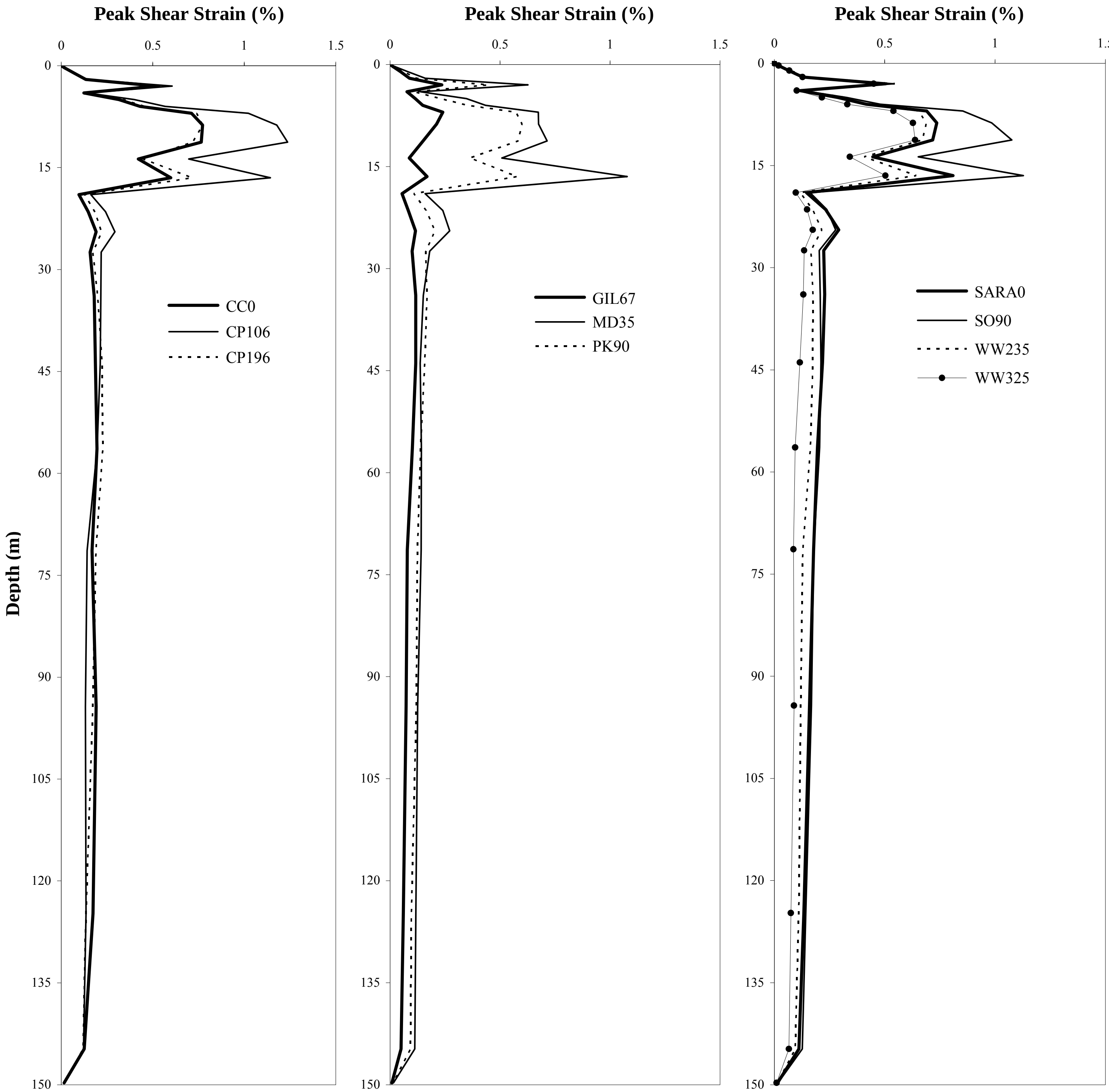


Figure 78. Peak Shear Strain profile

Figure 79 plots Peak Shear Stress profile in the soil deposit. These plots show that the shear stress increases smoothly with depth.

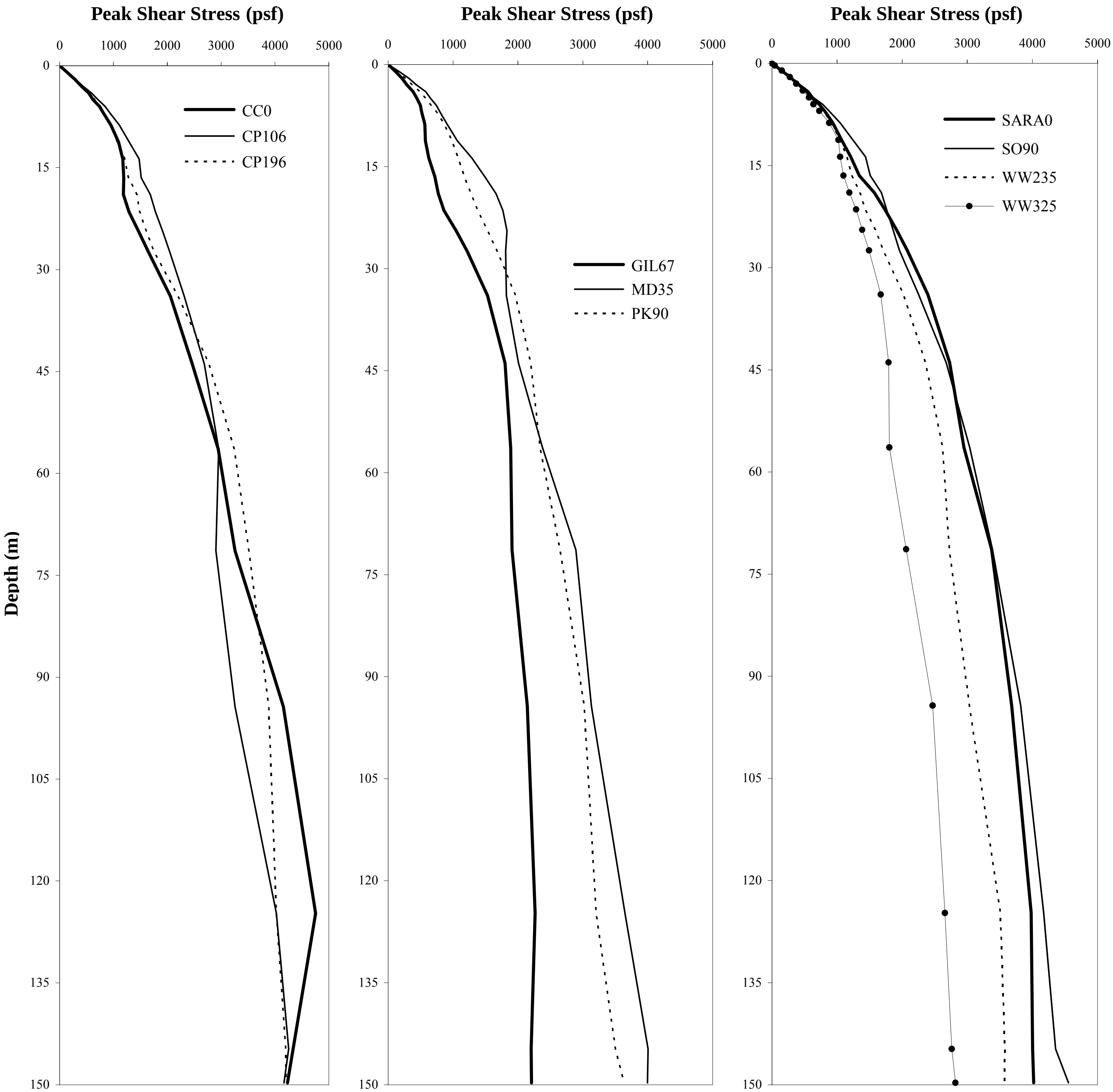


Figure 79. Peak Shear Stress profile

B. SENSITIVITY ANALISYS OF GROUND RESPONSE

Figure 80 plot the mean value of the Pseudo Velocity Response spectra at 0.6 m depth in the soil deposit considering 5 scenarios. Figure 81 shows that average amplification of the Pseudo Velocity at 1.5 s may vary from 1.6 to 2.3 (T = 1.5 s), approximately.

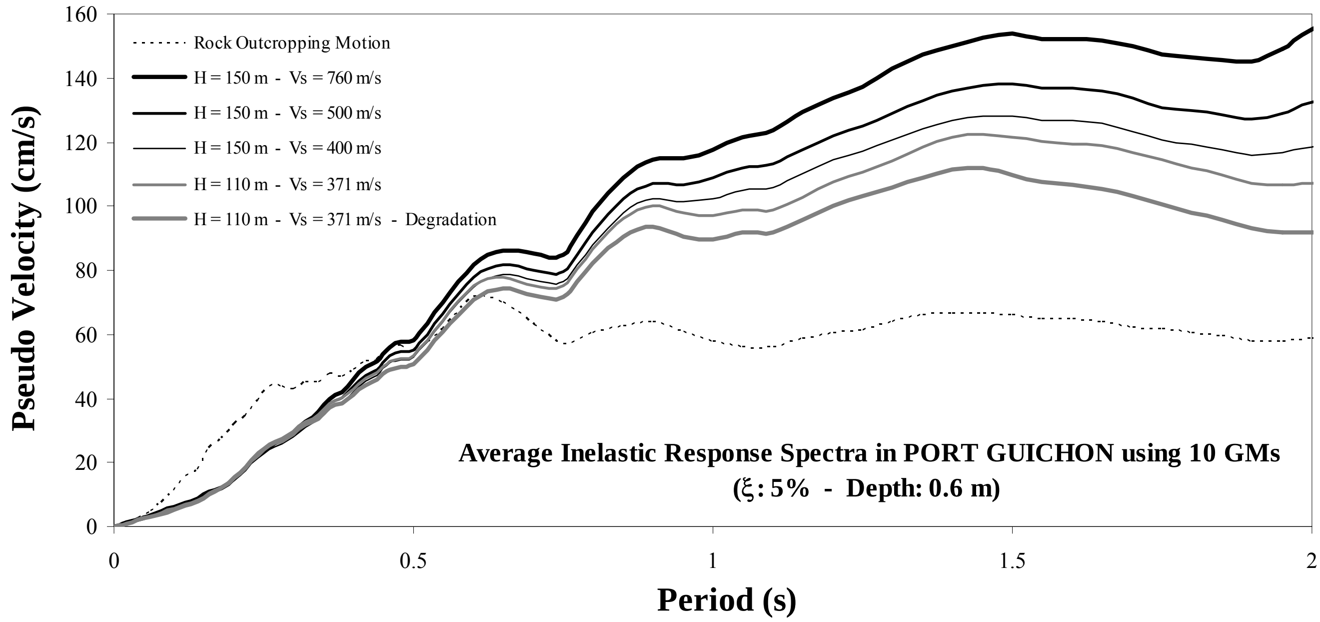


Figure 80. Average Inelastic Pseudo Velocity Response Spectra

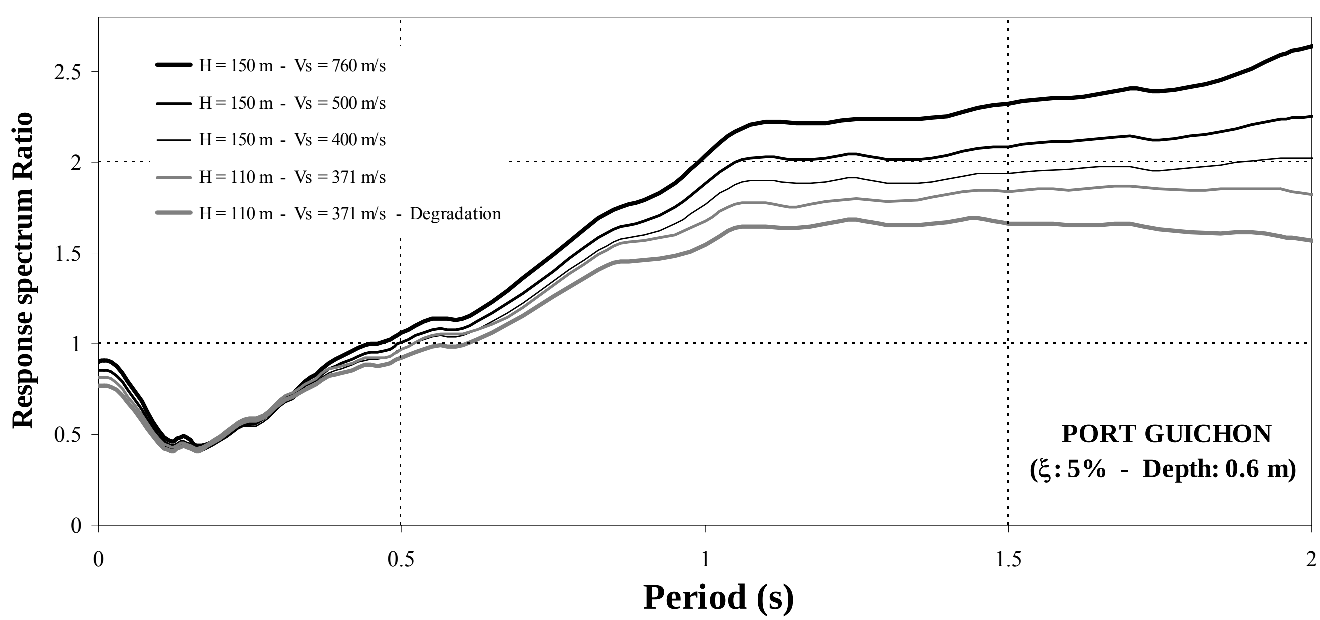


Figure 81. Average Inelastic Pseudo Velocity Response Spectrum Ratio

Table 1 indicates that considering Vs = 760 m/s and H = 150 m produces the biggest amplification in the ground response of the soil deposit.

Table 1. Average spectral pseudo velocity between 0.5 and 1.5 seconds

	OUTCROP	SOIL PROFILE					
		Depth H (m)	**150**	**150**	**150**	**110**	**110**
		Vs of Site C (m/s)	**760**	**500**	**400**	**371**	**371**
		Degradation of G	**NO**	**NO**	**NO**	**NO**	**Vucetic and Dobry**
Sv* (cm/s)	62		115	106	100	96	89
Ratio	1		1.9	1.7	1.6	1.5	1.4

Sv* : average spectra velocity between 0.5 and 1.5 seconds

APPENDIX J

DETAILED RESULTS FOR PORT GUICHON ELEMENTARY SCHOOL

CONTENTS

(1) Trow Associates Ltd. Report (SHAKE Analysis)

(a) Ground Response Analysis using 1-D Linear Model

Trow

MEMORANDUM

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From: **(Uthaya) M. Uthayakumar**

Total Pages (incl. this page): 16

SUBJECT: ***Port Guichon Elementary School Seismic Upgrade***

Please see the attached final version of Memorandum #1, titled Ground Response Analysis using 1-D Linear Model, for the above noted project. Also, please discard our earlier memo dated January 12, 2007 on this subject.

MEMORANDUM #1

GROUND RESPONSE ANALYSIS USING "1-D LINEAR MODEL"

PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE

Introduction

As required by the "Bridging Guidelines" for the Performance-based Seismic Retrofit B.C. Schools, a ground response analysis was carried out. This memorandum presents the results of the Phase 1 analysis. This analysis was carried out using a 1-D linear analysis method, utilizing the computer program SHAKE91 (Idriss and Sun, 1992). For Phase 2, 2-D non-linear analysis with liquefaction triggering, using the computer program FLAC (Itasca, 2005) will be used. The constitutive model UBCSAND or UBCTOT, developed at the University of British Columbia will be used in the FLAC analysis. Results of the Phase 2 analysis will be presented in a separate memorandum.

As described in the Bridging Guidelines, the objective of the ground response analysis is to develop a set of site-specific near-surface time history, which will be representative of ground motions at the base of the foundations. It is understood these near surface ground motions will be used by UBC and the structural engineers to analyze the response of the structure.

Our Geotechnical report dated February 22, 2006 provide the results of the Geotechnical drilling investigation, seismic assessment, including a previous set of ground response analyses, liquefaction assessment and site classification. Our May 17, 2006 report presents design recommendations for seismic upgrading of the school buildings and the foundation soils. Both of the above noted reports are attached to this memorandum. Please note that the soil model used for the earlier ground response analysis is slightly different from that used for the current analysis as described in the next section.

Brief discussion on soil profile, input acceleration records, method of analysis, and the results are given in the following sections.

Soil Profile and Properties

Description of the soil conditions from the drilling investigation is given in our February 22, 2006 report. The required soil properties for the analysis include shear wave velocity (V_s), unit weight, and relationships between shear modulus and damping with shear strain. Drawing 1 presents the soil profile and model used for the analysis.

The shear wave velocity (V_s) profile was developed using the following procedure:

- profile for the top 30 m - from the measured shear wave velocity at the Seismic Cone Penetration Test (SCPT) hole at the site;
- between 30 m and 150 m - V_s is proportional to $(\text{depth})^{0.33}$.
- The proposed mean shear wave velocity curve for Fraser Delta by Hunter et al (1998) was compared to the above noted profile and shown in drawing 2. Good agreement between the shear wave velocity profile used and that of Hunter et al (1998) can be noted.

The "Class C Firm Ground, glacial till-like soils" was assumed to be at 150 m depth. Top of this layer was assumed as elastic half space for the analysis. Shear wave velocity of the "firm ground" was assumed as 760 m/s in accordance with the recommendations given in the National Building Code, 2005.

Please note that some uncertainty exists as to the depth of the "till-like" soils or to the bedrock in the vicinity of this site. A computer generated contour map by Hunter et al (1999) show the "till-like" soils at this site could be at 150 m depth. This was based on interpolation of drill hole data or shear wave velocity data obtained at locations several kilometers from this site. Studies by Hunter et al (1999), Clague et al, 1998, Luternauer and Hunter, 1996, Hunter et al, 1996 indicate Tertiary bedrock in the vicinity of this site could be at 700 m to 800 m depth.

To investigate the effect of the depth to the elastic half space ("till-like" soils) we have carried out analyses with 150 m and 400 m deep soil columns. The results of this sensitivity analyses are described later in this report.

Shear modulus reduction and damping curves used in the analysis are from published data on similar soils (Seed and Idriss, 1970, and Vucetic and Dobry, 1991). For sand layers the upper bound modulus reduction and lower bound damping curves from Seed et al (1986) were used. For silt and clay layers data from Vucetic and Dobry (1991) for plasticity index (PI) of 30 was used. The Task Force report for Sesimic Design guidelines, Fraser River delta (in-draft) provides same recommendations for the selection of shear modulus and damping curves. Also, similar assumptions were made with respect to the modulus reduction and damping of soils for a recent bridge project across the Fraser River. For the earlier ground response analysis, shown in our February 22, 2006 report, we have used the same modulus and damping curves for the sands. However, for the silt and clay layers Vucetic and Dobry (1991) curves corresponding to PI of 15 (for silt crust) and 50 (for deeper silts and clays) were used.

Design Earthquake Motion and Response Spectra on "Firm Ground"

The Commentary to the Bridging Guidelines for the Performance – based Seismic Retrofit of British Columbia School Buildings, Second Edition (attached to this memorandum), provides the details of the ground motions to be used as input motions for the ground response analysis. Table C.4-1 of the above noted Commentary presents the details of the records. The input motions were scaled using the factors given in Table C.4-2 for Zone 4 multiplied by 1.0848 as recommended by UBC (see attached email from Graham Taylor, dated January 05, 2007). The new scaling factors are given in Table 1 in this report. For the ground response analysis the scaled input motions were applied at the outcropping Site Class C firm ground with a shear wave velocity of 760 m/s.

Table 1. Details of the original time histories and the scaling factors for "near-surface firm ground, Site Class C", Ladner, B.C. (Ref. Bridging Guidelines, 2nd Edition)

Record Name	Original time history					Time increment (s)	Acceleration in the record	Scaling factor for Ladner
	Station	PGA (cm/s^2)	PGV (cm/s)	PGD (cm)	S_v* (cm/s)			
SO90	Sherman Oaks – 105 deg	210	29.4	8.7	44.3	0.02	cm/s^2	1.36
WW235	Wadsworth - 235 deg	297	32.9	9.8	48.6	0.005	cm/s^2	1.24
WW325	Wadsworth - 325 deg	382	21.3	4.6	42.0	0.005	cm/s^2	1.43
CC0	Canyon Country – 0 deg	389	44.1	11.2	66.7	0.01	g	0.90
Sara0	Saratoga - 0 deg	495	32.6	17.2	69.3	0.02	cm/s^2	0.87
CP196	Canoga Park – 196 deg	381	59.8	12.4	78.0	0.01	g	0.77
CP106	Canoga Park – 106 deg	343	34.1	8.8	52.3	0.01	g	1.15
PK90	Pacoima Kagel – 90 deg	295	30.9	10.6	66.0	0.02	g	0.91
MD35	12520 Mulholland Drive - 35 deg	577	29.4	6.2	49.0	0.01	g	1.23
Gil67	Gilroy Gavilon College - 67 deg	349	22.8	5.7	39.6	0.02	cm/s^2	1.52

Note: Sv* is the average spectral pseudo velocity (5% damping) taken between 0.5-1.5 sec. *(BridgingGuidelines)*

Results of the Analysis

The results of the analysis in the form of acceleration response spectra, obtained for 5% damping are given in drawing 3. The response spectra were obtained for motions at the foundation level, at 0.6 m depth. Response spectra of the input motions, at outcropping firm ground (Site Class C) for 5 % damping are shown in drawing 4. The average of the acceleration response spectra in the above noted two drawings (i.e.: at 0.6 m depth and at the outcropping firm ground) are compared in drawing 5.

Drawing 5 also compares the effect of the depth to the "till-like (or firm ground)" soils. Depths of 150 m and 400 m were considered. The results clearly show the shallower soil column results in slightly higher response. The analyses also show a natural period of 2.03 seconds for the 150 m deep column and 3.92 seconds for the 400 m deep column.

As the response from the 150 m deep soil column is slightly higher than that from the 400 m deep column, and Hunter et al (1999) indicate the depth to the "till-like" soils may be 150 m at this site, further analysis and results given in this report are for the shorter 150 m deep soil column.

Time history at 0.6 m depth, in the form of acceleration is shown in drawings 6 and 7. Velocity history at 0.6 m depth is given in drawings 8 and 9, and the displacement history is provided in drawings 10 and 11.

Discussion

The results show significant portion of the high frequency content in the input motion is absent in the near surface history and believed to be damped out as the motion propagates through deep soil column. The output record of SO90 shows a couple of spikes after 32 seconds. This is believed to be a manifestation of the spikes in the corresponding input motion. If appropriate, this motion can be cut-off at 30 s.

The ratio acceleration response spectra at foundation level to that at the outcropping firm ground (Site Class C) increases from 0.6 at a period of 0.3 s to 1.0 at 0.5 s, and to 2.0 at a period of 1s. This trend is similar to the results previous studies carried out for sites in the Fraser River delta.

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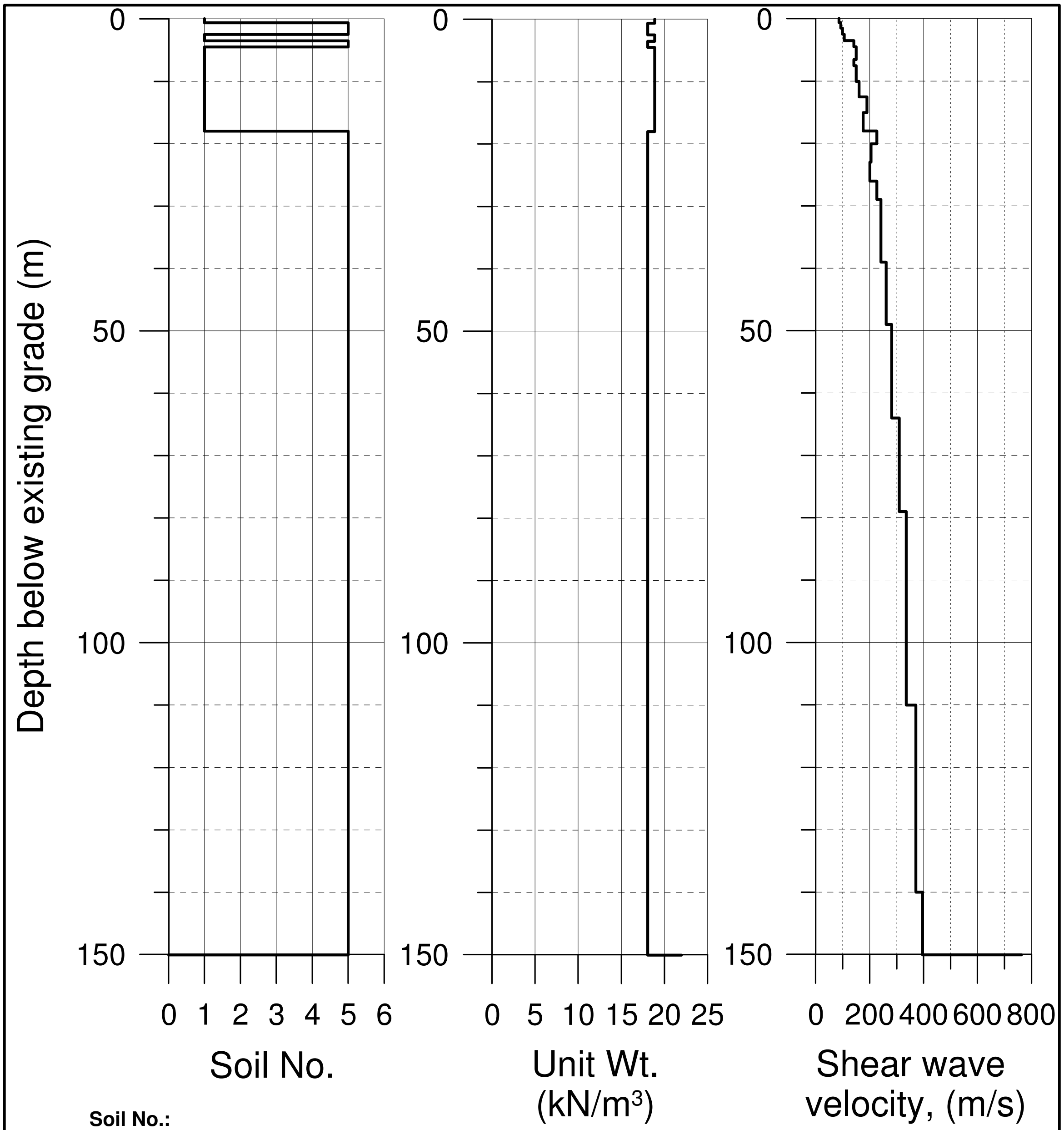

Soil No.:
1 - Sand;
5 - Silt/Clay, Plasticity Index = 30.

TROW ASSOCIATES INC. 7025 Greenwood Street Burnaby, B.C., V5A 1X7 Tel.: 604-874-1245 Fax.: 604-874-2358	CLIENT **SANDWELL ENGINEERING INC.** PROJECT **PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.**	TITLE **SOIL MODEL USED IN THE GROUND RESPONSE ANALYSIS**

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
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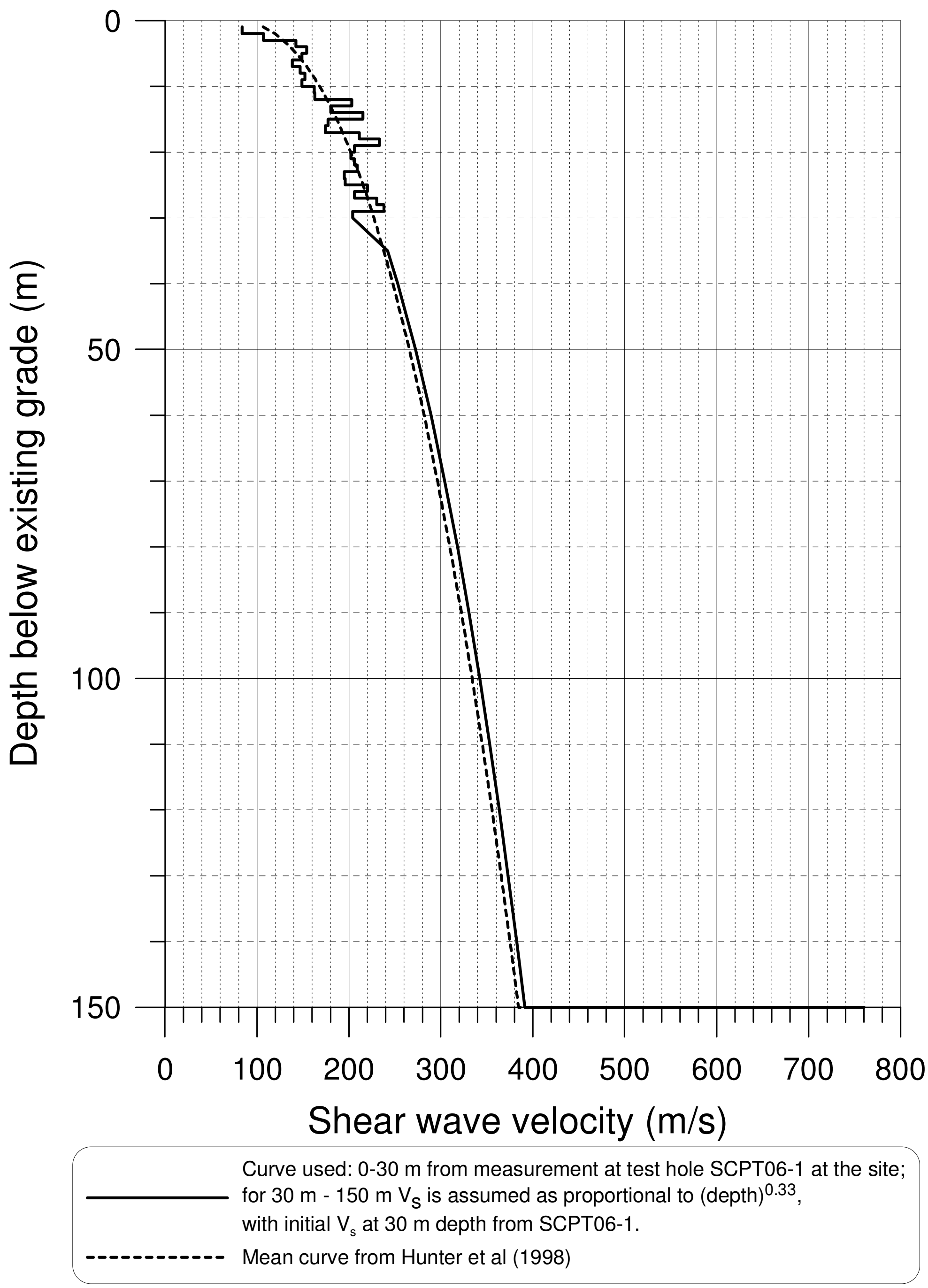

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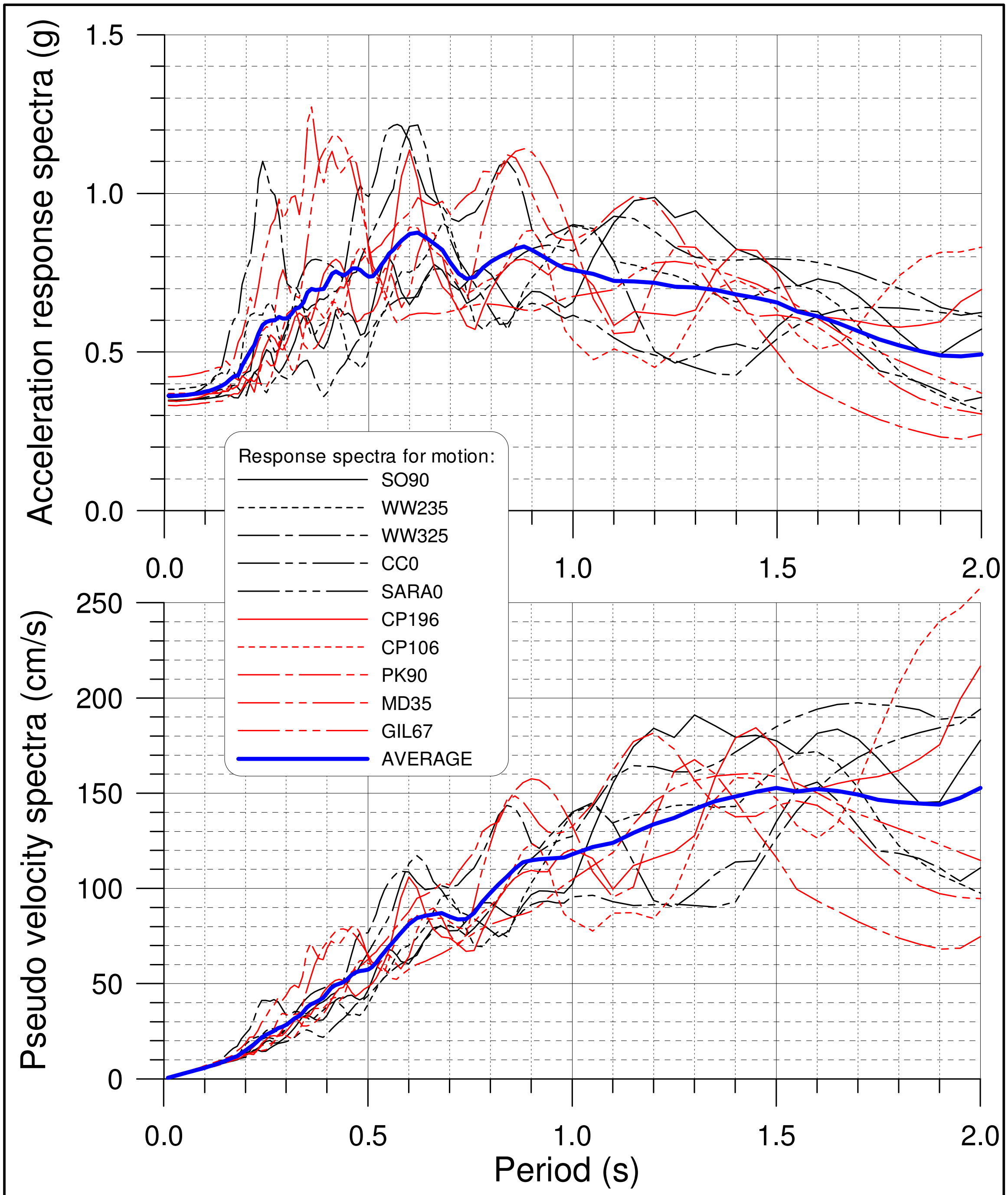
Tel.: 604-874-1245
Fax.: 604-874-2358

CLIENT **SANDWELL ENGINEERING INC.**			TITLE **SHEAR WAVE VELOCITY PROFILE USED IN THE GROUND RESPONSE ANALYSIS**
PROJECT **PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.**			
PROJECT No. **071-03006**	DFTR.	DSGN. **UMU**	CHK.
DATE **JAN., 2007**	SCALE	DWG. No. **2**	



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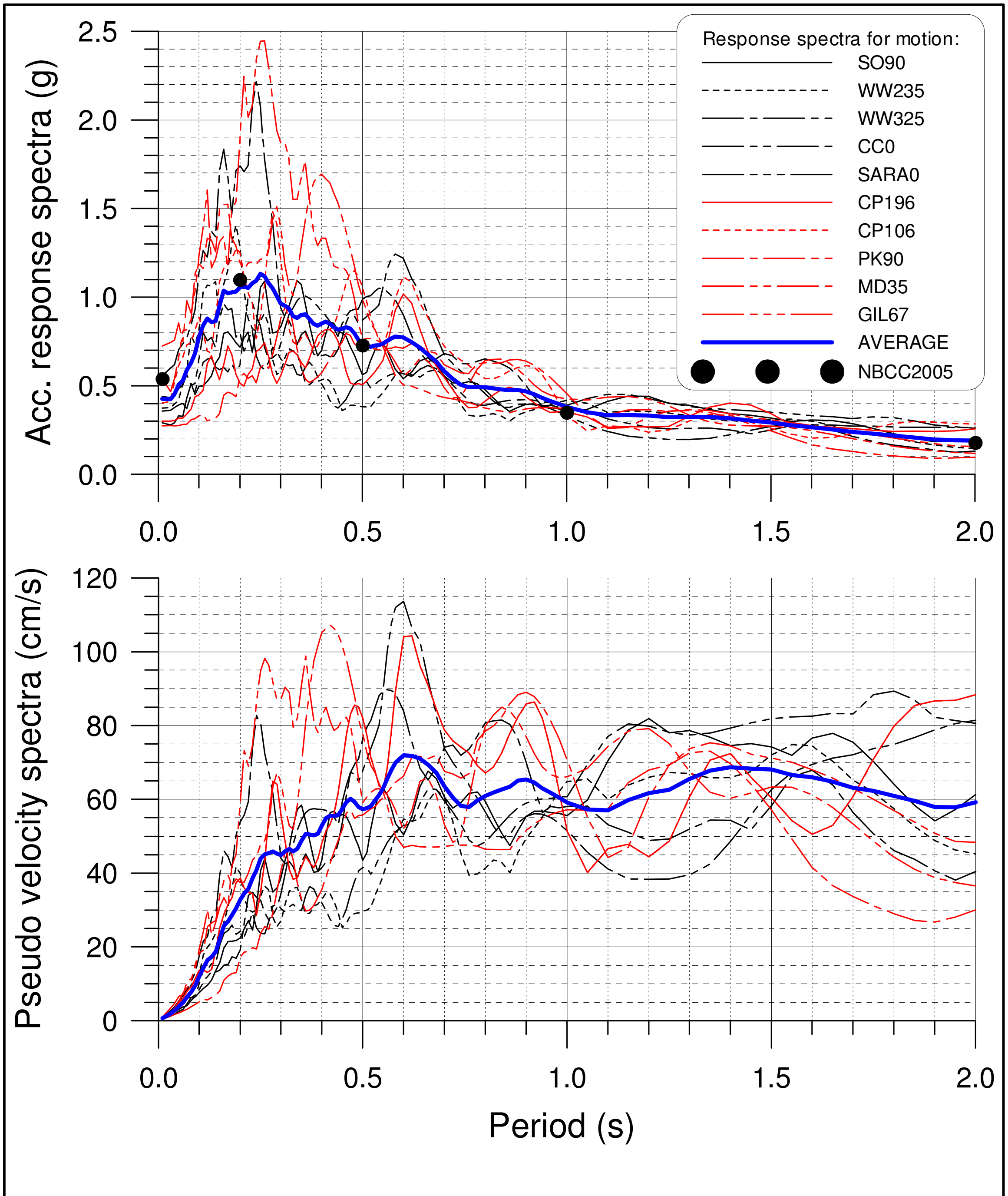
	PROJECT No. **071-03006**	DFTR.	DSGN. **UMU**	CHK.	DATE **JAN., 2007**	SCALE	DWG. No. **3**



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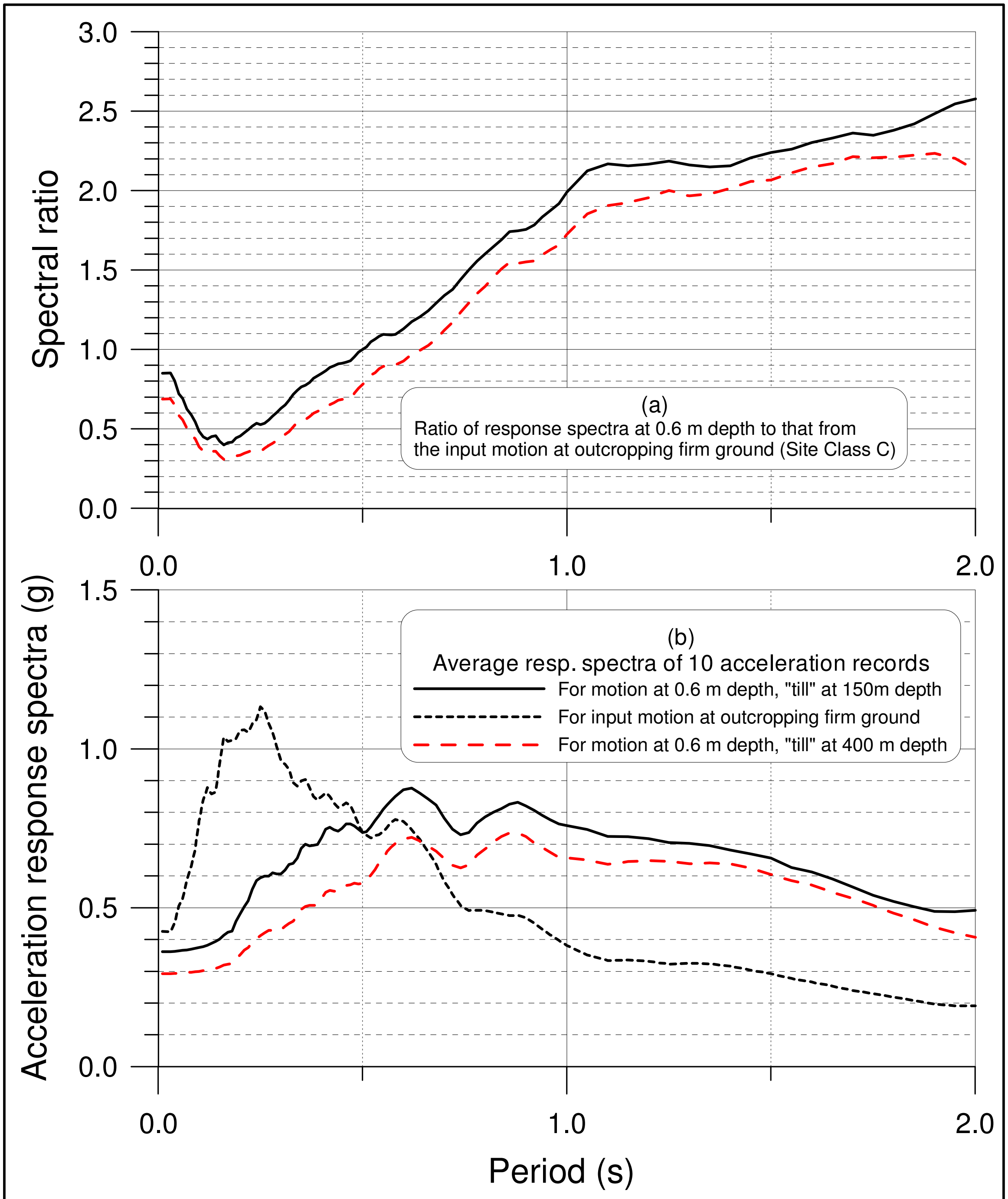
Fax.: 604-874-2358

CLIENT **SANDWELL ENGINEERING INC.**				TITLE **RESPONSE SPECTRA OF INPUT MOTION AT OUTCROPPING FIRM GROUND, FOR 5% DAMPING**		
PROJECT **PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.**						
PROJECT No. **071-03006**	DFTR.	DSGN. **UMU**	CHK.	DATE **JAN., 2007**	SCALE	DWG. No. **4**

Trow



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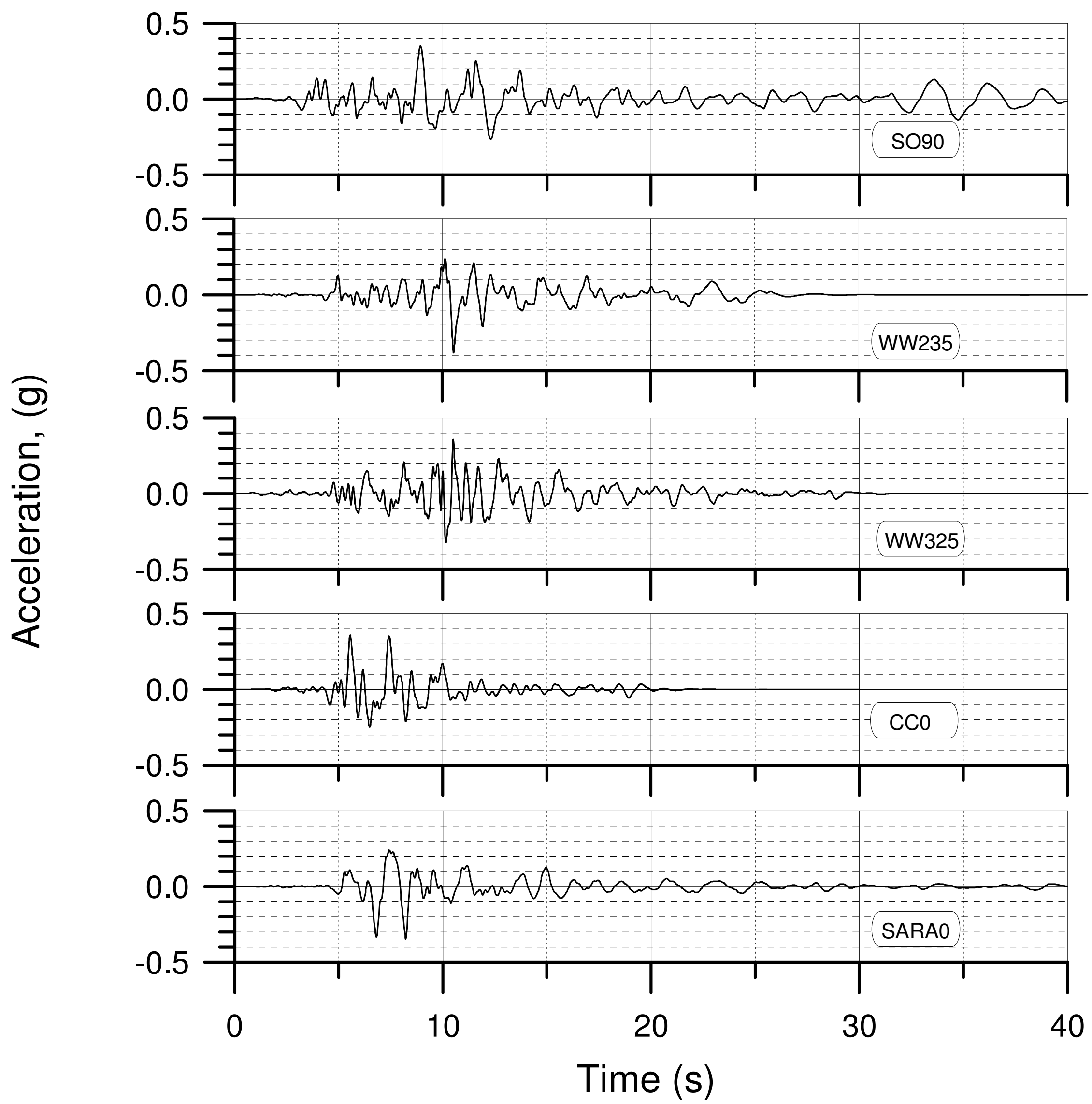
CLIENT
SANDWELL ENGINEERING INC.

PROJECT
PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.

TITLE
AVERAGE OF ACCELERATION RESPONSE SPECTRA OF MOTION AT 0.6 m DEPTH FOR 5% DAMPING

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
071-03006		**UMU**		**JAN., 2007**		**5**



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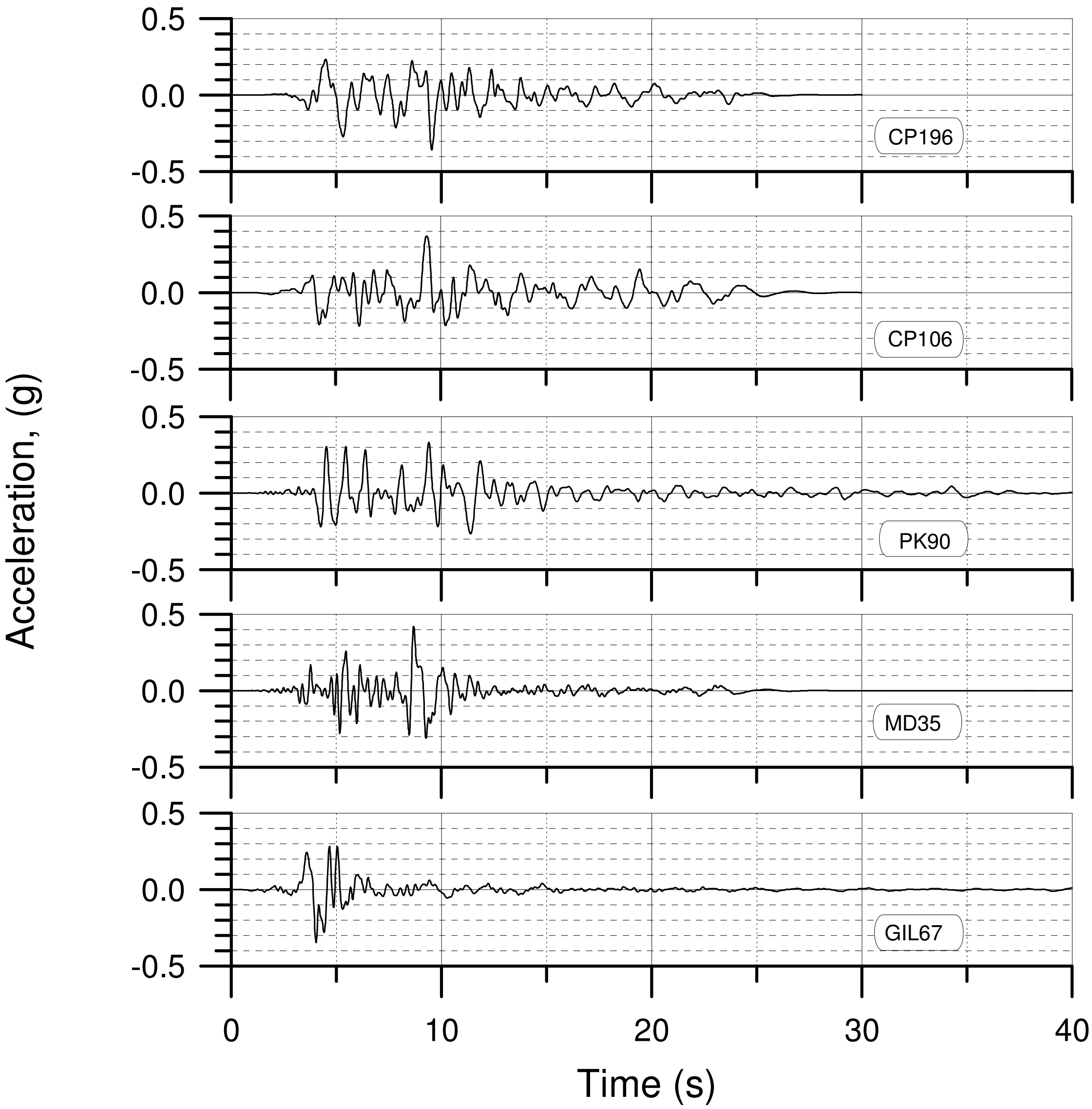
CLIENT
SANDWELL ENGINEERING INC.

PROJECT
PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.

TITLE
ACCELERATION RECORDS AT 0.6 m DEPTH, OBTAINED FROM SHAKE

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
071-03006		**UMU**		**JAN., 2007**		**6**



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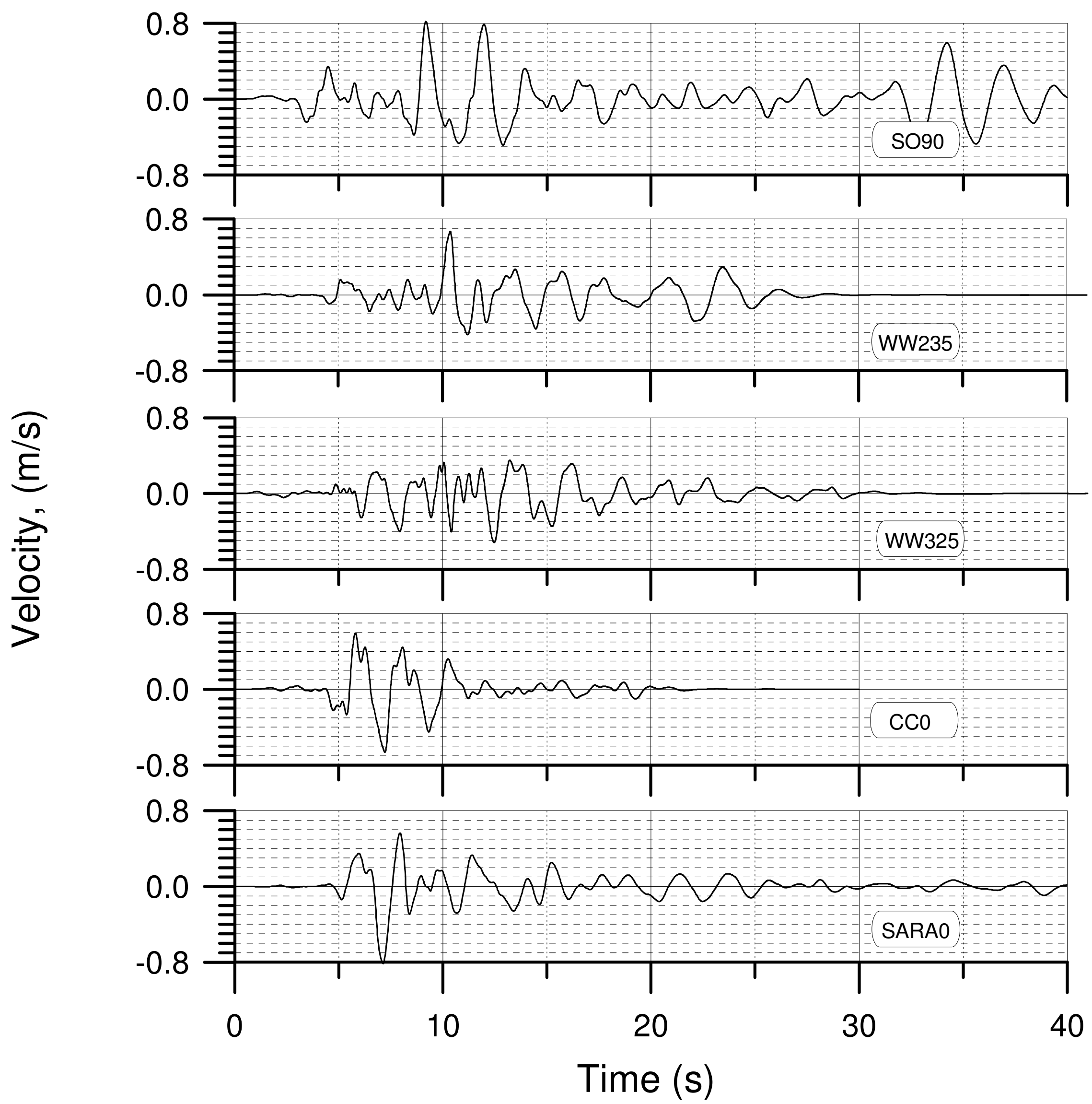

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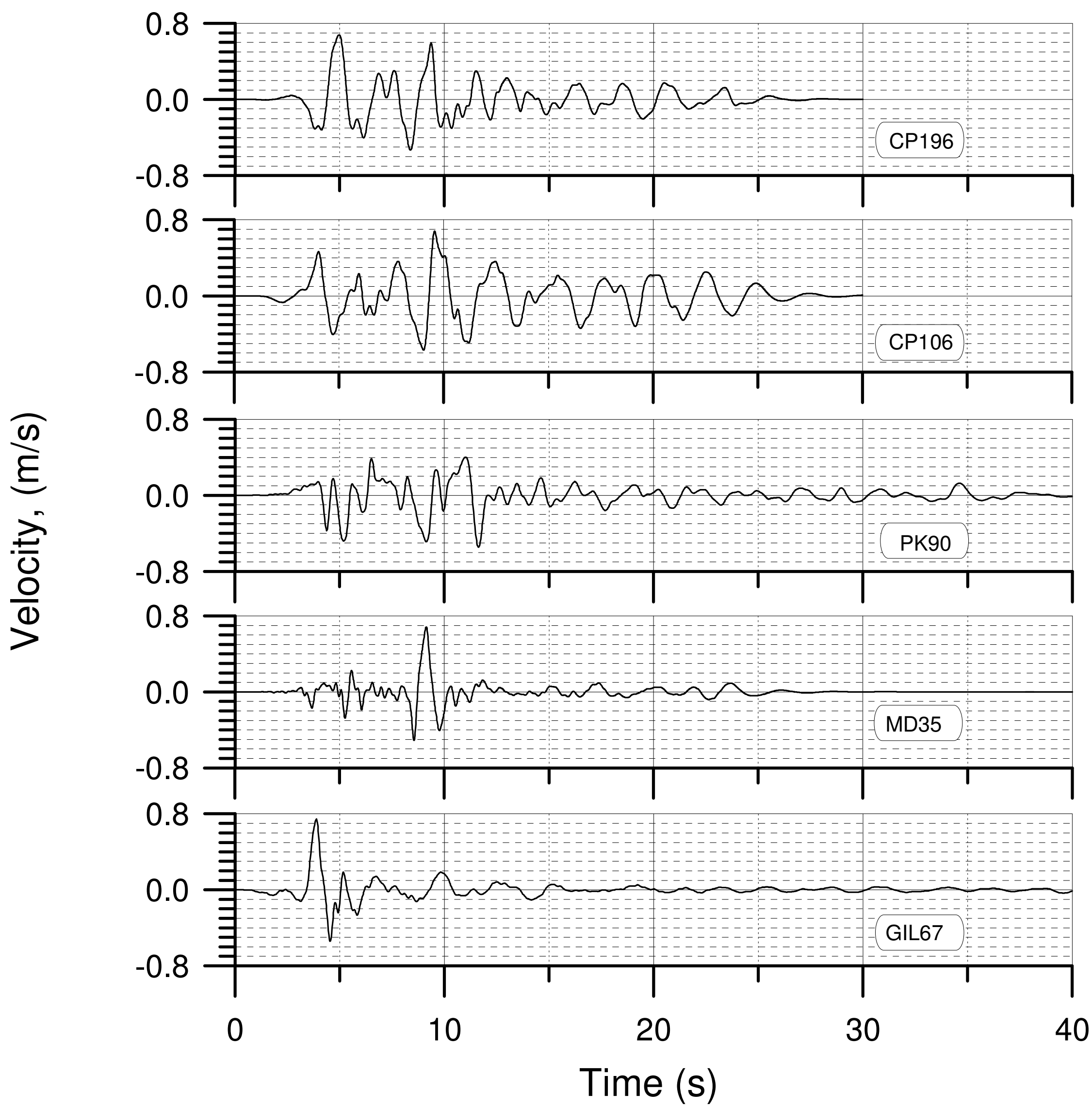
CLIENT
SANDWELL ENGINEERING INC.

PROJECT
PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.

TITLE
VELOCITY RECORDS AT 0.6 m DEPTH, OBTAINED FROM SHAKE

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
071-03006		**UMU**		**JAN., 2007**		**8**



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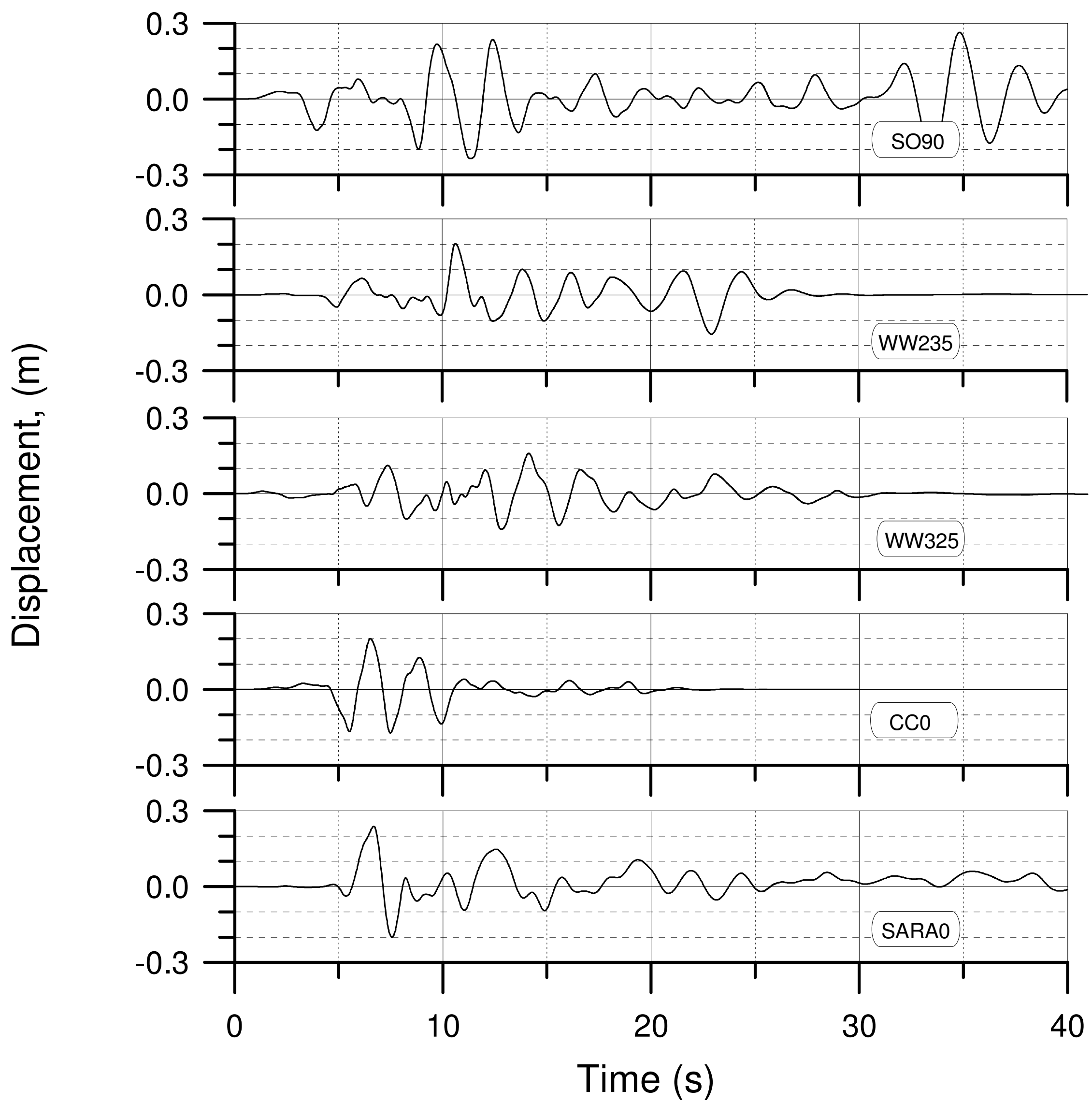
	PROJECT No. **071-03006**	DFTR.	DSGN. **UMU**	CHK.	DATE **JAN., 2007**	SCALE	DWG. No. 9

Trow



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CLIENT
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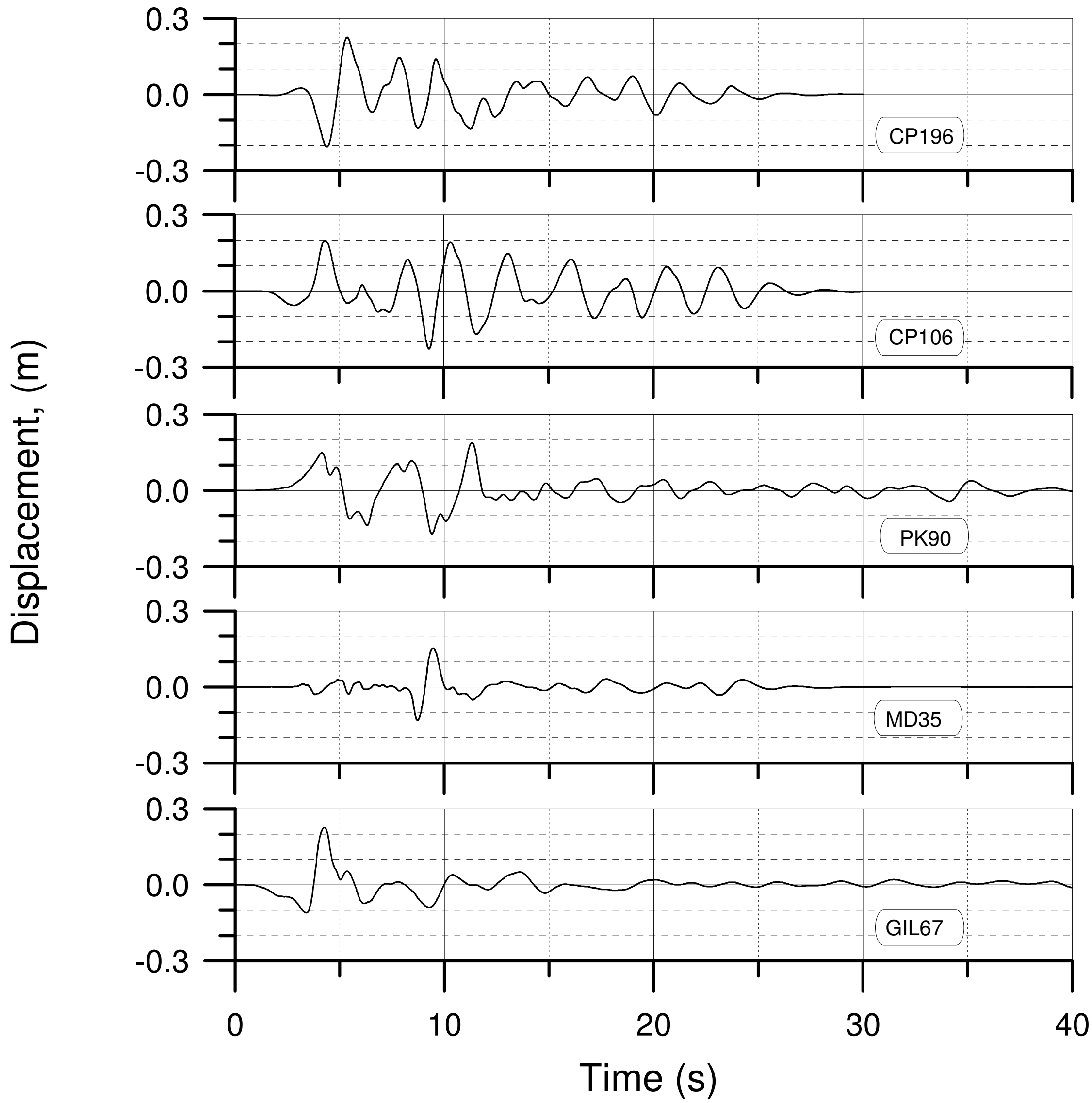
PROJECT
PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.

TITLE
DISPLACEMENT RECORDS AT 0.6 m DEPTH, OBTAINED FROM SHAKE

PROJECT No.	DFTR.	DSGN.	CHK.	DATE	SCALE	DWG. No.
071-03006		**UMU**		**JAN., 2007**		**10**



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PROJECT
PORT GUICHON ELEMENTARY SCHOOL SEISMIC UPGRADE DELTA, B.C.

TITLE
DISPLACEMENT RECORDS AT 0.6 m DEPTH, OBTAINED FROM SHAKE

PROJECT No. **071-03006** | DFTR. | DSGN. **UMU** | CHK. | DATE **JAN., 2007** | SCALE | DWG. No. **11**

(2) MEG Consulting Ltd. Report (DESRA Analysis)

(a) Summary of Results of Nonlinear Site Response Analysis

TBG Seismic Consultants Ltd
1945 Llewellyn Place
Sidney, B.C.
V8L 1G4

MEG

MARINE & EARTH
GEOSCIENCES

March 23, 2007

Attention: Dr. Graham Taylor, P.Eng.

Dear Graham:

RE: Summary of Results of Nonlinear Site Response Analysis Using DESRA-2C for Port Guichon School, Ladner, B.C.

As requested, one dimensional, nonlinear site response analysis of earthquake wave propagation was carried out for Port Guichon Elementary School, Ladner, B.C. The results of this analysis are summarized in the present letter report. We understand that this information will be used by TBG Seismic Consultants Ltd. and Sandwell Engineering Ltd. to facilitate structural design associated with seismic upgrading of the school.

The computer program DESRA-2C (Lee and Finn, 1978) was used for the present series of analyses to model earthquake wave propagation at the site. The model assumes that earthquake wave propagation is dominated by vertically propagating shear waves and considers nonlinear, cyclic hysteretic soil response with or without the influence of pore water pressure generation. Pore water pressure generation was not considered in the present analysis which is termed a "total stress" analysis. Neglect of pore pressure generation leads to prediction of the transmission of maximum shear stress and ground acceleration, and maximum inertial shaking of the school structure. This is most relevant to the early stages of ground shaking prior to the development of significant excess pore pressures in liquefiable soil deposits, or where ground densification is carried out so that soil liquefaction is mitigated.

With the DESRA-2C model, cyclic shearing stresses and shearing strains on the horizontal plane are computed at various depths versus time, as well as ground acceleration, velocity and displacement response. Since the model is one dimensional, non-level ground effects are not considered and ground motions are constrained to act only in the horizontal direction.

The use of nonlinear methods of site response analysis compared to the use of equivalent linear elastic methods (e.g. using the commonly used program SHAKE-91 computer program) generally leads to reductions in computed peak ground accelerations and response spectra. This is because nonlinear models limit the occurrence of elastic resonance since dynamic soil properties are continuously changing during shaking. This feature should lead to less conservatism in seismic retrofit design.

1.0 GENERAL SOIL AND GROUNDWATER CONDITIONS

We have reviewed geotechnical drill hole and electronic cone penetration test (CPT) data provided for the site by Trow Associates Inc. (reference their geotechnical report dated Feb. 22, 2006). This report indicates a 0.5 to 0.75m thick layer of silty sand and gravel fill underlain by stiff to firm, clayey silt with a thickness of about 2m, followed by 15.5 to 15.75m of Fraser River sand and silty sand deposits stratified with sandy silt. The latter are underlain by interbedded sandy silt, clayey silt, silty sand and sand, also deposited by the Fraser River, and extending to the maximum depth of CPT holes (= 30.0 m).

Downhole seismic CPT was carried out at one of the test hole locations (CPT-1) from which small strain, shear wave velocity measurements with depth were measured.

Groundwater levels were encountered at 1.5m depth at the time the site geotechnical investigations were carried out by Trow.

2.0 DYNAMIC SOIL PROPERTIES

Based on the above geotechnical information, Trow has carried out equivalent linear modeling of seismic site response using the computer program SHAKE91 (Idriss and Sun, 1992). This modeling was summarized in a memorandum dated January 18, 2007. In addition, nonlinear two dimensional modeling of seismic site response was carried out by Trow using the computer program FLAC (Itasca, 2005) primarily to estimate seismic ground displacements with consideration of cyclic pore pressure generation in the Fraser River sands down to about the 19m depth.

We have used the same soil layering profile as used by Trow in their SHAKE91 modeling to construct the DESRA-2C model. We have also used the same soil density, shear wave velocity (Vs) and small strain shear modulus (Gmax) parameters versus depth as used by Trow. Since Vs measurements (and Gmax computed using elasticity relationships from Vs) were not available for the site at depths in excess of 30m, it was necessary to use the published geophysical literature for the Fraser River Delta to estimate Vs and Gmax for greater depths, as described in the Trow January, 2007 memorandum. The soil layer thicknesses, soil densities, Vs and Gmax values used in the analysis are summarized in Table 1.

Since SHAKE91 is an equivalent linear model of site response, it does not directly consider the soil shear strength on the horizontal plane. From this point of view, it is possible for the SHAKE91 analysis to predict cyclic shearing stresses which exceed the soil shear strength. In order to use a nonlinear model such as DESRA-2C, it is necessary to calculate soil shear strength on the horizontal plane. These calculations were based on soil properties derived from the available CPT and drill hole data.

Drained shear strengths based on estimated values of peak friction angle and at rest earth pressure coefficient (K_0) have been used to calculate shear strengths in the near surface granular fills and Fraser River sand deposits above the 18m depth. The methodology proposed by Hardin and Drnevich (1972) has been used for this calculation. Undrained strengths have been used to characterize the dynamic strength of the surficial layer of clayey silt between the 0.6 and 2.3m depths. The static undrained strengths have been estimated based on the CPT data, and then increased by 20% to account for dynamic strain rate effects. Trow has recommended use of normally consolidated, undrained shear strengths (Su) for cyclic simple shear conditions for depths greater than 18m in the interlayered clayey silt, silt and sand deposits. The relation Su = 0.25 Φ'_{vo} has been used for this purpose. No strain rate effect has been considered for these deeper materials. The dynamic shear strengths on the horizontal plane used in the DESRA-2C modeling are summarized in Table 1.

The following contributions to soil damping were considered in the analysis:

(a) internal hysteretic energy losses within the soil mass
(b) elastic wave energy transmitted below the bottom boundary of the soil layer model

No additional radiation damping, commonly modeled using Rayleigh-type damping, was considered.

It is important to note that the seismic input motions specified by the UBC research group and the Geologic Survey of Canada (2003) are considered to be representative of motions occurring at the ground surface on a firm ground “outcrop”. Since firm ground representative of Site Class C conditions occurs at relatively large depth (150 metres or possibly greater), then some accounting for seismic wave energy dissipation into deeper materials below the 150 m depth must be made. Application of an interior seismic excitation combined with consideration of an energy absorbing bottom boundary reduces the effective seismic energy transmitted to the overlying soil layers. An energy absorbing bottom boundary was used in all DESRA-2C analyses presented herein based on the theory presented by Lee and Finn (1978). The energy absorption characteristics of the lower boundary were based on an average shear wave velocity of 400 m/sec derived from the Trow Vs profile extrapolated to the 150 m depth.

No attempt was made to alter the specified input motions by de-convolving these down to firm ground level at the 150 m depth through a “firm ground” Site Class C profile prior to propagating these altered motions back up through the actual soil profile.

3.0 “FIRM GROUND” SEISMIC INPUT MOTIONS

It is necessary to define input earthquake motions at “firm ground” level in order to carry out analysis of earthquake wave propagation for a particular site. These input motions will depend on seismic risk levels being considered for design. In the case of Port Guichon Elementary School, a seismic risk level having a 2% probability of being exceeded in 50 years has been adopted, consistent with the provisions of the 2005 National Building Code of Canada (NBCC).

The Geologic Survey of Canada (2003) report defining seismic ground motion parameters to be considered throughout Canada for the above seismic risk level states that "firm ground" is defined by materials having shear wave velocities in the range of 360 to 760 m/sec. Thus input earthquake motions selected for the study were placed at the 150 m depth where the soil materials had shear wave velocities in excess of 400 m/sec based on the available geophysical data for the area in the region of Port Guichon school. The 2005 NBCC defines these firm ground conditions as "Site Class C" soil conditions, representative of very dense soil, or soft rock.

The earthquake input motions (specified as horizontal accelerations versus time, termed an accelerogram) selected for seismic wave propagation analysis were supplied by the University of British Columbia Dept. of Civil Engineering (UBC) and TBG Seismic Consultants. The input motions were recorded at the ground surface during previous earthquakes at a variety of sites in California on soil conditions considered representative of Site Class C soils. The firm ground input motions were scaled from the original accelerograms so that after scaling their peak spectral velocity (PSV) averaged over the 0.5 to 1.5 second period range matched a target PSV (= 60.0 cm/sec) specified by the Geologic Survey of Canada (2003) for the Ladner area. The input accelerograms adopted for the present study and the scaling factors applied to the original accelerograms are presented in Table 2. The peak firm ground acceleration (PGA) and average peak spectral velocity (PSV) over the 0.5 to 1.5 second period range prior to scaling for each input motion are also presented in the table. Elastic response spectra computed for 5% structural damping after scaling of each accelerogram are shown in Figure 1.

TABLE 1 – DYNAMIC SOIL PROPERTIES USED IN TOTAL STRESS SITE RESPONSE ANALYSIS CARRIED OUT USING DESRA-2C

Layer No.	Soil Type	Thk. (m)	Avg. Depth (m)	Unit Weight (kN/m^3)	Vs (m/sec)	Gmax (kPa)	Drained Shear Strength (kPa)	Undrained Shear Strength (kPa)
1	granular fill	0.6	0.3	19	80	12395.5	1.3	
2	clay/silt	0.9	1.05	17	85	12520.4		28.4
3	clay/silt	0.8	1.9	16.4	85	12078.5		27.0
4	silty sand	1.2	2.9	19	108	22590.8	17.4	
5	silt/sand	1.5	4.25	18	145	38578.0	19.7	
6	FR sand	2.5	6.25	19	145	40721.2	29.6	
7	FR sand	4.5	9.75	19	150	43578.0	44.6	
8	FR sand	3	13.5	19	190	69918.4	65.4	
9	FR sand	2	16	19	180	62752.3	77.0	
10	FR sand	2	18	19	220	93741.1	90.3	
11	marine silt/sand	7	22.5	18	200	73394.5		52.4
12	marine silt/sand	4	28	18	225	92889.9		63.7
13	marine silt/sand	10	35	18	240	105688.1		78.0
13	marine silt/sand	10	45	18	260	124036.7		98.5
15	marine silt/sand	15	57.5	18	285	149036.7		124.1
16	marine silt/sand	15	72.5	18	300	165137.6		154.8
17	marine silt/sand	30	95	18	330	199816.5		200.9
18	marine silt/sand	30	125	18	370	251192.7		262.3
19	marine silt/sand	10	145	18	400	293578		303.2

Table 2 - Input Firm Ground Motions for Ladner Area

INPUT ACCELEROGRAM	SCALE FACTOR	PSV (cm/sec)	PGA (g's)
(1) Sherman Oaks – 105E	1.36	44.3	0.214
(2) Wadsworth – 235E	1.24	48.6	0.333
(3) Wadsworth – 325E	1.43	42.0	0.389
(4) Canyon Country – 0E	0.90	66.7	0.410
(5) Saratoga - 0E	0.87	69.3	0.504
(6) Canoga Park - 196E	0.77	78.0	0.434
(7) Canoga Park - 106E	1.15	52.3	0.355
(8) Pacoima Kagel – 90E	0.91	66.0	0.301
(9) 12520 Mulholland Dr. – 35E	1.23	49.0	0.597
(10) Gilroy Gavilon College - 67E	1.52	39.6	0.356

The above earthquakes have been recorded during earthquakes with magnitudes in the range of 6.5 to 7.5.

4.0 ANALYSIS RESULTS

Site response analysis results using total stress approaches are presented in the following figures:

Figure 2 – Peak ground surface acceleration versus depth.

Figure 3 – Cyclic stress ratios (CSR) versus depth. The CSR at a particular depth is computed as 0.65 times the peak cyclic shear stress, divided by the vertical effective overburden stress, consistent with geotechnical engineering practice.

Figure 4 – Elastic response spectral accelerations (peak spectral acceleration versus structural building period for 5% structural damping) and the mean spectra for all 10 input records. The spectra were obtained from computed horizontal accelerations at the 0.6m depth using the theory derived from a single degree of freedom oscillator. The computed spectra are compared with generic spectra provided in the 2005 NBCC for Site Class C, Site Class D and Site Class E soils. Based on the soil properties given in Table 1, the site would be classified as intermediate between Site Class D or E.

Figure 5 – Elastic response pseudo spectral velocities (spectral velocity versus structural building period for 5% structural damping) and the mean spectral velocity for all 10 input records.

The figures present analysis results for all 10 seismic input motions.

Computed acceleration time histories at the 0.6m depth have been provided to the UBC/TBG research group for further input into a structural model used to compute seismic base shears transmitted to buildings representative of those at Port Guichon Elementary School.

Examination of the above figures leads to the following observations:

- De-amplification of ground accelerations from the base of the soil column through the overlying denser materials to about the 10 m depth
- Slight amplification of ground accelerations above the 10 m depth
- Peak ground surface accelerations in the range of 0.17 to 0.22 g
- Peak CSR's in the range of 0.10 to 0.19 below the water table which are used to estimate liquefaction triggering potential for the granular soil layers (carried out by Trow)
- The computed mean acceleration response spectrum shows broad agreement for structural periods greater than 1.0 second with the 2005 NBCC Site Class D design spectrum
- The computed mean acceleration response spectrum and pseudo velocity response spectrum are significantly lower than presented by Trow using SHAKE-91 analysis (reference their January 18, 2007 report, figure 3)

It is recommended that the above results be checked against other nonlinear methods of analysis (e.g. the program FLAC).

5.0 CLOSURE

The report has summarized the results from seismic site response analyses performed using geotechnical data provided in geotechnical reports prepared by others. We have assumed that the information provided is correct and that it can be used for the purposes outlined in the text. Any concern as to the veracity of the information should be communicated to us so that we may revise our analyses and results in accordance with the observations.

MEG Consulting Limited has prepared this report in a manner consistent with a level of care and skill ordinarily exercised by members of the engineering and geosciences professions currently practicing in British Columbia, subject to the time limits and physical constraints applicable to this project and report. No other warranty, expressed or implied is made.

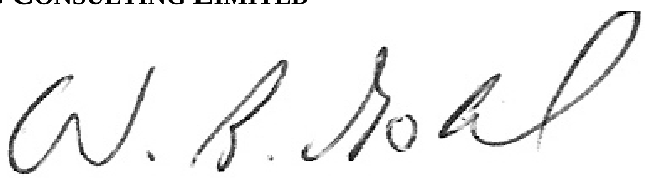
The report has been prepared for the specific site, design objective and development described to MEG. The factual data, interpretations and recommendations contained in the report are specific to this project as we understand it and are not applicable to any other project or site location. MEG can not be responsible for use of this report, or parts thereof, unless MEG is requested to review and, if necessary, revise the report.

The information, recommendations, estimates and opinions contained in the report are for the sole benefit of TBG Seismic Consultants and Sandwell Engineering. No other party may use or rely on this report without express written consent provided by MEG.

We hope that the information provided in the report meets your present requirements. Should you have any questions regarding the content of the report or any other detail, please contact the undersigned.

Sincerely,

MEG CONSULTING LIMITED



W. Blair Gohl, Ph.D., P.Eng.
Principal

LIST OF REFERENCES

Geologic Survey of Canada (2003), "Fourth Generation Seismic Hazard Maps of Canada: Values for Over 650 Canadian Localities Intended for the 2005 National Building Code of Canada", J. Adams & S. Halchuk, Open File Report 4459

Hardin, B.O. and V.P. Drnevich (1972), "Shear Modulus and Damping in Soils: Design Equations and Curves", ASCE, Journal of the Soil Mechanics and Foundations Division, vol. 98, no. SM7, pp. 667-692

Idriss, I.M. and J. Sun (1991), "User's Manual for SHAKE-91", Centre for Geotechnical Modeling, Dept. of Civil and Environmental Engineering, University of California, Davis, CA

Itasca Consulting Group, Inc. (2005), Fast Lagrangian Analysis of Continua (FLAC 5.0)

Lee, M.K.L. and W.D.L. Finn (1978), "DESRA-2C: Dynamic Effective Stress Response Analysis of Soil Deposits With Energy Transmitting Boundary Including Assessment of Liquefaction Potential", University of B.C., Faculty of Applied Science, June 1978

Figure 1: Elastic acceleration response spectra for firm ground (Site Class C) outcrop motions.

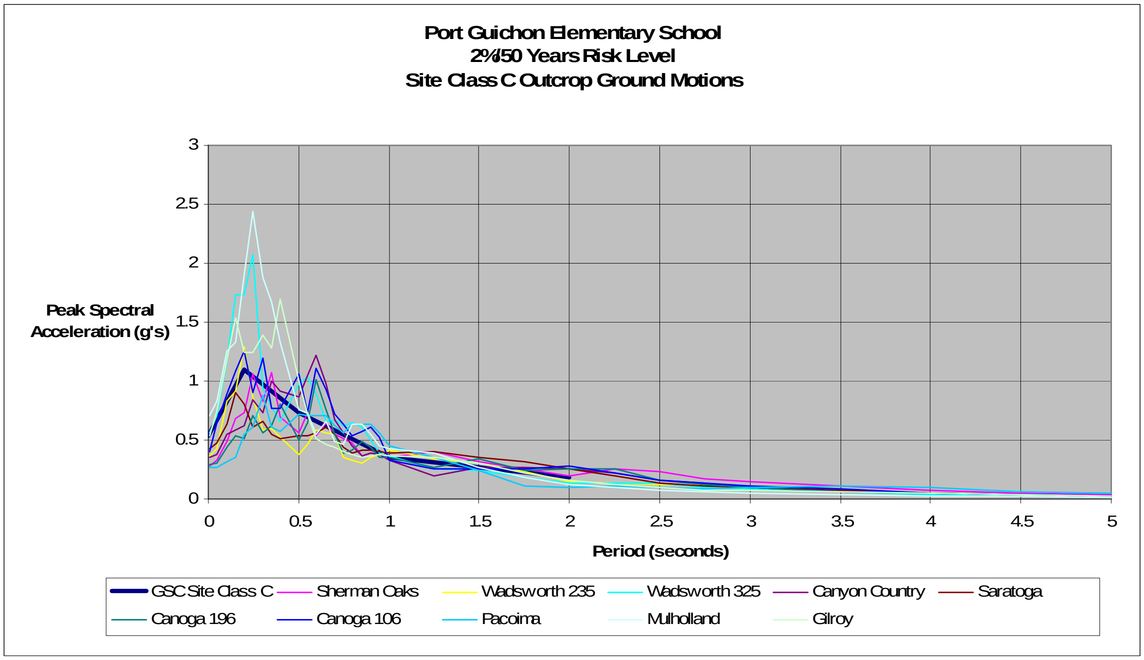


Figure 2: Peak ground acceleration versus depth computed using total stress site response analysis (program DESRA-2C).

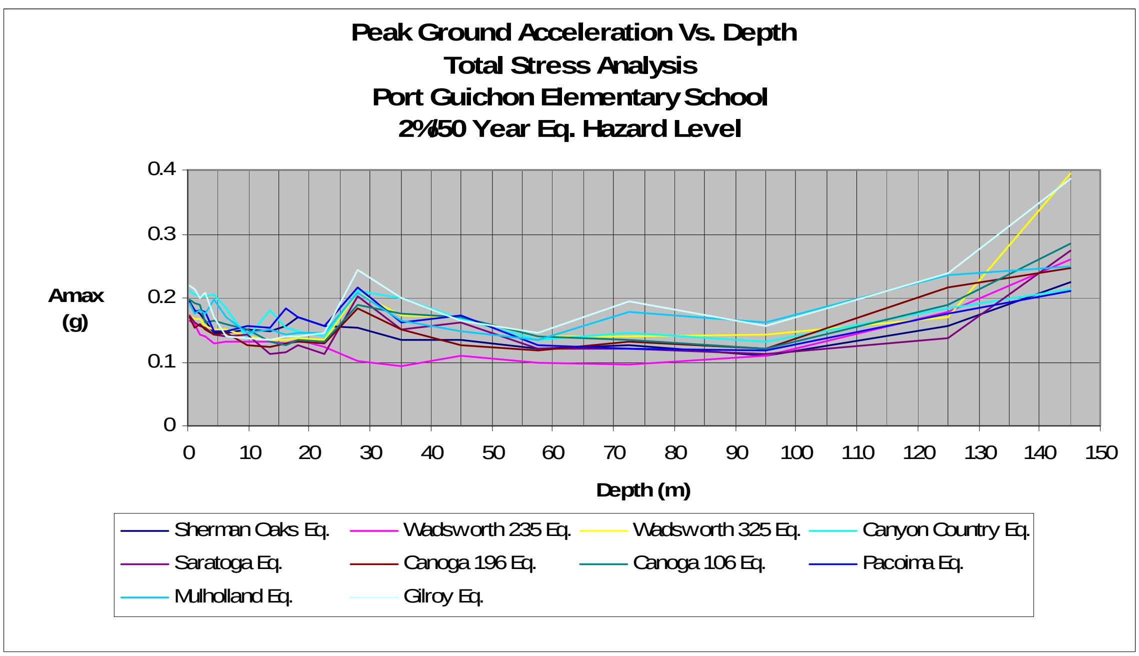


Figure 3: Effective cyclic shear stress ratio versus depth computed using total stress site response analysis (program DESRA-2C).

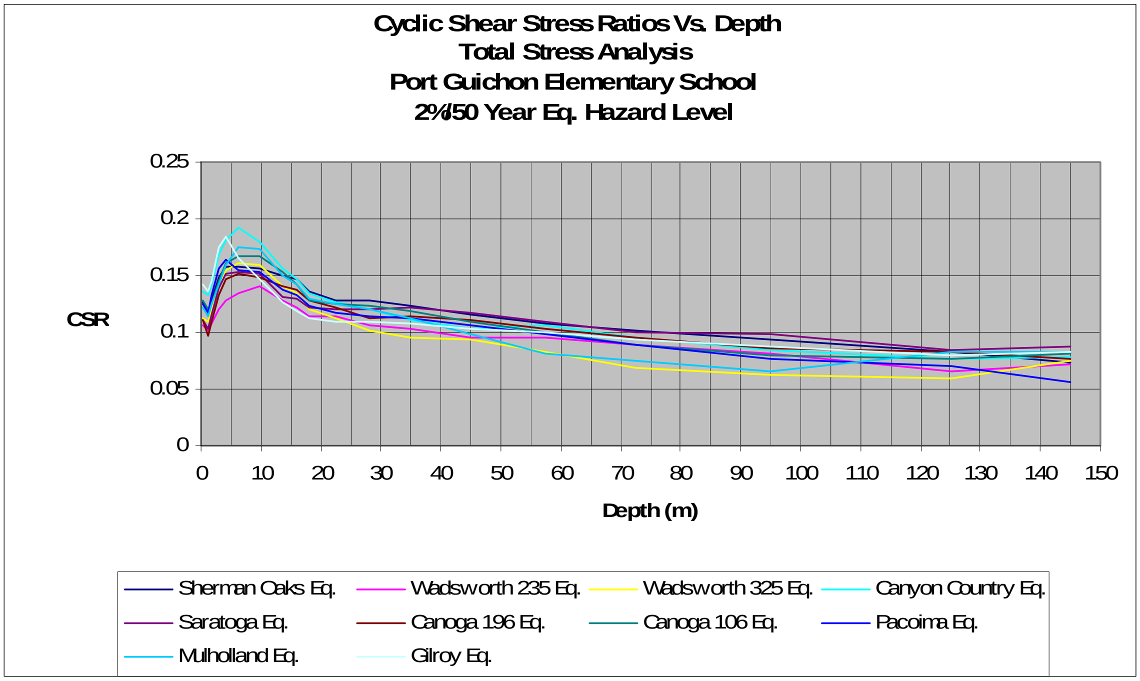


Figure 4: Elastic response peak spectral accelerations (5% damping case) for near surface motions at 0.6m depth computed using total stress analysis (program DESRA-2C).

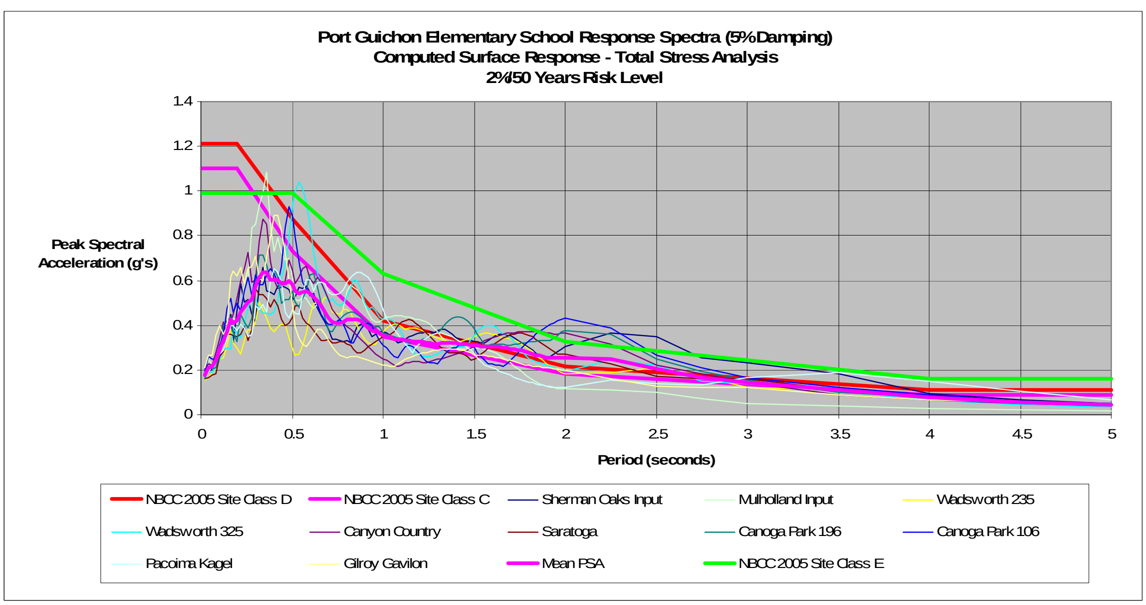
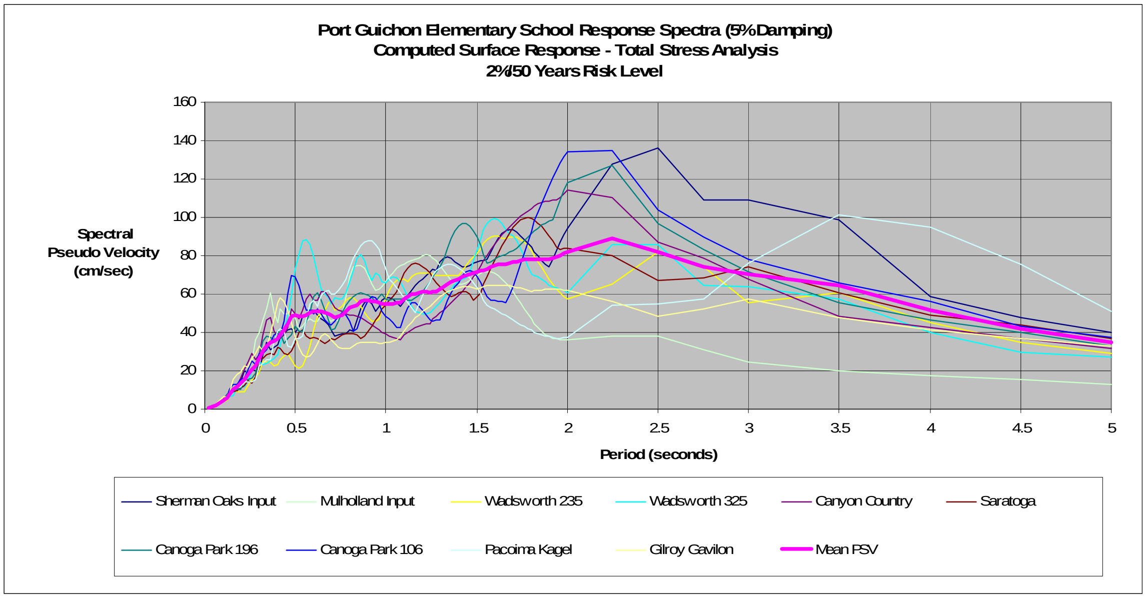
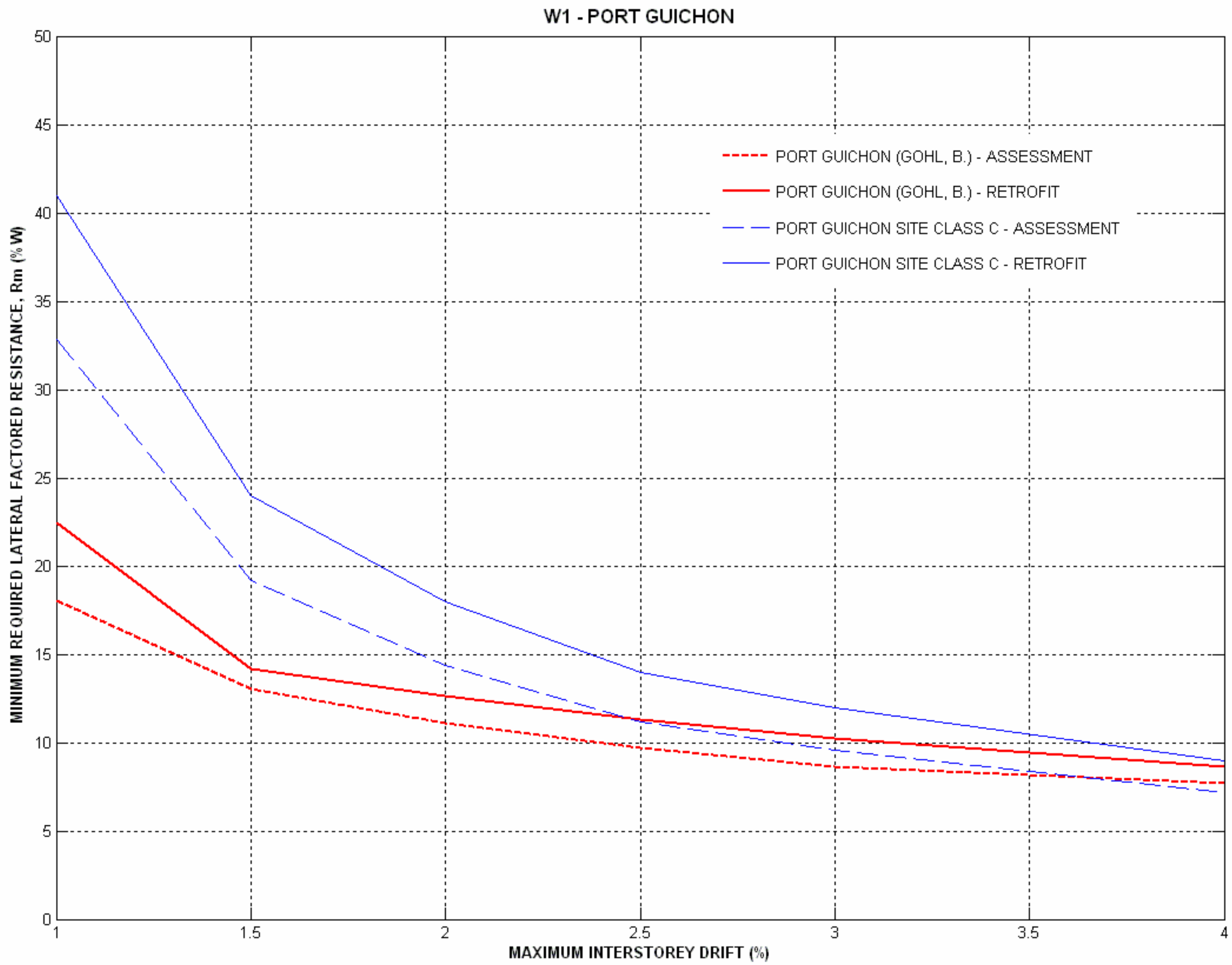


Figure 5: Elastic response pseudo spectral velocities (5% damping case) for near surface motions at 0.6m depth computed using total stress analysis (program DESRA-2C).



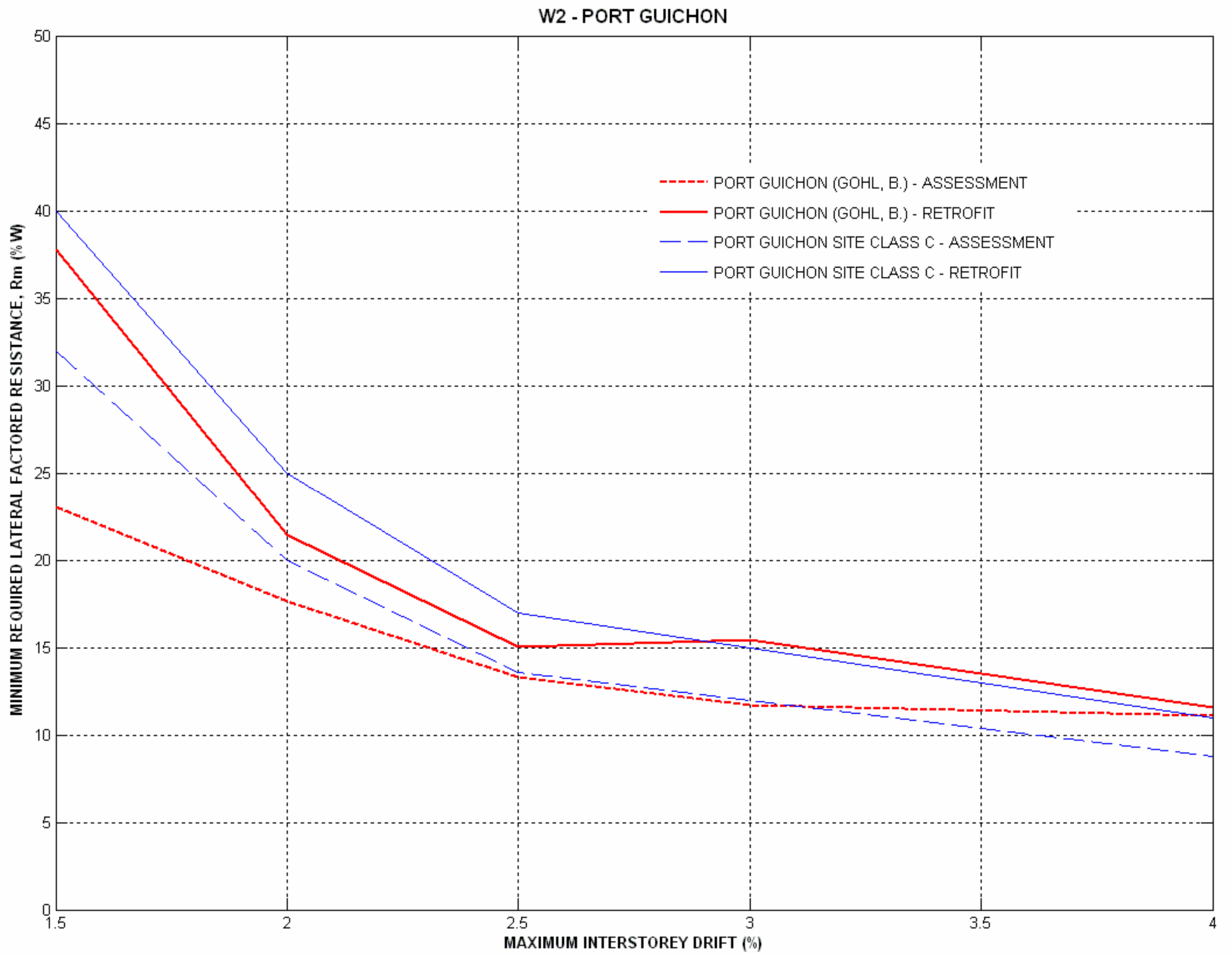
(3) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1


W1 - PORT GUICHON
MINIMUM REQUIRED LATERAL FACTORED RESISTANCE, Rm (% W)
MAXIMUM INTERSTOREY DRIFT (%)
PORT GUICHON (GOHL, B.) - ASSESSMENT
PORT GUICHON (GOHL, B.) - RETROFIT
PORT GUICHON SITE CLASS C - ASSESSMENT
PORT GUICHON SITE CLASS C - RETROFIT
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

(3) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for W-2


W2 - PORT GUICHON
MINIMUM REQUIRED LATERAL FACTORED RESISTANCE, Rm (% W)
MAXIMUM INTERSTOREY DRIFT (%)
PORT GUICHON (GOHL, B.) - ASSESSMENT
PORT GUICHON (GOHL, B.) - RETROFIT
PORT GUICHON SITE CLASS C - ASSESSMENT
PORT GUICHON SITE CLASS C - RETROFIT
0
5
10
15
20
25
30
35
40
45
50
1.5
2
2.5
3
3.5
4

APPENDIX K

DETAILED RESULTS FOR VICTORIA HIGH SCHOOL

CONTENTS

(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(a) E-mail dated March 23, 2007 - analysis assumptions

-----Original Message-----
From: Blair Gohl
Sent: Friday, March 23, 2007 12:42 PM
To: Graham Taylor (gwt@tbggsc.bc.ca)
Subject: Site Response Analysis for Victoria High School

Graham,

Based on the estimated soil properties for Victoria High School that Pat and Chris sent me (see attached), I have constructed a DESRA-2C model considering 4m of very stiff clay, over 2.5m of firm clay, underlain by dense till with a Vs of 475 m/sec. The Vs, soil density and undrained strength properties of the clay are as provided by Pat and Chris. I have not attempted to refine this information given the limited site specific geotechnical data for the site.

Using the 10 input Site Class C ground motions applied at the top of till level (6.5m depth) scaled to represent Zone 5 for Victoria, I have propagated the motions up from firm ground level. An energy absorbing bottom boundary was considered with a Vs of 475 m/sec and a density of 22 kN/cu.m. No additional Rayleigh-type damping was considered, over and above internal hysteretic damping of the soil.

The computed peak ground accelerations versus depth are provided in the attached Excel spreadsheet ("CSR&Amax") which indicates significant amplification through the very stiff clay layer for some of the input motions. This extreme amplification is due to the limited damping in this layer since cyclic shear stresses are well below undrained strengths of the materials. Cyclic shear strains are correspondingly small.

From the computed acceleration time histories at the 1m depth, I have computed elastic response spectra (spectral acceleration and velocity) as per the attached spreadsheet. Significant short period amplification is indicated for some of the input motions. I have also attached the computed acceleration time histories.

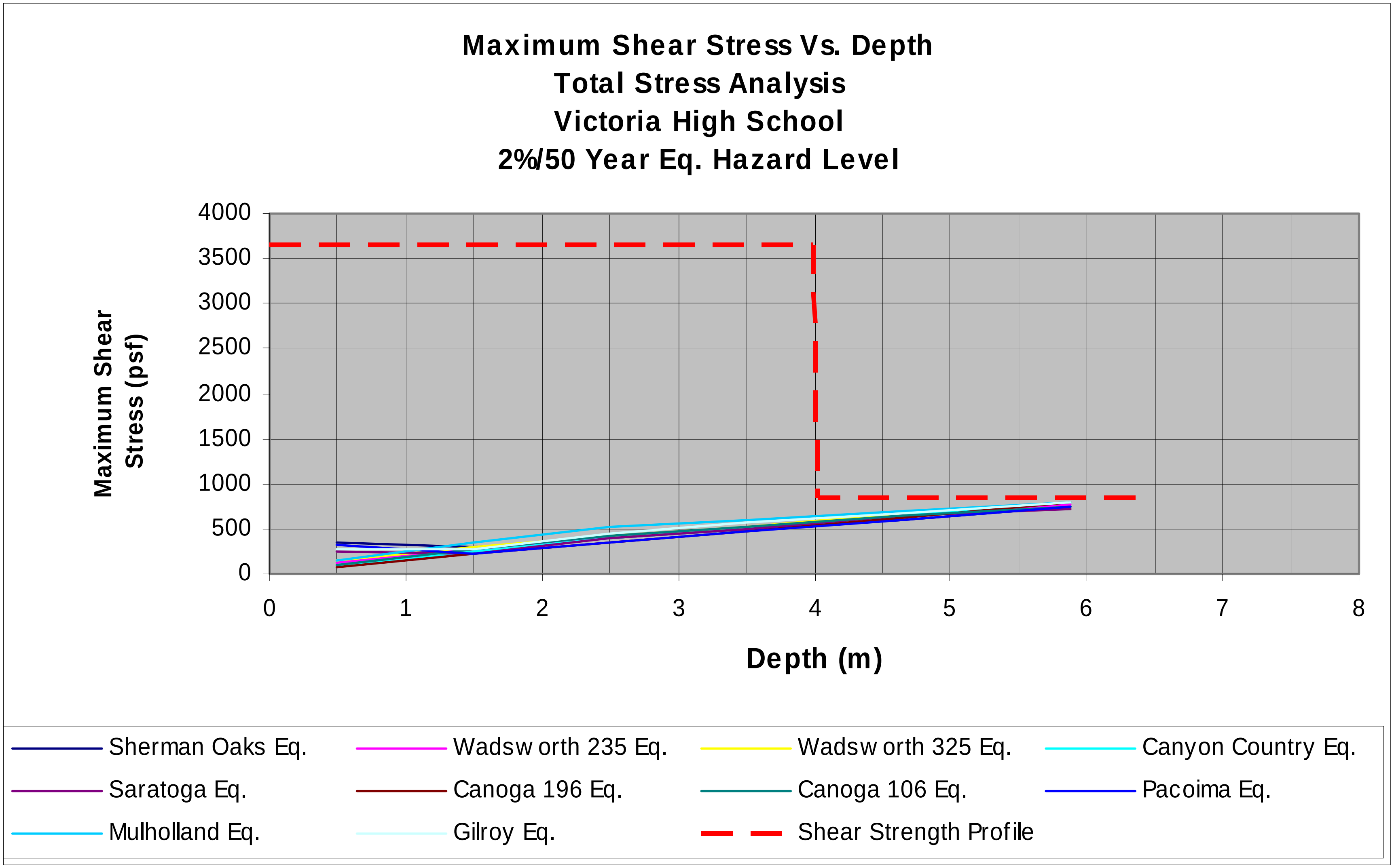
I hope this information suffices for now. As discussed, the significant short period amplification exceeds considerably the 2005 NBCC recommendations for Site Class C and D soils. A key reason for this amplification appears to relate to the very high stiffness and strength of the upper layer of very stiff clay which limits shear strains and damping in this layer. It would be necessary to confirm local geotechnical soil conditions prior to proceeding with final seismic retrofit design of Victoria High since these properties are critical to confirming whether this level of site amplification is real.

Regards,

Blair Gohl, Ph.D., P.Eng.
Principal
MEG Consultants Ltd.

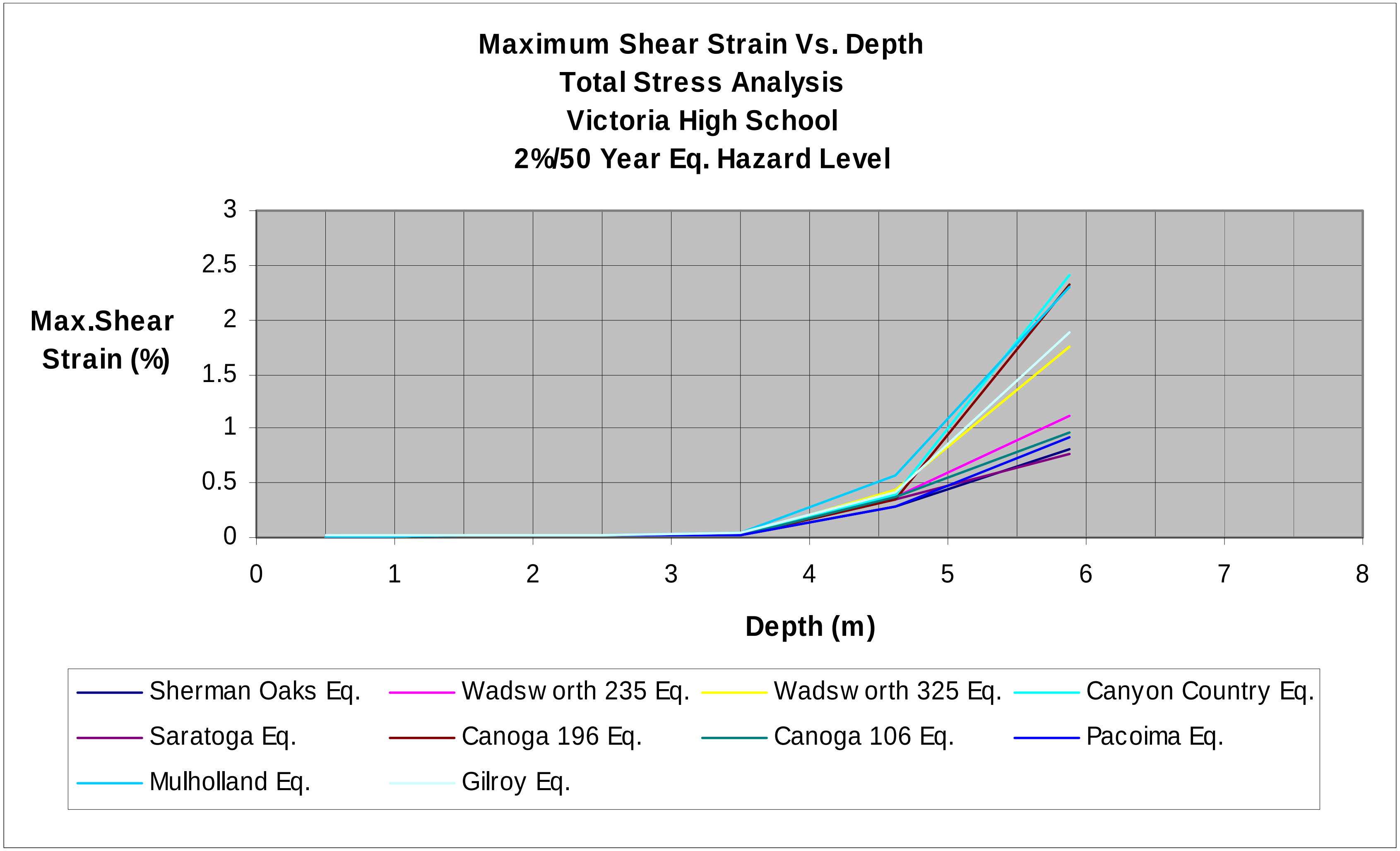
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Victoria High School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
4000
3500
3000
2500
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

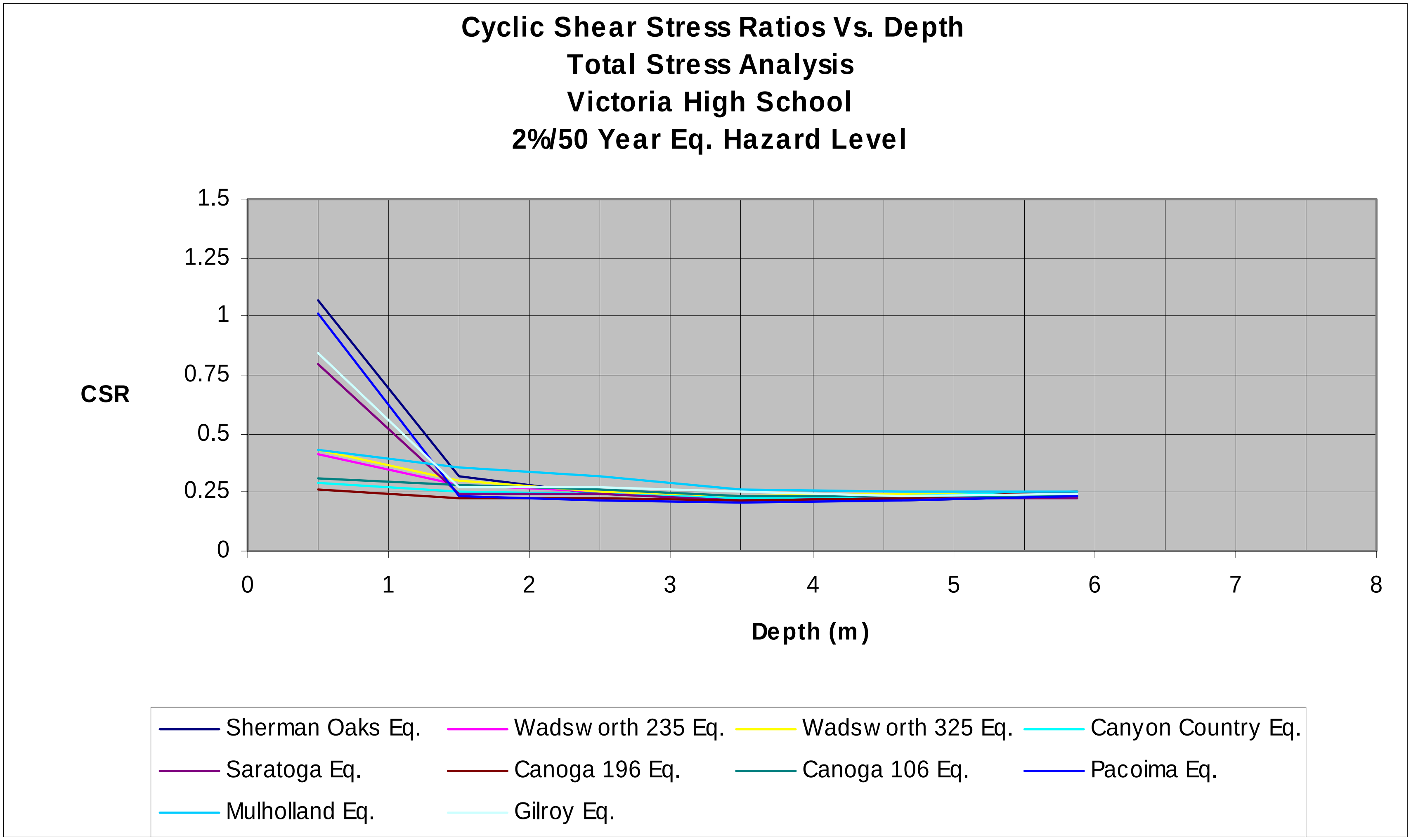
(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Victoria High School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
3
2.5
2
1.5
1
0.5
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

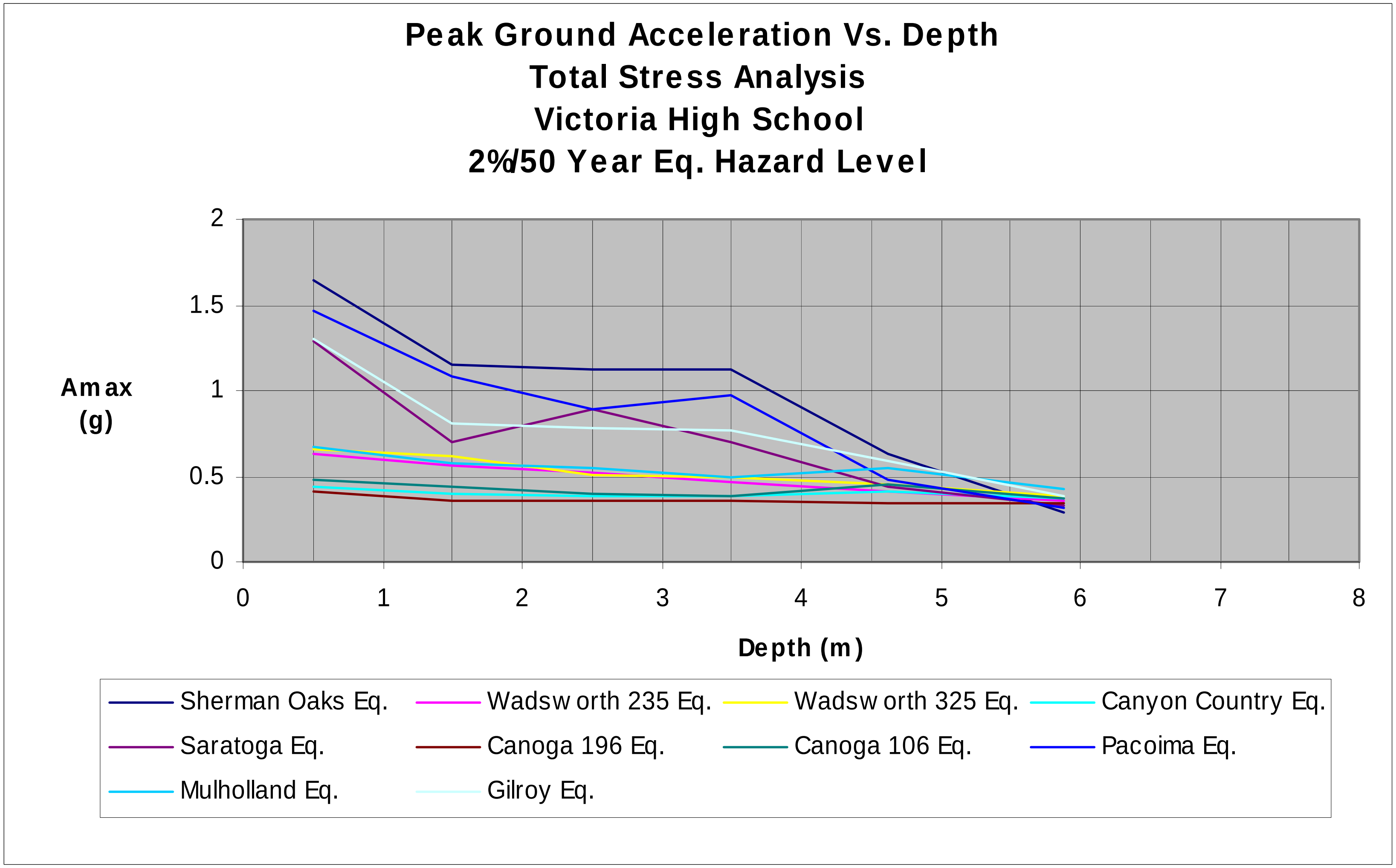
(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Victoria High School
2%/50 Year Eq. Hazard Level
CSR
1.5
1.25
1
0.75
0.5
0.25
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

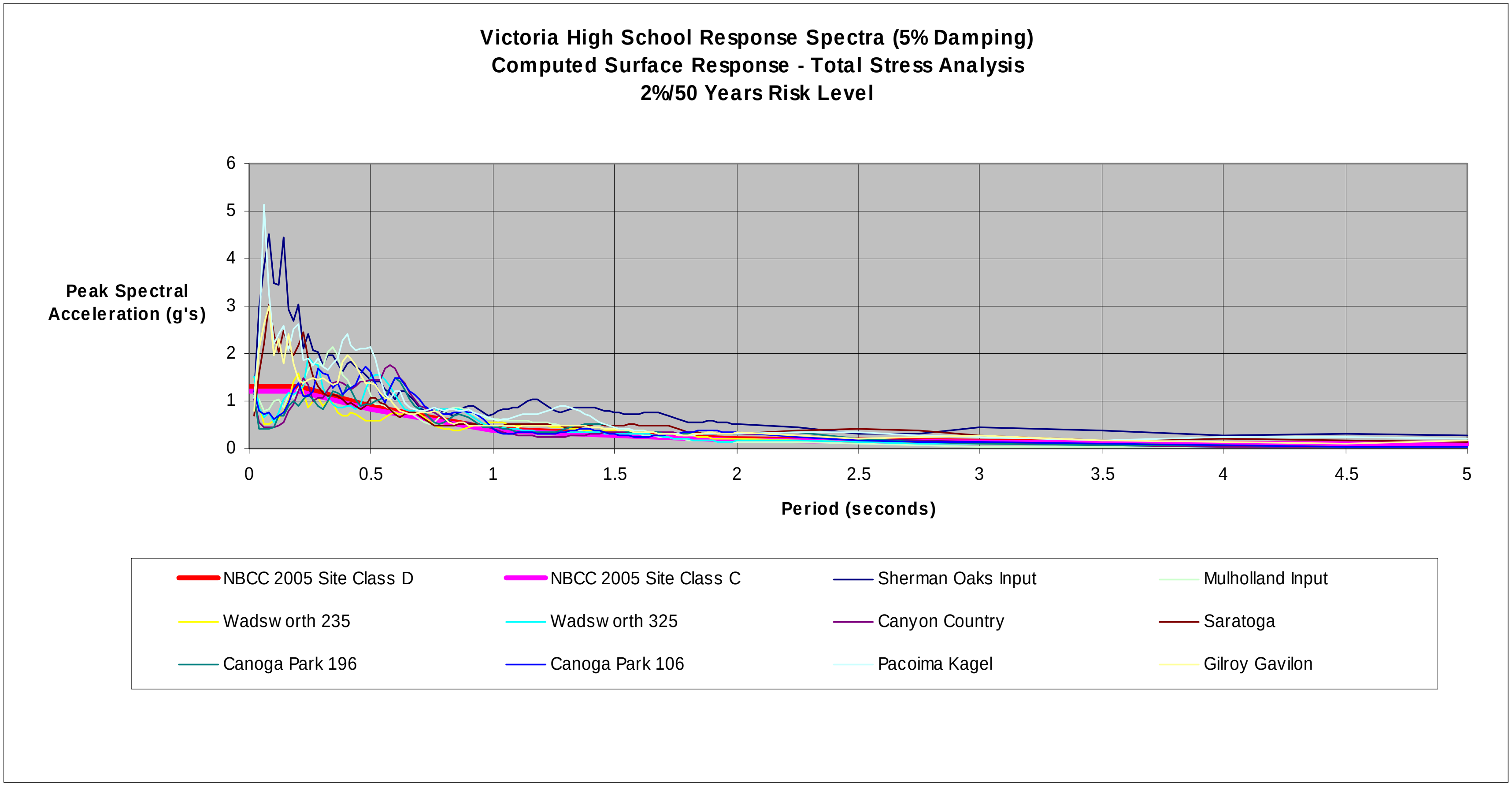
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Victoria High School
2%/50 Year Eq. Hazard Level
Amax
(g)
2
1.5
1
0.5
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

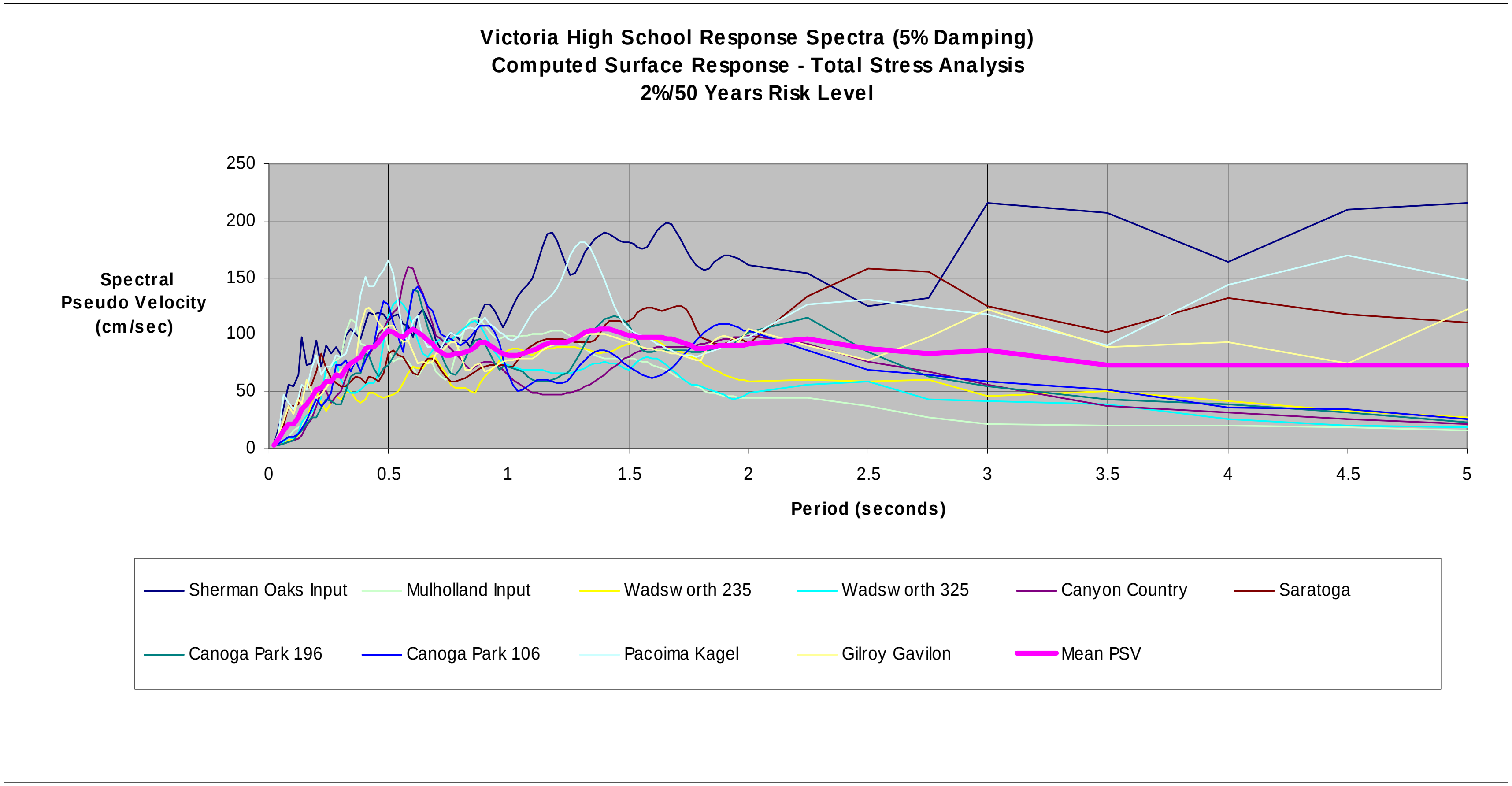
(f) Graph - surface acceleration response spectra



Victoria High School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

(1) MEG Consulting Ltd. Report (DESRA Analysis)

(g) Graph - surface velocity response spectra


Victoria High School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
250
200
150
100
50
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsworth 235
Wadsworth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSV

(1) MEG Consulting Ltd. Report (DESRA Analysis)

(h) Soil Column Profile

Victoria High Profile

BC Boundary

Layer number	stratigraphic unit	Soil type	Layer thickness m	Vs m/s	unit weight kN/m3	shear strength kPa	N160	Friction angle degrees (drained conditions)
1	Victoria Clay Brown Clay facies	very stiff clay	4	215	20	175	25	25
2	Till	very dense sand	1	475	22		>50	>45
3	Bedrock							

WT depth 3 m

Site Class C

1	Victoria Clay Brown Clay facies	very stiff clay	4	215	20	175	25	25
2	Victoria Clay Grey Clay facies	firm clay	3	133	20	40	4	22
3	Till	very dense sand	1	475	22		>50	>45
4	Bedrock							

WT depth 3 m

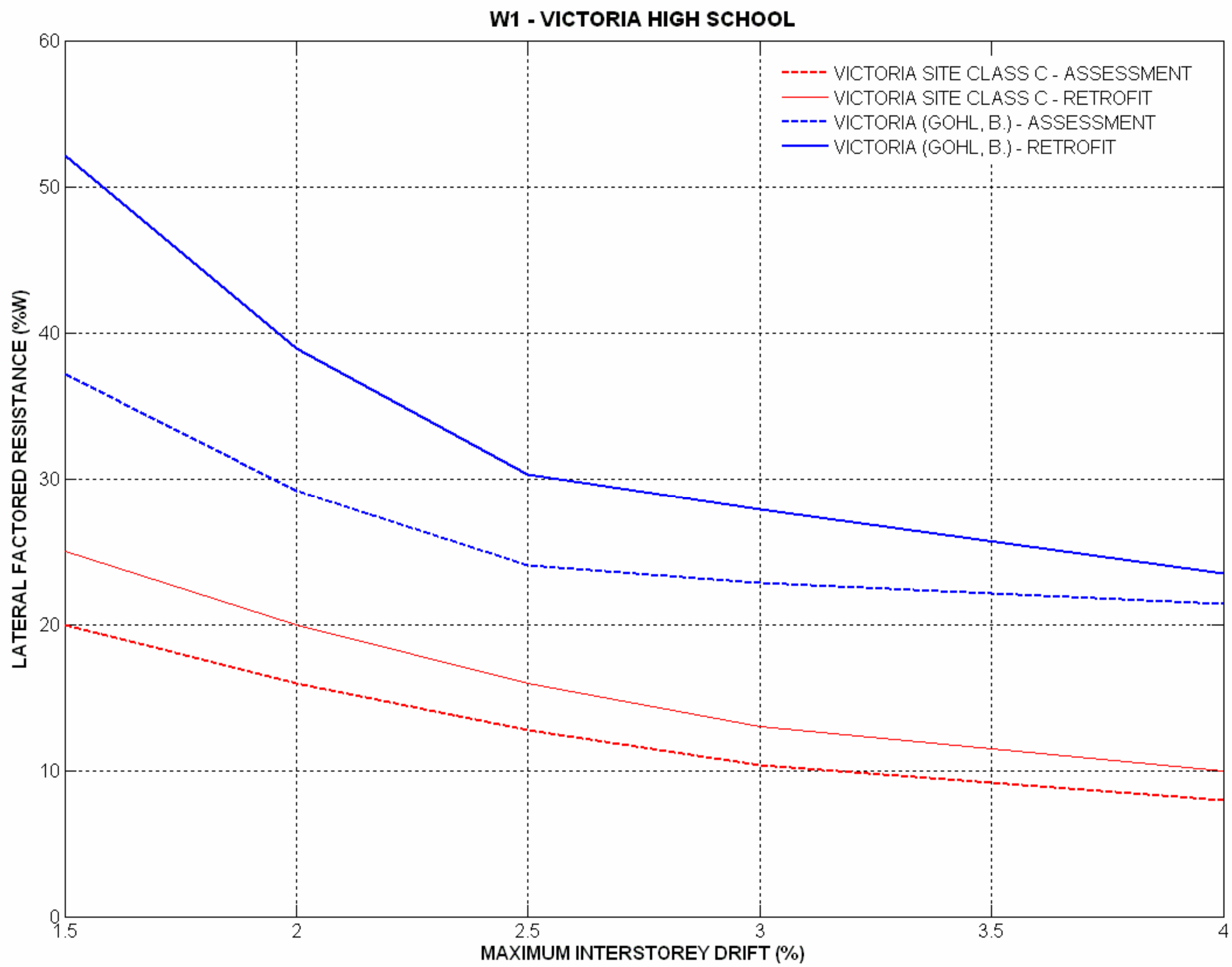
C-D?

1	Victoria Clay Brown Clay facies	very stiff clay	4	215	20	175	25	25
2	Victoria Clay Grey Clay facies	firm clay	2	133	20	40	4	22
3	Till	very dense sand	5	475	22		>50	>45
4	Bedrock							

WT depth 3 m

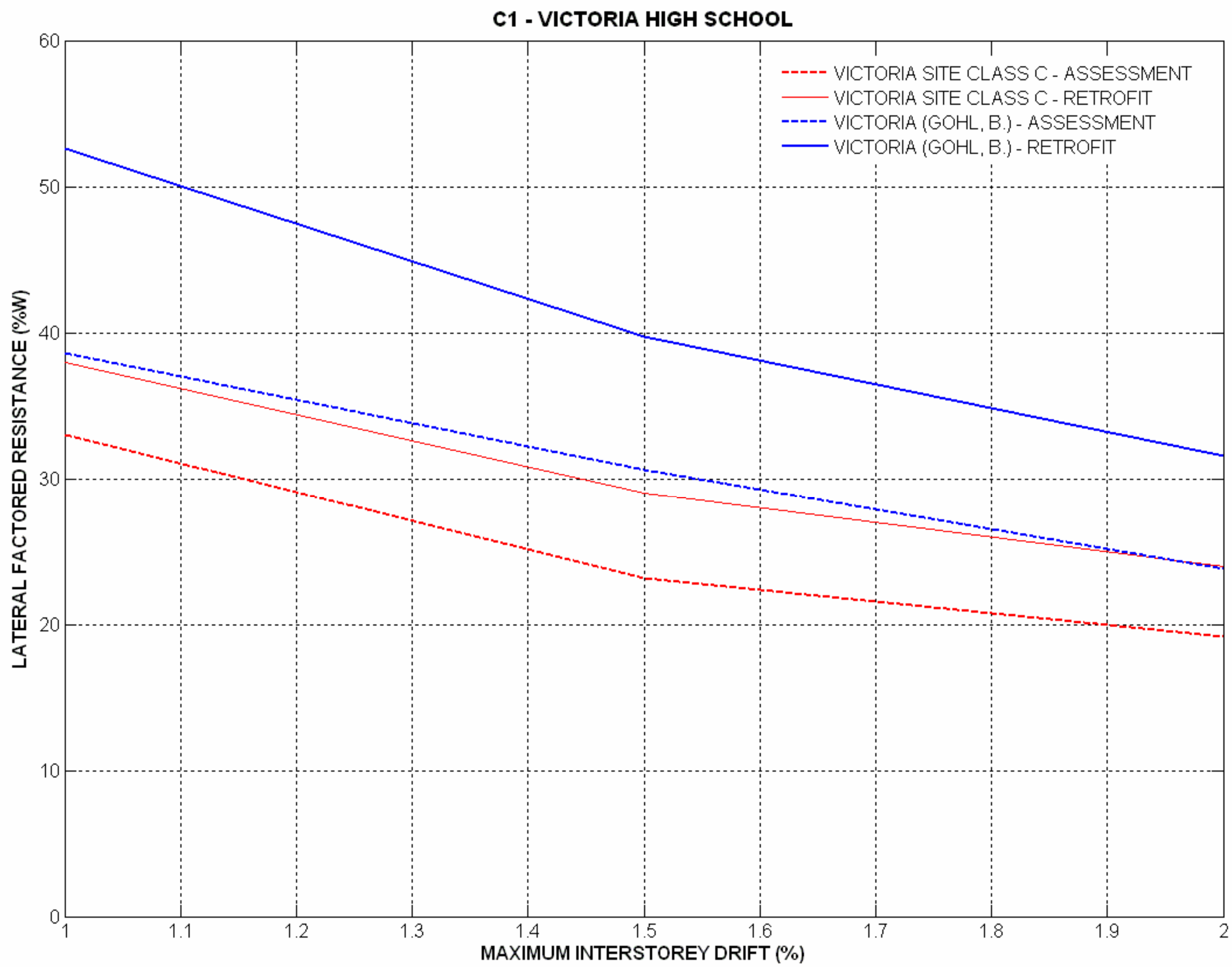
(2) <u>UBC Report (Structural Analysis)</u>

(a) Graph - lateral factored resistance versus drift for W-1


W1 - VICTORIA HIGH SCHOOL
VICTORIA SITE CLASS C - ASSESSMENT
VICTORIA SITE CLASS C - RETROFIT
VICTORIA (GOHL, B.) - ASSESSMENT
VICTORIA (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1.5
2
2.5
3
3.5
4

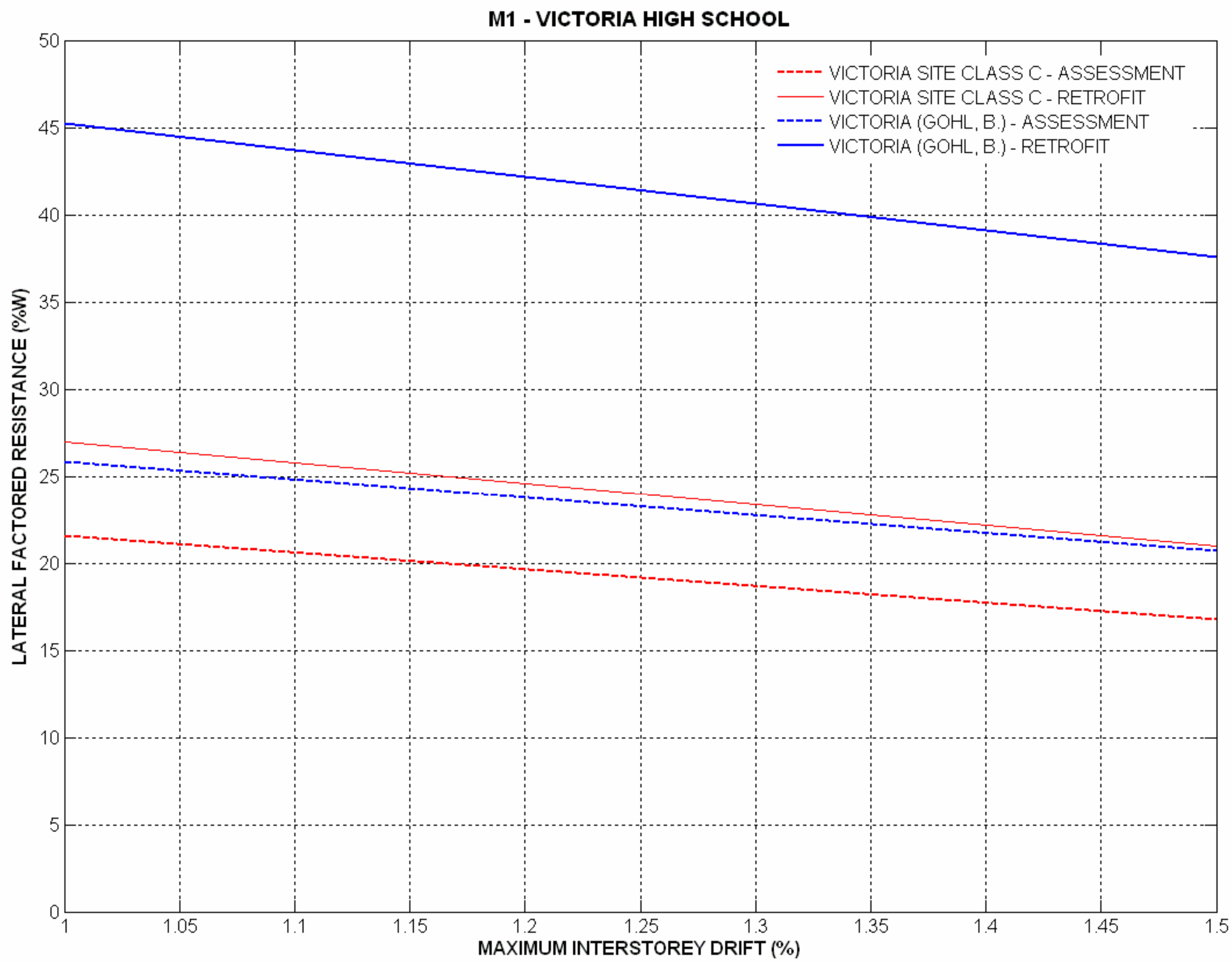
(2) <u>UBC Report (Structural Analysis)</u>

(b) Graph - lateral factored resistance versus drift for C-1


C1 - VICTORIA HIGH SCHOOL
VICTORIA SITE CLASS C - ASSESSMENT
VICTORIA SITE CLASS C - RETROFIT
VICTORIA (GOHL, B.) - ASSESSMENT
VICTORIA (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
10
20
30
40
50
60
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2

(2) <u>UBC Report (Structural Analysis)</u>

(c) Graph - lateral factored resistance versus drift for M-1


M1 - VICTORIA HIGH SCHOOL
VICTORIA SITE CLASS C - ASSESSMENT
VICTORIA SITE CLASS C - RETROFIT
VICTORIA (GOHL, B.) - ASSESSMENT
VICTORIA (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5

APPENDIX L

DETAILED RESULTS FOR WILLOWS ELEMENTARY SCHOOL

CONTENTS

(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(a) E-mail dated March 26, 2007 - analysis assumptions

Graham,

Based on the estimated soil properties for Willows School that Pat and Chris sent me, I have constructed a DESRA-2C model considering 4m of stiff clay over 4m of firm clay, underlain by very dense sandy till with an estimated Vs of 475 m/sec. The soil profile selected Pat defines as representative of "Soil Class C". The Vs, soil density and undrained strength properties of the clay are as provided by Pat and Chris based on available borehole data at the east end of the site. I have not attempted to refine this information given the limited site specific geotechnical data for the site.

Using the 10 input Site Class C ground motions applied at the top of till level (10m depth) scaled to represent Zone 5 for Victoria, I have propagated the motions up from firm ground level. An energy absorbing bottom boundary was considered with a Vs of 475 m/sec (appropriate for dense till) and a density of 22 kN/cu.m. No additional Rayleigh-type damping was considered, over and above internal hysteretic damping of the soil.

The computed peak ground accelerations versus depth are provided in the attached Excel spreadsheet ("CSR&Amax") which indicates de-amplification through the lower part of the firm clay profile followed by amplification in the upper 4m of stiff clay. Cyclic shear strains are relatively high in the lower half of the soil profile.

From the computed acceleration time histories at the 1m depth, I have computed elastic response spectra (spectral acceleration and velocity) as per the attached spreadsheet. Significant spectral amplification is generally indicated below the 1 second period range. The amount of amplification depends on the input record considered. I have also attached the computed acceleration time histories.

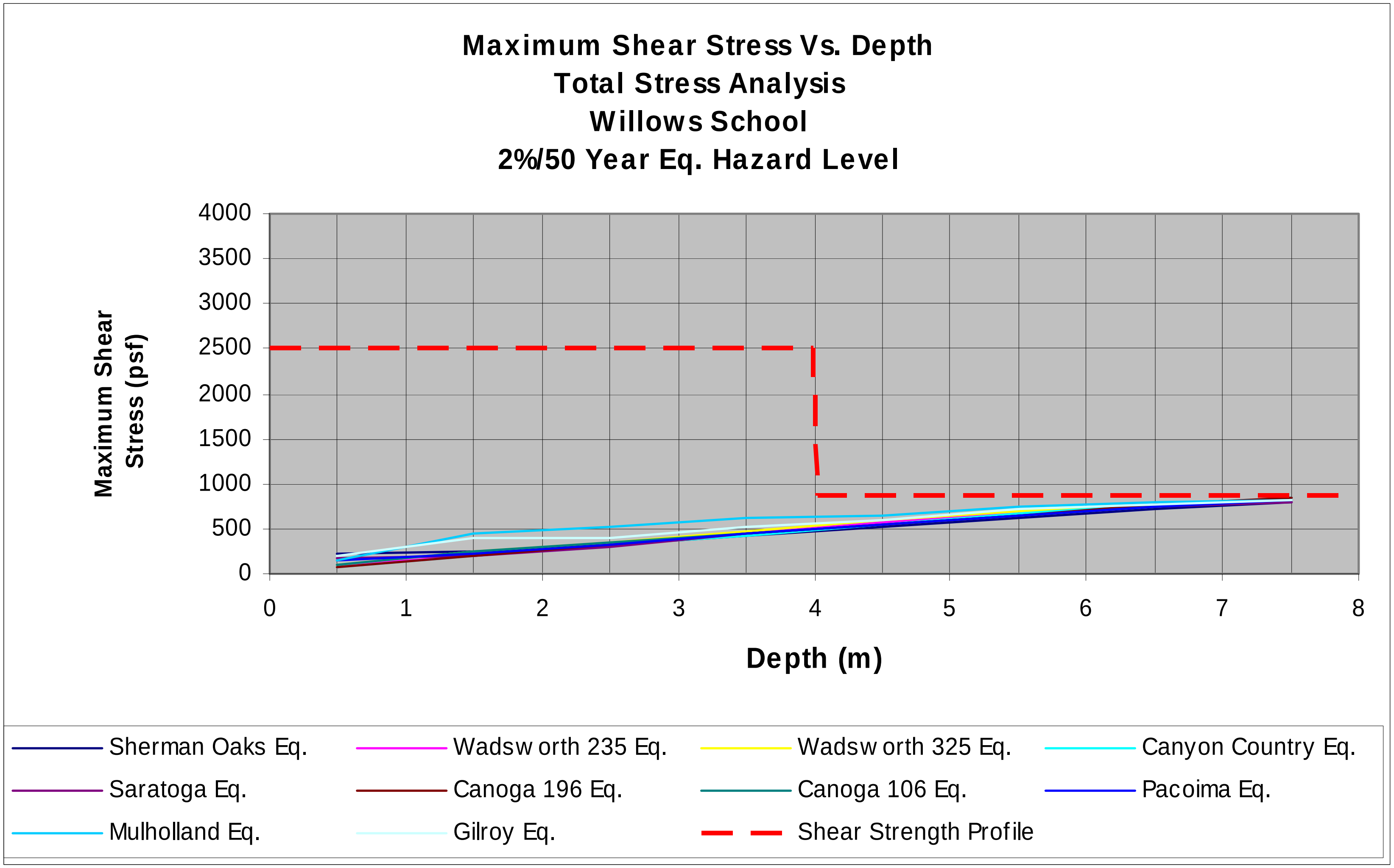
As noted earlier, it will be necessary to confirm local geotechnical soil conditions prior to proceeding with final seismic retrofit design of Willows School since these properties are critical to confirming the computed site response analysis.

Regards,

Blair Gohl, Ph.D., P.Eng.
Principal
MEG Consulting Ltd

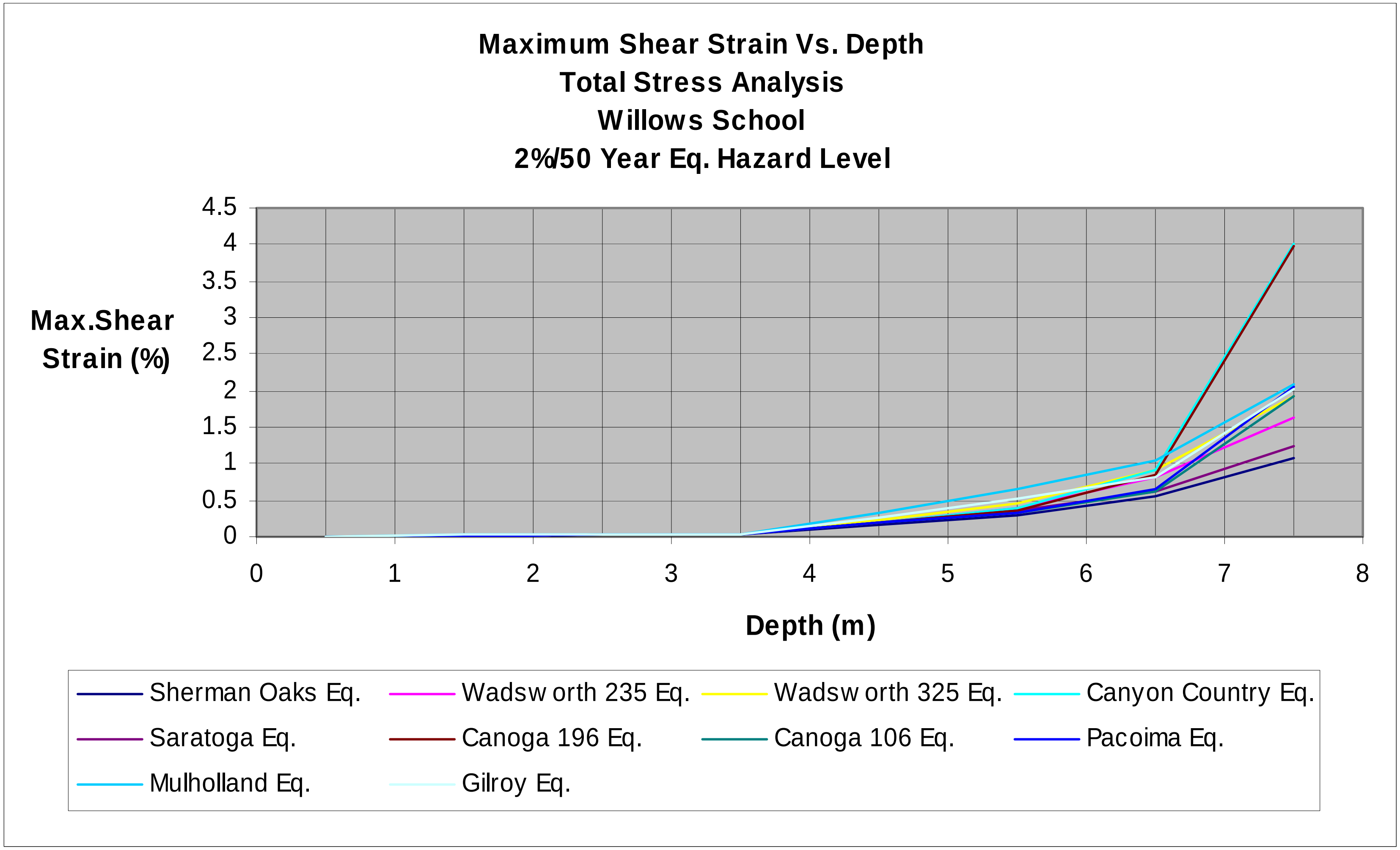
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(b) Graph - maximum shear stress versus depth


Maximum Shear Stress Vs. Depth
Total Stress Analysis
Willows School
2%/50 Year Eq. Hazard Level
Maximum Shear Stress (psf)
0
500
1000
1500
2000
2500
3000
3500
4000
Depth (m)
0
1
2
3
4
5
6
7
8
Sherman Oaks Eq.
Wadsw orth 235 Eq.
Wadsw orth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.
Shear Strength Profile

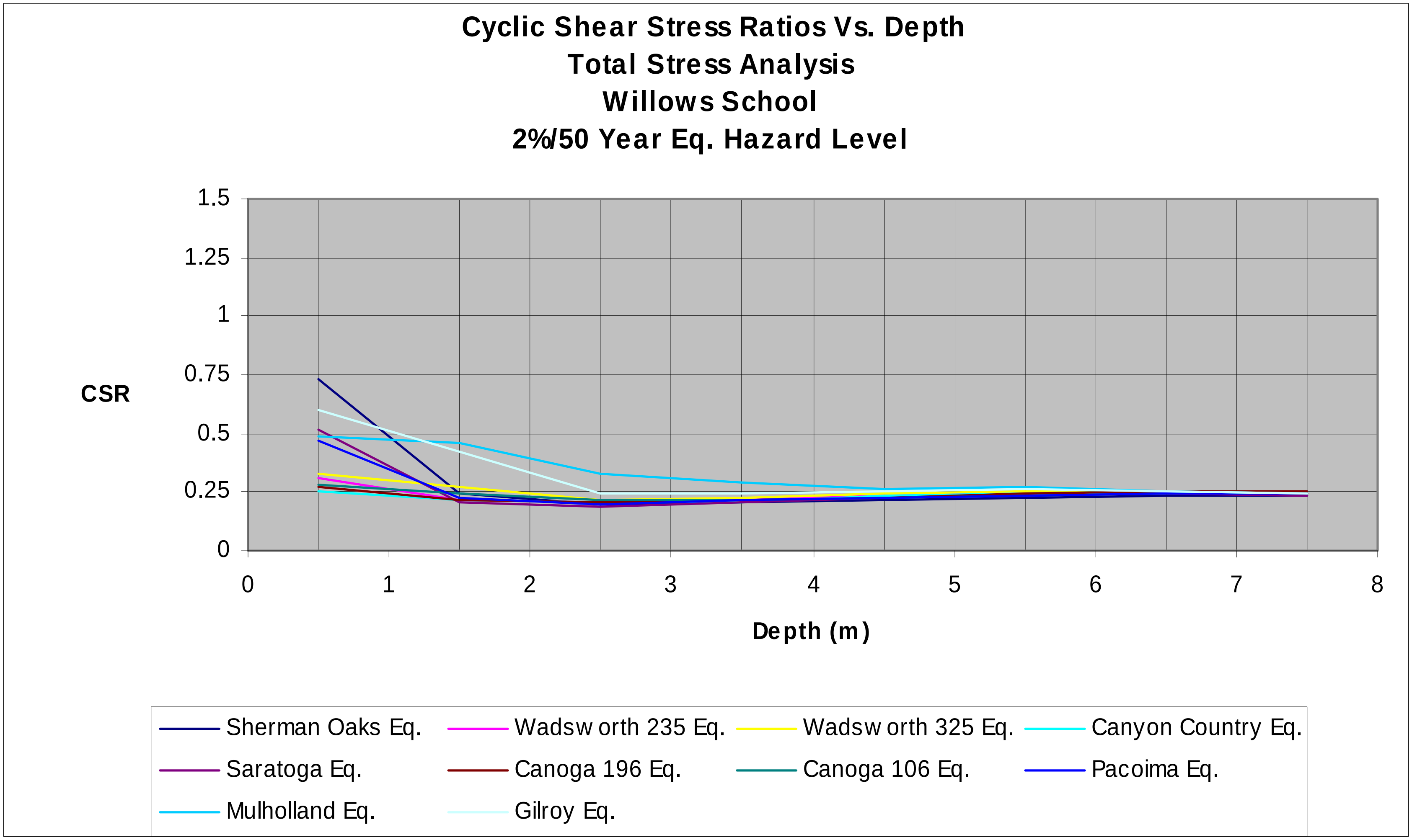
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(c) Graph - maximum shear strain versus depth


Maximum Shear Strain Vs. Depth
Total Stress Analysis
Willows School
2%/50 Year Eq. Hazard Level
Max.Shear Strain (%)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

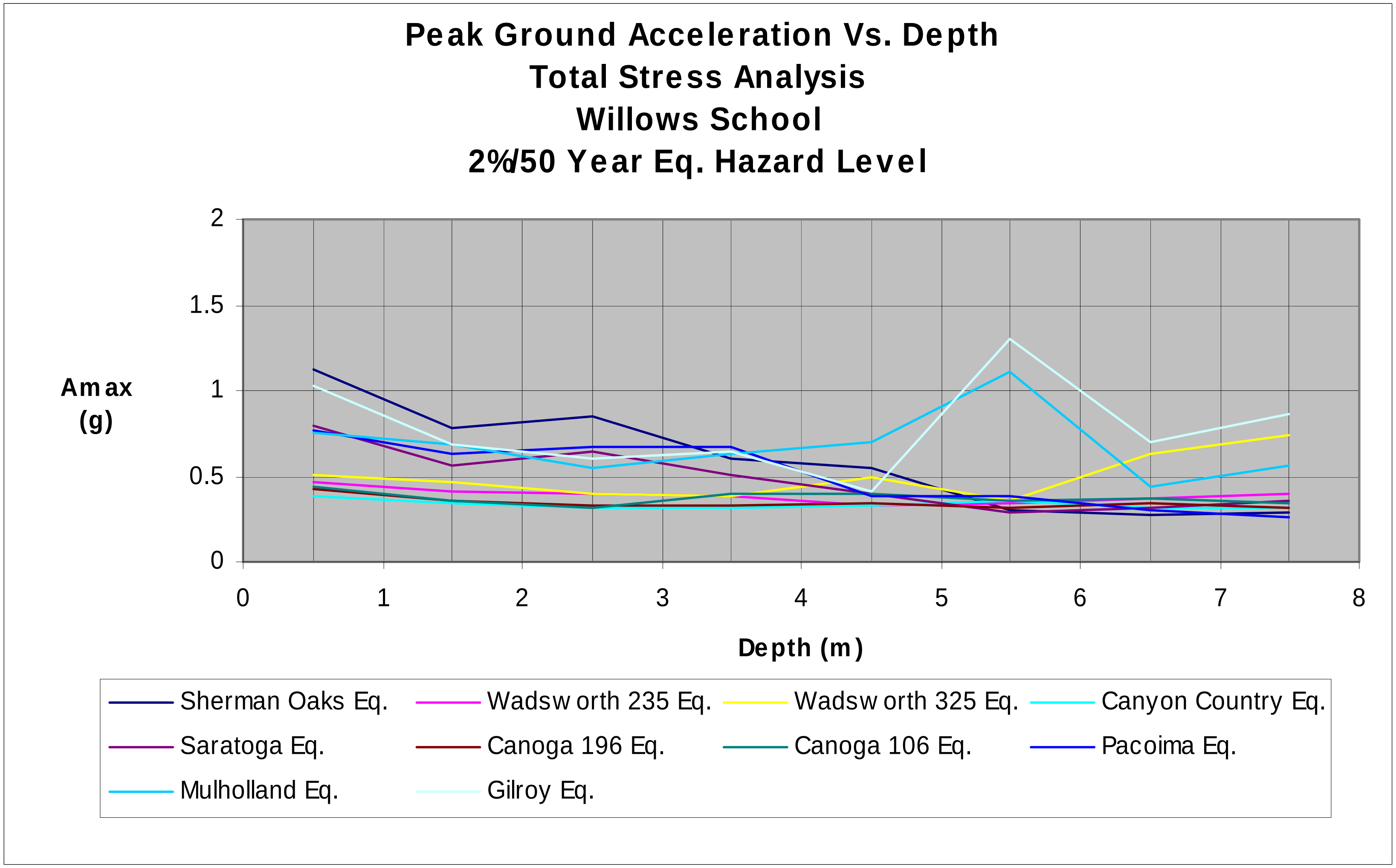
(1) <u>MEG Consulting Ltd. Report (DESRA Analysis)</u>

(d) Graph - cyclic shear stress ratio versus depth


Cyclic Shear Stress Ratios Vs. Depth
Total Stress Analysis
Willows School
2%/50 Year Eq. Hazard Level
CSR
1.5
1.25
1
0.75
0.5
0.25
0
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

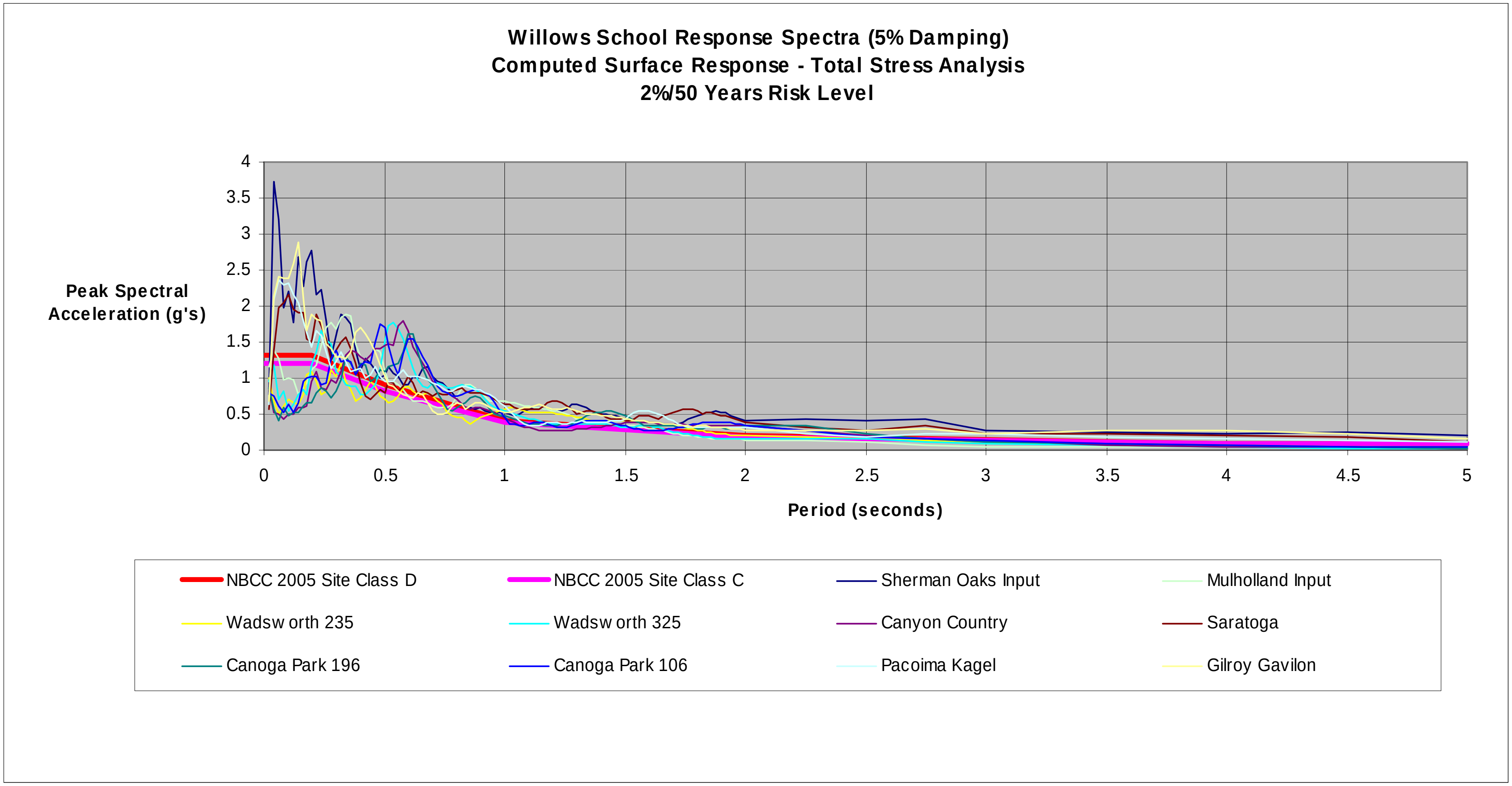
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(e) Graph - peak ground acceleration versus depth


Peak Ground Acceleration Vs. Depth
Total Stress Analysis
Willows School
2%/50 Year Eq. Hazard Level
Amax (g)
0
0.5
1
1.5
2
0
1
2
3
4
5
6
7
8
Depth (m)
Sherman Oaks Eq.
Wadsworth 235 Eq.
Wadsworth 325 Eq.
Canyon Country Eq.
Saratoga Eq.
Canoga 196 Eq.
Canoga 106 Eq.
Pacoima Eq.
Mulholland Eq.
Gilroy Eq.

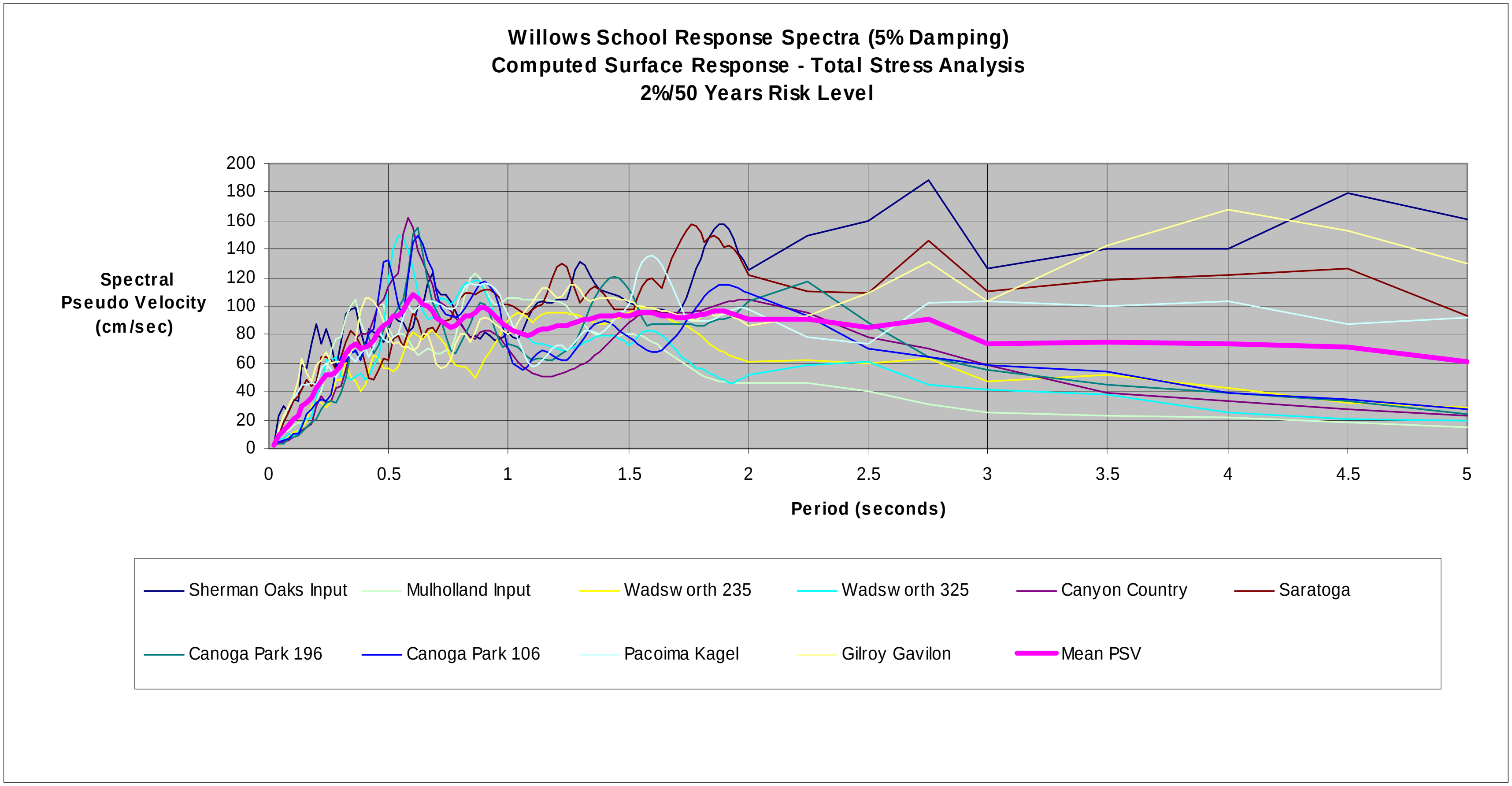
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(f) Graph - surface acceleration response spectra


Willows School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Peak Spectral Acceleration (g's)
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
NBCC 2005 Site Class D
NBCC 2005 Site Class C
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon

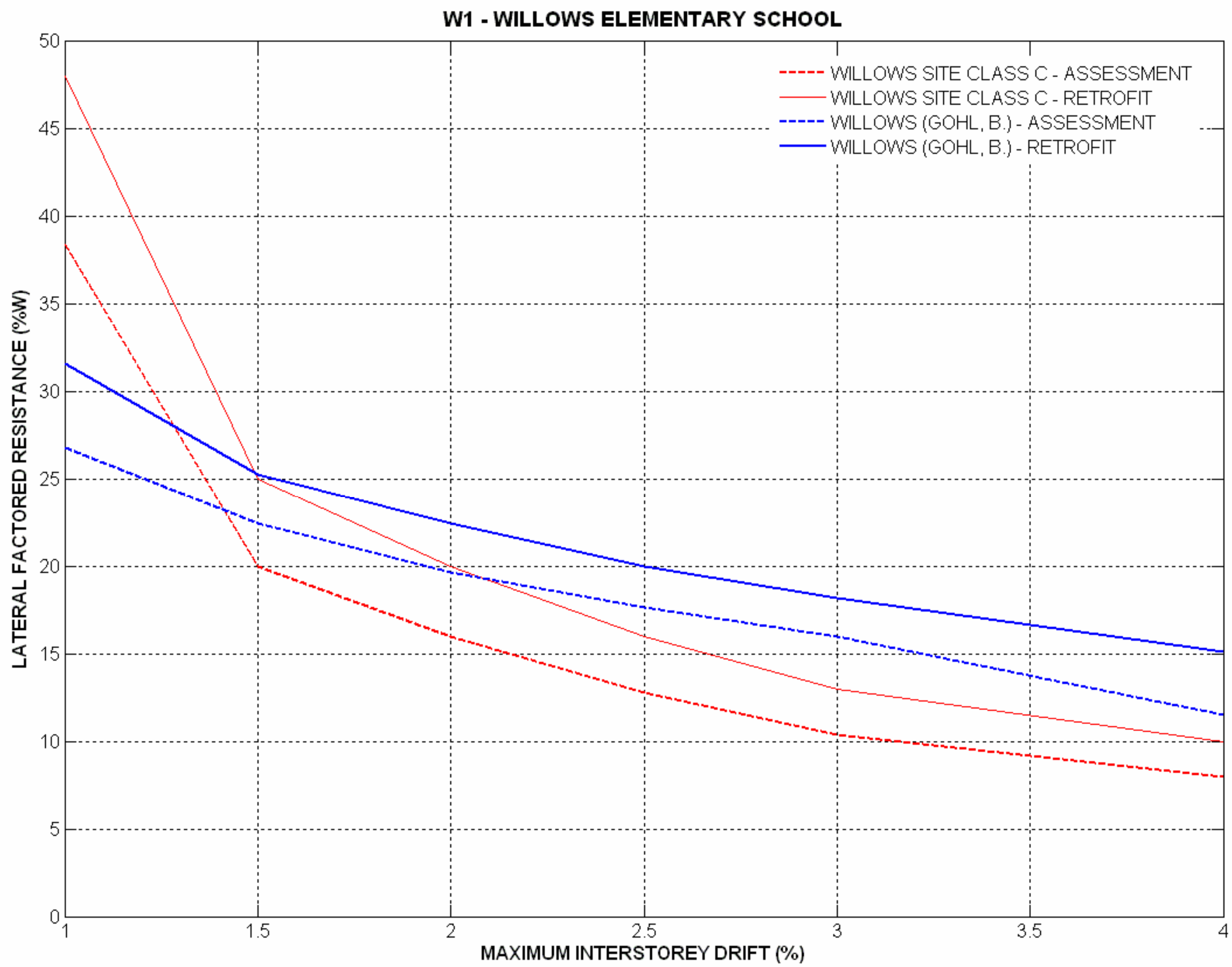
(1) MEG Consulting Ltd. Report (DESRA Analysis)

(g) Graph - surface velocity response spectra


Willows School Response Spectra (5% Damping)
Computed Surface Response - Total Stress Analysis
2%/50 Years Risk Level
Spectral Pseudo Velocity (cm/sec)
200
180
160
140
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Period (seconds)
Sherman Oaks Input
Mulholland Input
Wadsw orth 235
Wadsw orth 325
Canyon Country
Saratoga
Canoga Park 196
Canoga Park 106
Pacoima Kagel
Gilroy Gavilon
Mean PSV

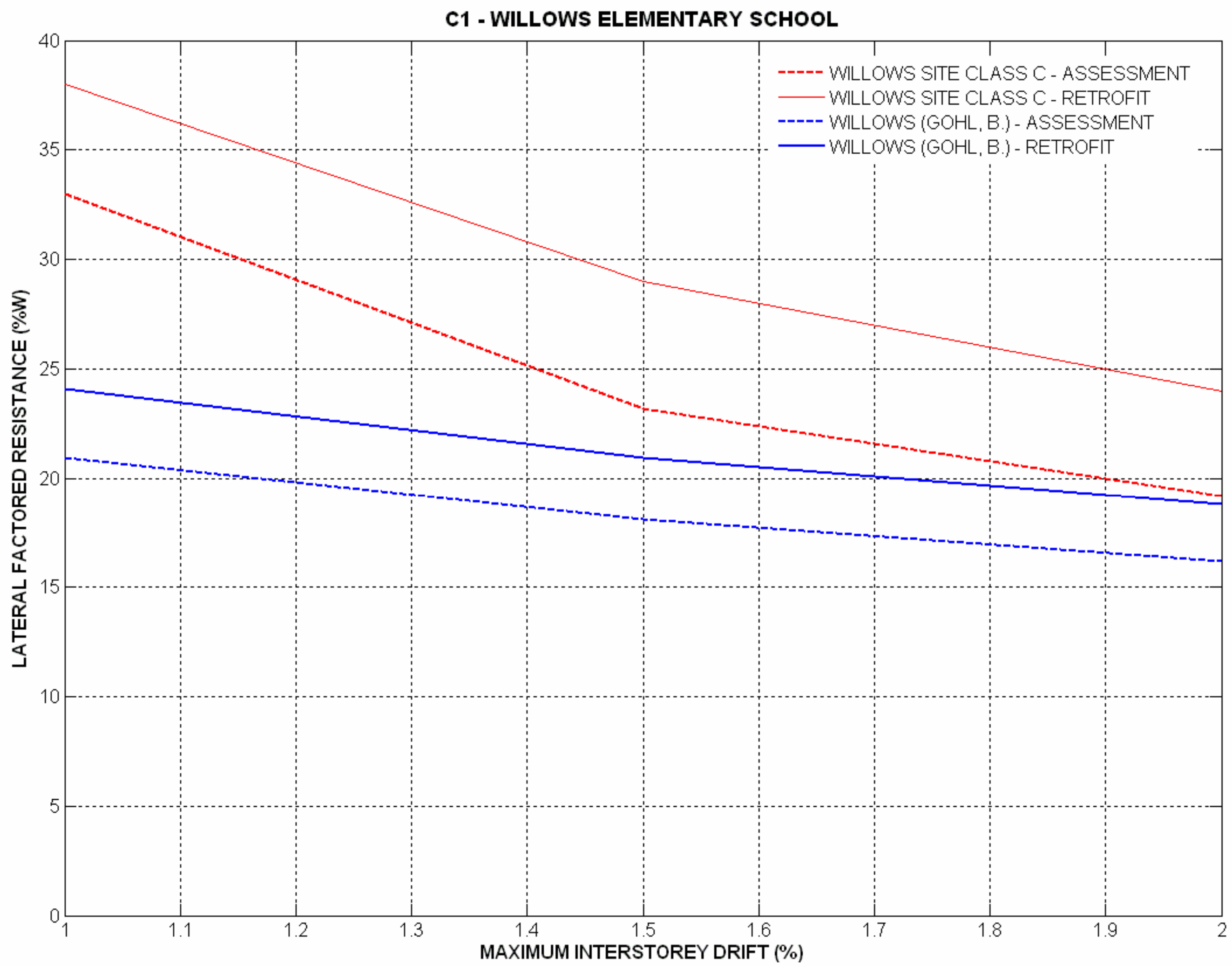
(2) UBC Report (Structural Analysis)

(a) Graph - lateral factored resistance versus drift for W-1


W1 - WILLOWS ELEMENTARY SCHOOL
WILLOWS SITE CLASS C - ASSESSMENT
WILLOWS SITE CLASS C - RETROFIT
WILLOWS (GOHL, B.) - ASSESSMENT
WILLOWS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
45
50
1
1.5
2
2.5
3
3.5
4

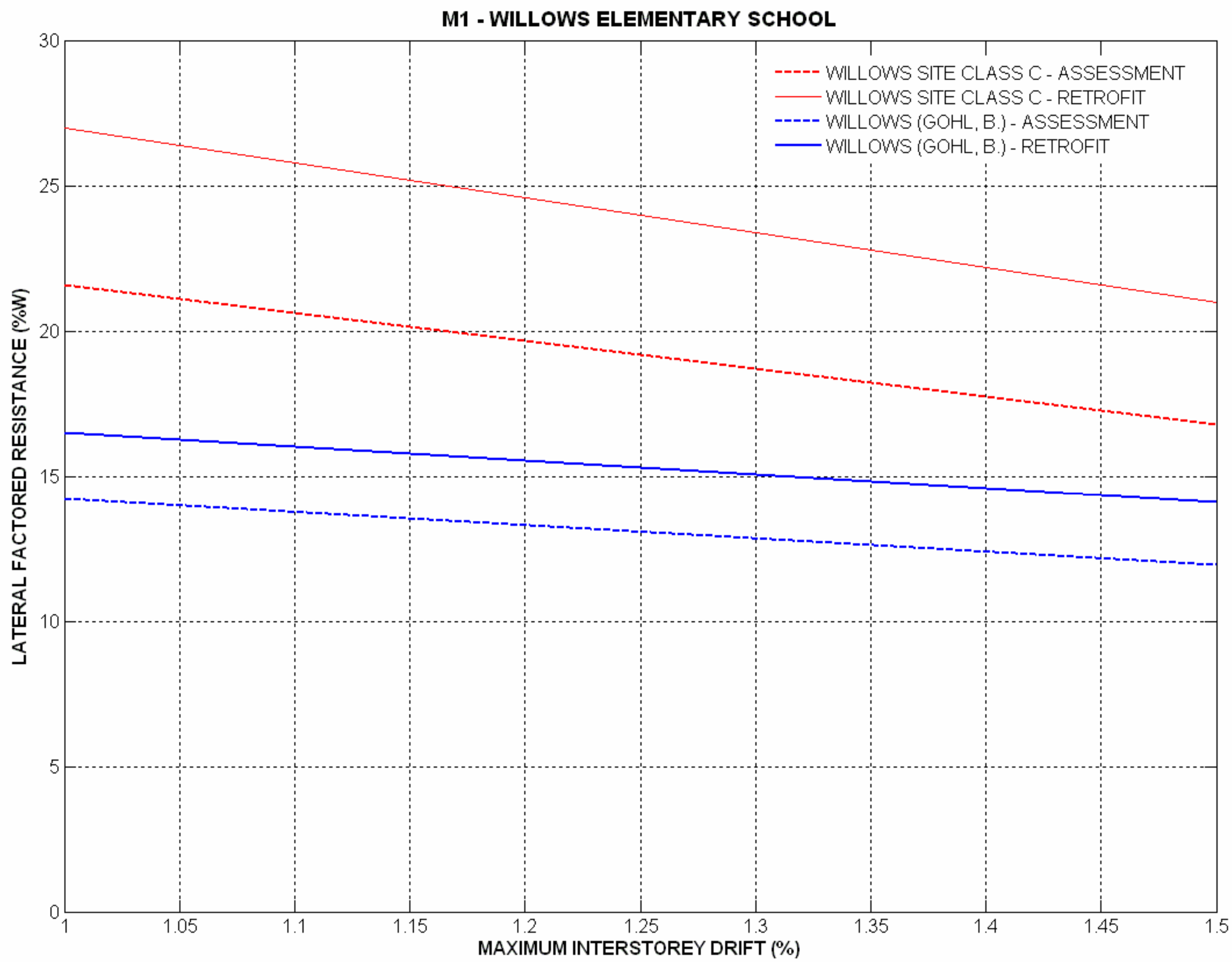
(2) UBC Report (Structural Analysis)

(b) Graph - lateral factored resistance versus drift for C-1


C1 - WILLOWS ELEMENTARY SCHOOL
WILLOWS SITE CLASS C - ASSESSMENT
WILLOWS SITE CLASS C - RETROFIT
WILLOWS (GOHL, B.) - ASSESSMENT
WILLOWS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
35
40
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2

(2) UBC Report (Structural Analysis)

(c) Graph - lateral factored resistance versus drift for M-1



M1 - WILLOWS ELEMENTARY SCHOOL
WILLOWS SITE CLASS C - ASSESSMENT
WILLOWS SITE CLASS C - RETROFIT
WILLOWS (GOHL, B.) - ASSESSMENT
WILLOWS (GOHL, B.) - RETROFIT
LATERAL FACTORED RESISTANCE (%W)
MAXIMUM INTERSTOREY DRIFT (%)
0
5
10
15
20
25
30
1
1.05
1.1
1.15
1.2
1.25
1.3
1.35
1.4
1.45
1.5