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Earthquake Ground Motions at Soft Soil Sites

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INTRODUCTORY REMARKS

This Special Session of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics has been dedicated to the Memory of the late Professor H. Bolton Seed. I had the privilege and the pleasure to work closely with him for about 25 years. For that reason, I was asked to provide in the opening remarks to this Special Session a glimpse of the "Life and Philosophy" of Professor H. B. Seed. It is with a mixture of sadness and a sense of loss, coupled with pride in the man we are honoring today, that I offer the following few comments regarding the life and philosophy of Professor Seed.

He was born in Bolton, England on 22 August 1922. He studied at Kings College, London University, receiving a B. S. in Civil Engineering in 1944 and a Ph. D. in Structural Engineering in 1947. Following two years as Assistant lecturer at Kings College, he came to the USA to study Soil Mechanics at Harvard with Karl Terzaghi and Arthur Casagrande.

In 1950, he came to the University of California at Berkeley where he stayed until his untimely death in April 1989.

Over that span of almost 40 years, he was engaged as a teacher, a public servant, a researcher in geotechnical engineering and as a consultant to numerous companies and governmental agencies on projects and technical issues throughout the world. He built the program in Geotechnical Engineering at UC Berkeley into one of the best (if not the best) and largest in the world by attracting and keeping excellent faculty colleagues and by attracting and educating excellent students.

As a Teacher, he was superb. He received all the honors a superb teacher receives both from his own institution and from many other campuses throughout the USA and other parts of the world.

As a Public Servant, he devoted large amounts of time and energy to the Technical Societies, to State and Federal Committees and Commissions. In all these assignments, he gave unselfishly of his time and talent and always made significant contributions.

As a Researcher, he had an enormous impact on every area of research activity in which he worked. In the 1950's and early 1960's he pioneered research on pile foundations, soil compaction, influence of methods of compaction on soil structure and on analytical methods of pavement design.

He created the field of Geotechnical Earthquake Engineering in the early 1960's and guided it throughout the remainder of his career. He pioneered research and applications in site response, dynamic soil properties, liquefaction, earthquake ground motions, soil-structure interaction, earthfill and rockfill dams ... etc (ie, just about each and every topic covered in this conference). This research resulted in the development of evaluation and design methods that revolutionized many aspects of engineering practice and thinking. One area of research that Professor H. Bolton Seed thrived in was the full utilization of case histories to create, calibrate or modify an evaluation procedure.

As a Consultant, he was involved in some of the most significant and tough projects throughout the USA and in many parts of the world.

For all his contributions, Professor Seed was honored on numerous occasions. He received 14 awards from the American Society of Civil Engineers (ASCE) and was selected to give several distinguished lectures, such as the Terzaghi, Rankine, Martin Kapp and many others. He became a member of the Academy of Engineering in 1970 and of the Academy of Science in 1986. He was elected an honorary member of ASCE and of the Earthquake Engineering Research Institute (EERI). He received the first honorary doctorate degree in engineering from the Ecole Nationale des Pontes et Chausses in Paris In 1987.

When we read the numerous writings of Professor Seed, we begin to glimpse the trend that is present in any topic he tackled. First and foremost, he always chose an important and relevant topic to work on. He started with the overall issue, divided it into solvable segments (never losing sight of the full picture, however), incorporated the lessons that can be learned from relevant case histories, and augmented each step with "good quality" analyses to arrive at an evaluation and design procedure for the issue at hand. At each step he tried to keep things as simple as possible, but 'no simpler' (as he wrote in his Rankine Lecture quoting from Albert Einstein).

An excellent example of this process is his work in the evaluation of earth- and rock-fill dams due to earthquake loading conditions. The basic issues were laid out in his paper titled "A Method for Earthquake Resistant Design of Earth Dams " which was published in the Soil Mechanics and Foundation Engineering Journal of ASCE in January 1966. These were followed by the evaluation of the failure of the Sheffield Dam and the slides in the San Fernando dams. Several studies later (including analytical and experimental developments as well as utilization of additional case histories) together with the re-evaluation of the slides of the lower San Fernando Dam in 1988 completed all the steps he had outlined in 1966.

This is only a glimpse that provides a slight insight into the approach that Professor Seed followed in his research. He wrote in his Rankine Lecture "it is extremely important that we take every opportunity that Nature provides to continually refine the procedures". This best expresses the importance he attached to case histories.

At the Memorial Symposium held in May 1990 'in recognition of 40 years of Extraordinary Contributions by Professor H. Bolton Seed to the Teaching, Research and Practice of Geotechnical Engineering', Professor J. K. Mitchell (who was a long time colleague of H. B. Seed and the Chairman of that Symposium) wrote "Harry Seed was truly a giant of his generation and all of us are richer for having had him among us." This statement truly reflects the feelings of all of us who knew Harry well.

This Special Session of this Conference is dedicated to summarizing the lessons learned from the recent Loma Prieta earthquake. This earthquake has been designated the "geotechnical earthquake" and two issues were particularly highlighted in this earthquake. One issue pertained to liquefaction, which was wide-spread in San Francisco, in Oakland and many other locations from Moss Landing in the south to Richmond in the north. The other issue was the variations of recorded ground motions with the local site conditions.

Professor H. Bolton Seed made giant contributions to both issues. Thus, it is very appropriate that this Special Session of this Conference is dedicated to the Memory of Professor H. Bolton Seed and to honoring Professor Seed and his contributions to the art and science of geotechnical earthquake engineering.

The remainder of my presentation at this Special Session is devoted to providing an assessment of the recordings obtained during this earthquake, with particular emphasis to those recorded at soft soil sites.

GROUND MOTIONS RECORDED DURING THE LOMA PRIETA EARTHQUAKE

The Loma Prieta earthquake occurred on 17 October 1989 at 5:04 pm PDT along a segment of the San Andreas fault in the Santa Cruz Mountains. The extent of the rupture zone was about 45 km. The hypocenter of the earthquake was at a depth of about 18 km; the rupture plane dips to the southwest at about 70 degrees thus the epicenter is located several kilometers west of the San Andreas fault trace (Fig. 1). The earthquake was assigned a surface wave magnitude, $M_S = 7.1$, and a moment magnitude, $M_W = 7$. The rupture was bilateral and thus the duration of shaking was only about one half what might be expected during a magnitude 7 earthquake.

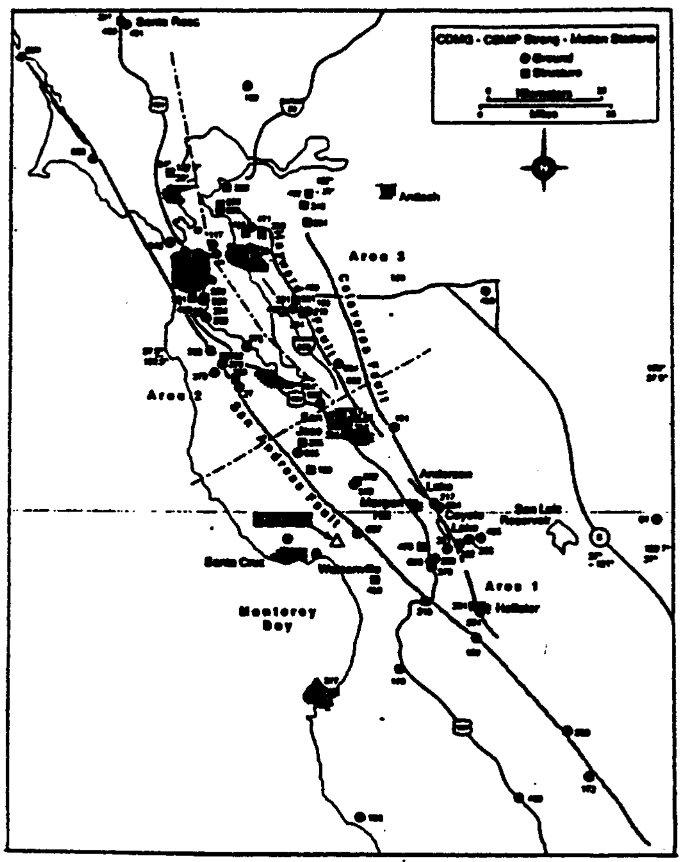

Fig. 1 Locations of CSMIP Stations that Recorded the 17 October 1989 Earthquake (from Shakal et al, 1989)

Ground motion were recorded at stations installed by the California Strong Motion Instrumentation Program (CSMIP) [Shakal et al (1989), CSMIP(1989) and Huang et al (1990)] and at stations installed by the U.S. Geological Survey (USGS) [Maley et al (1989)]. Figure 1 shows the location of the CSMIP stations that recorded this earthquake, together with the segment of the San Andreas fault on which this earthquake occurred.

The peak horizontal accelerations recorded at 33 rock and 70 soil sites (other than soft soil sites) are presented in Fig. 2a; the peak horizontal accelerations recorded at 9 soft soil sites are presented in Fig. 2b. (Note that both horizontal components from each station are shown in Figs. 2a and 2b). Also shown in both Fig. 2a and Fig. 2b are the median and the median ± one standard deviation from attenuation studies at rock sites for $M_W = 7$ (using the equations presented later in this paper). Examination of Fig. 2a indicates that the variations of peak horizontal accelerations at rock and at soil sites (other than those underlain by soft soils) can be reasonably estimated using typical attenuation relations. The peak horizontal accelerations recorded at soft soil sites, however, are significantly greater than those recorded at the other sites at distances of about 45 to 100 km as shown in Fig. 2b.

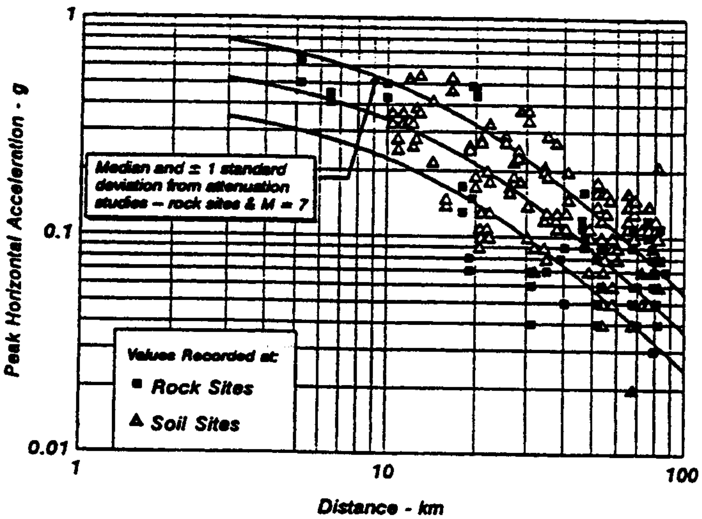

Fig. 2a Peak Horizontal Accelerations of Motions Recorded during the 1989 Loma Prieta Earthquake at Rock and Soil Sites (other than soft soil sites)

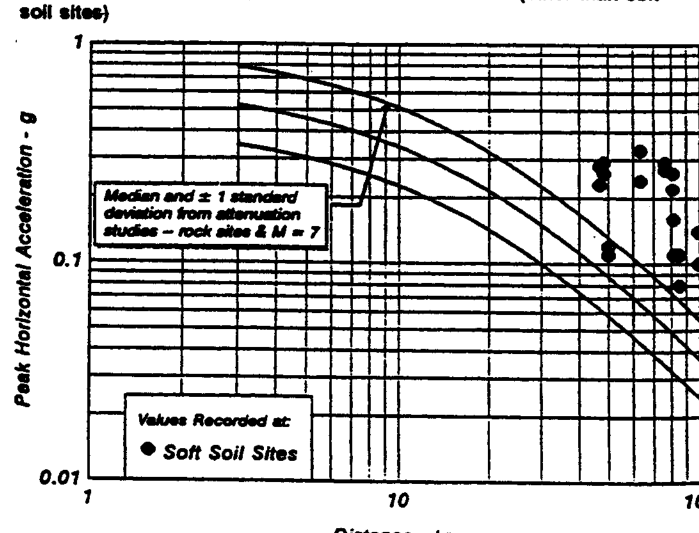

Fig. 2b Peak Horizontal Accelerations of Motions Recorded during the 1989 Loma Prieta Earthquake at Soft Soil Sites

The peak horizontal acceleration recorded at these soft sites during this earthquake ranged from slightly less than 0.1 g to a little over 0.3 g at distances ranging from about 45 to about 100 km. At comparable distances, the peak horizontal accelerations at rock sites ranged from about 0.05 g to about 0.18 g (Fig. 2a). These ranges are presented in Fig. 3 to illustrate the fact that there are no recorded data at soft soil sites where the peak accelerations at nearby rock sites exceed 0.2 g. Thus, to estimate levels of shaking at such soft soil sites for higher levels of shaking, it is necessary at this time to utilize results of analytical procedures to extend the range shown in Fig. 3.

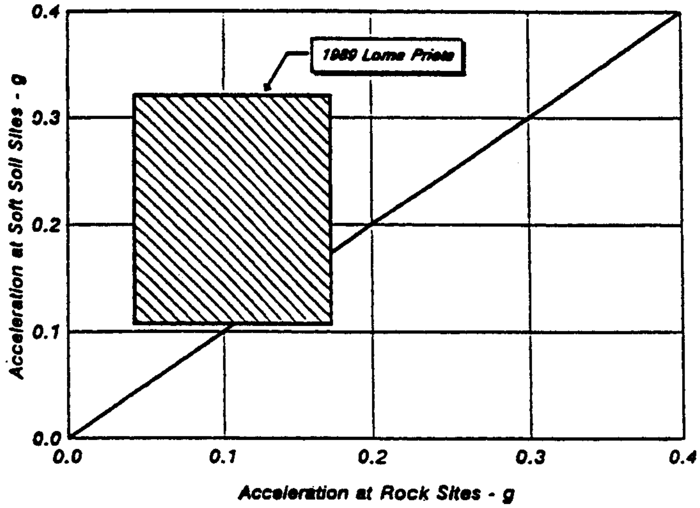

Fig. 3 Approximate Range of Peak Accelerations Recorded at Soft Soil Sites and at Nearby Rock Sites during the Loma Prieta Earthquake

GROUND RESPONSE ANALYSES

Ground response analyses, incorporating equivalent linear procedures, were conducted for several of the soft soil sites (eg, Idriss, 1990; Dickenson et al, 1991; Idriss et al, 1991) using the computer program SHAKE (Schnabel et al, 1972). These analyses showed reasonable agreement between recorded and calculated motions (for peak accelerations as well as for spectral ordinates) and hence offered a reasonable means for extending the range shown in Fig. 3. For example, Idriss (1990) had proposed the curve shown in Fig. 4 as a median relationship for use in empirical correlations. On-going research, at several institutions, using nonlinear analytical procedures, have shown similar results to those depicted in Fig. 4.

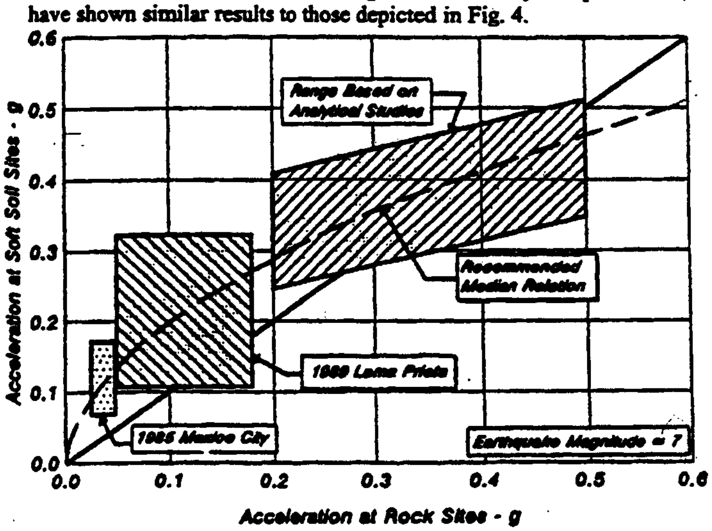

Fig. 4 Variations of Peak Horizontal Accelerations at Soft Soil Sites with Accelerations at Rock Sites

Selection of Input Rock Motions for Use in Site Response Studies

When conducting site response studies, it is essential that the appropriate input rock motion be selected for use in such studies. The key parameters that seem to affect the output include both the peak acceleration and the frequency content of this input rock motion. These parameters depend on the magnitude of the earthquake under consideration as well as the distance of the site to the earthquake source. Seed et al (1969) suggested the use of "predominant period" to represent the frequency content of the input rock motion. The predominant period is defined as the period at which the response spectrum (typically, at spectral damping of about 0.05) of a given accelerogram has the largest peak. For example, the two accelerograms shown in Fig. 5a were both obtained at rock sites during the Loma Prieta earthquake. The top accelerogram was recorded at Rincon Hill in San Francisco at a distance of about 80 km, and the lower accelerogram was recorded in Santa Cruz at a distance of about 20 km from the source. As can be readily noted in Fig. 5a, the two accelerograms appear to have significantly different frequency characteristics in addition to having significantly different peak accelerations.

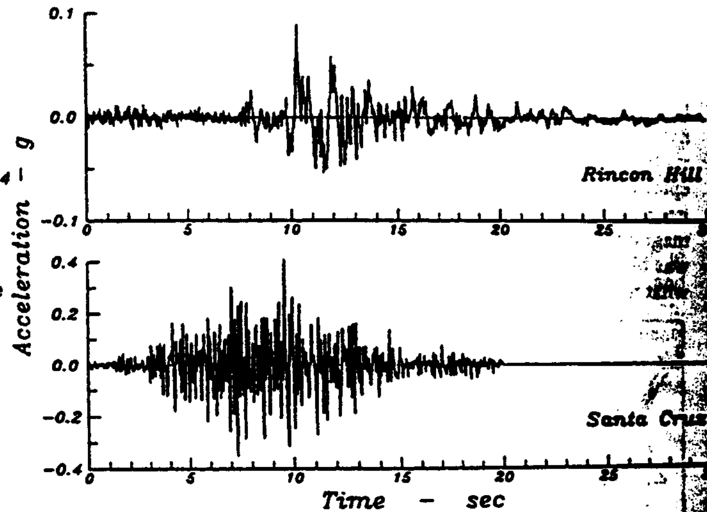

Fig. 5a East–West Components of Accelerograms Recorded at Santa Cruz and at Rincon Hill in San Francisco

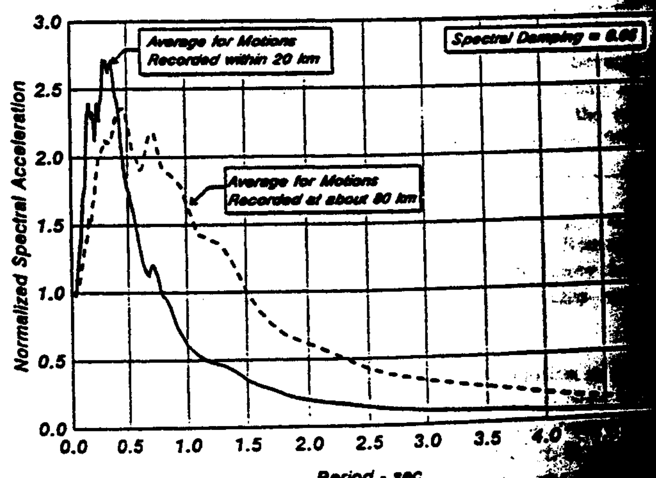

Fig. 5b Average Normalized Spectral Accelerations (Sp... for Rock Motions Recorded during the 1989 Loma Prieta...

Similar characteristics were observed for other rock accelerograms near the source (within about 20 km) and for other accelerograms recorded at distances of about 80 km. The average normalized spectral accelerations (ie, spectral acceleration divided by the peak acceleration) for these two sets of recordings are shown in Fig. 5b. (The accelerograms recorded at Corralitos, at Gilroy No. 1 and at Santa Cruz were used to obtain the average spectral shape for motions near the source. The accelerograms recorded at Diamond Heights, Rincon Hill, Pacific Heights, Telegraph Hill, Golden Gate, Cliff House, Presidio and Yerba Buena were used to obtain the average spectral shape for motions at a distance of about 80 km).

The predominant period for the recordings obtained within 20 km of the source is about 0.32 sec and that for the recordings at about 80 km is about 0.48 sec. The range of predominant periods at these distances and the average values together with the relationship originally proposed by Seed et al (1969) for M = 7 are shown in Fig. 6. The results presented in this figure indicate that the values initially proposed by Seed et al (1969) are reasonable at close distances from the source and that there is a significant distance-dependence of frequency content (as expressed by the predominant period) of rock motions. Based on the results obtained for Loma Prieta (as shown in Fig. 6), the predominant periods originally proposed by Seed et al (1969) as a function of magnitude and distance were slightly modified and are presented in Fig. 7.

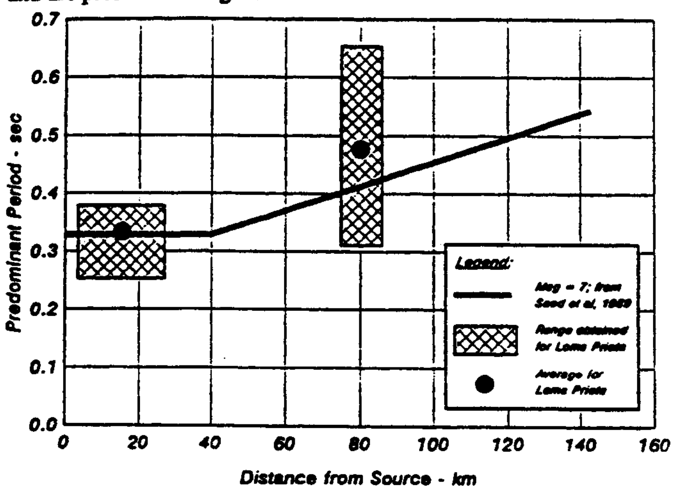

Fig. 6 Estimated Predominant Periods for Rock Motions - Earthquake Magnitude = 7

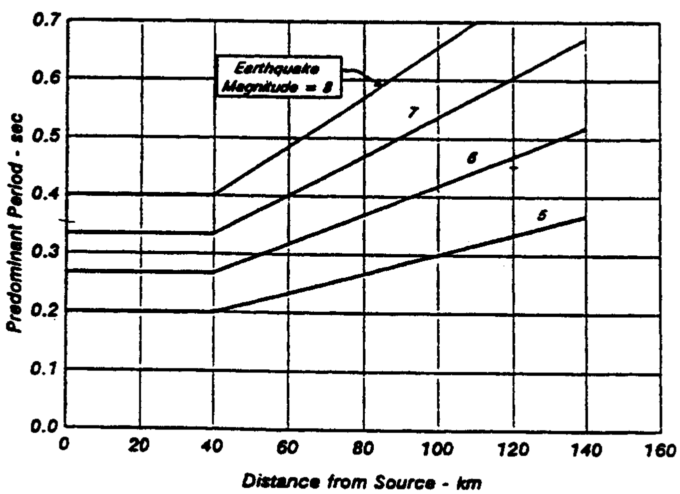

Fig. 7 Variations of Predominant Periods for Rock Motions with Distance and Earthquake Magnitude

Estimates of Response of Soft Soil Sites at Higher Levels of Shaking

Rock motions obtained at a distance of about 80 km were used to calculate the response at several soft sites in the San Francisco-Oakland area for levels of shaking up to about 0.25 g in rock. For higher levels of input rock motion, the earthquake was considered to occur closer to the site and recordings at rock sites obtained at closer distances were used for these calculations (including the accelerograms recorded at Santa Cruz and those recorded at Gilroy No. 1). The results of these analyses are presented in Fig. 8; note that these and similar results were used in developing the range designated as "range based on analytical studies" in Fig. 4. The results shown in Fig. 8 are for an earthquake magnitude of 7.

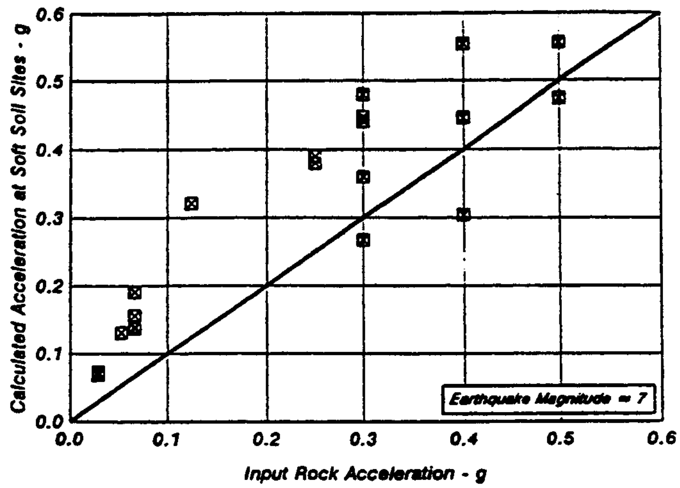

Fig. 8 Calculated Peak Accelerations at Soft Soil Sites Earthquake Magnitude = 7

Estimates of Response of Soft Soil Sites during Smaller Magnitude Earthquakes

The response at soft soil sites during an earthquake having a magnitude of about 5½ are presented in Fig. 9. The guidelines discussed above for the selection of appropriate input rock motions and the predominant periods for a magnitude 5½ were used in these analyses. (The accelerogram recorded at Golden Gate Park during the 1957 Daly City earthquake as well as somewhat modified accelerograms recorded near the source during the Loma Prieta earthquake were used in these analyses).

Based on the results shown in Fig. 9, an average relationship for the variation of peak horizontal acceleration at a soft soil site as a function of peak input rock acceleration was developed for an earthquake having a magnitude, M = 5½ and is presented in Fig. 10a. Also shown in Fig. 10a is the average curve developed for M = 7 (from Fig. 4). The amplification ratios (ie, the ratios of peak accelerations at a soft soil site divided by the corresponding peak accelerations at a nearby rock site) for each magnitude are presented in Fig. 10b. The amplification ratios for M = 7 shown in Fig. 10b are very similar to those reported by Jarpe et al (1989) based on measurements at Treasure Island and at Yerba Buena during the Loma Prieta earthquake and aftershocks. Similar amplification ratios were also reported by Borcherdt and Glassmoyer (1990) based on recordings from the Loma Prieta earthquake and aftershocks.

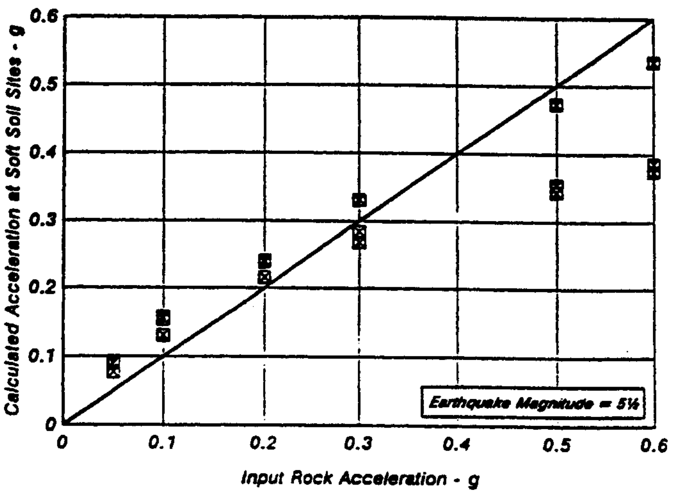

Fig. 9 Calculated Peak Accelerations at Soft Soil Sites
Earthquake Magnitude = 5½

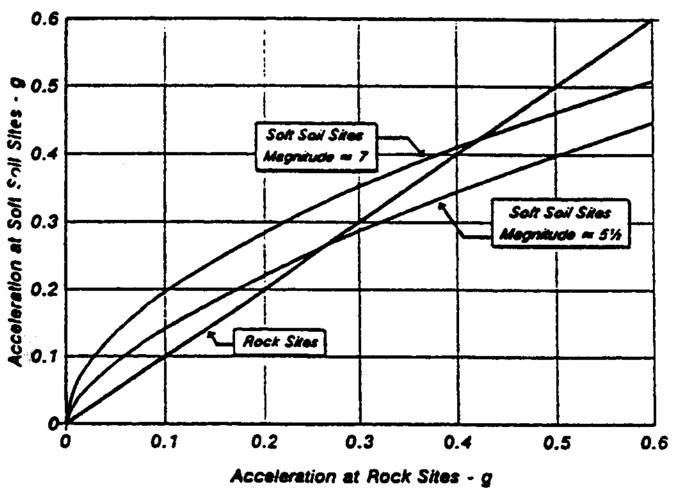

Fig. 10a Magnitude-Dependence Variations of Peak Horizontal Accelerations at Soft Soil Sites

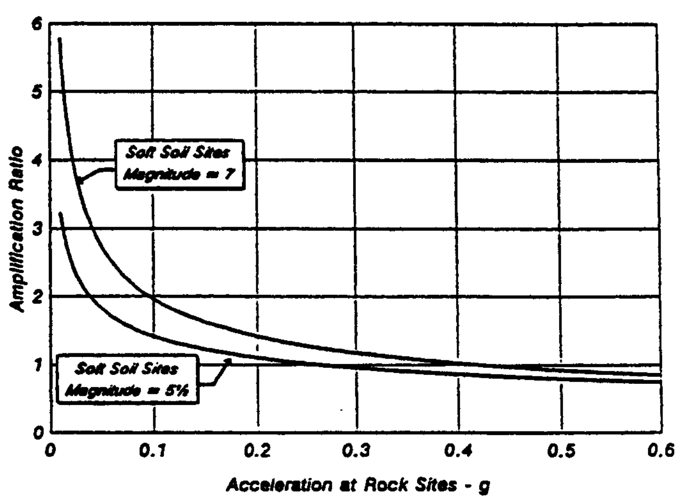

Fig. 10b Magnitude-Dependence Variations of Amplification of Peak Horizontal Accelerations at Soft Soil Sites

The curves presented in Figs. 10a and 10b were used in conjunction with median attenuation relationships for rock motions to estimate median peak horizontal accelerations at soft soil sites as discussed below.

MEDIAN PEAK HORIZONTAL ACCELERATIONS AT SOFT SOIL SITES

There are several attenuation relationships for estimating median peak accelerations at rock and at stiff soil sites. For example, Joyner and Boore (1988) include several of these relationships that were available as of 1988. Since that time, additional relationships have been proposed (eg, Tsai et al, 1990; Campbell, 1991; Idriss, 1991; Youngs et al, 1991). For example, the equations derived by Idriss (1991) for the median acceleration, a_r (in g's), at rock sites are the following:

For M ≤ 6

$$Ln\,(a_r) = -0.05 + exp(2.261 - 0.083{*}M) - exp(1.602 - 0.142{*}M){*}Ln(R + 20) \quad \textbf{[Eq. 1a]}$$

and, for M > 6

$$Ln\,(a_r) = -0.05 + exp(3.477 - 0.284{*}M) - exp(2.475 - 0.286{*}M){*}Ln(R + 20) \quad \textbf{[Eq. 1b]}$$

in which *Ln* is the natural logarithm, *exp* is the exponential function, M is essentially moment magnitude M_W and R is the closest distance to the source in km. The above equations were derived for earthquakes occurring on strike slip sources; for those occurring on reverse faults, the above median values should be increased by a factor of about 1.22.

The standard error terms, ε, associated with the above equations are magnitude-dependent and are given by the following expressions:

$$\varepsilon = 1.39 - 0.14{*}M \quad \text{for } M < 7\tfrac{1}{4} \quad \textbf{[Eq. 2a]}$$
$$\varepsilon = 0.38 \quad \text{for } M \geq 7\tfrac{1}{4} \quad \textbf{[Eq. 2b]}$$

These standard error terms are similar to those obtained by Tsai et al (1990) and are those derived by Youngs et al (1991).

Equations for estimating the median peak acceleration at soft soil sites can be derived by combining the above equations (Eqs. 1a and 1b) for median rock acceleration with the amplification ratios presented in Fig. 10b. The resulting equations for the median peak acceleration, a_s (in g's), at soft soil sites are the following:

For M ≤ 6

$$Ln\,(a_s) = exp(1.673 - 0.137{*}M) - exp(1.285 - 0.206{*}M){*}Ln(R + 20) \quad \textbf{[Eq. 3a]}$$

and, for M > 6

$$Ln\,(a_s) = exp(2.952 - 0.350{*}M) - exp(2.015 - 0.328{*}M){*}Ln(R + 20) \quad \textbf{[Eq. 3b]}$$

Standard Error Terms for Peak Horizontal Accelerations at Soft Soil Sites

Because of the limited amount of recorded data at low levels of shaking and lack of recorded data at high levels of shaking at soft soil sites, it is not possible to obtain a direct estimate of the standard error terms for the motions at these sites. To arrive at a reasonable estimate of the standard error terms, use is made of the limiting value of peak horizontal accelerations at these soft soil sites. This limiting value is obtained by equating the available shear strength at a particular depth in a soft soil layer to the shear stress that may be induced by the ground motion at that depth in the layer. (Note that the procedure outlined below does not take into account the potential for liquefaction in overlying or underlying soil layers at the site under consideration.)

Young Bay Mud underlies the soft soil sites considered in this paper. Most of the Young Bay Mud in the San Francisco Bay area is nearly normally consolidated to slightly over-consolidated; the over-consolidation ratio typically is less than about 1.5. The undrained shear strength, s_u, under static loading conditions is related to the effective vertical overburden pressure, p', by the following relationship: $s_u/p' = 0.3$ for a normally consolidated Young Bay Mud. Under dynamic conditions, this ratio may be increased by a factor of 1.3 to 1.5 to reflect the effect of rate of loading and by a factor of about 1.4 if the over-consolidation ratio is as high as 1.5. Thus, the undrained shear strength of Young Bay Mud under dynamic loading conditions could range from about $s_u/p' = 0.4$ to about $s_u/p' = 0.64$, with a reasonable overall value of about $s_u/p' = 0.55$.

The shear stress induced by the earthquake ground motions at a given depth in a soil profile can be estimated using a site response analysis or the equation originally derived by Seed and Idriss (1971) for the simplified liquefaction analysis. The latter equation is as follows:

$$\tau_{unf} = 0.65 * \sigma_v * a_{max} * r_d \qquad \text{[Eq. 4]}$$

in which τ_{unf} is the equivalent uniform shear stress induced by the earthquake, σ_v is the total vertical overburden at a given depth below the ground surface, a_{max} is the maximum ground surface acceleration and r_d is a depth correction factor that is equal to about 0.95 at a depth of about 20 ft and 0.85 at a depth of about 40 ft.

For example for a site underlain by 10 ft of fill (total unit weight of 125 pcf) and 40 ft of Young Bay Mud (total unit weight of 96 pcf) and the water table at the bottom of the fill, the following values are obtained at a depth of 20 ft:

$$\sigma_v = 10*125 + 10*96 = 2210 \text{ psf}$$

$$\tau_{unf} = 0.65*2210*0.95*a_{max} = 1365*a_{max} \text{ psf}$$

$$p' = 10*125 + 10*(96 - 64) = 1570 \text{ psf}$$

$$s_u = 0.55*1570 = 864 \text{ psf}$$

Thus, equating the average induced stress and the available shear strength gives a value of $a_{max} = 864/1365 = 0.63$ g. Similarly, at a depth of 40 ft, the corresponding value of a_{max} would be about 0.53 g.

Using other assumptions regarding the thickness of fill over the Young Bay Mud, total thickness of the Young Bay Mud, depth of the water table and total unit weights, the calculated values of peak horizontal acceleration range from about 0.4 g to about 0.7 g. Thus, *at this time a limiting value of 0.6 g is suggested as a first order approximation for use in empirical correlations.* For simplicity, and until additional recordings are obtained at soft soil sites, it is also suggested to use the same magnitude-dependent standard error terms as those given above for rock motions (ie, Eqs. 2a and 2b) with the provision that a limiting value of 0.6 g is used for soft soil sites. (Note that at any given site, a site- specific limiting peak horizontal acceleration can be readily derived using the above procedure or a more detailed site response evaluation to estimate the stresses induced by the earthquake ground motions and an appropriate laboratory testing program to estimate the strength of the soil).

The use of Eqs. 3 for estimating median peak accelerations for M = 7 is illustrated in Fig. 11a. This figure shows the peak accelerations recorded at soft soil sites during the Loma Prieta earthquake and the median and the median ± one standard deviation calculated using Eqs. 3b and 2a with M = 7. Also shown in this figure is the limiting peak acceleration of 0.6 g, which would not limit the calculated median value at any distance for M = 7; the median + one standard deviation for this magnitude earthquake, however, is limited to 0.6 g at distances less than about 10 km.

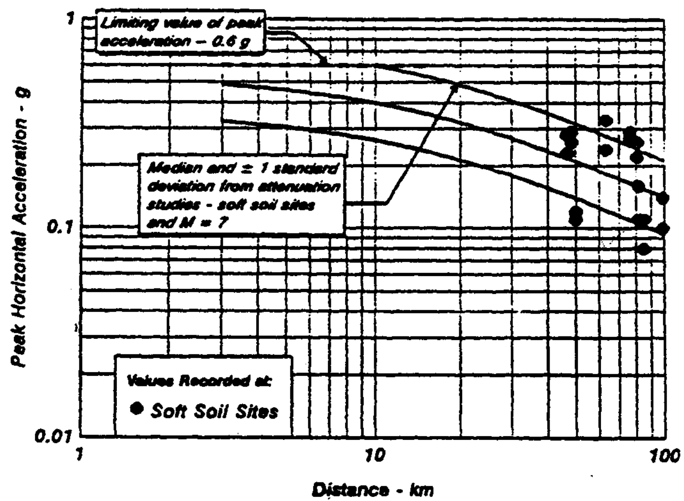

Fig. 11a Comparison of Peak Horizontal Accelerations of Motions Recorded during the 1989 Loma Prieta Earthquake at Soft Soil Sites and Those Derived from Attenuation Studies

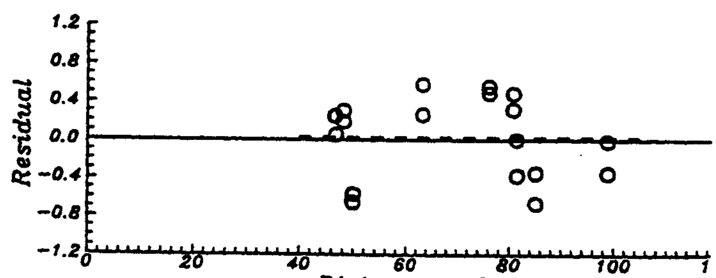

Fig. 11b Residuals for Recorded Horizontal Acceleratio Using Attenuation Relationship for Soft Soil Sites

The residuals for the recorded accelerations are presented in Fig. 11b; also shown in this figure is the average residual, which is only about 0.03. The information shown in Fig. 11b indicates that the equations (Eqs. 3) given above for calculating the median peak accelerations at soft soil sites appear to provide a reasonable estimate of the recorded values at distances of about 45 to 100 km. These equations are also suggested for use in estimating peak horizontal accelerations at soft sites at other distances and for other magnitude earthquakes.

The median peak horizontal accelerations calculated at rock sites using Eqs. 1 for earthquake magnitudes ranging from 5 to 7.5 (in increments of ½ magnitude) are presented in Fig. 12a. Corresponding values at soft soil sites (using Eqs. 3) are presented in Fig. 12b.

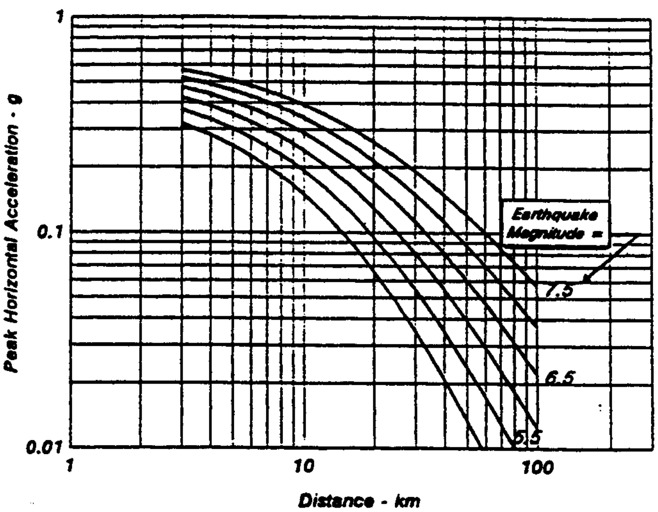

Fig. 12a Median Peak Horizontal Accelerations at Rock Sites

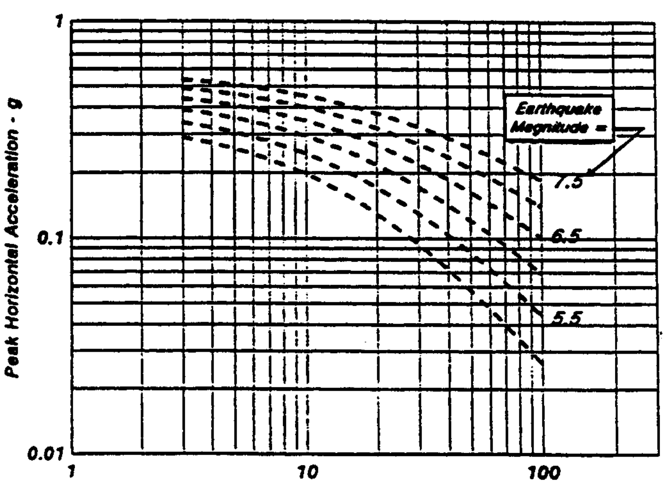

Fig. 12b Median Peak Horizontal Accelerations at Soft Soil Sites

CONCLUDING REMARKS

This Special Session is being held to honor the memory of the late Professor Harry Seed. In addition to this presentation on site effects, other invited speakers have also discussed at this Special Session various aspects of the Loma Prieta earthquake, including liquefaction, earth dams, lifelines and structural effects. Professor Harry Seed of course made giant contributions to each and every aspect of these topics. Many of the comments and conclusions he wrote in his numerous papers on these topics were observed during this earthquake.

It is hoped that the material presented in the preceding pages of this paper will provide a convenient means for incorporating the effects of local site conditions on ground motions at soft soil sites. Much of the work needed to fully evaluate this problem is still in progress by many investigators and the results will be reported in the coming months.

Finally, I would like to express a special appreciation to Dr. Shamsher Prakash for organizing this Special Session to honor the memory of Professor Harry Seed.

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Idriss (1991, 1994) Attenuation Relationship

In 1991 Idriss proposed the following model to predict the spectral accelerations at given periods.

$$\ln[S_a(g)] = a_o + \exp(a_1 + a_2 M) + [b_o - \exp(b_1 + b_2 M)] \cdot \ln(R_{rup} + c) + F_1\phi \quad \overset{\varepsilon}{[+\sigma]}$$

with:

$S_a(g)$ = Spectral acceleration in units of gravity

R_{rup} = closest distance to the rupture plane (km)

σ = standard error term, = 0 when calculating mean value

F_1 = style of faulting factor

= 0 for strike-slip, 0.5 for reverse/oblique, 1.0 for reverse faults

Other factors are found in the tables on the following pages.

In 1994 Idriss revised the factors for PGA (i.e. peak ground acceleration, S_a at T = 0.0), but did not revise the factors for the remainder of the response spectrum. To calculate spectral accelerations, scale the 1991 spectrum using the following relationship:

$$S_a(T) = \frac{S_{a,91}(T)}{PGA_{91}} \cdot PGA_{94}$$

Idriss (1991;1994) Coefficients for M < 6.0
Rock, Average Horizontal

period	a0	a1	a2	b0	b1	b2	c	ϕ	Sigma
PGA 94	0	1.127	0.011	0.000	1.126	-0.106	10	0.28	1.29 - 0.12M
PGA 91	-0.150	2.261	-0.083	0.000	1.602	-0.142	20	0.20	
0.030	-0.150	2.261	-0.083	0.000	1.602	-0.142	20	0.20	1.29 - 0.12M
0.050	-0.278	2.365	-0.092	0.066	1.602	-0.142	20	0.20	1.29 - 0.12M
0.075	-0.308	2.334	-0.081	0.070	1.602	-0.142	20	0.20	1.29 - 0.12M
0.100	-0.318	2.319	-0.075	0.072	1.602	-0.142	20	0.20	1.32 - 0.12M
0.110	-0.328	2.294	-0.070	0.073	1.602	-0.142	20	0.20	1.33 - 0.12M
0.130	-0.338	2.255	-0.062	0.075	1.602	-0.142	20	0.20	1.34 - 0.12M
0.150	-0.348	2.219	-0.055	0.076	1.602	-0.142	20	0.20	1.35 - 0.12M
0.200	-0.358	2.146	-0.042	0.078	1.602	-0.142	20	0.20	1.37 - 0.12M
0.250	-0.429	2.073	-0.030	0.080	1.602	-0.142	20	0.20	1.38 - 0.12M
0.300	-0.486	2.010	-0.020	0.082	1.602	-0.142	20	0.20	1.39 - 0.12M
0.350	-0.535	1.977	-0.016	0.087	1.602	-0.142	20	0.20	1.40 - 0.12M
0.400	-0.577	1.921	-0.009	0.092	1.602	-0.142	20	0.20	1.41 - 0.12M
0.500	-0.648	1.818	0.003	0.099	1.602	-0.142	20	0.20	1.42 - 0.12M
0.600	-0.705	1.704	0.017	0.105	1.602	-0.142	20	0.20	1.43 - 0.12M
0.700	-0.754	1.644	0.022	0.111	1.602	-0.142	20	0.20	1.44 - 0.12M
0.800	-0.796	1.593	0.025	0.115	1.602	-0.142	20	0.20	1.45 - 0.12M
0.900	-0.834	1.482	0.039	0.119	1.602	-0.142	20	0.20	1.46 - 0.12M
1.000	-0.867	1.432	0.043	0.123	1.602	-0.142	20	0.20	1.47 - 0.12M
1.500	-0.970	1.072	0.084	0.136	1.602	-0.142	20	0.20	1.47 - 0.12M
2.000	-1.046	0.762	0.121	0.146	1.602	-0.142	20	0.20	1.47 - 0.12M
3.000	-1.143	0.194	0.191	0.160	1.602	-0.142	20	0.20	1.47 - 0.12M
4.000	-1.177	-0.466	0.280	0.169	1.602	-0.142	20	0.20	1.47 - 0.12M
5.000	-1.214	-1.361	0.410	0.177	1.602	-0.142	20	0.20	1.47 - 0.12M

Idriss (1991;1994) Coefficients for M ≥ 6.0
Rock, Average Horizontal

period	a0	a1	a2	b0	b1	b2	c	ϕ	Sigma M<7.25	Sigma M≥7.25
PGA 94	0	2.763	-0.262	0.000	2.215	-0.288	10	0.28	1.29 - 0.12M	0.42
PGA 91	-0.050	3.477	-0.284	0.000	2.475	-0.286	20	0.20		
0.030	-0.050	3.477	-0.284	0.000	2.475	-0.286	20	0.20	1.29 - 0.12M	0.42
0.050	-0.278	3.426	-0.269	0.066	2.475	-0.286	20	0.20	1.29 - 0.12M	0.42
0.075	-0.308	3.359	-0.252	0.070	2.475	-0.286	20	0.20	1.29 - 0.12M	0.42
0.100	-0.318	3.327	-0.243	0.072	2.475	-0.286	20	0.20	1.32 - 0.12M	0.45
0.110	-0.328	3.289	-0.236	0.073	2.475	-0.286	20	0.20	1.33 - 0.12M	0.46
0.130	-0.338	3.233	-0.225	0.075	2.475	-0.286	20	0.20	1.34 - 0.12M	0.47
0.150	-0.348	3.185	-0.216	0.076	2.475	-0.286	20	0.20	1.35 - 0.12M	0.48
0.200	-0.358	3.100	-0.201	0.078	2.475	-0.286	20	0.20	1.37 - 0.12M	0.50
0.250	-0.429	3.034	-0.190	0.080	2.475	-0.286	20	0.20	1.38 - 0.12M	0.51
0.300	-0.486	2.982	-0.182	0.082	2.475	-0.286	20	0.20	1.39 - 0.12M	0.52
0.350	-0.535	2.943	-0.177	0.087	2.475	-0.286	20	0.20	1.40 - 0.12M	0.53
0.400	-0.577	2.906	-0.173	0.092	2.475	-0.286	20	0.20	1.41 - 0.12M	0.54
0.500	-0.648	2.850	-0.169	0.099	2.475	-0.286	20	0.20	1.42 - 0.12M	0.55
0.600	-0.705	2.803	-0.166	0.105	2.475	-0.286	20	0.20	1.43 - 0.12M	0.56
0.700	-0.754	2.765	-0.165	0.111	2.475	-0.286	20	0.20	1.44 - 0.12M	0.57
0.800	-0.796	2.728	-0.164	0.115	2.475	-0.286	20	0.20	1.45 - 0.12M	0.58
0.900	-0.834	2.694	-0.163	0.119	2.475	-0.286	20	0.20	1.46 - 0.12M	0.59
1.000	-0.867	2.662	-0.162	0.123	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60
1.500	-0.970	2.536	-0.160	0.136	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60
2.000	-1.046	2.447	-0.160	0.146	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60
3.000	-1.143	2.295	-0.159	0.160	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60
4.000	-1.177	2.169	-0.159	0.169	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60
5.000	-1.214	2.042	-0.157	0.177	2.475	-0.286	20	0.20	1.47 - 0.12M	0.60