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An Example of the Use of 3D Slope Stability Analyses

(and of simplified calculation of expected seismic displacements)

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General Background

While there is some published literature on the difference between 2D and 3D slope stability analyses, the truth is that, lacking suitable tools for routinely conducting 3D analyses, no-one really knows.

The conventional wisdom seems to be that 3D effects are generally small. This is based in part on analyses of the failure of the Kettleman Hills landfill in California. This failure occurred at a hazardous waste landfill with a slippery liner system and back slopes that widened out, so that in 3D the potential sliding mass got an extra push. That extra push only amounted to something like 8 percent of the driving forces but was still significant in back-calculations of the failure.

But the effect of 3D geometry can be much more significant in the other direction. In 1989 the writer was approached about a problem in the design of the Canyon Nine landfill at Puente Hills in the Los Angeles area. Canyon Nine is a “bottleneck canyon” where the mouth of the canyon closes in like the abutments of a dam site. Conventional 2D slope stability analyses could not show that the planned landfill sitting on a slippery liner system would be stable even though common-sense argued otherwise. The writer then wrote a simple 3D slope stability program using the Method of Columns (analogous to the Method of Slices in 2D) which demonstrated the obvious, namely that if the 3D geometry was taken into account, the landfill would be more than adequately stable.

That program has now evolved into the next-generation slope stability program TSLOPE, which can be used to perform either 2D or 3D analyses using either the Method of Columns or Spencer’s Method. The program also has a modern interface which facilitates import of data from other programs and from 3D geological modelling packages. While it is particularly applicable to the more efficient design of open pit slopes where detailed 3D geometry is normally available from geologic modelling packages, it is equally applicable to other geotechnical engineering applications.

Use of the new program has turned up some surprising results. It is not just bottleneck canyons where 3D results are significantly different from 2D results.

One such surprise is the difference between a 2D circular failure and a 3D spherical failure in a homogeneous cohesive slope that is described in a note by Peter Wood, which is also posted on this web site. This is in effect an extension of the well-known technical note by Baligh and Azzouz (1975) on end effects. The 2D factor of safety for the problem analysed by Peter is 1.08 and the 3D factor of safety obtained by Peter and other workers cited in his note is in the order of 1.40, a 30 percent difference!

Another surprise is the difference between a 2D failure and a more realistic 3D failure in a zoned earth dam, as described in the note by Ian Brown that is posted on this web site. And this can be true no matter how long the dam is. The 2D failure surface overweighs the core material because the proportion of the 3D failure surface that cuts through the shell is much greater than its participation in the 2D failure surface.

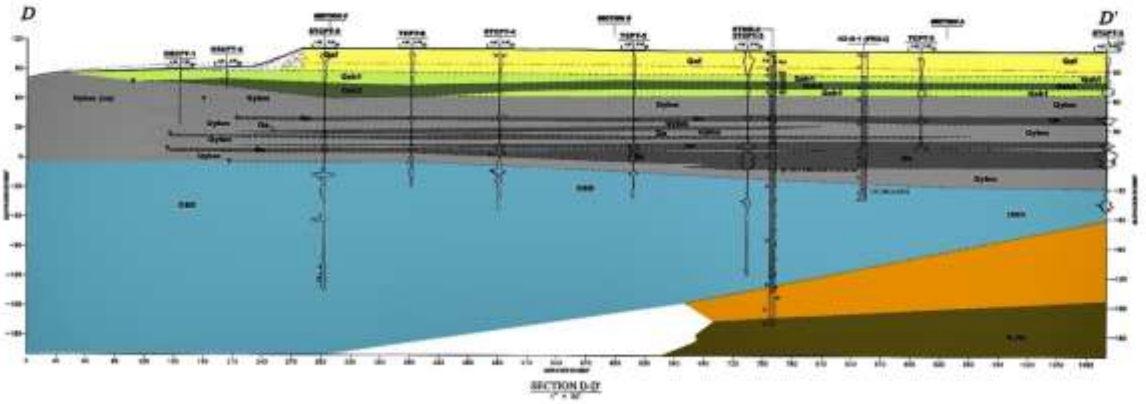
And a third surprise is the effect that a wall or revetment can have on the stability of a long slope. This falls in the category of things that are obvious once they are pointed out to you, but were not so obvious previously. The point is that a wall or revetment often does not participate in a 2D slope stability analysis because the critical slip surface dives under the wall or revetment. However, a 3D failure surface has to cut through the wall or revetment. This effect will be greatest when the slide is narrow in the direction along the wall, that is, it has an aspect ratio of less than one. As the aspect ratio of the 3D slide surface increases and it encompassed more of the length of the slope, the effect of the 3D failure will decrease and the computed factor of safety will approach but never reach the 2D value. But most, although not all, natural landslides have an aspect ratio of less than one and 3D effects can be significant.

The overall point is that you will never know unless you check. The following real-world example shows the difference that 3D analyses make in both static slope stability analyses and in simplified seismic deformation analyses that require computation of the yield acceleration using a slope stability program. Extension of TSLOPE to more accurately calculate seismic displacements using site-specific acceleration histories is currently underway.

Real-World Example

This example involves Treasure Island, a man-made island in San Francisco Bay, which was originally intended to serve as an airport, but, after the completion of the 1939 World's Fair, the island was taken over by the US Navy. It is presently being redeveloped for civilian use. The sand fill that was place to form the island will be densified to mitigate possible liquefaction, and prefabricated vertical drains and surcharging will be used to limit future settlement of the underlying young Bay Mud. The final grades will be raised slightly to allow gravity flow of stormwater for the foreseeable future. The cross section below and the soil properties are taken from publically-released bid documents.

In part because an initial layer of sand fill was placed prior to the construction of the perimeter rock dikes, as can be seen in Section D-D’ below, there remains some concern about the stability of the perimeter of the island, particularly in earthquakes since the site sits in between the active San Andreas and Hayward faults. Likely this sand layer has been compacted to some extent since its original placement by the Loma Pieta earthquake and repeated wave loadings, and at least the sands under the original rock dikes can be further densified to some extent if dynamic compaction is used to compact the sand layer up to the edge of the revetment and specialist techniques are used to reach in under these dikes. It may not be possible to further compact all of the sand layer in Section D-D’ that continues under the rip-rap, however, my subsequent calculations suggest that this is not critical to the stability of the revetment since the critical failure surface in a conventional stability analysis lies within the young Bay Mud.



Section D-D’

The shoal materials which underlie the sand fill are clayey sands that generally contain more than 20 percent fines and have measurable PIs. These materials are not liquefiable in any conventional sense and they were very resistant to densification by vibratory loading in trials that were performed at the site. The properties of the shoal materials are discussed in greater detail subsequently, but on the face of it, if the hydraulically placed sand fill is densified, the shoal materials do not show a loss of strength under vibratory loadings, the young Bay Mud is consolidated not only under the weight of the existing fill but under additional surcharge loads, and the rock revetment is composed of free-draining, competent rock, there is no obvious concern about shoreline stability at this site, even given its proximity to the San Andreas and Hayward faults.

Nonetheless, in the bid documents there are brief descriptions of work done by the project’s geotechnical consultant using simplified methods of analysis, which may be commonly used but are now increasingly being recognized as being inadequate - see for instance Pyke (2015) and Boulanger (2016) – and these analyses have indicated a potential shoreline stability problem . These simplified methods are at best screening analyses, but the two methods used by the project geotechnical consultant are so flawed that it is doubtful whether they are useful even for that. In addition to the large uncertainty which comes from their being based on large collections of earthquake records, the Bray and Travasarou procedure has the curious feature that when the stiffness is increased, the deformation increases rather than decreases, and the NCHRP method is independent of earthquake magnitude or duration, which cannot be correct.

There are several references in the bid documents to a further deformation analysis which makes use of the finite element program PLAXIS to conduct nonlinear deformation analyses with site-specific earthquake acceleration histories as input, but that report is not included in the bid documents and it does not appear that the necessary properties to conduct an accurate analyses, even in 2D, are readily available. In any case, the bid documents indicate that the PLAXIS analyses were only two-dimensional and I will demonstrate subsequently why 2D analyses are inadequate to address the Treasure Island shoreline stability problem, or at the very least, why they are excessively conservative.

Apart from the difficulty of identifying a nonlinear soil model that is capable of developing the correct deformations under complex cyclic loadings, there are two critical inputs that are required for nonlinear site response analyses whether they be 1D, 2D or 3D. These are the low strain modulus or shear wave velocity and the “modulus reduction curve” (which is a concept developed for equivalent linear analyses but is commonly used as the “backbone” for nonlinear soil models). Of these, the modulus reduction curve is the easier to grapple with because it can often be based on previous studies. That is what the project geotechnical consultant has done in this case. Certainly for the equivalent linear site response analyses that are described in the bid documents, modulus reduction and damping curves taken from Vahdani et al. (2002) were used. But, while these curves might be appropriate for the other materials, the curve used for the shoal materials is incorrect, in the sense that it degrades too quickly for a clayey sand, and there is no basis for its use in the reference that is cited. I should know because I was a co-author of that reference! A more appropriate modulus reduction curve could be chosen from those provided in Pyke et al. (1993).

It is unclear what shear wave velocities were used in either the simplified methods or the PLAXIS analyses, but the more fundamental problem is that the necessary data does not appear to exist. While the bid documents include a nice plot of pre and post densification cone tip resistances which is Attachment 1 to this note, there is no equivalent plot of shear wave velocities. I have therefore constructed my own plot from data that are in the bid documents and that is included at Attachment 2. The two pre-test measurements, which were both outside the actual test area, are linked by cross hatching. Post-test measurements within the test area are shown for 21 and 35 days after the test for one location, and 21, 35 and 49 days after the test in the other location. Several interesting things can be seen in these data. One is that the post-test measurements indicate shear wave velocities that are decreasing with time, suggesting that pore pressure dissipation and reorientation of particles is still going on at up to 49 days after the field trial, suggesting that measurements should have been made at additional intervals after the test.

Another is that while the measured shear wave velocities in the sand fill increased as a result of densification, they did not increase in proportion nearly as much as the cone tip resistances. This is readily explainable because the penetration resistance is a more direct function of the packing of the particles whereas the shear wave velocity is to some extent a function of that but also very much a function of the condition of the particle contacts. Densification by vibration disrupts the particle contacts and in some materials causes a decrease in the shear wave velocity even though the density has been increased. This phenomenon has been recognized for some years and is addressed in the Terzaghi Lectures of both Mitchell (1986) and Schmertmann (1991). An estimate of the long term shear wave velocity can still be made in the case of the sand fill because the SPT blow counts implied by the cone tip resistances are in the order of 30, which translates using standard relationships to shear wave velocities in the order of 1000 to 1200 ft/sec, which is what one would expect for a very dense sand.

The third interesting thing is that while anecdotal reports and the cone tip resistances suggest that even heavy vibration had no impact on the density of the clayey sands that comprise the shoal materials, the measured shear wave velocities decreased. Again this is readily explainable. Even if the packing of the shoal materials, as seen in electron micrographs, and the distribution of clay particles is such that heavy vibration (or earthquake shaking) does not densify these materials, the particle contacts will still be disrupted. These shear wave velocities will surely recover with enough time, but whether long term they will equal or even exceed the original values is not known and can only be established by measurements that are made over a period of several years or more.

So, not only does the basic data required to perform more sophisticated deformation analyses not exist, but it might take several years to acquire such data. This also applies to any properties that need to be determined from laboratory tests. In addition to any errors caused by sample disturbance or seating issues, laboratory tests performed on samples taken shortly after the trial densification will be in error because the particle contacts have not settled down and aged.

This raises the question of whether in the meantime there is any screening analysis that is appropriate for this site. The short answer is yes, there is. As explained by Harry Seed in his Rankine lecture (Seed, 1979), for materials that do not undergo a loss of strength and stiffness as a result of cyclic loading, pseudo-static analyses are not too bad. That in turn raises the question of what seismic coefficient should be used in a pseudo-static analysis but a robust answer to that is provided in Pyke (1991), which drew on the work of Makdisi and Seed (1978). Or, alternately one can use Makdisi and Seed (1978), which has less shortcomings than any other simplified method for computing deformations. Strictly speaking Makdisi and Seed only applies to dams ranging in height from 50 to 250 feet, but the Treasure Island revetment falls near the lower end of this range.

Both pseudo-static and Makdisi and Seed analyses require knowledge of the expected peak acceleration and the earthquake magnitude. Conservatively assuming up to a magnitude 7.3 earthquake on the combined Hayward – Rogers Creek fault and an up to magnitude 8.1 earthquake on the San Andreas fault, the project geotechnical consultant computed a peak acceleration for the site of 0.46 g using the computer program SHAKE to perform an equivalent linear site response analyses.

In order to compute both the static factor of safety and the yield acceleration (the seismic coefficient that reduces the factor of safety to unity – the factor of safety for a specified seismic coefficient can be derived from this) for Section D-D' I have used the computer program TSLOPE. TSLOPE is a new slope stability program which allows 3D analyses as well as 2D analyses. It is available for a free trial at http://tagasoft.com. For Section D-D', when a circular slip circle is transformed to a spherical or ellipsoidal slip surface, two things happen. One is that the slip surface now has to cut through the rock revetment, rather than diving under it – this will increase the factor of safety. The other is that relatively more of the slip surface will be in the young Bay Mud – this might either reduce or increase the factor of safety.

I have used both the methods of solution that are available in TSLOPE – the Ordinary Method of Columns (OMC), which I prefer, and Spencer's Method, which is more generally accepted, or rather I should say wrongly pushed by academics. The geometry and the properties that I have used are recorded in the input files, which I will be happy to make available to interested parties. Because hard data are not available for many of the required properties, I have used my judgement, based on 40 plus years of local practice, where necessary. For the young Bay Mud layer I have adopted the project geotechnical consultant's value of 0.3 for the ratio of the undrained shear strength divided by the effective vertical stress and I have divided the young Bay Mud layer into four zones for purposes of computing its undrained shear strength. For the zone that is going to be overconsolidated by wicking and surcharging I have increased the undrained shear strength by 50 percent, corresponding to an OCR value of a little more than 1.5. For the other zones I have assumed that the Bay Mud is normally consolidated under the current overburden. I have conservatively assumed that the lower strength sand layer extends to under the heel of the lower triangle of rockfill, even though I believe that some or all of this material can be densified. For the "static" loading case I have used undrained strengths in the young Bay Mud and drained strengths in the other materials.

For the "seismic" loading case I have used undrained strengths for all materials below the water table, except for the rockfill in the revetment. I have also corrected these strengths for rate of loading effects in order to represent the short rise time of an earthquake pulse. These corrections are based on UC Berkeley Ph.D. theses by Gerry Thiers and Willie Lacerda as well as other data reported by Pyke (1981) and Bea (1999). For the shoal materials I assumed a base undrained shear strength of 2000 psf based on the test data shown in Attachment 3. Note that UU tests normally give undrained shear strengths on the low side. The project geotechnical consultant has subdivided the shoal materials into two layers but I do not see convincing evidence for that and have modelled it as a single layer. The logs of a pair of adjacent borings and three more widely spaced borings are shown in Attachments 4 and 5.

The critical circular slip surfaces obtained using Spencer's method and the "static" and "seismic" properties are shown in Attachments 6 and 7. These are both for "static" analyses without the application of a seismic coefficient. The critical circular slip surface obtained in the "static" analysis with "seismic" properties was then used in subsequent searches for the yield acceleration.

The critical 2D failure surface was also used as the basis for constructing three 3D failure surfaces, as shown in Attachment 8. The center 3D slip surface is a sphere, which has an aspect ratio of 1.0. In addition there are two further ellipsoids that have aspect ratios of 0.5 and 2.0. The larger the aspect ratio, the more the 3D solution approaches the 2 D solution. Again, the reason that the 3D factors of safety are higher than the 2D is that in 3D you have to cut through the revetment, rather than diving under it as you do in 2D. Of the four cases, the one with the aspect ratio of 0.5, which gives the highest factor of safety, is probably the most like an actual landslide.

OMC	Spencer	
Static analyses		
2.23	**2.51**	**2D FoS**
2.59	**2.95**	**3D FoS aspect ratio = 2.0**
2.44	**2.97**	**3D FoS aspect ratio = 1.0**
2.57	**3.52**	**3D FoS aspect ratio = 0.5**
Seismic analyses		
0.22g	**0.26g**	**2D yield acceleration**
0.27g	**0.31g**	**3D yield acceleration – aspect ratio = 2.0**
0.29g	**0.33g**	**3D yield acceleration – aspect ratio = 1.0**
0.35g	**0.39g**	**3D yield acceleration – aspect ratio = 0.5**

The results are shown in the above table. As expected for a slope that has been stable for many years and would have been at greatest risk at the end of construction, the static factors of safety are healthy enough to suggest that there is some margin of safety to accommodate earthquake loadings.

Recall that the design peak acceleration for the site is 0.46g. At most, the seismic coefficient that should be used in a pseudo-static analysis is half that, or 0.23g (see Pyke, 1991). That would be for a magnitude 8 earthquake on the San Andreas fault. Except for the 2D analysis using the OMC, all the yield accelerations (the seismic coefficient that reduces the factor of safety to unity) are greater than 0.25, implying factors of safety in pseudo-static analyses of more than 1.1, which is the accepted standard in California for passing a "screening analysis". And, if you meet the screening analysis criteria, you are not required to attempt more detailed or sophisticated analyses. Taking the yield acceleration for the aspect ratio of 0.5 and the OMC, which in my judgment is the "best" answer, the implied factor of safety using the highest possible seismic coefficient in a pseudo-static analysis is actually 1.5! There is no rational argument for requiring any further analyses even if an extensive field and laboratory investigation were to be undertaken to acquire the kind of data that would be needed.

I have taken the results below one step further and illustrated the expected seismic displacements in Attachment 9, using Figure 1 from the "McCrink letter" (Pyke, 1991, which is attached). The computed ratios of the yield acceleration divided by the peak acceleration range from about 0.5 to about 0.75, with the higher values being likely more correct. Thus the expected displacements from both San Andreas and Hayward fault events are small, less than 1 foot, and in the worst case the San Andreas displacements might be something like 2 feet. These seem like entirely reasonable results for the site after densification of the sand fill and wicking and surcharging of the yBM. There is no precedent for failure of a fill over young Bay Mud many years after the initial construction when the young Bay Mud has fully consolidated, let alone when it has been overconsolidated by wicking and surcharging, even in the Loma Prieta earthquake which generated strong ground motions around at least parts of the Bay.

Conclusions

Simplified analyses using conventional procedures and 2D slope stability analyses are unnecessarily conservative, and in this case suggest that there is a problem where no problem actually exists.

Furthermore, any effort that is made to improve shoreline stability by introducing relatively stiff soil-cement walls or cells is likely to worsen the situation rather than improving it. Such measures would surely result in longitudinal cracking as a result of strain incompatibility and would likely increase the tendency for the revetment to shed into the Bay.

And, requiring the construction of soil-cement walls or cells when they are not in fact needed, will often have adverse effects on schedule and introduce unnecessary headaches with regard to construction quality control.

Next generation software tools such as TSLOPE (and our forthcoming pile and wall analysis program TPILE) are intended to provide geotechnical engineers not with more sophisticated tools just for the sake of it, but with tools that better match both reality and common-sense and lead to designs which are both more economical and safer.

References

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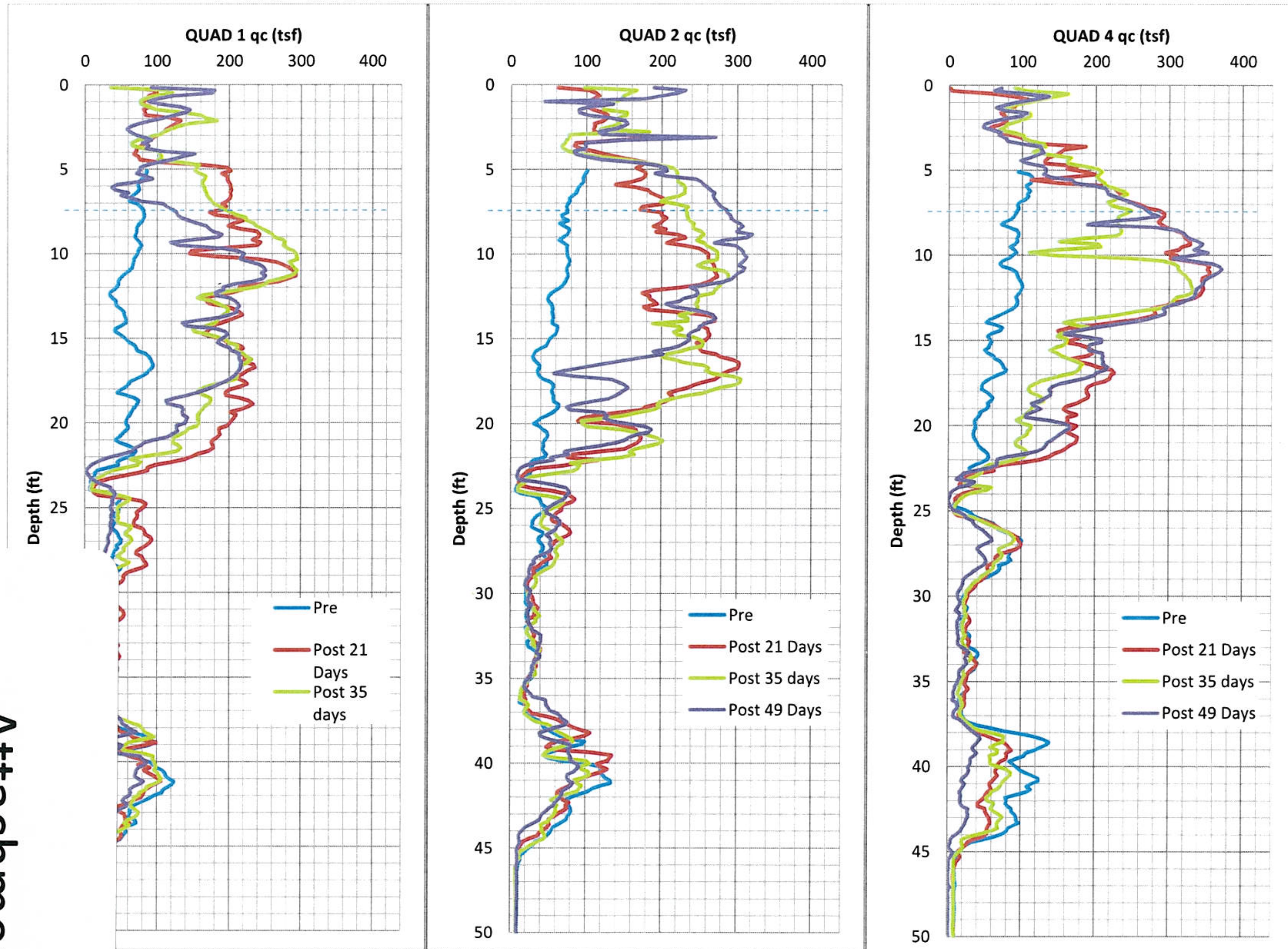
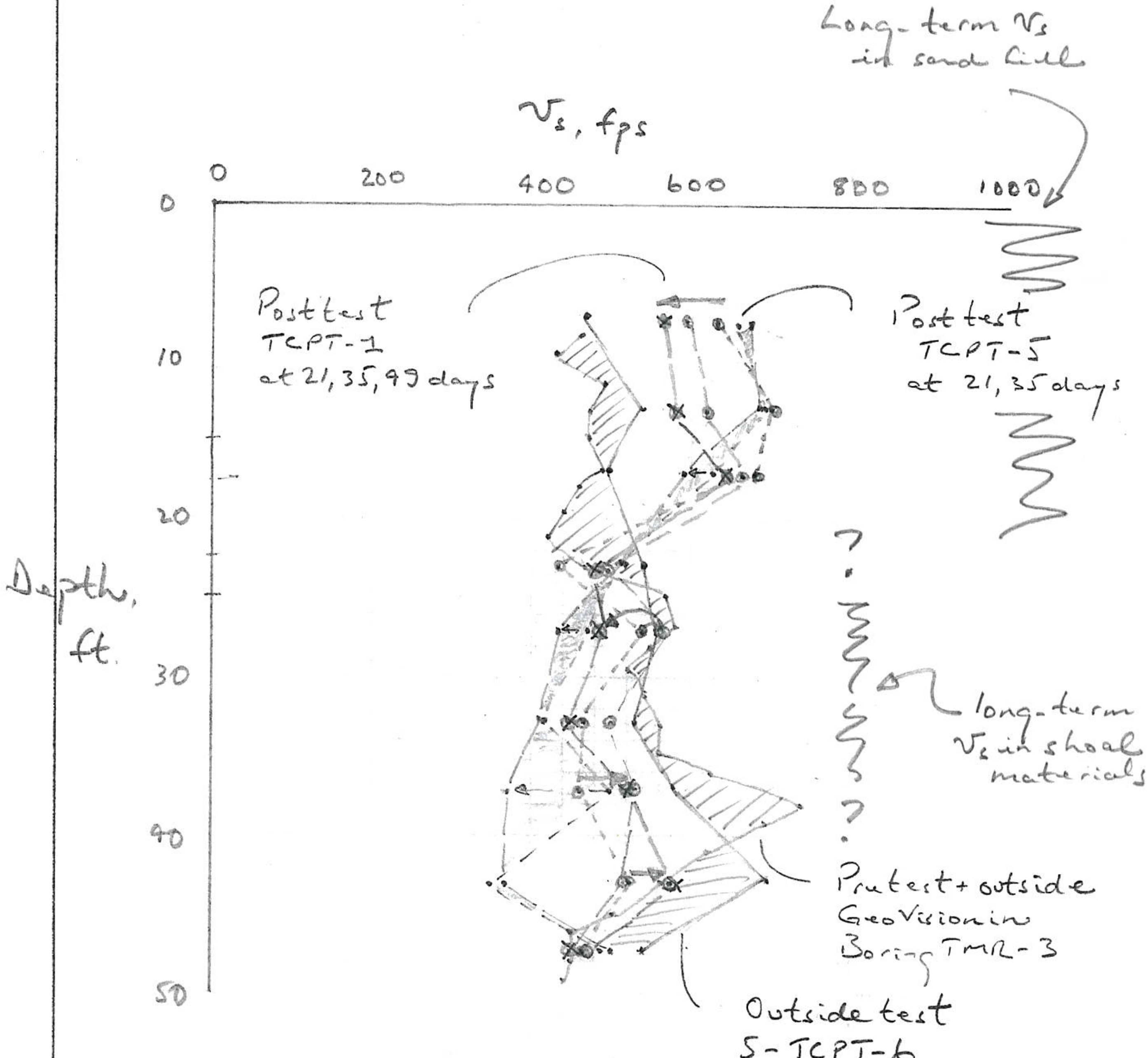


POST IMPROVEMENT CONE RESISTANCE
TI MP1 SUB-PHASES 1B, 1C, 1E
San Francisco, California

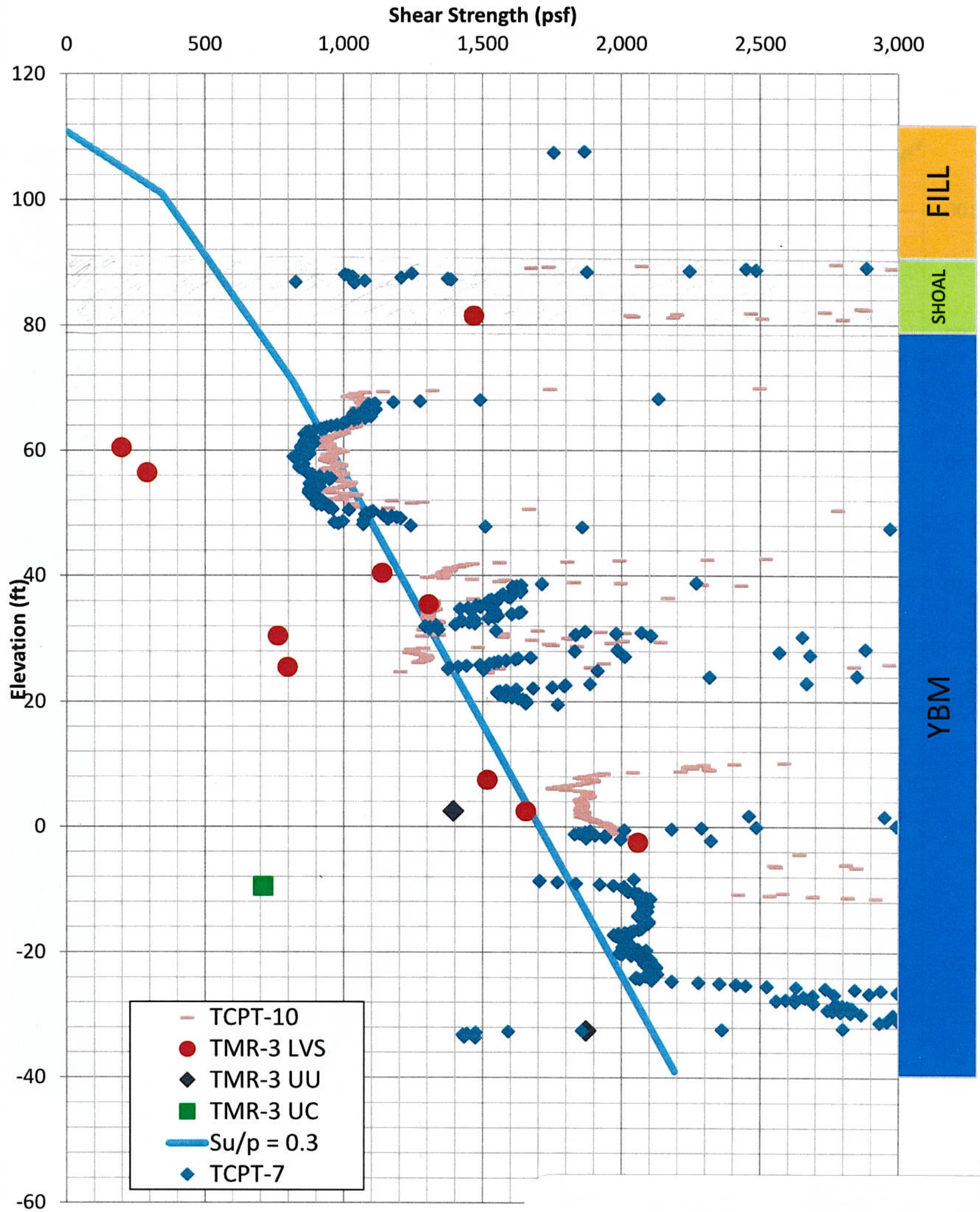
Figure No.:	T3-F1
Job No.:	7091.000.000

Attachment 1

RMP 10/09/2016



Attachment 2



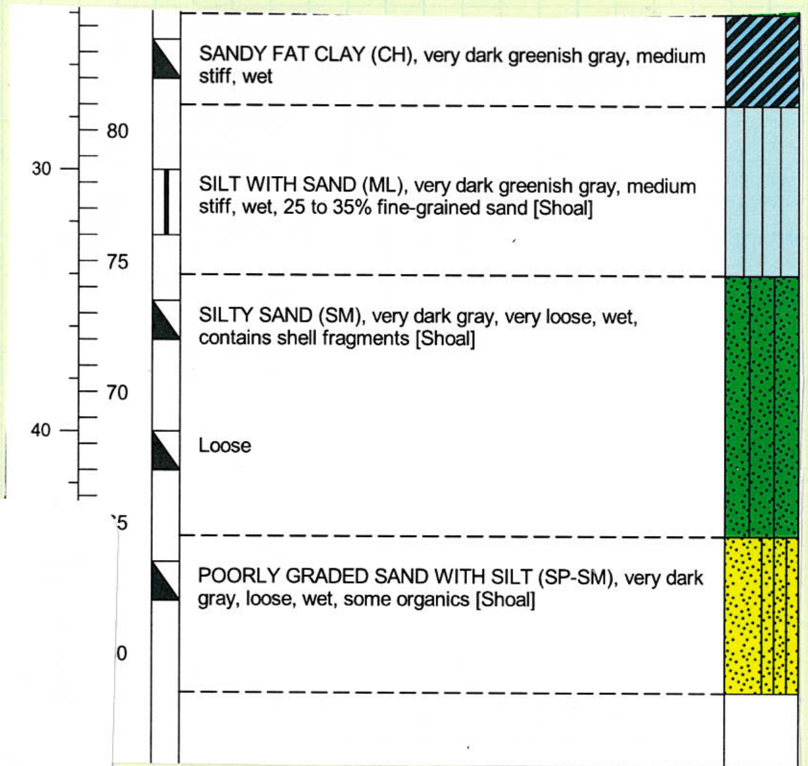
UNDRAINED SHEAR STRENGTH

TI MP1 SUB-PHASES 1B, 1C

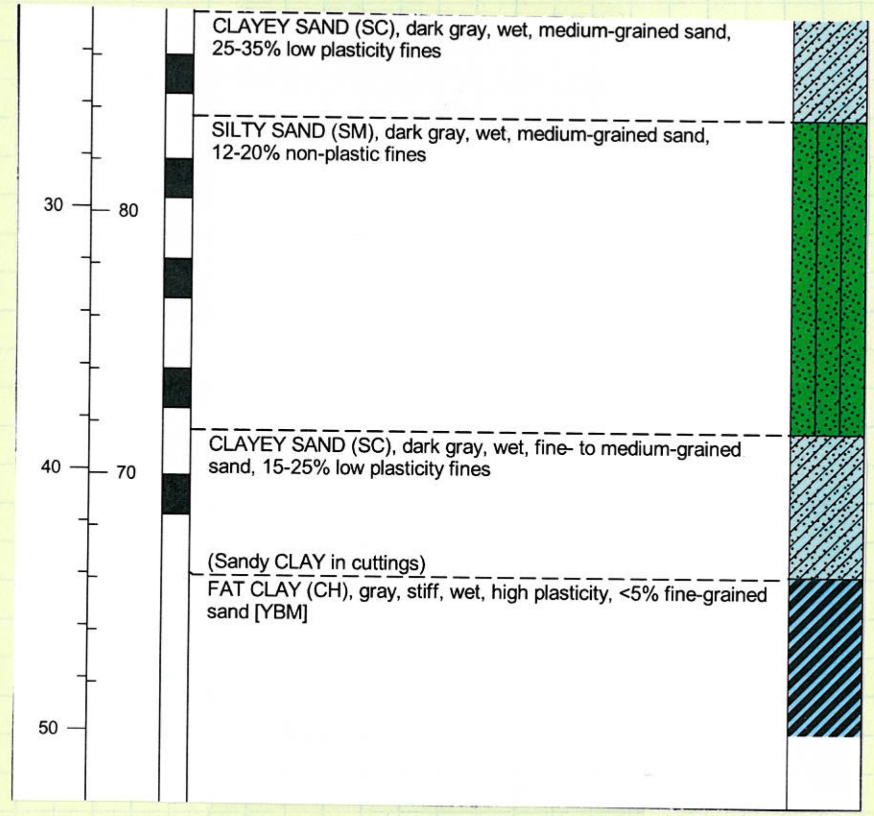
San Francisco, California

Attachment 3

Same elevation



Boring TMR-2



Boring 6-TMR-2

Attachment 4

Attachment 5

Same depth below ground surface

TMR-2

30, 40

80, 75, 70

SANDY FAT CLAY (CH), very dark greenish gray, medium stiff, wet

SILT WITH SAND (ML), very dark greenish gray, medium stiff, wet, 25 to 35% fine-grained sand [Shoal]

SILTY SAND (SM), very dark gray, very loose, wet, contains shell fragments [Shoal]

Loose

) SAND WITH SILT (SP-SM), very dark
me organics [Shoal]

Boring TMR-3

POORLY GRADED SAND (SP), dark gray, loose to medium dense, wet, [Shoal]

SANDY CLAY (CH), dark gray, wet, fine-grained sand, contains shell fragments

CLAYEY SAND (SC), dark gray, wet, fine-grained sand, contains shell fragments [Shoal]

POORLY GRADED SAND WITH SILT (SP-SM), dark gray, very loose, wet, contains shell fragments [Shoal]

Medium dense

FAT CLAY (CH), dark gray, medium stiff, wet, contains shell fragments

Boring TMR-4

SANDY LEAN CLAY (CL), very dark greenish gray, medium stiff, wet, 35 to 45% fine-grained sand

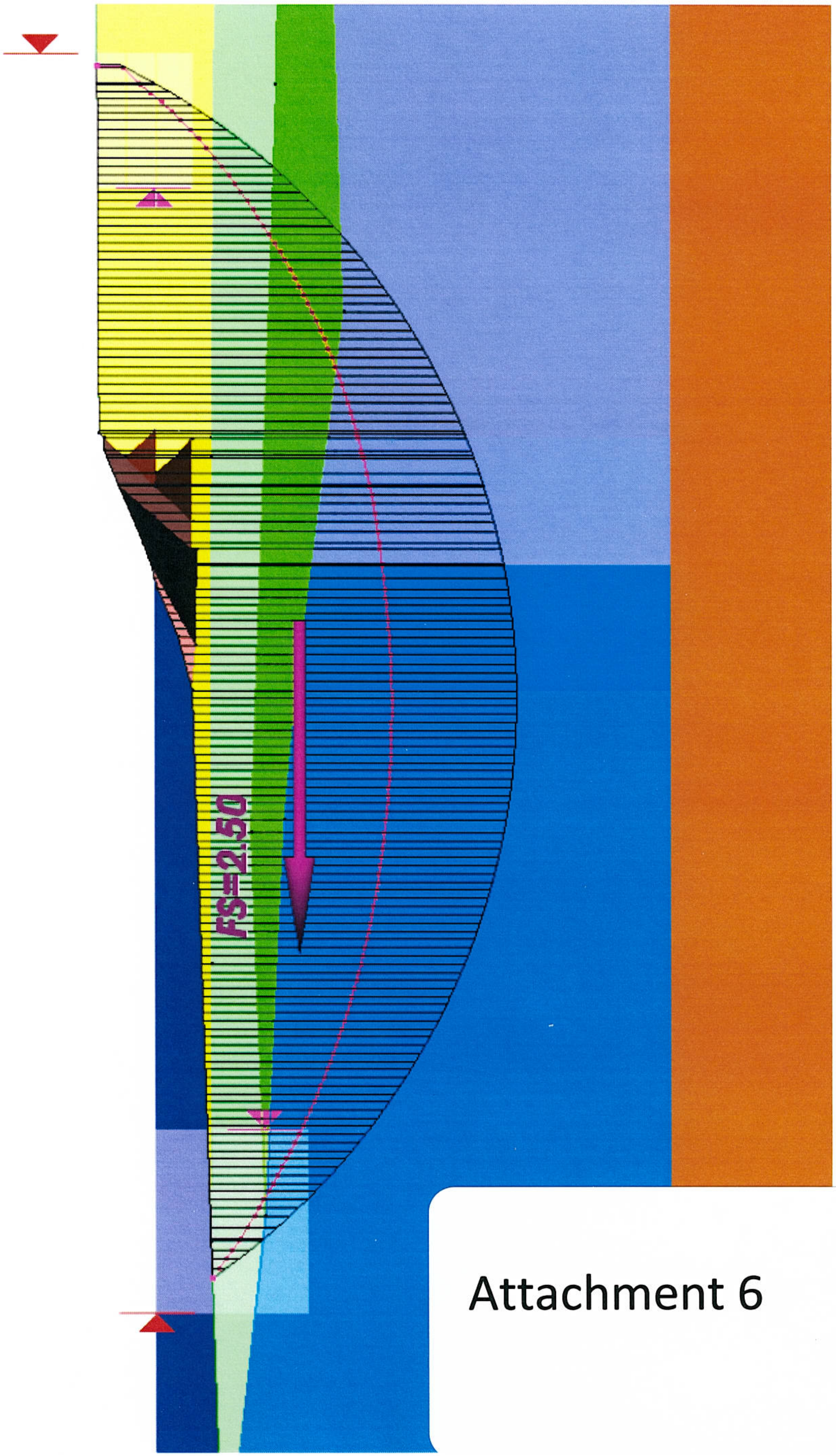
SILTY SAND (SM), dark grayish brown, very loose, wet, fine- to medium-grained sand, contains shell fragments [Shoal]

SANDY LEAN CLAY (CL), very dark greenish gray, medium stiff, wet, 35 to 45% fine-grained sand

SILTY SAND (SM), dark grayish brown, very loose, wet, fine- to medium-grained sand, contains shell fragments [Shoal]

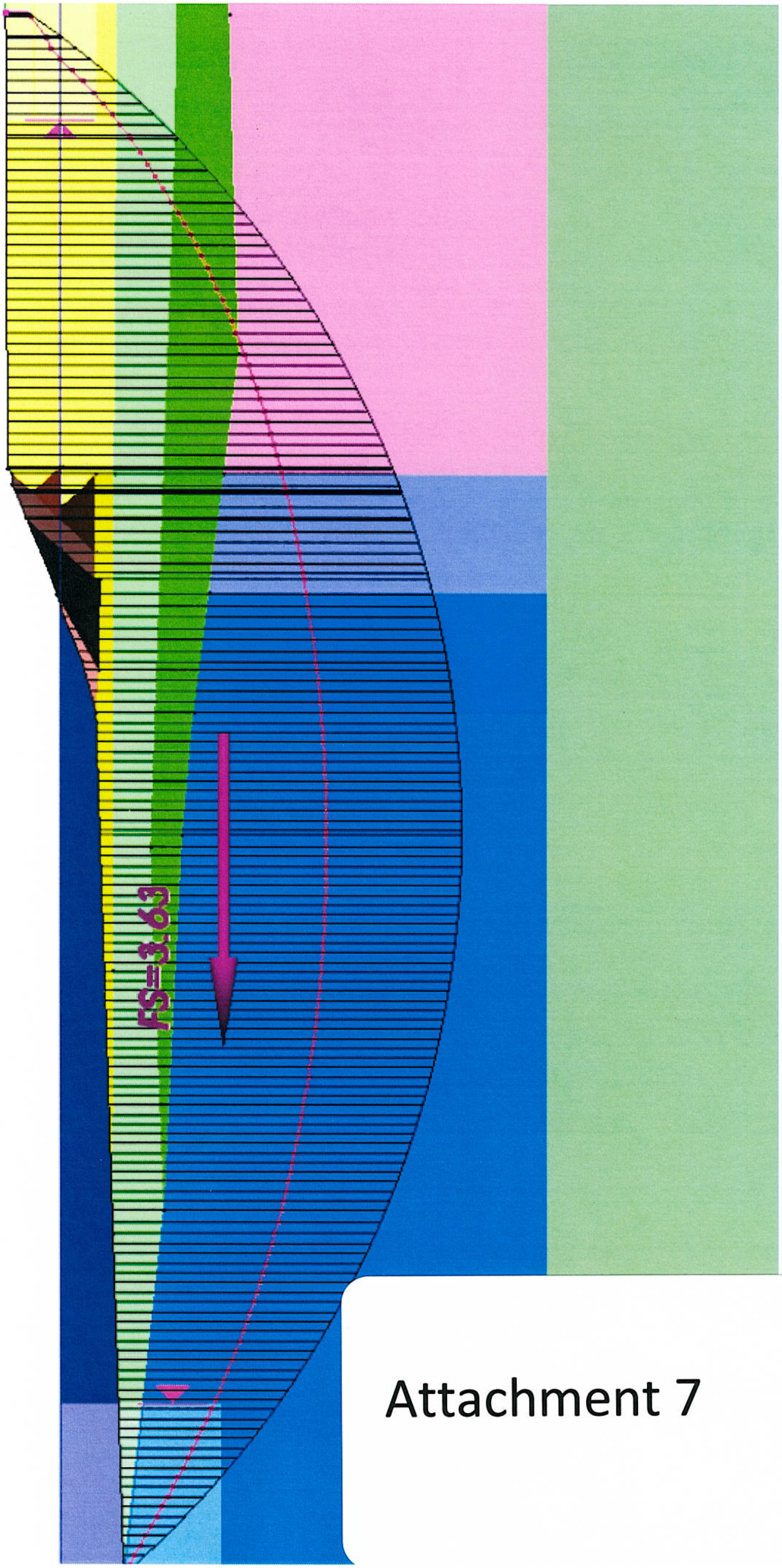
SANDY LEAN CLAY (CL), very dark greenish gray, medium stiff, wet, 35 to 45% fine-grained sand

FAT CLAY (CH), very dark greenish gray, stiff, wet, <5% fine-grained sand



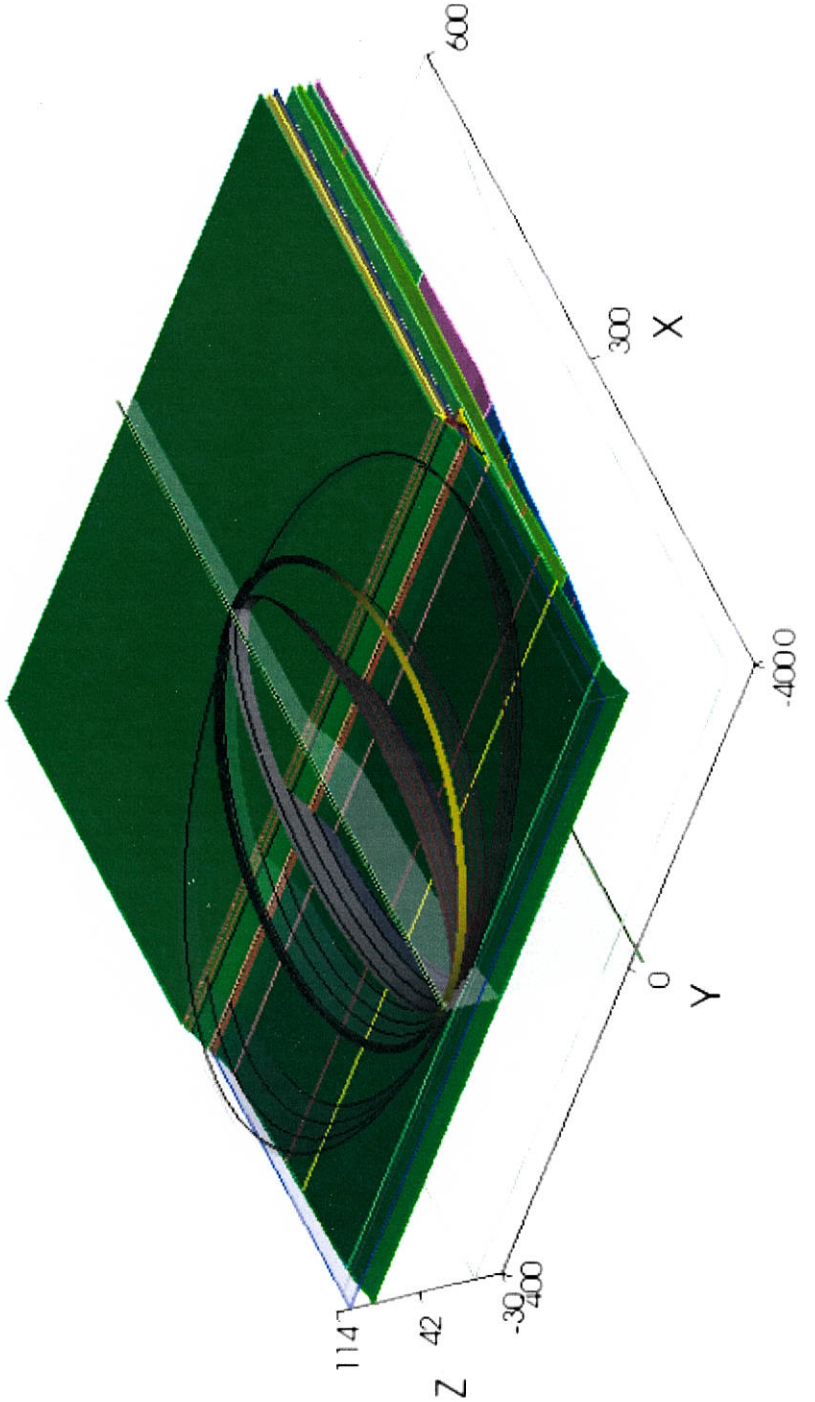
Critical Slip Surface for "Static" Properties

Attachment 6



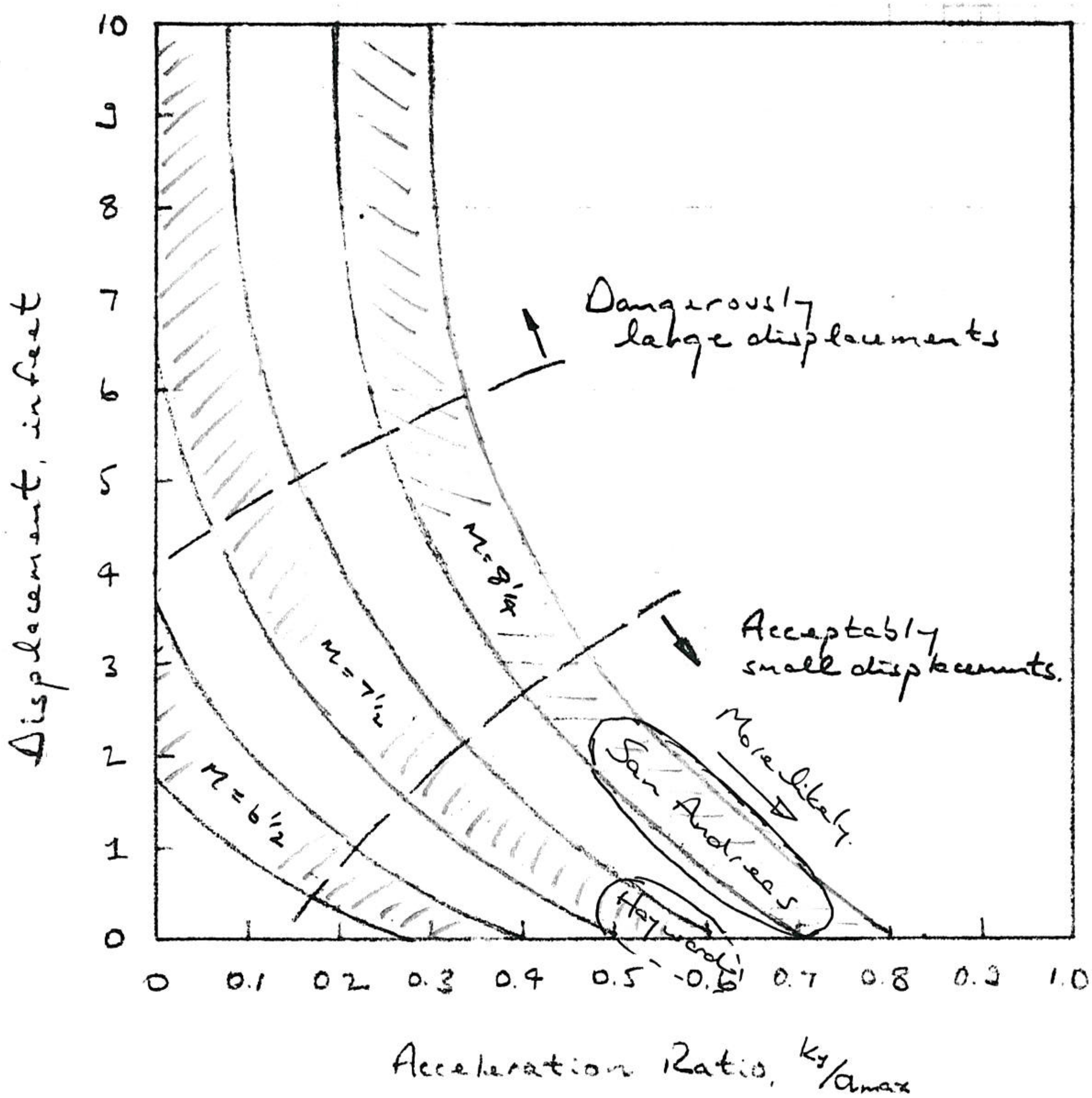
Critical Slip Surface for "Seismic" Properties

Attachment 7



Three-dimensional failure surfaces.

Attachment 8



Typical Displacements Computed
by Newmark

after Makdisi a

Attachment 9

Robert Pyke, Consulting Engineer

October 04, 1991

Tim McCrink
California Division of Mines and Geology
630 Bercutt Drive
Sacramento, CA 95814

Re: Selection of Peak Acceleration Values and Seismic Coefficients

Dear Tim,

As discussed by telephone, I am pleased to provide this summary of my understanding of better practice regarding the selection of peak acceleration values for use in either the design of buildings and other engineered facilities or the assessment of geotechnical problems such as slope stability and liquefaction and the separate, but related, question of the selection of seismic coefficients for use in pseudo-static analyses of walls or slopes.

Ideally, the same value or values of peak acceleration should be used for both the design of buildings and geotechnical assessments, and the peak acceleration should be just one component of a more complete description of the design earthquake motion which should also contain information regarding the frequency content and duration of the motions. In the general case, more than one motion and hence more than one peak acceleration might be specified, both because damaging motions from more than one source might affect a given site and because the owner and his engineers might be willing to accept or may wish to design for different levels of risk.

Either deterministic or probabilistic methods, or some combination of these, can be used to evaluate the design motions for a particular site. In the deterministic approach design magnitudes are assigned to significant seismic sources and the expected peak accelerations at the site are determined for these magnitudes. In the probabilistic approach, a mathematical model of the various seismic sources is constructed and the probabilities that given levels of motions at the site will be exceeded are computed.

The probabilistic approach has been used principally for building design in those cases where the owner desires or is required to use design loadings which are more site-specific than the basic provisions of the building code. A common design practice has been to size structures so that they do not exceed code allowable stresses and deflections for motions that have a 50 percent probability of exceedance in 50 years and to check that they will not collapse for motions that have a 10 percent probability of exceedance in 50 years. More stringent requirements now apply to state and public school buildings which the State of California requires be designed to prevent structural damage for motions that have a 40 percent chance of exceedance in 100 years and to hospitals and essential services buildings which have to be designed to prevent damage for motions that have only a 20 percent chance of exceedance in 100 years. Such probabilistic analyses may be conducted directly in terms of response spectrum amplitudes or they may be conducted in terms of the peak acceleration which is then used to scale an appropriately shaped elastic response spectrum.

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Regardless of whether it is arrived at deterministically or probabilistically, the peak acceleration that is used to scale an elastic response spectrum (sometimes called the zero period acceleration) should be the peak acceleration of the expected motion within the range of frequencies of engineering interest. This acceleration is sometimes referred to as an effective peak acceleration and is generally similar to the peak accelerations of recorded motions with the exclusion of high frequency spikes that are outside the range of frequencies that will affect the structure of interest. No hard and fast rule can be given regarding the possible decrease from recorded peak accelerations to effective peak accelerations since this is very much a function of individual recorded motions. When expected peak accelerations are obtained from formulae or charts that average the recorded values of peak accelerations for an appropriate suite of records, no adjustment is necessary.

While the probabilistic approach could also be used to select ground motions for use in the evaluation of geotechnical problems such as slope stability and liquefaction, this has not been common practice and there are no readily-available guidelines for their use. This results in part from the fact that the results of both slope stability and liquefaction evaluations have usually been expressed as a factor of safety and many authorities require that the factor of safety be not less than unity even for the ground motions corresponding to the "maximum credible" earthquake. More widespread use of the probabilistic approach awaits further development and implementation of procedures for computing deformations rather than factors of safety so that meaningful acceptance criteria can be established for different probabilities of occurrence.

It is clear, however, that if reference is made to peak accelerations in general descriptions of the seismic environment, then those accelerations should be not less than the effective peak accelerations with a probability of occurrence of 10 percent in 50 years. It is inappropriate to cite some lower value such as the "repeatable high ground acceleration" of Ploessel and Slosson (California Geology, September 1974) without reference to specific problems and methods of analysis. The reduction of the peak acceleration to the "repeatable high ground acceleration" of Ploessel and Slosson is in fact similar to the reduction of the peak shear stress to an average shear stress in the procedures for evaluating the potential for liquefaction developed by the late Professor H. Bolton Seed and his colleagues at the University of California, Berkeley, and such a reduction is appropriate in that context.

Selection of Seismic Coefficients

Perhaps the best example of a situation where the peak acceleration should <u>not</u> be used directly is provided by pseudo-static analyses of slope stability. These are very much simplified analyses in which a constant lateral force is expected to somehow equivalence the cyclic loading and possible variation in material properties that actually take place during earthquake shaking. Historically, the choice of an appropriate seismic coefficient (and a corresponding acceptable factor of safety) has been a matter of acceptable local practice without any particular rational including linkage to expected peak accelerations or the duration of strong shaking.

In recent years a basic point of reference has been that the U.S. Army Corps of Engineers manual for seismic design of new dams (which are generally considered to be among the more critical civil engineering facilities) requires use of a seismic coefficient of 0.1 in Seismic Zone 3 and 0.15 in Seismic Zone 4, in conjunction with a minimum factor of safety of 1.0. In California, many state and local agencies also require the use of a seismic coefficient of 0.15 but impose the slightly more conservative requirement that the minimum computed factor of safety be not less than 1.1. Clearly, however, engineering judgement must be still be applied as to the applicability of pseudo-static analyses and the acceptable factor of safety might be varied with the uncertainties involved in a particular analysis.

Further, it is now possible to make an approximate but rational connection between the seismic coefficient that is used in a pseudo-static stability analysis and the expected amplitudes and duration of ground motion by working backwards through the method for computing displacements of slopes that was originally suggested by Newmark (1965). This approach was first explored by Seed (1979) who drew the general conclusion that for embankments composed of materials which show no significant loss of strength as a result of cyclic loading, "it is only necessary to perform a pseudo-static analysis for a seismic coefficient of 0.1 for magnitude 6.5 earthquakes or 0.15 for magnitude 8.25 earthquakes and obtain a factor of safety of the order of 1.15 to ensure that displacements will be acceptably small".

While Seed simplified his conclusion to make it independent of the peak acceleration, the procedure that he suggested can be used to make more site specific evaluations of appropriate seismic coefficients by referring to the attached figure which is based on the same study by Makdisi and Seed (1978) that Seed used in his 1979 lecture and paper. The figure shows displacements computed by the Newmark method (specifically for embankments ranging in height from 50 to 250 feet, but generally applicable to earth slope with depths to bedrock in that order, and generally conservative for shallower depth to bedrock) as a function of the acceleration ratio, k_y/a_{max}, where k_y is the critical seismic coefficient (that is, the seismic coefficient that reduces the factor of safety to unity) and a_{max} is the expected peak acceleration. Ranges of the most likely displacements are indicated for magnitudes 6.5, 7.5 and 8.25 (magnitude being an indicator of duration of strong shaking) and likely displacements for intermediate magnitudes can be interpolated. The predicted displacements should necessarily be small for magnitudes less than about 6.5 since field experience indicates that smaller magnitude, shorter duration earthquakes do not usually cause significant slope failures. While there are number of approximations made in the Newmark method and in the construction of the attached figure, if the acceleration ratio and magnitude are such that they fall below the line marked "acceptably small displacements", the slope involved might generally be considered to be safe from failure. Thus, for a magnitude 8.25 earthquake, non-failure conditions are indicated if the critical seismic coefficient is at least equal to half the expected peak acceleration. Conversely, if a pseudo-static analysis using a seismic coefficient equal to one-half the peak acceleration yields a factor of safety greater than 1.0, the displacements are likely to be acceptably small. Similarly, for magnitudes 7.5, 7.0 and 6.5, if the seismic coefficient is taken as one-third, one-fourth and one-fifth of the expected peak acceleration, and the computed factor of safety is greater than 1.0, the displacements are likely to be acceptably small. The seismic coefficients obtained in this way are shown as a function of peak acceleration and magnitude in the attached Figure 2.

Sincerely,

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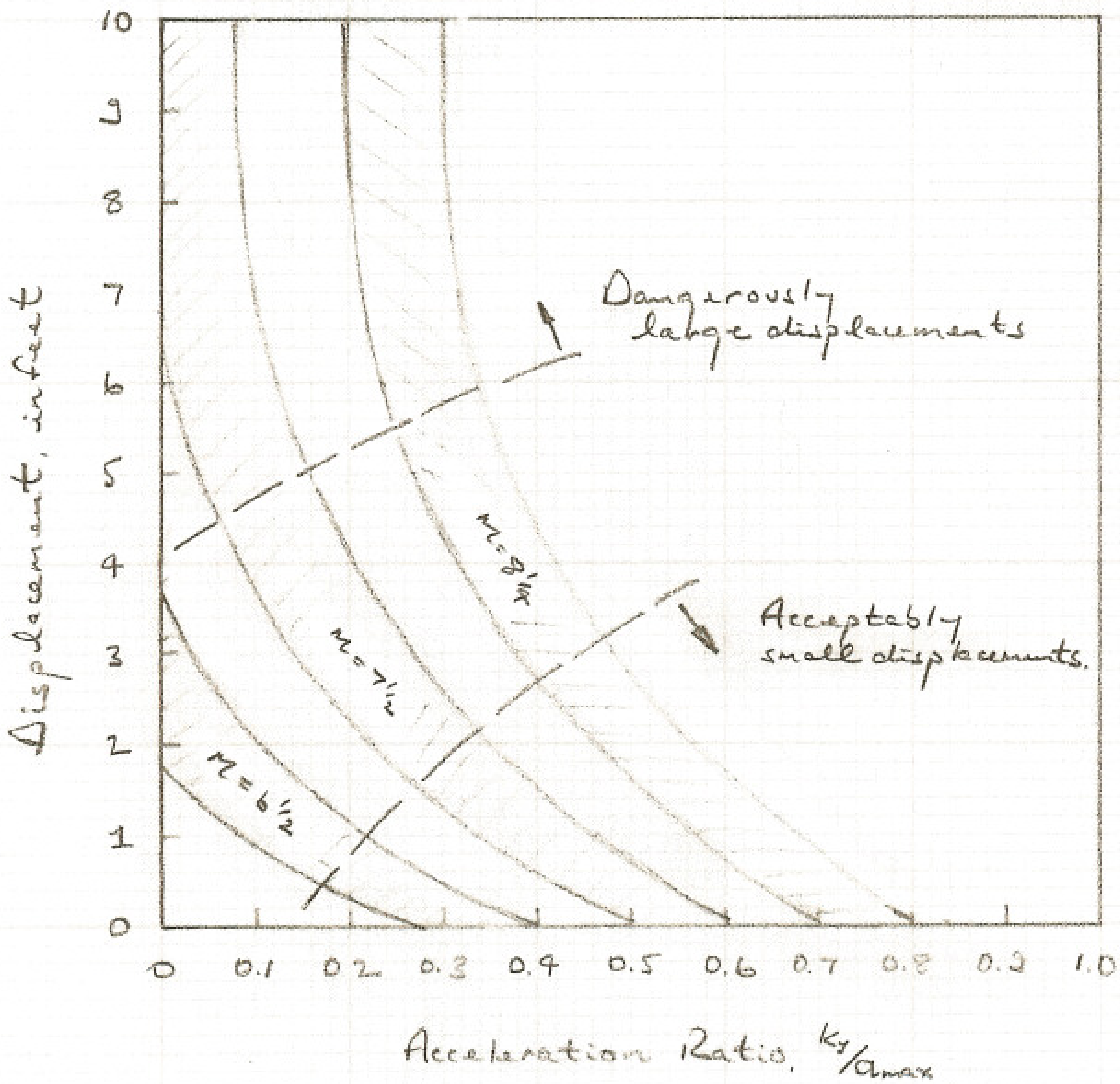

Typical Displacements Computed by Newmark Method

after Makdisi and Seed (1978)

Fig. 1

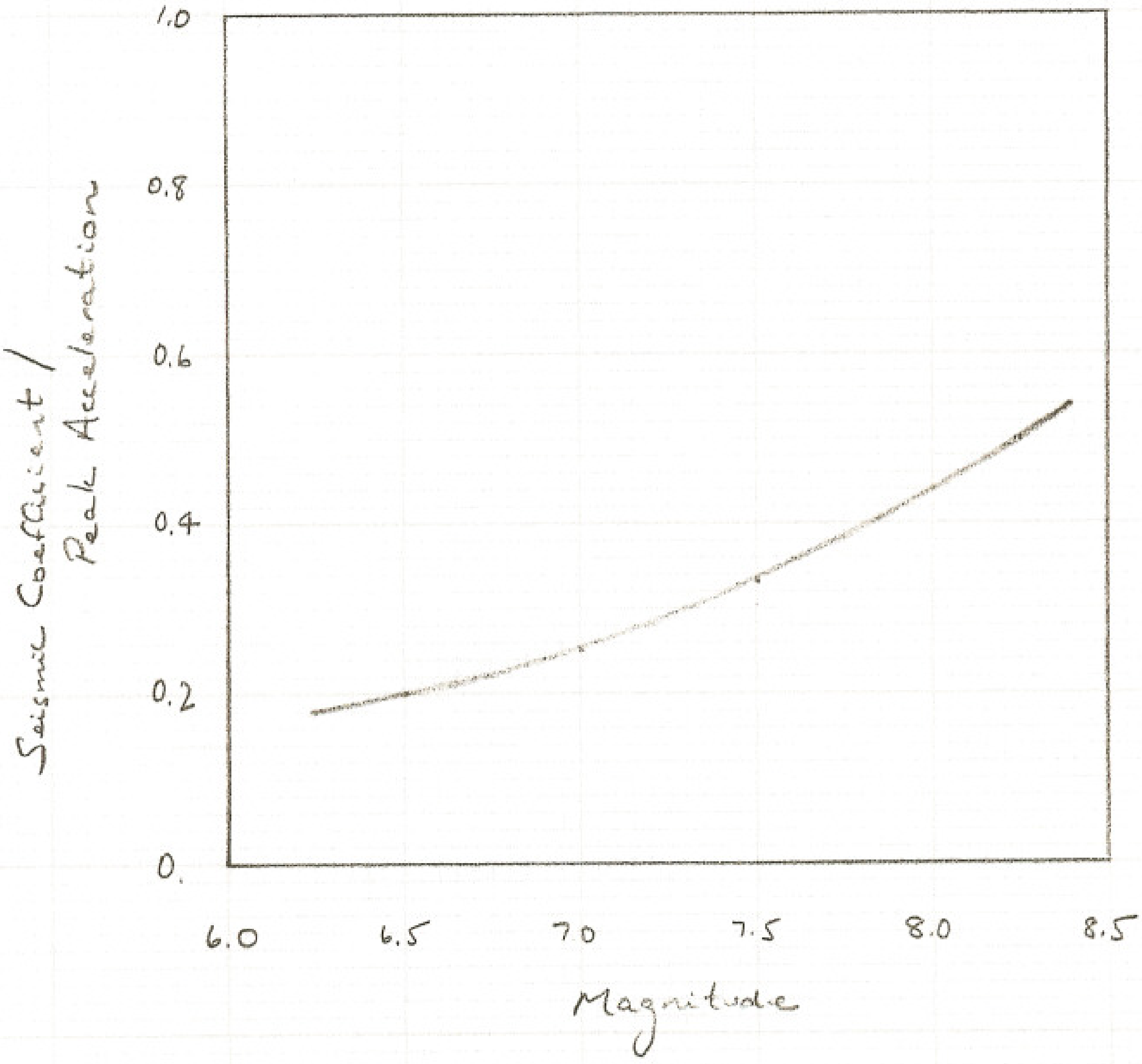
Seismic Coefficient /
Peak Acceleration
1.0
0.8
0.6
0.4
0.2
0.
6.0
6.5
7.0
7.5
8.0
8.5
Magnitude

Fig. 2