San Diego, CA., July 17th, 2003.

LAN LIM & NASCIMENTO ENGINEERING CORP.
Attn. Dr Mohan S. Char

Ref: MIDDLE CHANNEL HUMBOLDT BAY BRIDGE

Dear sirs,

As per your request to Prof Joel P Conte, please find enclosed the documents regarding The Middle Channel Bridge.

Regards,

Gabriel Acero
Graduate Student – UCSD

State of California | Business, Transportation and Housing Agency

Memorandum

To: MR. RAY ZELINSKI
Division of Structure Earthquake Engineering

Attention: Mr. Patrick Hipley

Date: December 1, 2000

File: O1-Hum-255-0.7
01-296701

Middle Channel Bridge
Bridge No. 04-0229

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Geotechnical Earthquake Engineering

Subject: Foundation Design Recommendations

As per your request, this report is prepared to present a summary of the foundation design recommendations provided by our Office in various memoranda during the design phase of the Middle Channel retrofit project, Earthquake Retrofit Project No. 601A. These memoranda were prepared in response to requests from your Office to address specific geotechnical issues during design, and forwarded to the design engineer as completed. On September 25, 2000, copies of these memoranda were forwarded to your Office under one cover letter titled "Geotechnical Reports". A list of these memoranda is also included at the end of this report.

Project Description

The proposed project involves seismic retrofitting of the Middle Channel Bridge (Br. No. 04-228) located in Humboldt County. The Middle Channel Bridge was constructed in 1968 and is one of the three bridges crossing the Humboldt Bay. This bridge is a 329.7-m (1081.5 m) long and 10.4-m (34 feet) wide, has nine spans and consists of pre-stressed I-girders. The bridge is supported by two seat-type abutments and eight single column type piers founded on deep foundations. The deep foundations are consisted of driven pre-cast pre-stressed concrete piles and caps. The abutments are supported on 356-mm (14-inch) square 400-kN (45-ton) piles. The piers M-2, M-8 and M-9 are supported on 356-mm (14-inch) square 625-kN (70-ton) piles and the piers M-3 through M-7 are supported on 1372-mm (54-inch) diameter 1800-kN (200-ton) piles.

Based on the information provided by your Office, the proposed retrofit strategy includes strengthening the foundations of the piers M-2 through M-9 by installing additional four 915-mm (36-inch) diameter cast-in-steel shell (CISS) piles with new or enlarged caps. The retrofit piles at the piers M-2, M-8, and M-9 will be installed with a new cap with bottom elevation at +0.92 m (+3.0 feet). The retrofit piles at the piers M-3 through M-7 will be installed by enlarging the existing pile caps with the bottoms at elevation +0.92 m (+3 feet). The proposed seismic retrofit also includes strengthening the existing concrete columns by installing steel casings.

Scope of Work

In preparing the referenced memoranda and this report, we have reviewed the available in-house geologic data including the 1967 As-Built Log of Test Borings (LOTB) and the results of an additional subsurface exploration conducted in 1994. The 1967 As-Built LOTB consisted of 10 rotary auger borings drilled to a maximum depth of about 30 m (100 feet) below existing ground surface. The borings were located at near the centerline of the bridge. The 1994 subsurface exploration conducted by the Office of Structural Foundations (OSF) included drilling one rotary auger boring to a depth of about 64 m (210 feet) below existing ground surface. Electric, Gamma and P-S Suspension Logging (for Compression and Shear wave velocity measurements) were performed in this boring. The measured shear wave velocities were used in a site-specific dynamic ground response analysis. We also reviewed the available subsurface data including the laboratory test results for the nearby Eureka Channel Bridge (Br. No. 04-228) and the Samoa Channel Bridge (Br. No. 04-0230). Data obtained during the review were analyzed and interpreted, and the referenced memoranda and this report were prepared to present the results of our findings and recommendations for foundation design.

Subsurface Conditions

Based on the above-referenced subsurface data, a generalized soil profile was developed and included in our memo dated March 10, 1999. The bridge alignment, to the maximum explored depth, is underlain by Tertiary and Quaternary Alluvial deposits. A 1.5-m (5-foot) to 3.0-m (10- foot) thick soil layer consisting of mainly soft to very soft organic silt with clay and some construction debris blanket the entire bridge alignment. The surficial layer from near the abutment M-1 to near the pier M-3 is underlain by about 10.8 m (35 feet) of medium dense to dense silty sand and sand with some organic matter. This surficial layer from the vicinity of the pier M-7 to the vicinity of the abutment M-10 is underlain by a layer of soft or loose sandy silt or silty sand with organic matter. The thickness of this layer varies significantly and ranges from about 1.5 m (5 feet) near the pier M-7 and more than 15 m (50 feet) near the abutment M-10. The surficial layer along the remainder of the alignment in the middle is underlain by about 9.2 m (30 feet) of mainly dense silty sand and sand. A 7.6-m (25-foot) to 10.7-m (35-foot) thick layer of mainly very dense sand underlies the above soil layers along the entire alignment. This very dense sand layer is underlain, to the maximum explored depth, by mainly medium dense organic silt and sandy silt and stiff silty clay. Flammable gas was noted in some of the borings drilled in 1967. Detailed soil descriptions are included in the LOTB prepared for this project.

Groundwater

Groundwater was encountered at ground surface in the land borings drilled in 1967. The water level in the channel was at elevation 0.92 m (+3.0 feet) in 1968. Groundwater should be anticipated at sea level, which is likely to vary based on season, time of the day, and weather conditions.

Corrosion

No corrosion study was performed for this project. However, it is our understanding that the designer is aware of the fact that the piles will be exposed to the marine environment and are likely to be subjected to significant corrosion. The Corrosion Technology Branch with the Division of Materials Engineering and Testing Services should be contacted for detailed corrosion recommendations

Faulting and Seismicity

The Humboldt Bay Bridges are located in an area of complex tectonic interaction among the Gorda, the North American and Pacific Plates. The "Little Salmon" fault, which is categorized as one of the principal active fault in California by the California Department of Mines and Geology (CDMG), is the nearest seismic source from the site. This fault is located about 5 km (3 mi) from the Humboldt Bay Bridges and is capable of generating a Maximum Credible Earthquake of Moment Magnitude M_w=7.5. Geomatrix Consultants (1994) conducted a site-specific seismic ground motion study for Caltrans. Based on this study, the Peak Bedrock Acceleration (PBA) at this location was estimated to be about 0.9g. The Geomatrix Consultant report may be consulted for more information on the geology, faulting and seismicity.

Seismic Hazards

The site is not considered susceptible to surface rupture due to fault movement during seismic events since no known fault crosses or extends toward the site.

Site specific seismic ground response analyses were conducted by this Office to develop design Acceleration Response Spectrum (ARS) for the Humboldt Bay Bridges. The recommended design ARS corresponding to the mean (50^{th} percentile) PBA of 0.7g was presented in our memo dated February 24, 1999. Based on this analysis, the peak horizontal ground acceleration at the site was estimated to be on the order of 0.4g.

In addition to a strong ground shaking, seismic hazards at the site include soil liquefaction, seismically-induced total/differential settlement, slope stability, and lateral spreading. Results of a detailed study conducted to evaluate these hazards were presented in our memo dated March 10, 1999. Based on our analysis, the abutment M-1 and the pier M-2 are underlain by potentially liquefiable soils between approximate elevations of –2.4 m (–8 feet) and -10 m (–33 feet). The seismically-induced total ground settlement is estimated to be on the order of 100 mm (4 inches) to 125 mm (5 inches). Surficial non-liquefiable layer and the sloping liquefiable soils in this area are considered prone to lateral spreading. It is estimated that in the even of soil liquefaction, the piles at Abutment M-1 and Pier M-2 may experience an additional lateral load on the order of 700 to 790 $kN/m/m^3$ (4.5 to 5.0 $kip/ft/ft^2$).

A pseudo-static slope stability analysis was performed to evaluate the stability of the approach embankment at Abutment 1. The critical acceleration for the slope corresponding to a factor of safety of 1.0 was found to be 0.13g. As stated above, the 50^{th} percentile peak horizontal ground acceleration was estimated to be 0.4g for this site. The corresponding permanent slope displacement was estimated to be on the order of 0.61 m (2 feet) to 0.91 m (3 feet).

Due to the location of the bridge, the potential for hazards associated with tsunami generated by the movements of the nearby or distant off shore faults exist at this site.

Foundation Design Recommendations

Foundation recommendations were presented by this Office in various memoranda during design on an as needed or as-requested basis. As stated previously, site soils underneath the abutment M-1 and the pier M-2 are susceptible to liquefaction during earthquakes. It is recommended that the shear strength of the potentially liquefiable soils be neglected in evaluating pile axial capacity under earthquake loading conditions. Furthermore, additional lateral load due to lateral spreading as recommended above should be considered in the evaluation of the lateral load capacity of both the existing and the proposed retrofit piles.

Soil parameters required to run the COMP624 computer program were presented in a memo dated April 15, 1999. An analysis was performed to evaluate the axial capacities (compression and tension) of the existing piles with and without considering liquefaction. Negative skin friction due to post-liquefaction ground settlement was considered in this analysis. The estimated axial pile capacities under both compression and tension (uplift) were presented in our memo dated May 18, 1999.

An analysis was also performed to develop p-y curves for the existing piles in accordance with the general procedure suggested in the documentation for the computer program GROUP (version 3.0). The resulting p-y curves and the co-ordinates of the corresponding data points were presented in our memo dated June 21, 1999.

Preliminary foundation stiffnesses based on a soil-foundation interaction analysis were presented in our memo dated May 4, 2000. As the design evolved, additional soil-foundation interaction analyses were performed to evaluate foundation stiffnesses for both the existing and the proposed retrofit foundations with different pile diameters. Results of these analyses were presented in our memoranda dated May 30, 2000 and June 1, 2000. The recommended axial load- settlement curves for the existing piles and the proposed 610-mm (24-inch) diameter CISS piles were presented in a memo dated June 5, 2000. The corresponding curves for the proposed 915-mm (36-inch) diameter CISS piles were presented in our memo dated July 17, 2000.

The recommended pile tip elevations based on a laterally loaded pile analysis for the proposed 915-mm (36-inch) diameter piles are presented in Table 1 below. These tip elevations are based on obtaining at least two zero moment points along the pile length during laterally load analysis. Note that both the design and the specified tip elevations are based on lateral response (load) only. However, the geotechnical axial (both compression and tension) capacities of the piles when founded at the recommended tip elevations are estimated to be either equal to or exceed the corresponding nominal resistances shown in Table 1. These results were also presented in our memo dated September 18, 2000.

Construction Considerations

This project will involve over-water construction in the bay requiring the use of trestle or spud barges to access the mudflats, and deep water barges. Restricted headroom conditions exist for pile driving at the locations of the piers M-2, M-8 and M-9. Welded pile extensions may be required at these locations. Hard pile driving conditions should be anticipated in the dense and very dense sandy soils, which may require drive shoes (driving tips) to facilitate driving and/or to ensure pile integrity during hard driving. Center relief drilling may be used if piles reach refusal before the design tip elevation. Such drilling may be allowed to extend to a maximum of 0.6 m (2 feet) below the bottom of the CISS pile except within 2 m (5 feet) of the design tip elevation. Jetting or predrilling can result in significant reduction in the lateral capacity of the piles and should not be allowed. Pile acceptance shall be based on reaching the design tip elevation. Verification of the axial nominal resistance is not required for pile acceptance. Because of high groundwater conditions, adequate water/slurry pressure head may need to be maintained in the shell to prevent soil or concrete plug blowout in artesian-like conditions from occurring inside the steel shell. Prior to pouring concrete, the inside surface of the CISS piles shall be cleaned of all substances and contaminants that can inhibit bonding. There is restriction on sound level during breeding season for birds on Indian Island that may limit the time of the year when piles can be driven for this bridge. The excavated soils from the CISS piles will need to be dewatered in sediment bins prior to disposal off-site.

Table 1. CISS Pile Tip Elevations Based on Laterally Loaded Pile Analysis

Location/ Type	Design Loading (Service), kN (kips)	Nominal Resistance, kN (kips)		Cut-off Elevation, m (ft)	Design Tip Elevation, m (ft)	Specified Tip Elevations, m (ft)
		Compression	Tension			
Pier M-2/ PP 915- mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-16.8 (-55)**	**-16.8 (-55)**
Pier M-3/ PP 915- mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-19.8 (-65)**	**-19.8 (-65)**
Pier M-4/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-19.8 (-65)**	**-19.8 (-65)**
Pier M-5/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-19.8 (-65)**	**-19.8 (-65)**
Pier M-6/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-19.8 (-65)**	**-19.8 (-65)**
Pier M-7/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-19.8 (-65)**	**-19.8 (-65)**
Pier M-8/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-16.8 (-55)**	**-16.8 (-55)**
Pier M-9/ PP 915-mmx12.7-mm (36-inchx1/2-inch)	**N/A**	**2670 (600)**	**1335 (300)**	**+0.92 (+3.0)**	**-18.3 (-60)**	**-18.3 (-60)**

If you have any questions or comments, please call Mohammed S. Islam at (916)227-7094 or Abbas Abghari at (916)227-7172.

MOHAMMED S. ISLAM
Transportation Engineer

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED S. ISLAM
NO. 2485
EXP. 06/30/03
GEOTECHNICAL
STATE OF CALIFORNIA

A. Abghari
ABBAS ABGHARI, Chief
Office of Geotechnical Earthquake Engineering

Attachment: List of Memoranda

List of Memoranda

1. *Site Specific ARS Curves*, prepared for the Foundation Studies North Section, dated December 20, 1994.

2. *Acceleration Response Spectra for the Humboldt Bay Bridges*, prepared for the Office of Structure Design (OSD), dated January 20, 1999.

3. *Acceleration Response Spectra for the Humboldt Bay Bridges*, prepared for the OSD, dated February 24, 1999.

4. *Soil Liquefaction Potential Evaluation for the Middle Channel Bridge*, prepared for the OSD, dated March 10, 1999.

5. *Soil Parameters for COM624 Computer Program for the Middle Channel Bridge*, prepared for the OSD, dated April 15, 1999.

6. *Pile Bearing Capacity under Axial Loading for the Middle Channel Bridge*, prepared for the OSD, dated May 18, 1999.

7. *p-y Curves for the Middle Channel Bridge*, prepared for the OSD, dated June 21, 1999.

8. *Preliminary Foundation Stiffness*, prepared for the Office of Earthquake Engineering (OEE), dated May 4, 2000.

9. *Final Foundation Stiffness*, prepared for the OEE, dated May 30, 2000.

10. *Foundation Stiffness for the Existing and the New Pile Groups*, prepared for the OEE, dated June 1, 2000.

11. *Axial (Compression and Tension) Load-Settlement Curves*, prepared for the OEE, dated June 5, 2000.

12. *Axial (Compression and Tension) Load-Settlement Curves for 36" Diameter Pile*, prepared for the OEE, dated July 17, 2000.

13. *Pile Tip Elevations Based on Laterally Loaded Pile Analysis*, prepared for the OEE, dated September 18, 2000.

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. PATRICK HIPLEY
Office of Earthquake Engineering

Date: September 25, 2000

File: 01-Hum-255-0.7
01-296701

Middle Channel Bridge
Bridge No. 04-0229

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations – MS 5
Office of Geotechnical Earthquake Engineering

Subject: Geotechnical Reports

As per your request, dated September 20, 2000, enclosed are copies of the geotechnical reports prepared by this Office for the Middle Channel Bridge (Br. No. 04-0229) Earthquake Retrofit Project No. 601A. The following reports are included:

1. *Site Specific ARS Curves*, prepared for the Foundation Studies North Section, dated December 20, 1994.

2. *Acceleration Response Spectra for the Humboldt Bay Bridges*, prepared for the Office of Structure Design (OSD), dated January 20, 1999.

3. *Acceleration Response Spectra for the Humboldt Bay Bridges*, prepared for the OSD, dated February 24, 1999.

4. *Soil Liquefaction Potential Evaluation for the Middle Channel Bridge*, prepared for the OSD, dated March 10, 1999.

5. *Soil Parameters for COM624 Computer Program for the Middle Channel Bridge*, prepared for the OSD, dated April 15, 1999.

6. *Pile Bearing Capacity under Axial Loading for the Middle Channel Bridge*, prepared for the OSD, dated May 18, 1999.

7. *p-y Curves for the Middle Channel Bridge*, prepared for the OSD, dated June 21, 1999.

8. *Preliminary Foundation Stiffness*, prepared for the Office of Earthquake Engineering (OEE), dated May 4, 2000.

9. *Final Foundation Stiffness*, prepared for the OEE, dated May 30, 2000.

10. *Foundation Stiffness for the Existing and the New Pile Groups*, prepared for the OEE, dated June 1, 2000.

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12. *Axial (Compression and Tension) Load-Settlement Curves for 36" Diameter Pile,* prepared for the OEE, dated July 17, 2000.

13. *Pile Tip Elevations Based on Laterally Loaded Pile Analysis,* prepared for the OEE, dated September 18, 2000.

If you have any questions or comments, please call Mohammed S. Islam at 227-7094 or Abbas Abghari at 227-7172.

MOHAMMED S. ISLAM
Transportation Engineer

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED S. ISLAM
NO. 2485
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STATE OF CALIFORNIA

ABBAS ABGHARI, Chief
Office of Geotechnical Earthquake Engineering

Attachments

State of California

Business, Transportation and Housing Agency

Memorandum

To : MR. RICHARD W. FOX, Chief
Foundation Studies North Section

Date : December 20, 1994

File No. : 01-HUM-255-0.2/1.2
01-296700
Seismic Retrofit
Humboldt Bay Bridge

Br. Nos. : 04-0228, 04-0229, 04-0230

From : DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
Office of Structural Foundations

Subject : Site Specific ARS Curves

This memo presents the results of site response analysis for Humboldt Bay Bridges and outlines the main assumptions and limitations considered in this analysis.

The Humboldt Bay Bridge lies in an area of complex tectonic interaction among the Gorda, North American and Pacific plates. Geomatrix Consultants has performed a seismic ground motion study of this area for Caltrans. The results can be found in the report "Seismic Ground Motion Study for Humboldt Bay Bridges on Route 255" in which they developed three component rock motions for the controlling fault, "Little Salmon," with a maximum credible earthquake of moment magnitude 7.5. This fault is located 5 km (3 mi) from Humboldt Bay Bridges. The peak bedrock acceleration at this location was estimated about 0.9g. Three rock motions (longitudinal, transverse, and vertical) with a response spectrum matching the magnitude 7.5 target spectrum for these bridges were developed by Geomatrix. For information about the tectonics, seismicity, and geology of the area refer to the above mentioned report by Geomatrix Consultants.

A vicinity map of the Humboldt Bay Bridge is shown on Figure 1. Also shown in the figure are the locations of borings where shear wave velocities were measured. Site response analyses were conducted based on the information from these borings including shear wave. However, our design recommendations are based only on measurements conducted in August 1994 because of the higher reliability of the Suspension P-S Logging System and its information about deep soil layers. For more information about subsurface investigation refer to Ken Cole's report "Shear Wave Velocity Measurements at Humboldt Bay Bridges."

According to the Geomatrix report, the bedrock is estimated to be at a depth ranging from 300 m (1000 ft) to 600 m (2000 ft). Since the deepest site for which "Shake" program results have been compared to earthquake recorded data is about 150 m (500 ft), special sensitivity analysis techniques were applied in the analysis of this deep site. Within the last year, the Electrical Power Research Institute (EPRI) has developed a set of modulus reduction and damping curves for deep sites up to 300 m (1000 ft). A series of sensitivity analysis was conducted in order to determine the range of variation of soil response when a combination of three different site depths (244 m (800 ft), 366 m (1200 ft), and 458 m (1500 ft) with three different modulus reduction and damping curves (Dobry, et.al. 1987, Idriss 1992, EPRI 1993) is used. In terms of depth, this sensitivity analysis showed very little differences in site response due to the high measured shear wave velocities, about 600 m/sec (2000 fps), beyond 214 m (714 ft) depth. The variation of the modulus reduction curve showed also very little impact on the results. However, the soil response proved to be very sensitive to the damping curve used. The EPRI damping curves produced an unusual high acceleration level for structure periods of 1.5 to 2.5 seconds. The EPRI results were not included in the design because they induced soil strains on the order of 10% well above the validation limits of the program "Shake."

In another set of sensitivity analysis, different intensities of the input motion from 10% to 100% were applied to all three sites. The Dobry 1987 damping curves resulted in consistent spectral shape for the output motions. When using EPRI 93 damping curves anomalous amplifications occurred, accompanied by excessively high strains.

The "Shake" results can be found on Figures 2 to 4. Due to the similarity of these curves, only one design ARS curve is recommended for all three bridges. This curve is shown on Figure 5. These acceleration response spectra were computed 1.5 m (5 ft) below ground surface at the approximate location of the pile cap for all bents on land. Caution should be applied when using these results for the bents in the middle of the channel, where the pile cap is at sea level elevation, many feet above ground surface. In this case, we recommend a dynamic soil-pile interaction analysis to determine the response at the pile cap elevation which we can provide, if requested.

If you have any questions or comments, please call Angel Perez-Cobo at 227-7167 or Abbas Abghari at 227-7172.

Angel Perez-Cobo

ANGEL PEREZ-COBO
Transportation Engineer
Earthquake Engineering Section

A. Abghari

ABBAS ABGHARI, Acting Chief
Associate Materials & Research Engineer
Earthquake Engineering Section

REGISTERED PROFESSIONAL ENGINEER
ABBAS ABGHARI
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Exp. 12/31/94
CIVIL
STATE OF CALIFORNIA

Attachments

AP-C/jlm

cc: ELeivas

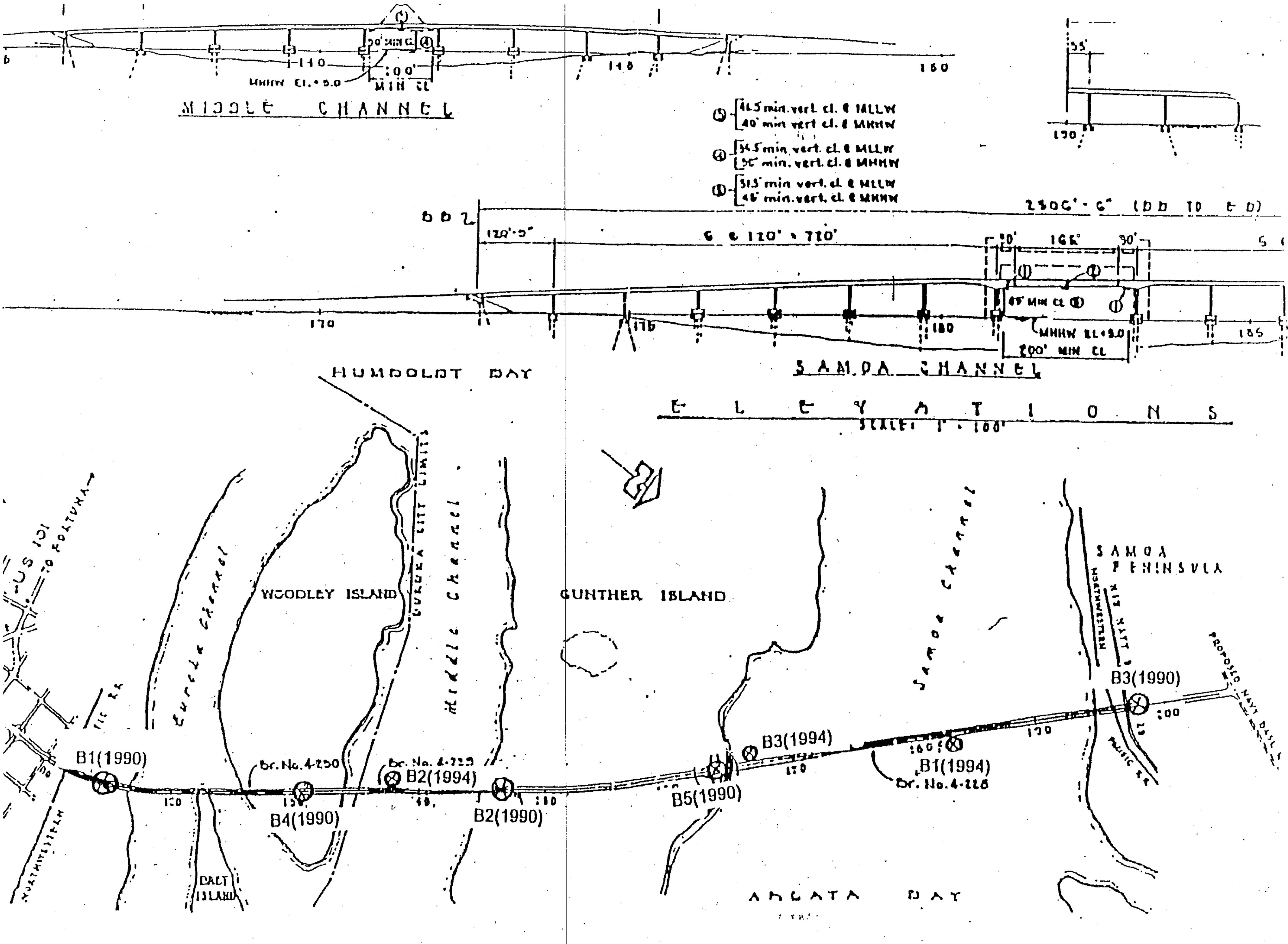

Figure 1. Site Plan of Humboldt Bay Bridges

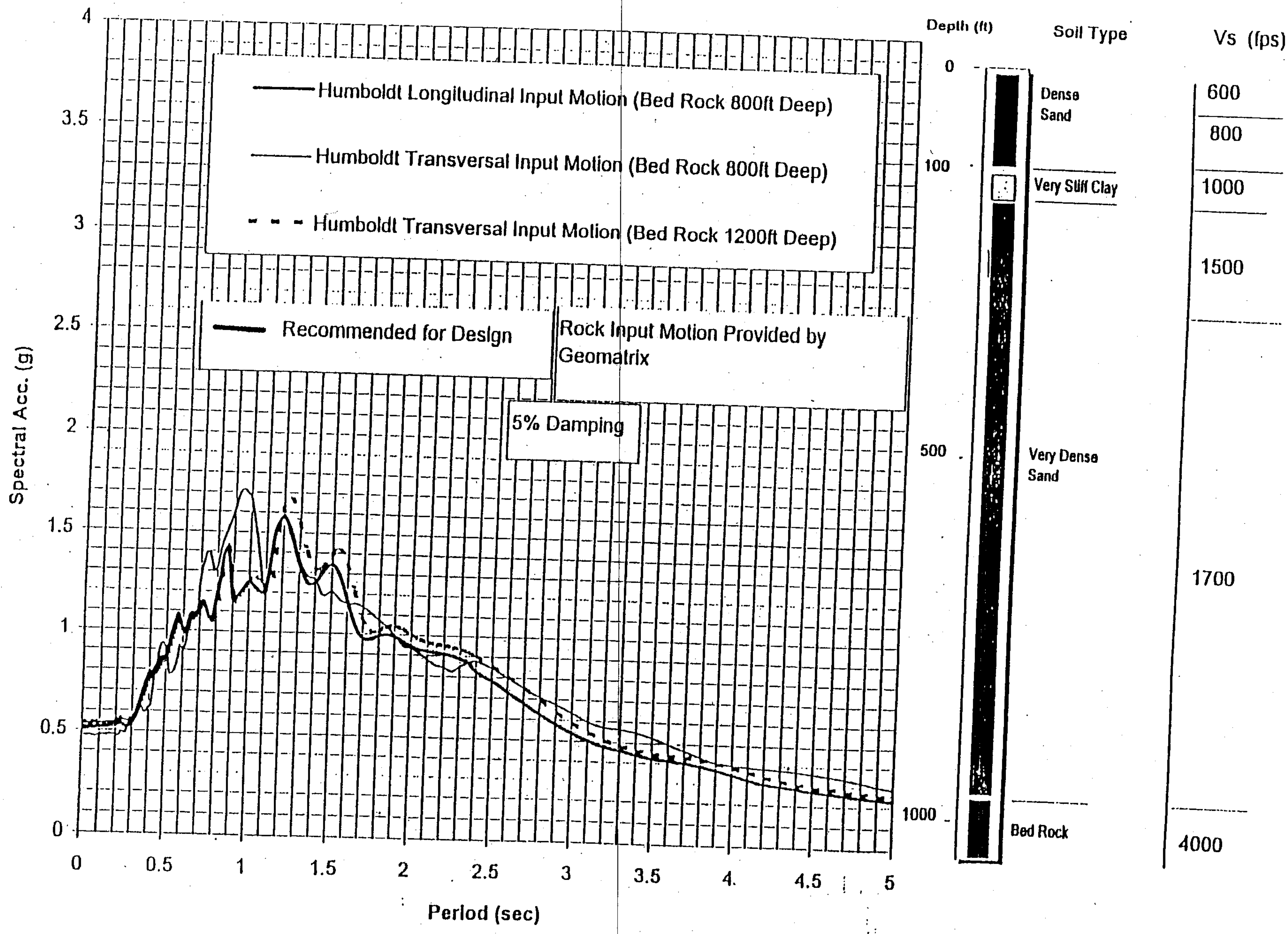

Figure 2. Acceleration Response Spectra for Humboldt Bay Bridge at Samoa Channel

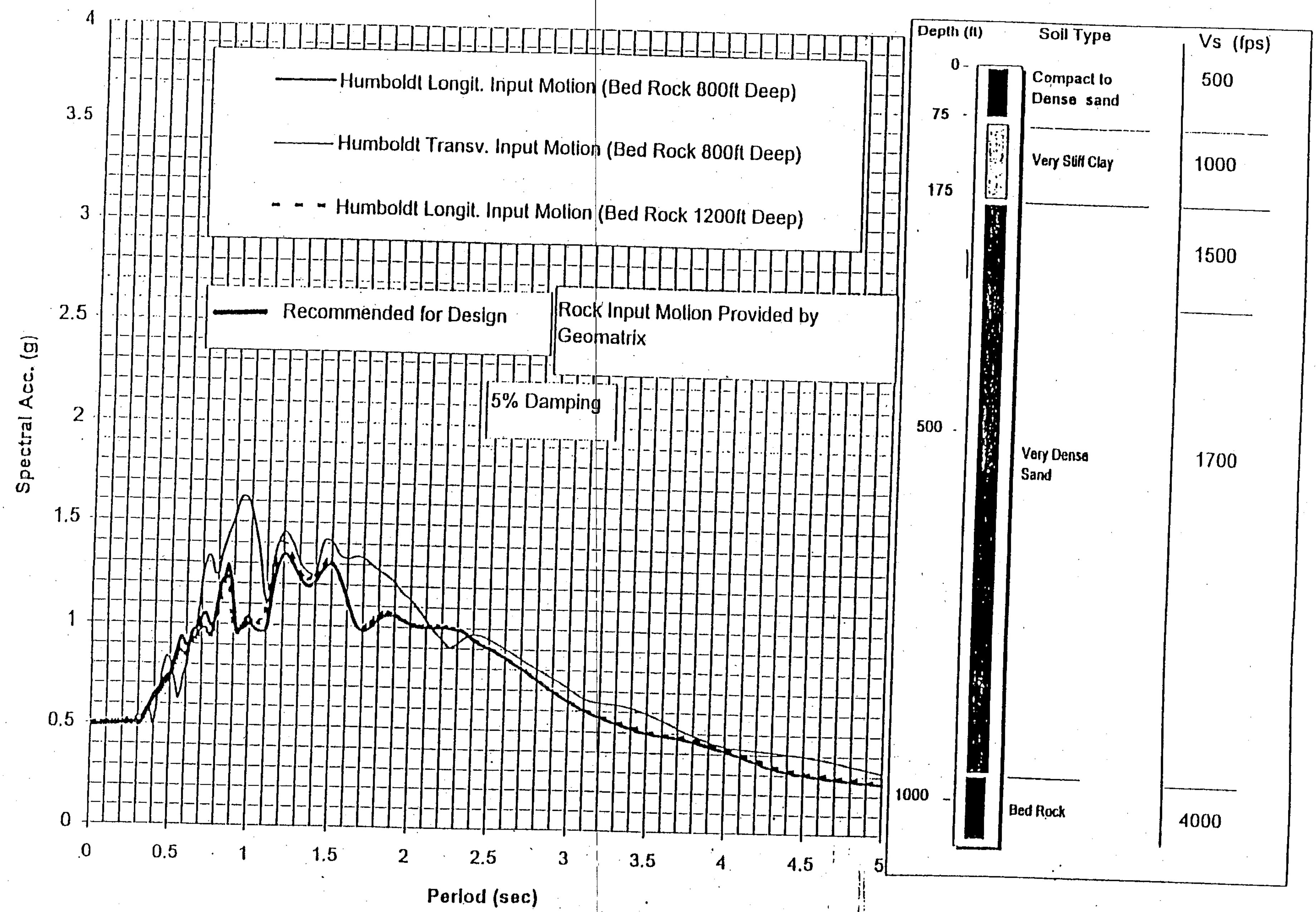

Figure 3. Acceleration Response Spectra For Humboldt Bay Bridge at Middle Channel

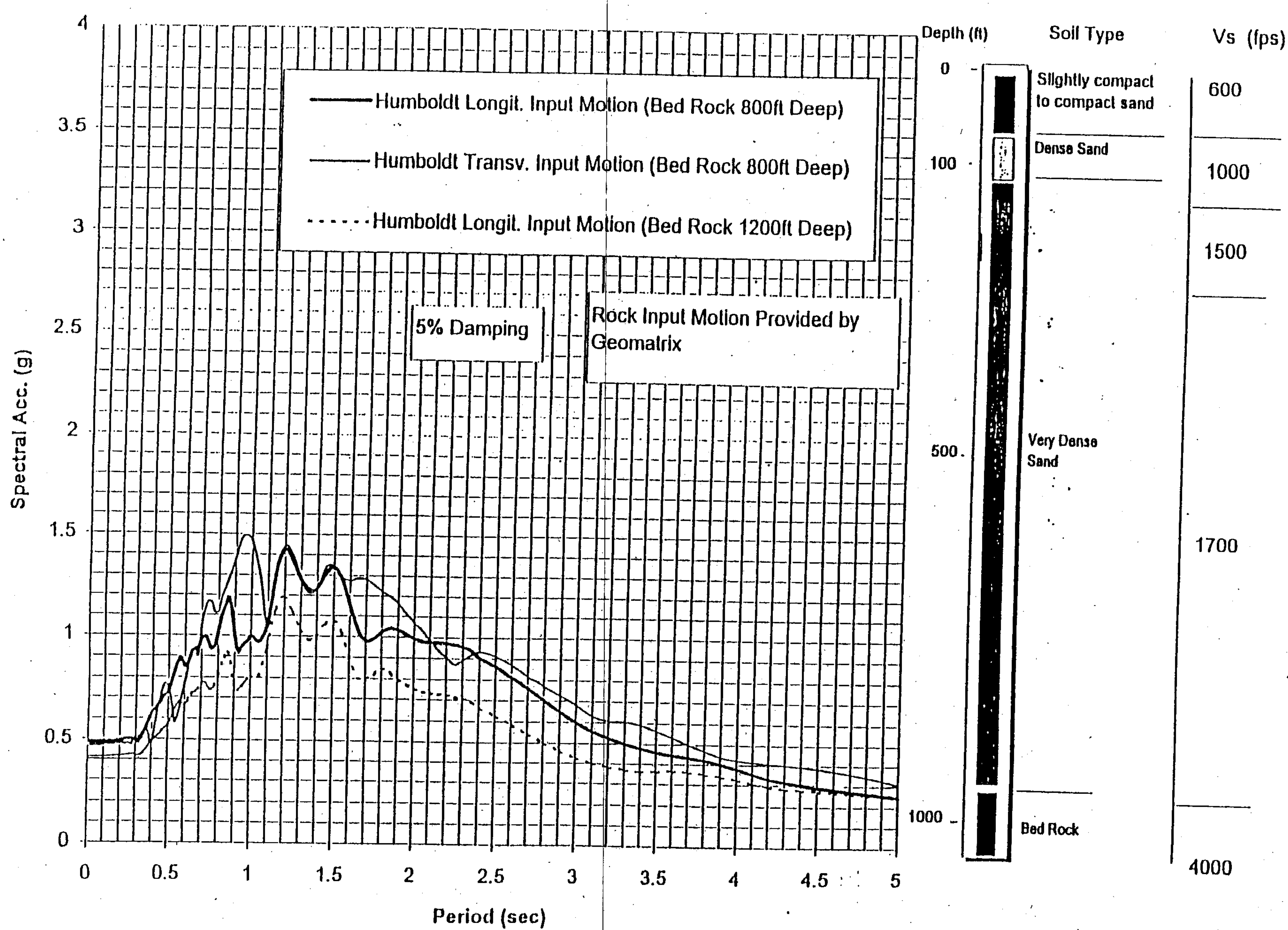

Figure 4. Acceleration Response Spectra for Humboldt Bay Bridge at Gunther Island (Boring B2-Samoa)

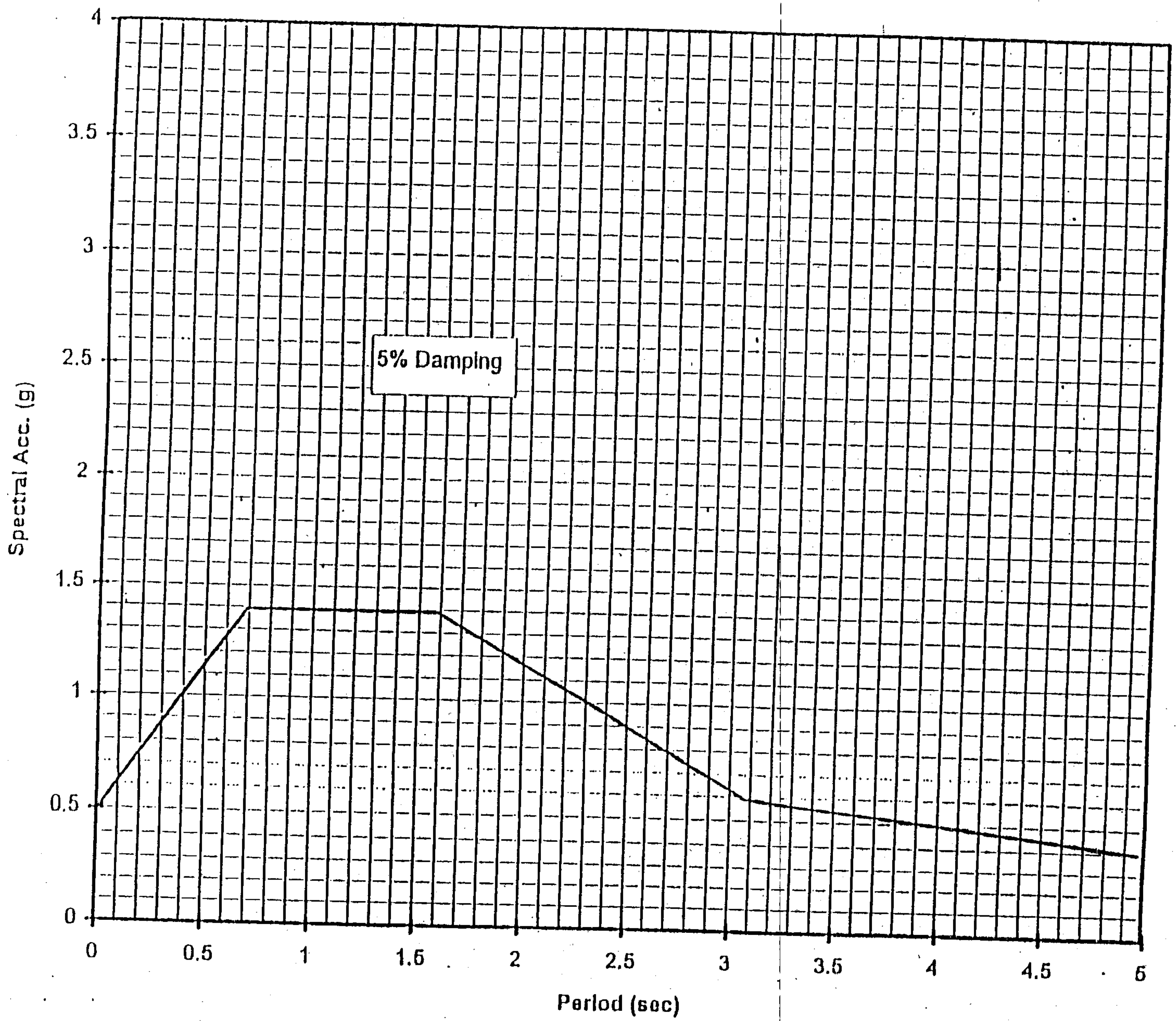

Figure 5. Design Acceleration Response Spectrum for Humboldt Bay Bridges

Memorandum

To : MR. TOM POST
Office of Structure Design

Attention: Mr. Saad El-Azazy

Date : January 20, 1999

File : 01-HUM-255-0.2/1.2
01-296701
Humboldt Bay Bridge

Br. No : 04-0228
04-0229
04-0230

From : **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials and Foundations
Structure Foundations Branch

Subject : Acceleration Response Spectra for the Humboldt Bay Bridges

Site specific Acceleration Response Spectra (ARS) for the Humboldt Bay Bridges were provided in our memo dated December 20, 1994. These ARS were developed using the rock motions from the Geomatrix, Inc. report entitled "Seismic Ground Motion Study for Humboldt Bay Bridges on Route 255. Contract No. 59N772" dated March 1994. Based on this report the controlling fault for these bridges is the Little Salmon with a Maximum Credible Earthquake (MCE) of magnitude 7.5 located at a distance of approximately 5 km from these bridge. The rock target spectra corresponded to the mean plus one standard deviation (84th percentile) as outlined in Section 5.5 of the Geomatrix report. Therefore, the site specific ARS provided in 1994 corresponded to the 84th percentile MCE. The 84th percentile spectra have been used for the evaluation of important bridges and also for the safety evaluation of the toll bridges. However, other bridges have been retrofitted for mean events. Thus, the design ARS for Humboldt Bay Bridges may be revised to correspond to the mean (50th percentile) event on the controlling fault.

If you have any questions, please call me at 227-7172.

A. Abghari

ABBAS ABGHARI, Chief
Geotechnical Earthquake Engineering Section

cc: ELeivas – SFB
RZelinski - OSD

State of California | Business, Transportation and Housing Agency

Memorandum

To : MR. RAY ZELINSKI
Office of Structure Design

Attention: Saad El-Azazy

Date : February 24, 1999

File : 01-HUM-255-0.2/1.2
01-296701
Humboldt Bay Bridges

Br. Nos.: 04-0228, 04-0229, 04-0230

From : **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials and Foundations
Structure Foundations Branch

Subject : Acceleration Response Spectra for Humboldt Bay Bridges

This memo presents the results of our site response analysis for the Humboldt Bay bridges corresponding to different Peak Bedrock Acceleration (PBA).

Site specific design curves for the mean plus one standard deviation were provided in our memo dated December 20, 1994 (copy attached) which included a description of the geology and seismicity of the site. Site response analyses for different PBA were conducted by scaling the rock motion provided by Geomatrix in the report entitled " Seismic ground motion study for Humboldt Bay Bridges on Route 255." The results are shown in Figure 1. The recommended design Acceleration Response Spectrum (ARS) corresponding to the mean (50th percentile) rock motion (PBA=0.7g) is shown in Figure 2.

If you have any questions, please call Angel Perez-Cobo at 227-7167 or Abbas Abghari at 227-7172.

ANGEL PEREZ-COBO
Associate Materials & Research Engineer
Earthquake Engineering Section

ABBAS ABGHARI, Chief
Earthquake Engineering Section

Attachments

cc: ELeivas - SFB

REGISTERED PROFESSIONAL ENGINEER
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No. C 053543
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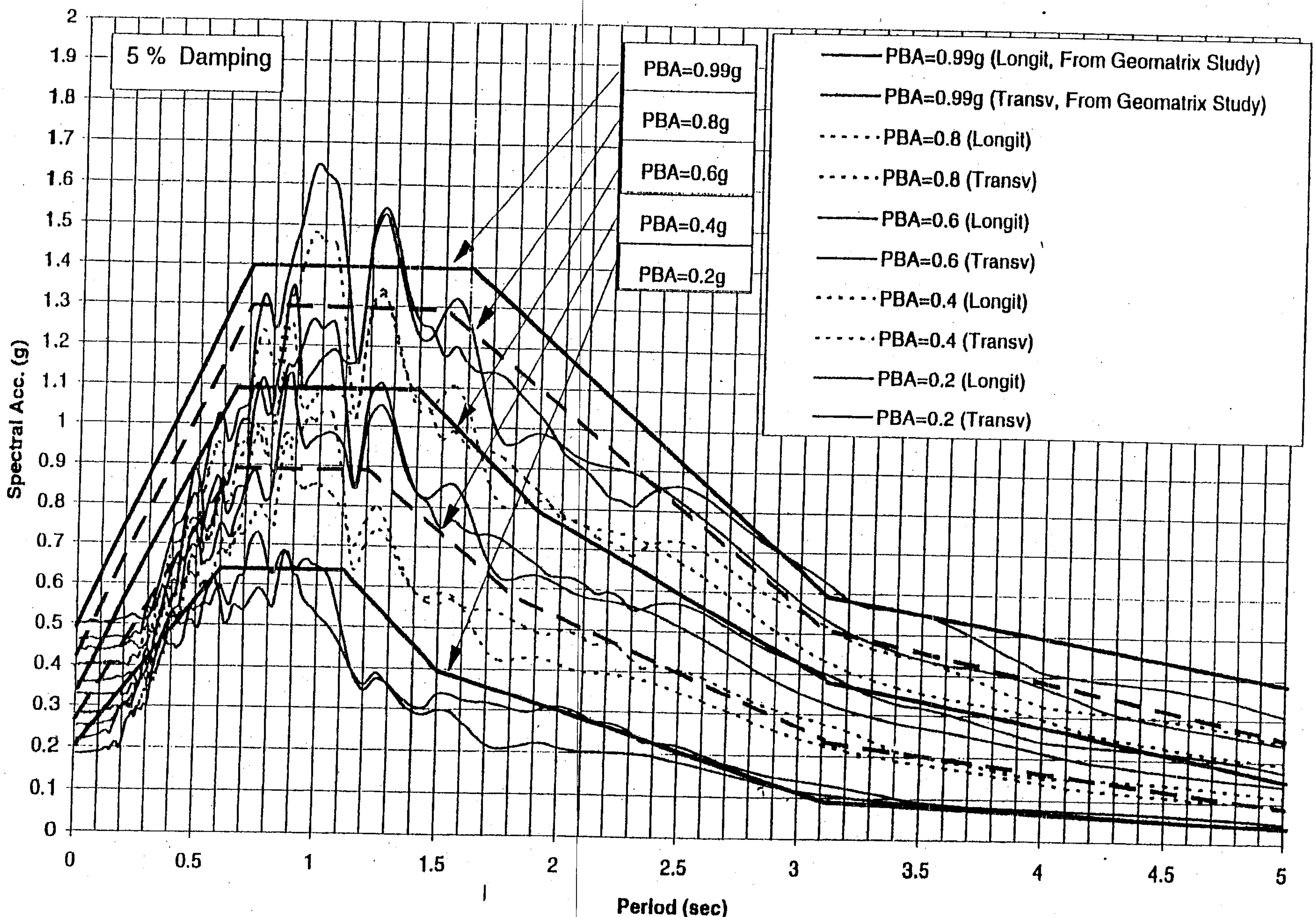

Figure 1. Comparison of Acceleration Response Spectra for Different Peak Bedrock Accelerations at Humboldt Bay Bridge.

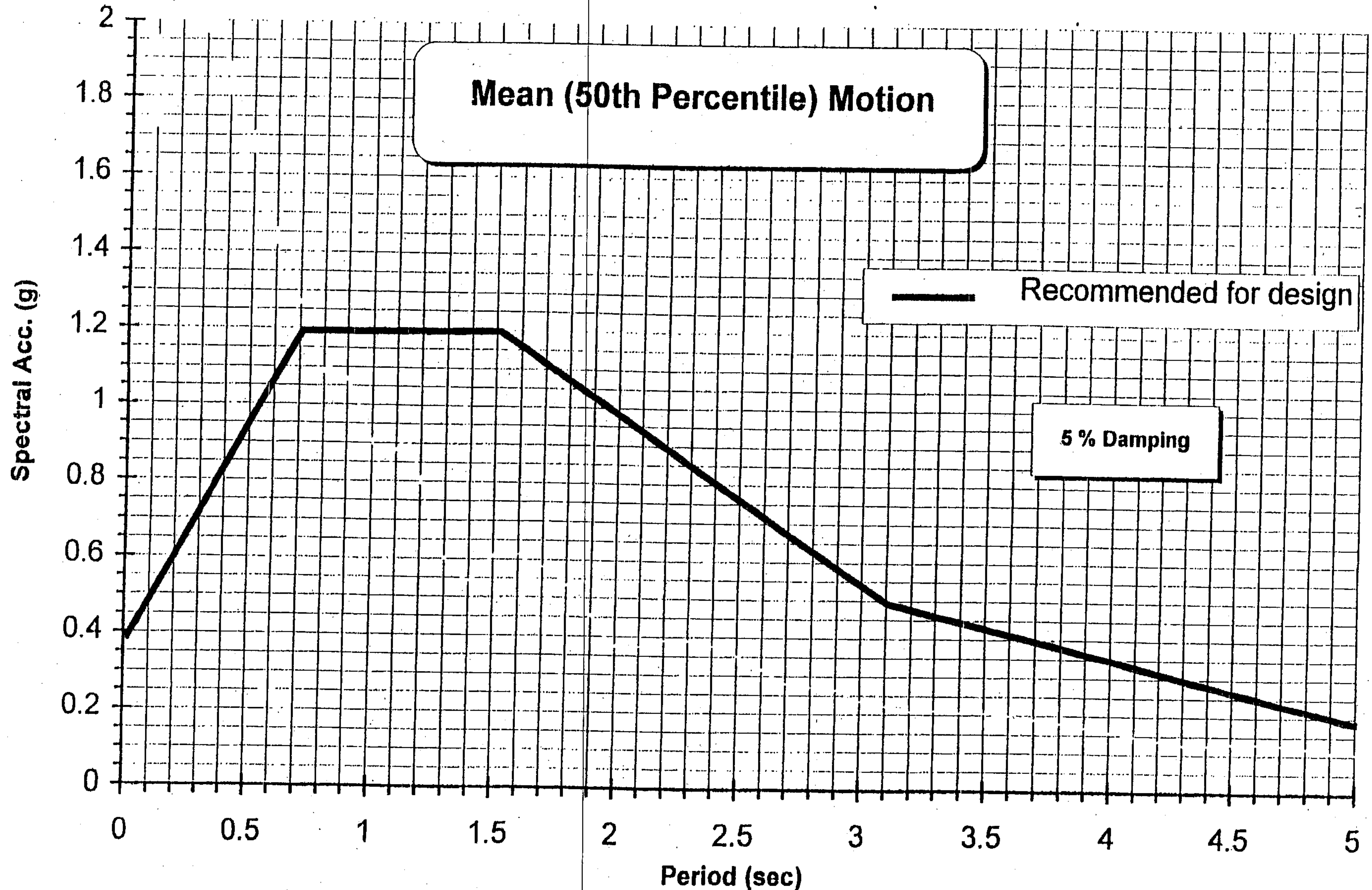

Figure 2. Recommended Design ARS for the Humboldt Bay Bridges.

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. TOM POST
Office of Structure Design

Attention: Mr. Saad El-Azazy

Date: March 10, 1999

File: 01-HUM-255-0.7
01-296701

Middle Channel Bridge
Bridge No. 04-0229

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials and Foundations - MS 5
Structure Foundations Branch

Subject: Soil Liquefaction Potential Evaluation for the Middle Channel Bridge

This memo presents the results of our analysis performed to evaluate soil liquefaction potential at the site of the Middle Channel Bridge (Br. No. 04-229).

A vicinity map of the Middle Channel Bridge is shown on Figure 1. A description of the site geology and seismicity including site-specific Acceleration Response Spectra (ARS) was provided in our memo dated December 20, 1994 (attached). Site-specific ARS were later revised and presented in a memo dated February 24, 1999 (attached). The recommended design ARS was revised to correspond to the mean (50th Percentile) Peak Bedrock Acceleration (PBA=0.70g) generated by a Maximum Credible Earthquake (MCE) of moment magnitude, M_w = 7.5 associated with the. Little Salmon fault. This controlling fault is located approximately 5.0 km from the bridge site. Based on the recommended design ARS, a peak horizontal ground surface acceleration of 0.40g should be anticipated at the site due to the mean PBA of 0.70g. Therefore, our liquefaction analysis was based on a mean peak horizontal ground surface acceleration of 0.40g.

A generalized soil profile along the approximate centerline of the Middle Channel Bridge was developed based on nine (9) soil borings drilled in 1967. The generalized soil profile depicting our interpretation of the subsurface conditions at the site is shown on Figure 2. The soil profile at each boring location was analyzed for liquefaction potential in accordance with the procedure suggested by Seed et al (1985).

Results of our analysis indicate the presence of a liquefiable soil layer at the location of Abut M-1 and Pier M-2, as shown on Figure 2. The approximate elevation limits and the total thickness of potentially liquefiable soils at each support location are presented in Table 1. Also included in Table 1 is seismically -induced ground surface settlement estimated based on the procedure suggested by Tokimatsu and Seed (1987) and lateral spread susceptibility at each support location. The elevations in Table 1 are based on the As-Built Log of Test Borings (LOTB), Drawing No. 0229-42, and Contract No. 01-077504.

Table 1, Summary of Liquefaction Potential Evaluation for the Middle Channel Bridge

Support Location	Approx. Elevation of Liquefiable Soils (ft)	Approx. Thickness of Liquefiable Soils (ft)	Estimated Seismically-Induced Ground Surface Settlement (inch)	Lateral Spread Susceptibility	Recommended Lateral Load per Unit Pile Width per Unit Pile Length (Kips/ft/ft)	Recommended Elevation Range for the Application of Uniformly Distributed Lateral Load (ft)
Abutment M-1	-8 to –33	25.0	5.0	Moderate	5.0	+1.0 to –15.0
Pier M-2	-10 to –28	18.0	4.0	Moderate	4.5	-6.0 to –15.0
Pier M-3 thru M-9 and Abutment M-10	None	None	Minimal	None	0.0	N/A

The shear strength of the potentially liquefiable soils should be neglected in evaluating pile axial capacity under earthquake loading. Additional lateral loads due to lateral spreading of the near surface soils should be applied to the Abut. M-1 and Pier M-2 piles as recommended in Table 1. Alternatively, the potentially liquefiable soils can be mechanically improved to increase their resistance to liquefaction and lateral loading. If requested, this office can provide recommendations regarding soil improvement at this site.

If you have any questions or comments, please call Mohammed S. Islam at 227-7094 or Abbas Abghari at 227-7172.

MOHAMMED S. ISLAM
Transportation Engineer

A. Abghari

ABBAS ABGHARI, Chief
Geotechnical Earthquake Engineering Section

Attachments

cc: ELeivas - SFB

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED S. ISLAM
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STATE OF CALIFORNIA

References

Tokimatsu, K. and Seed, H. B. (1987), "Evaluation of settlement in sands due to earthquake shaking," Jnl. Geotech. Engrg., ASCE, 113(8), 861-878.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. (1985), "Influence of SPT procedures in soil liquefaction resistance evaluation," Jnl. Geotech. Engrg., ASCE, 111(12), 1425-1445.

State of California

Business, Transportation and Housing Agency

Memorandum

To : MR. RAY ZELINSKI
Office of Structure Design

Attention: Saad El-Azazy

Date : February 24, 1999

File : 01-HUM-255-0.2/1.2
01-296701
Humboldt Bay Bridges

Br. Nos. : 04-0228, 04-0229, 04-0230

From : **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials and Foundations
Structure Foundations Branch

Subject : Acceleration Response Spectra for Humboldt Bay Bridges

This memo presents the results of our site response analysis for the Humboldt Bay bridges corresponding to different Peak Bedrock Acceleration (PBA).

Site specific design curves for the mean plus one standard deviation were provided in our memo dated December 20, 1994 (copy attached) which included a description of the geology and seismicity of the site. Site response analyses for different PBA were conducted by scaling the rock motion provided by Geomatrix in the report entitled "Seismic ground motion study for Humboldt Bay Bridges on Route 255." The results are shown in Figure 1. The recommended design Acceleration Response Spectrum (ARS) corresponding to the mean (50^{th} percentile) rock motion (PBA=0.7g) is shown in Figure 2.

If you have any questions, please call Angel Perez-Cobo at 227-7167 or Abbas Abghari at 227-7172.

Angel Perez

ANGEL PEREZ-COBO
Associate Materials & Research Engineer
Earthquake Engineering Section

A. Abghari

ABBAS ABGHARI, Chief
Earthquake Engineering Section

Attachments

cc: ELeivas - SFB

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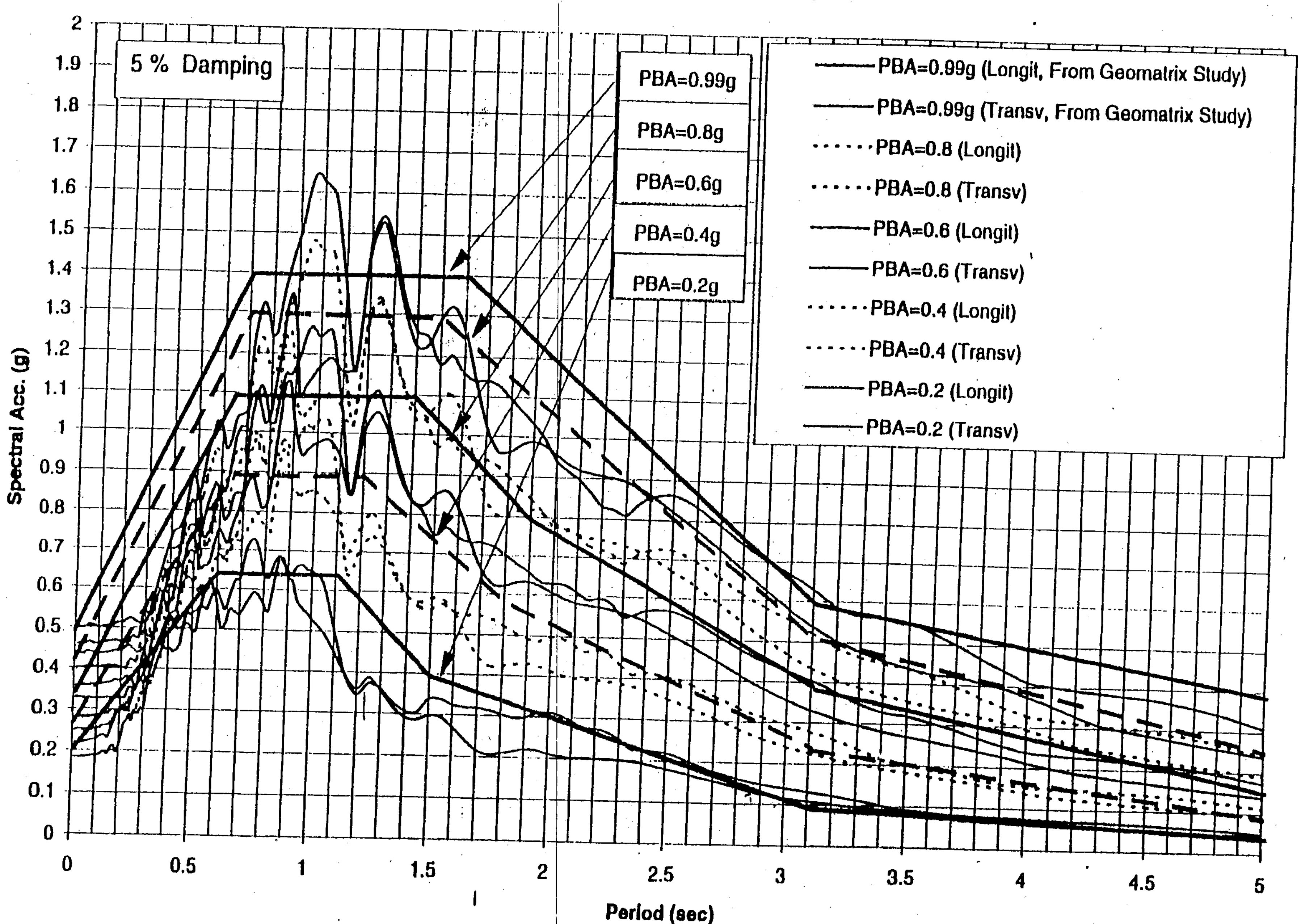

Figure 1. Comparison of Acceleration Response Spectra for Different Peak Bedrock Accelerations at Humboldt Bay Bridge

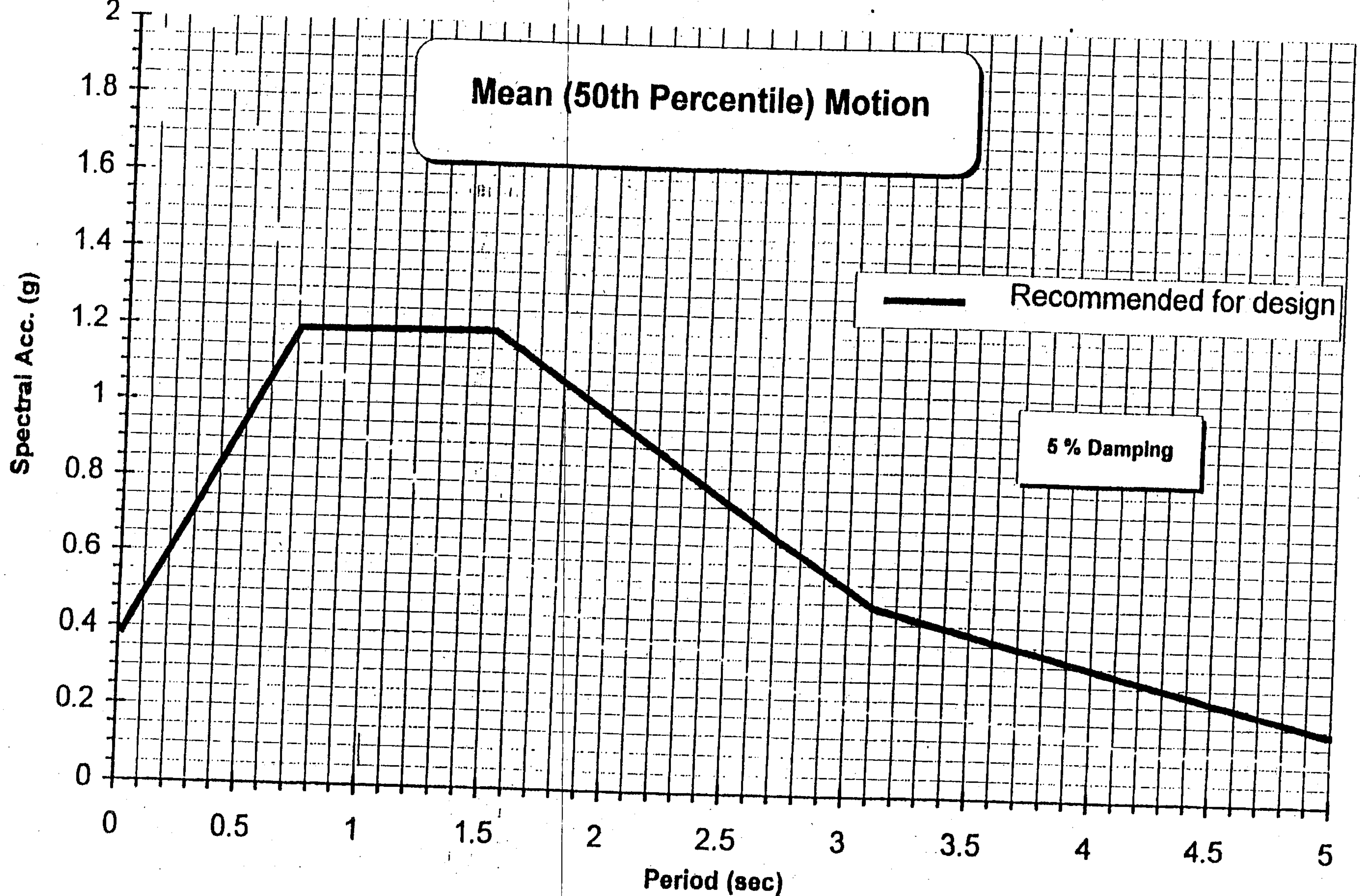

Figure 2. Recommended Design ARS for the Humboldt Bay Bridges.