CHAPTER 7 RELIABILITY

Introduction

This chapter is based on some of the work reported in detail in a technical report for the Navy Facilities Engineering Service Center by Ferritto and Putcha (1995b). This chapter presents an approach for evaluating the seismic reliability of a typical element of waterfront construction, wharves, but it is applicable to all types of construction. The reliability evaluation of a structure for various limit states, especially when these limit states are non-linear, is a complex problem by itself. This becomes even more involved when the structure is subjected to seismic excitation. A good amount of work has been done in the general area of seismic reliability analysis and the reader may refer, among others, to work done by Hwang et al.(1987), Hwang and Jaw(1990), Ang (1990), Tung and Kermidjian (1991), Moller and Rubinstein (1992), Hwang and Hsu (1993), and Wen et al. (1994) . These studies dealt with structures such as buildings, water tanks and nuclear power plants. Some of these studies in the literature , for example, the study by O'Connor and Ellingwood (1987) also dealt with reliability of non-linear structures under seismic loading. In the work by O'Connor and Ellingwood (1987) reference was also made to an earlier equivalent static analysis used for the reliability analysis of structures subjected to seismic forces by Ellingwood et al. (1980). The corresponding safety indices, β, were given along with the probability of failure, P_f, for each of the limit state. An important point to be noted is that the number of specific studies on the seismic reliability analysis of waterfront construction involving soil-structure interaction problems like wharves reported in the literature is limited.

Reliability Analysis - General Methodology

The reliability analysis methodology that is being proposed is general in nature for all structures subjected to seismic forces even though it is discussed with reference to wharves. The probability of failure of a wharf can be evaluated for each of the applicable limit states such as strain or ductility limit exceedance and yielding of piles, excess lateral displacement, etc. Then the bounds on the probability of failure of the wharf can be established, if need be, using the methods proposed by Ang and Tang (1984). The limit state function of a wharf is given by

$$g = R - L \qquad (7\text{-}1)$$

where,

R = component of resistance capacity
L = component applied load

The above limit state uses the basic premise that the probability of failure is defined as:

$$P_f = P(g < 0) \qquad (7\text{-}2)$$

$$P_f = P(R < L) \quad (7\text{-}3)$$

Once P_f is calculated then the reliability can be evaluated from the following equation as:

$$\text{Reliability} = 1 - P_f \quad (7\text{-}4)$$

It has been common practice presently to express reliability in terms of a reliability index β, which is expressed as,

$$\beta = \Phi^{-1}(1 - P_f) \quad (7\text{-}5)$$

Where Φ^{-1} is the inverse of a standard normal cumulative distribution function. To be specific, β is the First Order Second Moment Reliability index, defined as the minimum distance from the origin of the standard, independent normal variable space to the failure surface as discussed in detail by Hasofer and Lind (1974), Ellingwood et al. (1980), and Ang and Tang (1984). The above relation is exact if the limit state function is linear and all probability distributions are jointly normal or lognormal.

There have been several applications of the First Order Second Moment (FOSM) and also the Advanced First Order Second Moment (AFOSM) methods in the literature, Ayyuba et al. (1984), Ellingwood et al. (1980), Galambos et al. (1978a), Galambos (1978b), and Hoeg et al. (1974), to name a few.

The safety index β is also expressed as,

$$\beta = \frac{\bar{g}}{\sigma_g} \quad (7\text{-}6)$$

where,

$$\bar{g} = g(\bar{X}_1, \bar{X}_2, - - - - - \bar{X}n) \quad (7\text{-}7)$$

$$\sigma_g^2 = \Sigma \, (\partial g/\partial X_i)^2 \, \sigma^2_X \quad (7\text{-}8)$$

where the bar over the variable indicates the mean value. The partial derivatives are evaluated at the corresponding mean value of the variable. If g is defined by equation 7-1 then, for R and L being normal variables, β can be expressed as,

$$\beta = \frac{\overline{R} - \overline{L}}{\sqrt{\sigma_R^2 + \sigma_L^2}} \tag{7-9}$$

If R and L are assumed to have lognormal distribution then β can be expressed as,

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{L}}\right)}{\sqrt{V_R^2 + V_L^2}} \tag{7-10}$$

where, V_R and V_L represent the coefficient of variation of R and L respectively.

Knowing β the probability of failure P_f can be obtained from the following equation for each limit state:

$$P_f = \Phi(-\beta) \tag{7-11}$$

The above equation is exact if the limit state function is linear and all probability distributions are jointly normal or lognormal(Ang et al. (1984), Ellingwood et al. (1980), Warner et al. (1968).

The general approach can be applied to a specific case study by evaluating the probability of site acceleration based on procedures developed for performing site seismicity studies Ferritto (1993). The site ground motion should be based on historical and geologic data for the region and reflect local site soil conditions. The ultimate capacity of the structure must be determined. Measures of uncertainty need to be established for both the load and the capacity.

Reliability Methodology For Seismic Loads

Wen et al.(1994) suggest the calculation of probability of failure based on the following equation. This is similar to the equation developed by Ang and Tang (1984)

$$P_f = \Phi\left(-\frac{\ln(SA_c / SA_r)}{\left(\beta_c^2 + \beta_r^2\right)^{1/2}}\right) \tag{7-12}$$

where, $\Phi\ (\ )$ is the standard cumulative normal distribution function. SA_c and SA_r are the median values of spectral acceleration of structural capacity and load of a lognormal distribution respectively. β_c and β_r are logarithmic standard deviation for structural capacity and load corresponding to a lognormal distribution.

In this case the median value of spectral acceleration is determined by means of the capacity spectrum method Freeman (1978), Wen et al. (1994). The median value of the spectral acceleration of load SA_r given by Wen et al. (1994),

$$SA_r = (SA_n) * (A_p) \qquad (7\text{-}13)$$

A_p is the value of peak ground acceleration (PGA) and SA_n is the median normalized spectral acceleration determined from response spectra in the Tri-services guidelines (5). The calibration of seismic structural design parameters as related to reliability based design is discussed in detail elsewhere, Han et al. (1994), Wen (1994).

Detailed Development Of Methodology For Seismic Loads

The following outlines steps suggested for use as a general procedure for the seismic reliability analysis of waterfront construction and used in the case study of a wharf reported in following sections.

1. The uncertainty in structural loading is obtained by first identifying the level of the earthquake. There are two levels of earthquake that are used for waterfront structures. One is the Level 1 event often termed OLE (Operating level earthquake) and the other is the level 2 event often termed CLE (Contingency level earthquake) . The first one has a probability of exceedance of 0.5 in 50 years and the second one has a probability of exceedance of 0.1 in 50 years. Using this information on acceleration, obtain the corresponding mean value of acceleration and the 95% confidence limits from cumulative acceleration plots. This uses the general procedures for computing site seismicity and seismic hazard analysis, Ferritto (1993), Sykora (1989).

2. From the mean value and 95% confidence limits of acceleration calculate the corresponding standard deviation of acceleration for a normal distribution. This will be $\overline{L}$ and σ_L .

3. Identify the limit state of the structure which controls capacity.

4. Identify the random parameters in the structure capacity. The uncertainty in structural capacity is obtained by first identifying all the random properties to be included. The geometric properties may be treated as deterministic variables, as was done in the case study in the following section. The material properties are treated as random variables. In the case study the two random variables are-- M_y (yield moment of each pile), subgrade soil stiffness(K).

5. For a set of random parameters of the variables, use an automated analysis program to compute response and collapse load. This will give a random value of the collapse load.

6. Repeat the process illustrated in step 5 for 1000 samples of random values which in turn will give 1000 random collapse loads. Monte Carlo simulation, Ang et al. (1984), Warnet et al. (1968) is used for this purpose.

7. For each random collapse load calculated in steps 5-6 calculate the corresponding random value of capacity acceleration.

8. Calculate the mean and standard deviation of all the random values of accelerations. This gives $\overline{R}$ and σ_R.

9. From the results of steps 2 and 8 calculate the safety index β , for the collapse limit state considered, from Equation 7-9 for normal distribution.

10. The probability of failure is also obtained from Equation 7-12 or Equation 7-10 if the distributions of accelerations for capacity and loading are assumed as lognormal.

Wharf Reliability Demonstration

The Navy has recently completed design for dredging and construction of a carrier wharf at the Naval Air Station, North Island, California. This project is typical of wharf design and was used in a simple form as a demonstration study to illustrate the procedures discussed above. Since the wharf model incorporated a number of simplifications and assumptions where actual soil data was not available, it should not be looked upon as a performance evaluation of the actual construction project.

Regional Seismicity Required

A reliability analysis requires quantification of the seismic load environment and its associated uncertainty. To that end a seismicity study must be performed. The results of such a study are discussed in this section.

The seismicity and regional geologic structure of the San Diego area can be interpreted in light of current plate tectonic theory. California lies on the junction of two relatively rigid plates of the earth's crust that respond to movement of subcrustal material. The main evidence of this juncture is the San Andreas fault. These same forces that tend to move the portion of California on the westerly side of the San Andreas fault northward have resulted in the formation of other faults, such as the San Jacinto, Whittier-Elsinore and Newport-Inglewood faults. Distant faults that must be considered significant to the site region include the Elsinore and San Jacinto fault zones to the northeast and the San Clemente fault zone to the west. Local faults include the Rose Canyon and La Nacion. The San Andreas fault zone is not considered very significant because of its great distance from the study area. This section is based on a detailed study performed by Ferritto (1994).

The San Diego Bay contains Cretaceous, tertiary, and quaternary strata, which is generally flat but locally folded and cut by normal and right lateral faults. This area is called the Rose Canyon zone Lee et al. (1988). A bottom survey of the bay revealed numerous faults which were difficult to correlate. The quaternary deformations observed along the Rose Canyon fault zone attest to the tectonic importance of the zone. Although no major earthquakes have occurred near San Diego recently, several earthquakes of about magnitude 3.5 have been recorded during the past 41 years. Eleven took place near the Rose Canyon fault. The magnitude 3.5 earthquake is associated with a fault rupture length of 1 km. The geologic structure of this area shows evidence of previous movement. Surface traces of more than 24 km in length and vertical separation of hundreds of feet are visible. Table 1 shows the key faults and the maximum credible earthquake.

Probability Analysis

The bounds of the study area are 115.0 to 119.0 W longitude, 34.0 to 32.0 N latitude. The coordinates of the site are 117.18N, 32.705N. A set of historical data was prepared for the site containing over 6,000 events with magnitudes of 3 or greater. Figure 7-1 shows the region of interest with the epicenters plotted. Figure 7-2 shows a similar plot with only the faults shown. Figure 7-3 shows the total probability of not exceeding the acceleration for a 50-year exposure.

The best estimate of site seismic exposure from all sources is as follows:

1000 year	0.60 g
500 year	0.42 g
250 year	0.28 g
100 year	0.18 g

For the purpose of engineering analysis the causative events are as follows:

The 1000 year earthquake is a magnitude 6.5 event at 1 to 3 miles from the site.

The 500 year earthquake is a magnitude 5.5 event at 1 to 3 miles from the site or a magnitude 6. to 7 event at about 10 to 20 miles from the site.

The 250 year event is a magnitude 5 event at about 2 miles from the site.

The seismicity at the site is totally dominated by the Rose Canyon fault. Generally the causative events associated with ground motion return times specified are caused by magnitude 5 to 5.5 earthquakes close to the site. These events would not have durations as long as those associated with magnitude 6 to 7 events. As noted there is the possibility of magnitude 6 to 7 events 10 or more miles from the site which would produce longer duration shaking. To support this study a

Table 7-1
Fault Systems of Interest

Fault	Maximum Credible Magnitude
Coyote Creek	7.0
Elsinore	7.5
Imperial	7.0
La Nacion	6.8
Malibu	7.5
Newport-Inglewood	7.0
Palos Verdes	7.0
Pinto Mountain	7.5
Raymond Hills	7.5
Rose Canyon	7.1
San Clemente	7.7
San Gabriel	7.7
San Jacinto	7.5
Santa Susana	6.5
Sierra Madre	6.5
South San Andreas	7.5
Superstition Mountain	7.0

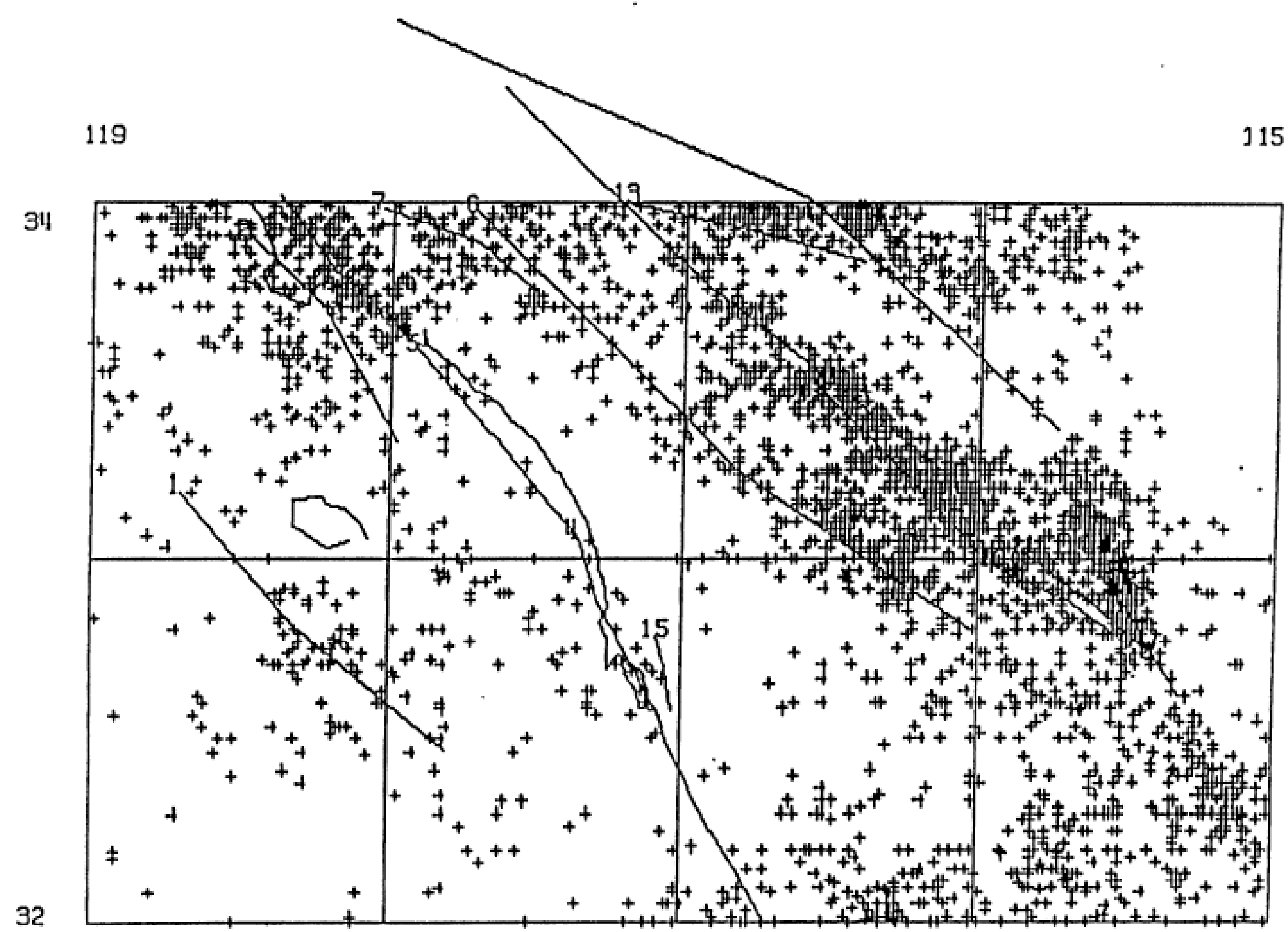

Figure 7- 1. Region surrounding site showing faults and historical epicenters.

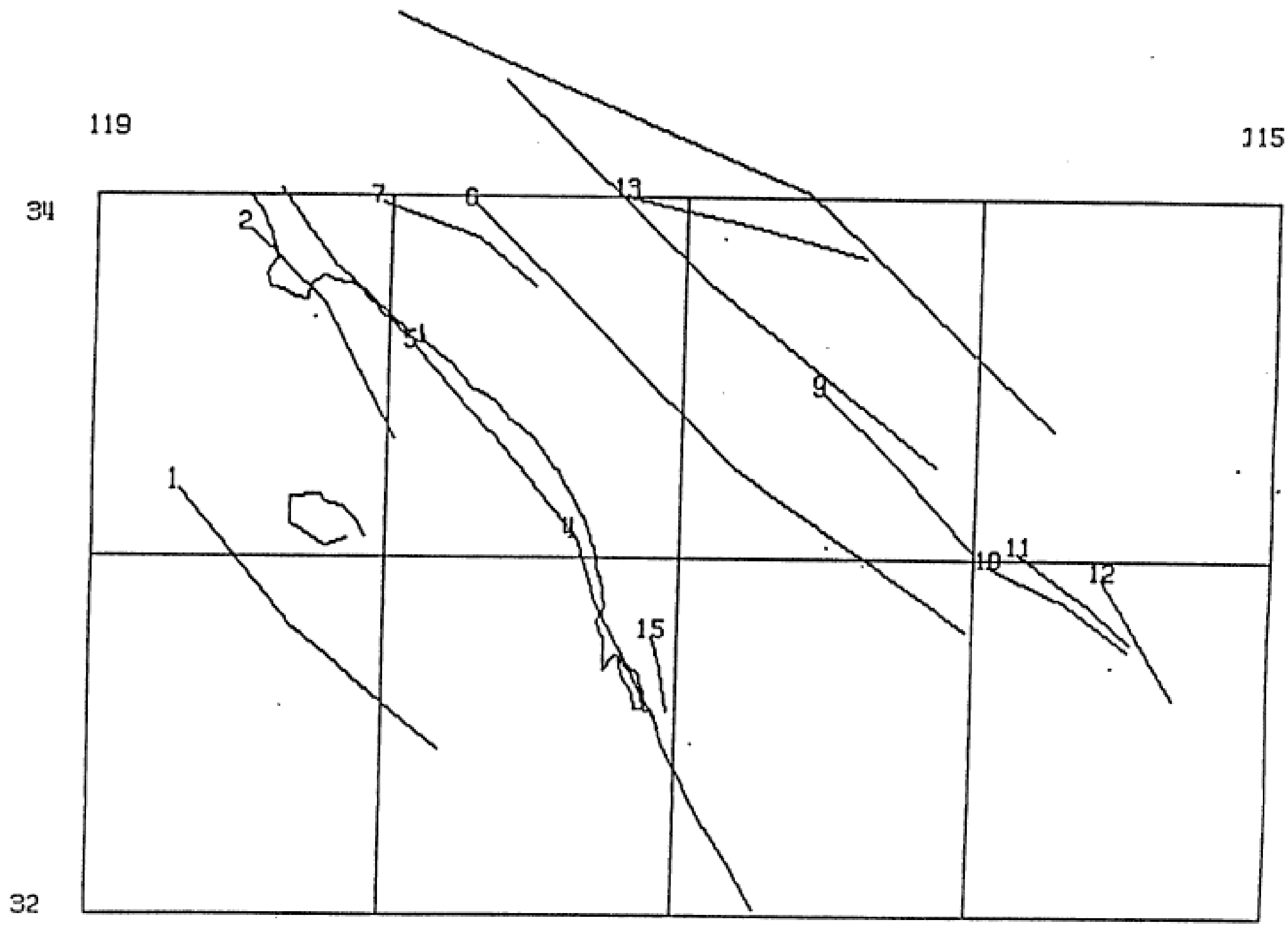

Figure 7-2. Region surrounding site showing faults as defined in Table 1.

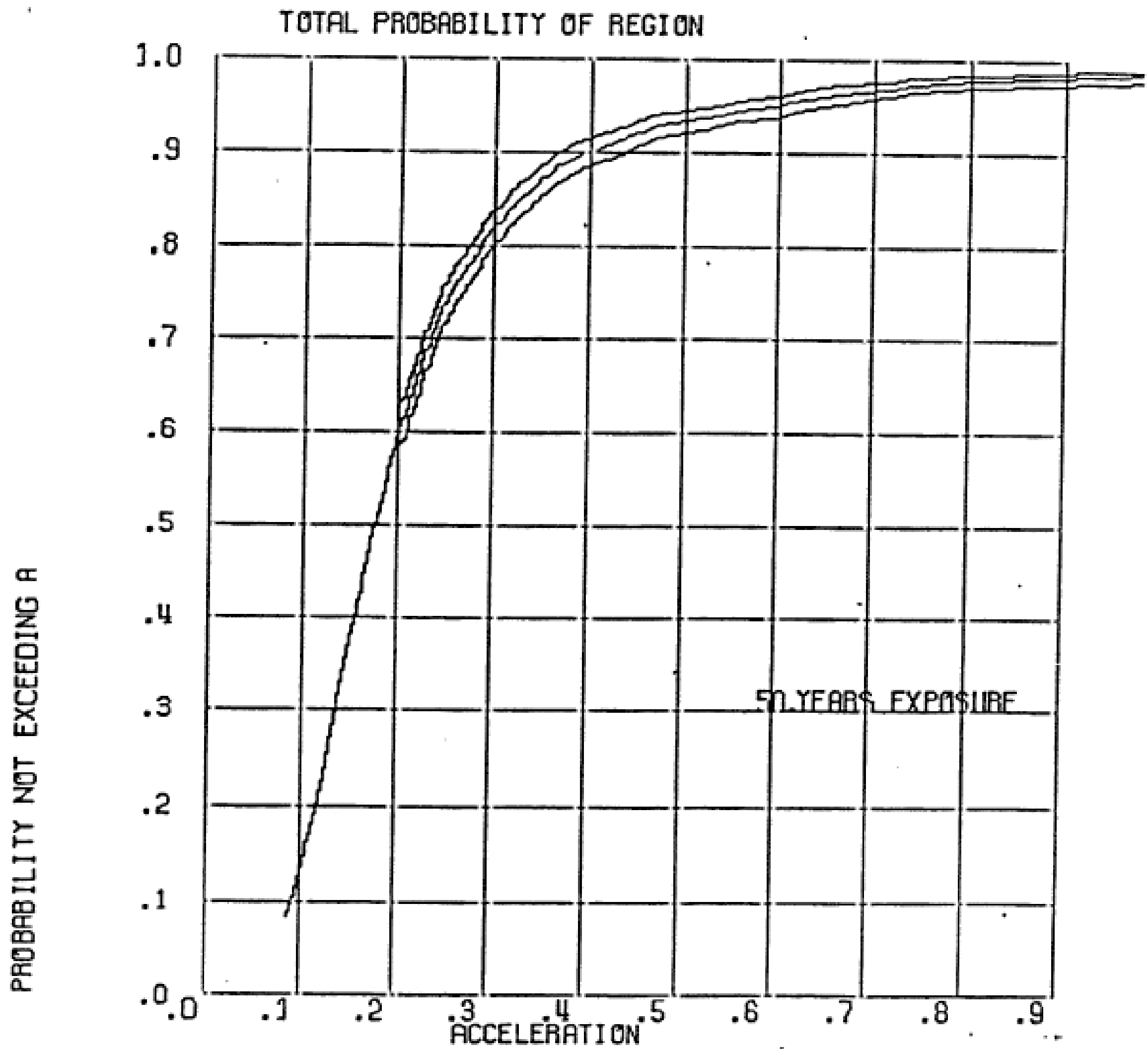

Figure 7-3. Probability distribution of site peak acceleration.

number of earthquake time history records were selected using procedures documented in Ferritto (1992).

Site Soil Model For Ground Motion Amplification/Attenuation

The following site soil foundation model was developed for this study to model a section of the dike below the wharf and soil layers below.

Profile	Thickness ft	Density Lb/cu ft	Blow Count	Shear Modulus fps
Rock Dike	50	140		1500
Bay Point Formation Layer 1	10	120	40	1040
Bay Point Formation Layer 2	10	120	60	1200
Bay Point Formation Layer 3	600	120	80	1400-3000

The blow count data was used to establish the shear velocity and shear modulus using data from Sykora (1989). The shear modulus was allowed to increase with depth to bedrock. It was also decided to use mean values for the shear modulus and damping relationships as a function of strain rather than lower bound values. A one dimensional wave propagation analysis was performed to estimate the acceleration time history in the rock dike using the established level of seismicity as a bedrock acceleration. A series of records were used to represent possible ground motion variation and the variance determined. **This example is based on an existing project which used the 1,000 year event as the design earthquake rather than the 500 year event suggested for use in the criteria section**. It is recommended that the events shown in the criteria be used and the data used herein is intended only to demonstrate the methodology. The 1,000 year peak acceleration earthquake level motion using the 1-dimensional wave propagation analysis was computed for each of the records; the average acceleration is 0.5g with a standard deviation of 0.14 g. The motion is seen to be transmitted to the surface with some attenuation from the rock motion of 0.6g. The uncertainty of the level of this motion was computed. This uncertainty was combined with the uncertainty of value from the seismicity study The values to be used for the reliability analysis are:

1,000 Year Peak Acceleration Mean Value 0.5g

1 σ Standard Deviation 0.147g

Example Wharf and Lateral Resistance Structural Model

The example wharf is shown in Figure 7-4. It was decided to model the structure in two dimensions using the typical cross-section shown in Figure 7-4. The structure is composed of a reinforced concrete deck supported on pile caps. The first pile on the land side is a 28-inch diameter steel pipe pile filled with concrete. The next four piles are 24-inch octagonal prestressed concrete piles and the outboard fender pile is a square 24-inch prestressed concrete pile. A

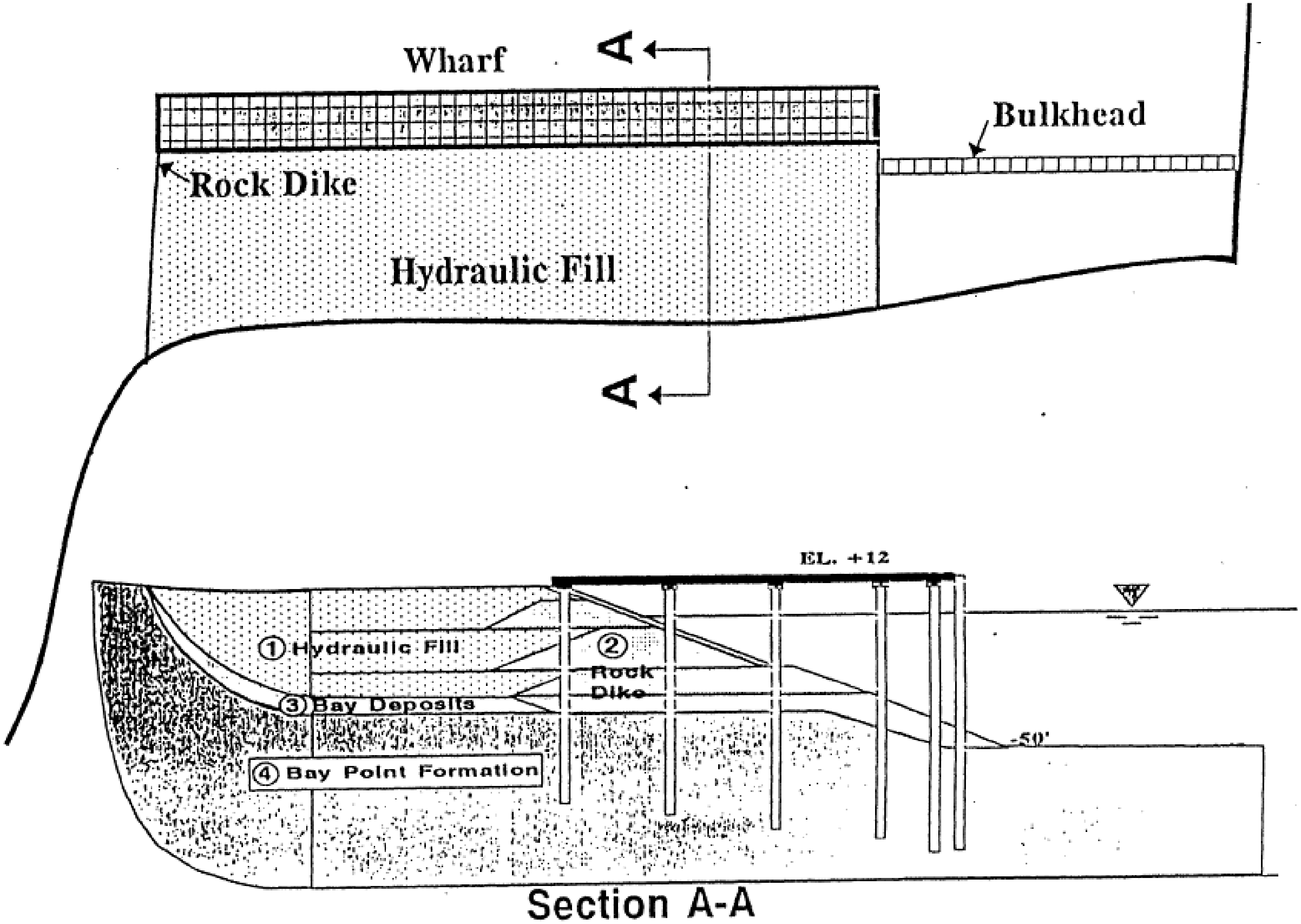

Figure 7-4. Plan view of wharf and dike used in study.

preliminary analysis showed that 90 percent of the lateral resistance was provided by the 28-inch diameter steel pipe pile filled with concrete. The lateral force model of the wharf could thus be simplified to model the pile as a series of beams, constrained by the deck and supported by lateral springs representing the dike. The structural model and procedure used for this study, utilized the ultimate moment - thrust capacity of the pipe pile to define the lateral force capacity. The mass of the structure including deck, piles and restraining soil around the piles was computed. A reliability analysis requires computation of the mean value of capacity and an estimate of its uncertainty. The variance of pile capacity was estimated to be 0.15 and the variance in soil stiffness subgrade modulus for lateral pile restraint was estimated to be 0.10, Arbabi et al. (1991), Ellingwood et al. (1980). The weights of the wharf itself could be estimated with a high degree of reliability. An amount of soil representing the lateral spring stiffness of the dike was included; this could only be determined approximately. The uncertainty of this soil mass was set by giving it a variance of 0.5.

Results Of Reliability Analysis And Discussion

For the analysis conducted the following was found:

OLE Operating Level Event
100 year return time ground motion

Loads, 0.18g σ = 0.07g
Capacity 1.248g σ = 0.382g

β = 2.72 P_f = 0.003
For normal distribution

β = 4.09 P_f = 0.00002
For lognormal distribution

CLE Contingency Level Event
1000 year return time ground motion

Loads, 0.5g = 0.147g
Capacity 1.248g = 0.382g

= 1.827 P_f = 0.034
For normal distribution

= 2.20 P_f = 0.014
For lognormal distribution

The reason for choosing the normal distribution for either structural capacity or loads is mainly based on the principle of maximum entropy , Harr (1987). Based on this principle the normal distribution is to be assumed if the expected value and standard deviation of a distribution are the known parameters. Further it has been stated by Ang and Tang (1984) that the normal or lognormal distribution is frequently used to model non-deterministic problems even when there is no clear basis for such a model. Since almost all of the random data for structural capacity and load are positive it was decided to use a lognormal distribution in addition to the normal distribution. The decision to use a lognormal distribution in addition to a normal distribution is also based on the recent research by Wen et al. (1994) wherein they advocate use of the lognormal distribution for structural capacity and loads in connection with seismic studies. The lognormal distribution computes lower probabilities of failure and is thought to be a more accurate estimate of the results for the seismic study which is consistent with current practice Turkstra et al. (1978), Wen et al. (1994).

The results indicate that the probability of failure under the operating load to be about 0.003 or 0.3 percent for the normal distribution and 0.00002 or 0.002 percent for the lognormal distribution. The probability of failure under the collapse level of loading is about 0.034 or 3.4 percent for normal distribution and 0.014 or 1.4 percent for lognormal distribution. The uncertainty in both loading and capacity was found to be significant as can be seen by the high coefficient of variation values. The uncertainty levels for structural capacity and loading computed in this report are in the same range as in recent report by Wen et al. (1994). A major element in the uncertainty is the manner of wharf -dike coupling. The procedure developed requires the computation of the mean collapse level capacity and its uncertainty. This can not be computed directly in a closed form manner and use of a finite element program is required. This project is of limited scope, intended to demonstrate the feasibility of a general procedure; the analysis options were constrained by available project duration and funding. Determination of the collapse load can be performed by an equivalent static lateral load model as was done here or a more elaborate dynamic soil structure interaction model. The Monte Carlo procedure requires repetition of the analysis varying the strength parameters to evaluate the mean and variance of the capacity. Typically repetitions on the order of 1,000 are used. This poses a problem for implementation of a dynamic finite element approach. The equivalent static approach is thought most appropriate. A major factor in the analysis as shown by the sensitivity of results to the variance in capacity is the estimate of the mass of the system to be used to compute the equivalent capacity acceleration. The effective mass of the soil coupled to the pile was estimated to be a region associated with the pile about 1.5 pile diameters wide by about 3 pile diameters long for the length of the pile. A 50 percent uncertainty was assigned to this soil weight to account for this uncertainty. It should be noted that the more soil mass that is included the lower is the equivalent lateral force capacity. This aspect should be given additional study using a dynamic soil-structure model of the problem to verify the mass effect.

The results show that for the two load conditions specified the probability of collapse is between 0.00002 and 0.003 under the operating level and between 0.014 and 0.034 percent under the contingency level. The wharf design allowed possible major repairable damage under the CLE. This case study is meant only as an illustrative example and is loosely based on the design of the wharf at Naval Air Station, North Island. Thus direct conclusions about the actual wharf should not be made. From this simplified model, it would appear that the wharf in this case study would be expected to perform well under both the OLE and the CLE event.

The safety index β values, for the capacity and load having normal and lognormal distributions, have been calculated using uncertainties in these parameters for a typical wharf structure subjected to seismic loads. Both the OLE (Operating Level Event) and the CLE (Contingency Level Event) are considered in this study. The uncertainties in capacity and load parameters reported in this study are consistent with other work dealing with seismic loads by Wen et al. (1994). A high value of safety index β is found to be for the OLE while a low level of safety index β is found to be for CLE. This is consistent with the fact that for CLE the safety index β should be low as it is a contingency level event.

Limitation in Analysis and Need for Additional Study

Limitations in the scope of this effort necessitated use of a simplified wharf model. A major element of uncertainty is the wharf dike response. It is possible that dike slope deformations can induce additional curvature into the piles. This aspect of soil structure interaction could not be addressed in the model used in this study. It is expected that this would be of concern only for the CLE. As suggested above a more detailed study could better evaluate the effect of dike deformation on wharf capacity.

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