Seismic Risk Mitigation Designs For Two East Caribbean Infrastructure Projects

A. Wightman [1], B.G. Chin [2] and P.W. Henderson [3]
1. Klohn-Crippen Consultants Ltd.
Vancouver, British Columbia
Canada.
2. Klohn-Crippen Consultants Ltd.
Calgary, Alberta
Canada

Abstract

This paper describes the approach to aseismic design for two civil engineering projects in the Eastern Caribbean; the Roseau River dam project in St. Lucia, and the Frigate Bay Sewerage Development in St. Kitts. The Roseau River Dam was rated as a high hazard structure because of the potential for severe damage and loss of life downstream in the event of sudden breaching due to earthquake. The Frigate Bay sewage treatment plant on the other hand is a facility with little or no risk to life associated with sudden operational failure due to ground shaking or ground rupture.

The levels of engineering effort in first defining the seismic hazard and design ground motions and, secondly, in the design of the mitigative measures, were quite different. In both cases, the direct costs of engineering and construction to provide for acceptable post-earthquake performance were modest relative to the total capital costs.

Introduction

The design of civil engineering projects in the Caribbean is influenced by the improved awareness and understanding of the seismic hazards in the region. The assessment of the likely behaviour of important structures and their foundations, both during and after earthquake shaking, should now be an integral part of the design process to ensure that such performance will be within acceptable limits. It is also important that the level of engineering and construction effort expended should be consistent with the potential consequences of failure in order to avoid undue expense to the project.

The Roseau River dam is currently under construction. Primary funding is being provided to the Government of St. Lucia, through the Water and Sewerage Authority, by the Canadian International Development Agency (CIDA) with added funding by the Organization of Petroleum Exporting Countries (OPEC). A key feature of the dam is a spillway over the crest of the embankment near mid-valley. The downstream consequences of failure of the Roseau River dam are sufficiently severe to place the dam in a high hazard category. This requires that it be designed to safely pass the Probable Maximum Flood (PMF) and withstand the Maximum Credible Earthquake (MCE) without uncontrolled release of the reservoir. The dam type (rockfill) is inherently seismically stable, but there were concerns about possible excessive deformations of the parapet wall and spillway crest and chute slab. A dynamic analysis indicated that the amount of deformation under the MCE could be limited to a more acceptable level by incorporating horizontal steel reinforcement embedded into the rockfill.

The Frigate Bay sewerage development includes 4 km of trunk sewers, 14.6 km of collection mains, and a 1.5 million gpd treatment plant that will service a residential and tourist/commercial area near the southern tip of St. Kitts. Design and tender documents for the project have been funded by CIDA. The soft ground conditions and a high water table render the treatment plant site susceptible to ground failure by liquefaction. However, in contrast to the Roseau River dam, the consequences of failure of the plant during an earthquake are not catastrophic or life threatening, so that a building code approach to quantifying the earthquake hazard and design ground motion was considered appropriate. Because of the low hazard rating for the facilities, earthquake resistance was only provided for the control building which might be occupied when an earthquake occurs.

Seismotectonic Setting and Seismic Hazard Assessments

St. Lucia and St. Kitts are part of the Lesser Antilles island arc formed by volcanic activity associated with subduction of Atlantic Oceanic lithosphere beneath the Caribbean plate (Tomblin, 1975). Towards the end of the Miocene era, the part of the arc north of Guadeloupe was uplifted and volcanic activity shifted to the west, resulting in the growth of an inner arc of volcanic islands with the outer arc consisting of limestone capped volcanoes. The majority of the contemporary and historic seismic activity in the region is associated either directly with the subducting plate at depths ranging from near surface, at 100 to 150 km east of the islands to 200 km at a distance of 50 km west of the islands; or with active shallow secondary faulting, or with volcano-seismic swarms in areas of recent volcanic activity (Shepherd, 1989a).

A detailed probabilistic seismic hazard study was commissioned for the Roseau River dam. For the Frigate Bay project seismic design parameters were inferred from the Caribbean Uniform Building Code (CUBiC).

Roseau River Dam

Project Description

The Roseau Basin Water Development Program includes a water storage reservoir, a system of pipelines, a river intake and a water treatment plant to supply the needs of the capital city Castries, and the northwest portion of St. Lucia, to the year 2025. Construction of a dam on the Roseau River, just below the Village of Millet, started in April of 1993.

The dam (Figure 1) is of compacted rockfill, with concrete face on the upstream slope connected to a parapet wall at the dam crest and sealed onto the abutment rock with a reinforced concrete plinth wall. The dam is 43 m high and 250 m along the crest, and has side slopes of 1.3H:1V upstream and 1.5H:1V downstream. A low-level outlet, comprising a 609-mm-diameter steel pipe, passes through the concrete plug in the diversion tunnel and discharges through a hollow-cone valve located at the downstream portal. This provides a facility to drain the reservoir either for scheduled maintenance or in an emergency, and to flush the lower river channel as a means of controlling schistosomiasis. It also provides a way for maintaining minimum flow in the river channel below the dam to meet irrigation and riparian demands.

The spillway, located on the embankment fill near mid-valley, is a 36m-wide free-overflow broad-crested weir discharging down a concrete chute that tapers uniformly to a 21m wide flip bucket. Preliminary designs considered a more conventional layout with a spillway crest and chute bearing on sound rock on the right abutment. The steep abutment slopes would have required costly excavation and slope flattening and there were concerns regarding the risk of heavy rain or earthquake-induced landsliding blocking the spillway channel. The final spillway configuration was selected after consideration of several factors including cost, environmental impact, and safety.

Foundation Conditions

St. Lucia is of volcanic origin, with the oldest exposed rocks on the island being about 7 million years old. The Roseau River dam site is located in the middle Tertiary Central series identified by Newman (1965), consisting of more recent, moderately folded volcanic

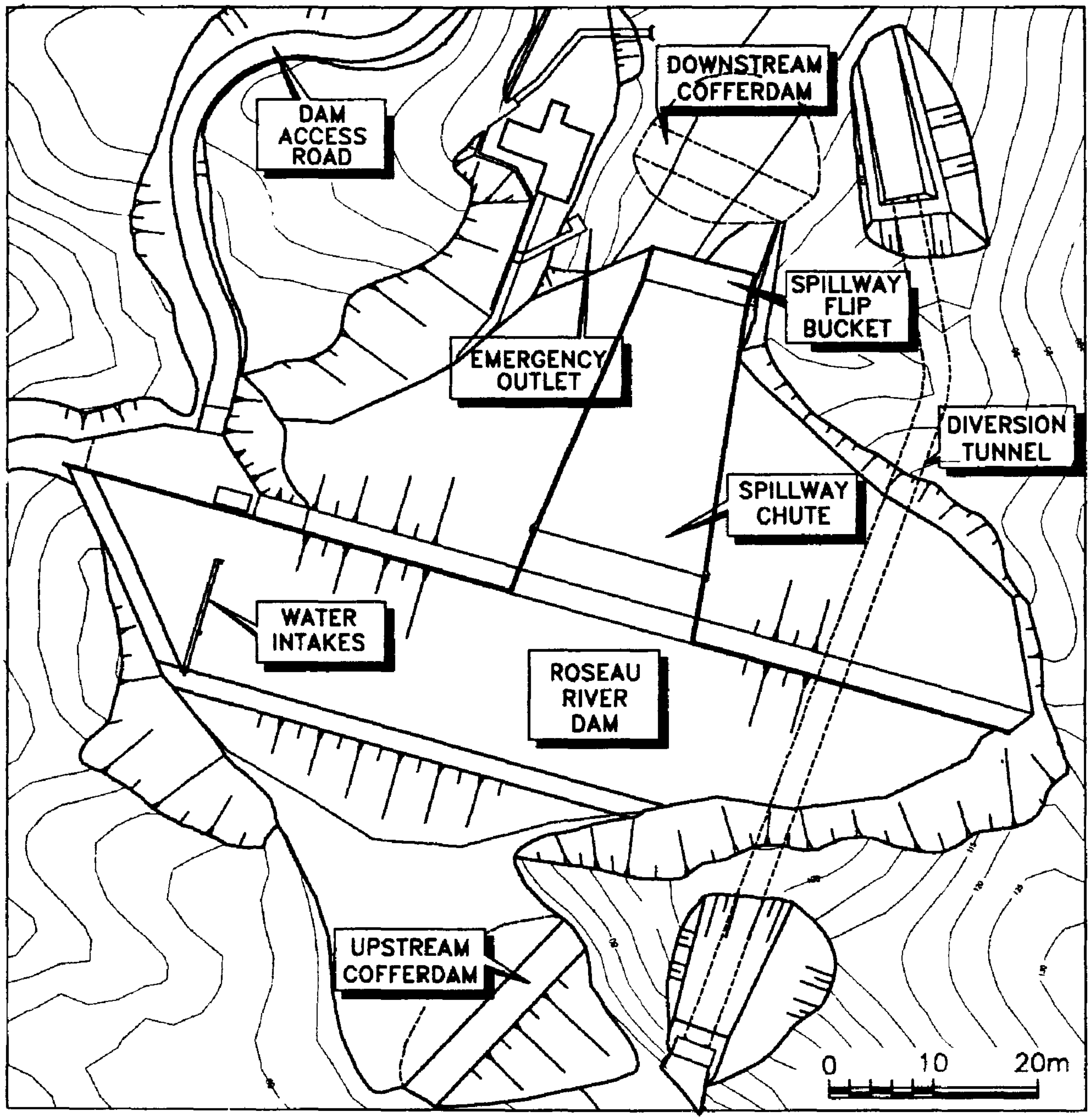


Figure 1: Roseau River Dam Layout

rocks of predominantly andesitic composition. At the dam site, unweathered andesite and andesite porphyry rocks are exposed at river level, with residual soil overlying the unweathered rock higher on the abutments. The weathering profile on the abutments extends to depths of up to 10 m but is mostly in the 4 m to 6 m range at the dam axis.

Seismic Hazard and Design Ground Motions

In 1983, the International Commission on Large Dams (ICOLD) published Bulletin 46 - Seismicity and Dam Design- which recommended a two-tier approach to seismic design of dams, referred to as the Design Basis Earthquake (DBE) and the Maximum Design Earthquake (MDE). This concept can be combined with a hazard rating based on the consequences of failure such as that given by the United States Bureau of Reclamation (USBR).

Given the likelihood of future agricultural and residential development downstream, a "high" hazard rating was adopted for this dam. Consequently, the DBE ground motions should be those with an annual probability of exceedance of 0.0021, while the Maximum Design Earthquake is synonymous with the Maximum Credible Earthquake (MCE), which can be defined either deterministically or, as was the case for the Roseau River dam, probabilistically.

The methods used for estimating the ground motion for design of the dam and spillway are described by Aspinall et al. (1994). The hazard was evaluated by the probabilistic method using the computer program PRISK (Principia Mechanica, 1985). This program allows seismic sources to be defined not only as area zones but also as specific geological structures including arcuate subduction zones having curvature in both the strike and dip directions. The source zones adopted for the evaluation are shown in Figure 2 together with epicentres for earthquakes from 1980 to 1990. The model includes northern and southern subduction zone sources that are curved in cross section with depth ranging from 10 km to 200 km. The distribution of hypocentres in these three dimensional source zones is based on well constrained data recorded between 1980 and 1990 and confirms the conclusions drawn by Wadge and Shepherd (1984) that the subduction zone is segmented between the islands of St. Lucia and Martinique. In addition to the two subduction zone sources, a third source is included to model shallow activity in the overriding plate. An extensive review of the literature was made when selecting suitable attenuation relationships for the model, since no data and no widely accepted relationships exist for the region. Attenuation relationships by Woodward-Clyde Consultants (1982) which fit Japanese and Alaska data were adopted for the base case, best estimate, calculations.

For design of the dam and spillway the expected values at the 10^{-4} per annum exceedance level were adopted as representative of the MCE, i.e., PGA = 0.51 g; PGV = 0.28 m/s (Figure 3). A design spectrum was established after considering these peak ground motion bounds and published standard spectral shape relationships, and best judgement.

Performance of Rockfill Dams in Earthquakes

Concrete face rockfill dams are considered by many to be "inherently" safe against earthquakes because (a) the interior of the dam is dry and there can therefore be no cyclic degradation and loss of strength due to pore pressure build up, and (b) the resultant of the water load on the upstream face acts downwards into the foundation thereby increasing stability, and (c) compacted rockfill founded on sound bedrock is highly interlocked so that shearing failure is virtually impossible. The actual field performance data for rockfill dams subjected to earthquakes is sparse. The compilation of 14 observed rockfill dam responses by Bureau et al. (1985) shows that most field data is limited to base rock motion of 0.3 g or less. The data base includes only 5 concrete face rockfill dams and at least three of these pre-date the advent of modern rockfill compaction methods. These dams experienced crest deformations in the range cf 60 to 380 mm vertical and 40 to 50 mm horizontal. The highest recorded base acceleration of 0.41 g, for Leroy Anderson dam,

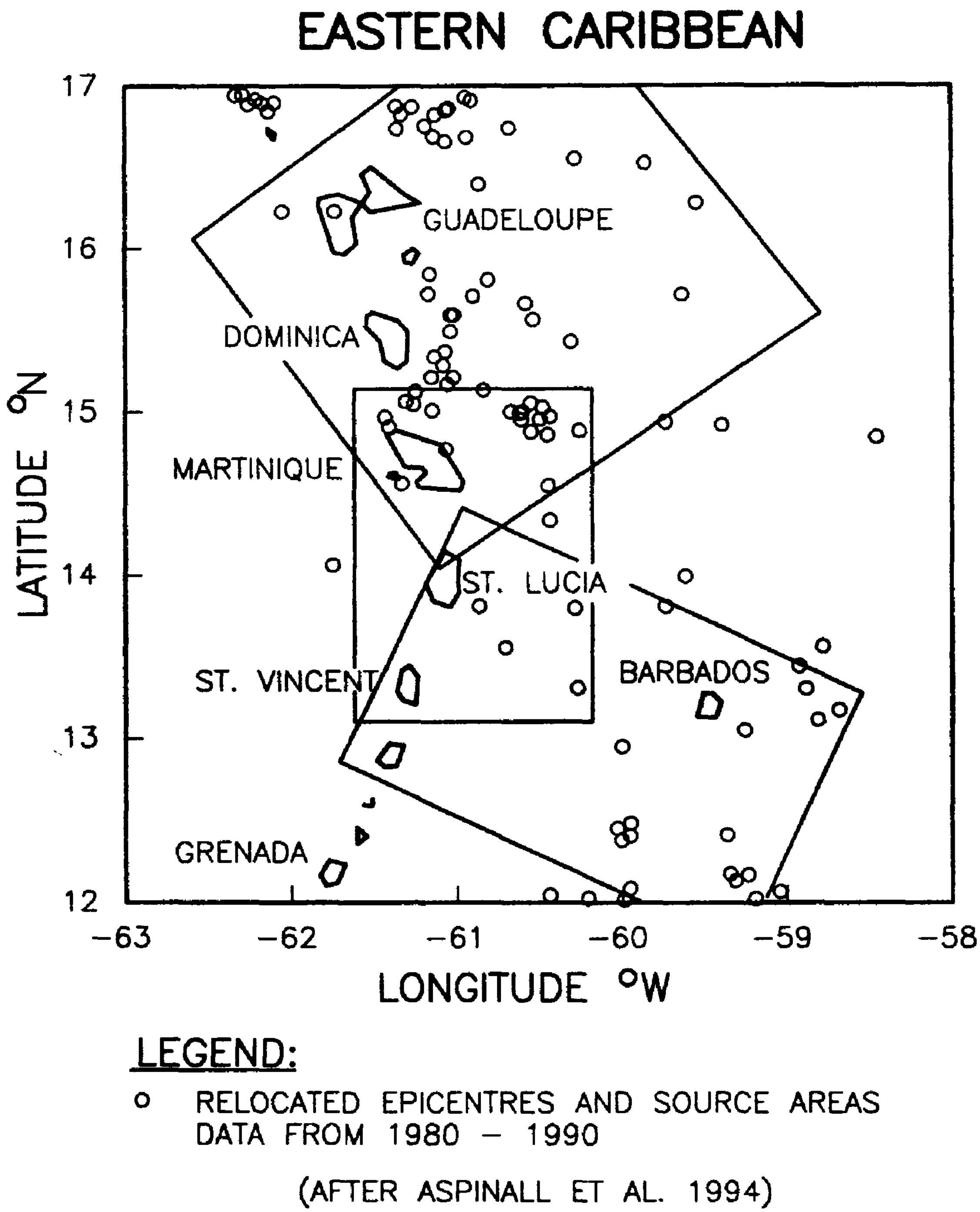


Figure 2: Seismogenic Source Zones

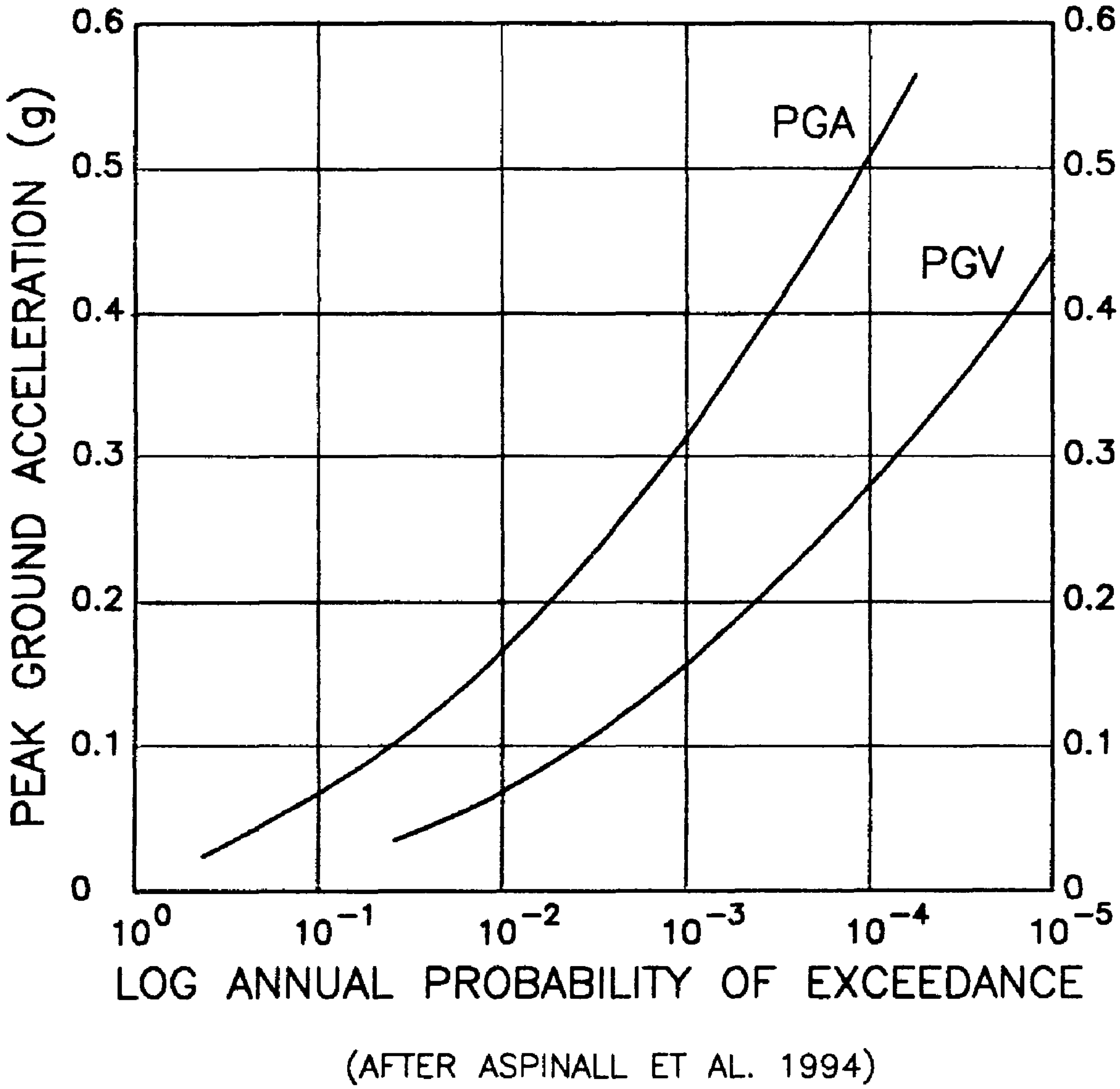


Figure 3: Peak Ground Acceleration and Velocity Annual Probability of Exceedance

produced 15 mm and 9 mm vertical and horizontal displacement respectively. These are very modest displacements and would not be any concern for a modern rockfill dam.

A plot of the crest/base amplification factors for the data presented by Bureau et al. (op. cit.) supplemented by recent data from the Loma Prieta earthquake (M_s 7.1) of 1989, the Edgecombe earthquake (M_L 6.3) of 1987, and the M_s 8.1 earthquake in Mexico in 1985, as reported by Maley (1989), Gillon (1988), and Elgamal et al. (1990), is shown in Figure 4. Unfortunately, none of these are concrete face rockfill dams. Nevertheless, this data indicates a strong trend for reduced amplification with increasing base excitation. This is thought to be due to non-linear behaviour of the rockfill at higher strain levels leading to more internal damping and lengthening of predominant period that might tend to shift the response away from the significant period of the base rock motion. At low base accelerations, non-linear effects would be minimal, resulting in the higher amplification ratios. Figure 4 also shows the upper bound of observed behaviour for several California earthfill dams as reported by Harder (1991), which agrees very well with the rockfill data.

A recent state-of-the-art presentation by Gazetas and Dakoulas (1991) presents a more cautious view of the dynamic response. Based on numerical analysis they argue that because of the very stiff characteristics of modern compacted rockfill dams, combined with 3-dimensional canyon effects, non-linearity at high accelerations may not be as dominant as suggested by the limited data on Figure 4, and that higher amplification factors can be experienced. They indicate that base to crest amplification of 5 times is possible. Unfortunately, to date, there appears to be no field data or large scale centrifuge test data to confirm these analytical results.

Dynamic Response and Deformation Analysis

Estimating the potential deformations of the spillway slab and parapet wall started with determining the response of the rockfill dam to the bedrock earthquake motion. State-of-practice two-dimensional analysis was used in which the dam is considered to respond synchronously in the upstream-downstream plane. Three-dimensional effects were approximated based on published two- and three-dimensional response characteristics of rockfill dams in narrow canyons (Mejia and Seed, 1983). One-dimensional and two-dimensional analyses were performed. For brevity, only the two-dimensional analyses using QUAD4 (Idriss et al., 1973) are reported.

Figures 5 and 6 show a dam-section through the spillway and the finite element mesh used for analysis. Properties of the rockfill for the response analyses were obtained from field tests on compacted gravel material of the Jordanelle Dam (Ake, 1990). These properties correlated well with those obtained by back analysis of the response of the Oroville Dam to a weak seismic shaking (Mejia et al., 1991). Figure 6 shows the peak acceleration at selected nodal points for the MCE with a peak acceleration of 0.51 g at bedrock level. Similar analysis for the DBE, with a peak base input acceleration of 0.27 g, resulted in a crest acceleration of 0.56 g. Both analyses indicate amplification by a factor of about 2.

Based on the published analytical results of Mejia and Seed (1983), the response characteristics of the Roseau River dam and the response spectra of the bedrock motions, three-dimensional effects are expected to increase the computed two-dimensional accelerations by 46% and 57% for the MCE and DBE, respectively. This implies amplification factors of 2.8 for the MCE and 3.2 for the DBE.

Under earthquake shaking, a distributed slumping or plastic deformation rather than failure along a distinct surface is considered to occur, with concentration of movement in the upper third of the dam (Bureau et al., 1985). Finite element programs capable of computing the plastic deformations in the dam due to earthquake shaking are not readily available. Therefore, it is customary to estimate deformations along "failure wedges" using a simple stick-slip sliding block model. The procedure is to carry out static stability analyses for critical "failure" wedges, wherein the horizontal acceleration is varied until the instantaneous sliding safety factor is unity. This acceleration is termed the yield acceleration. For each failure wedge, a spatially averaged acceleration time history is

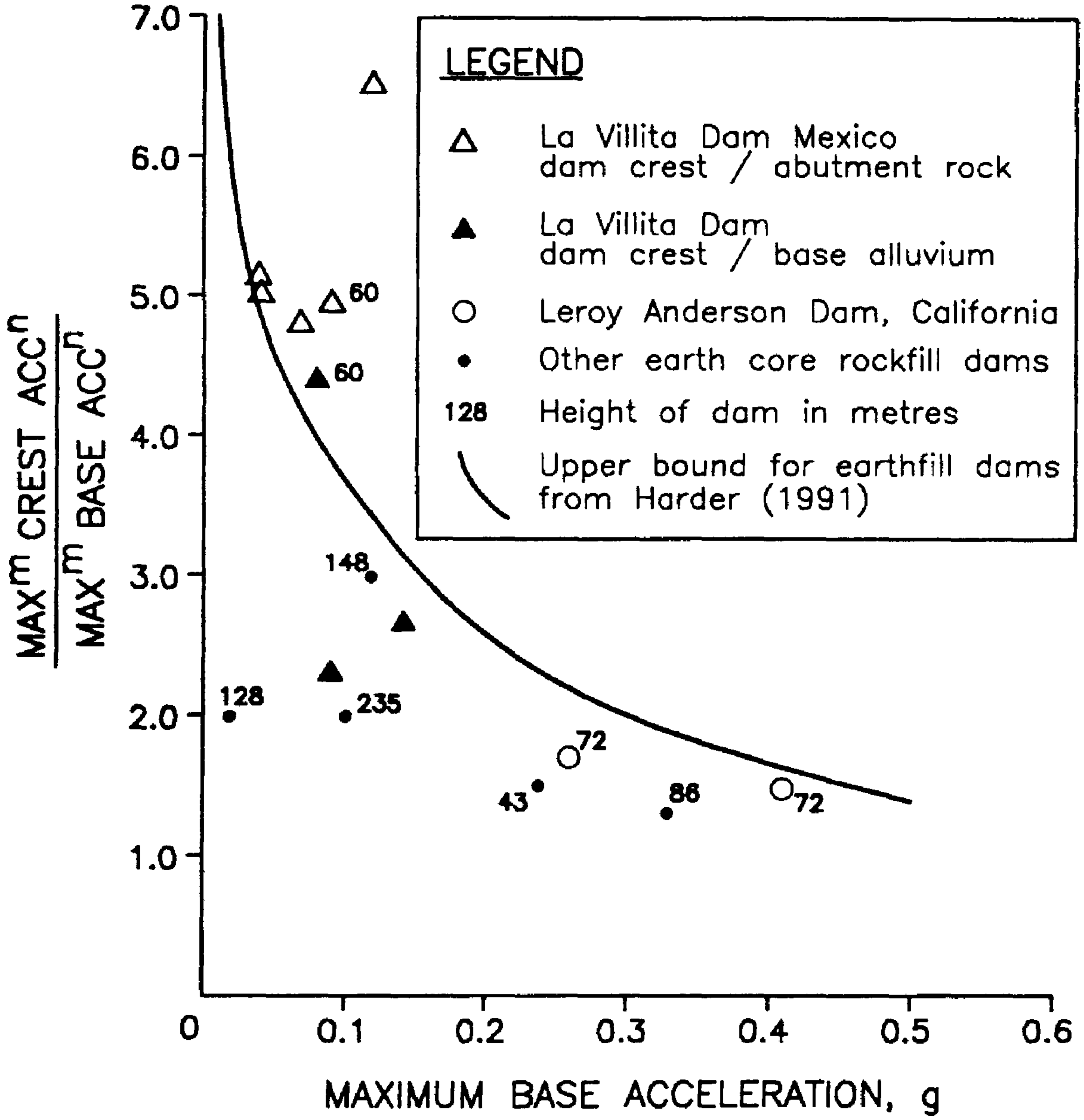


Figure 4: Measured Amplification for Earth and Rockfill Dams

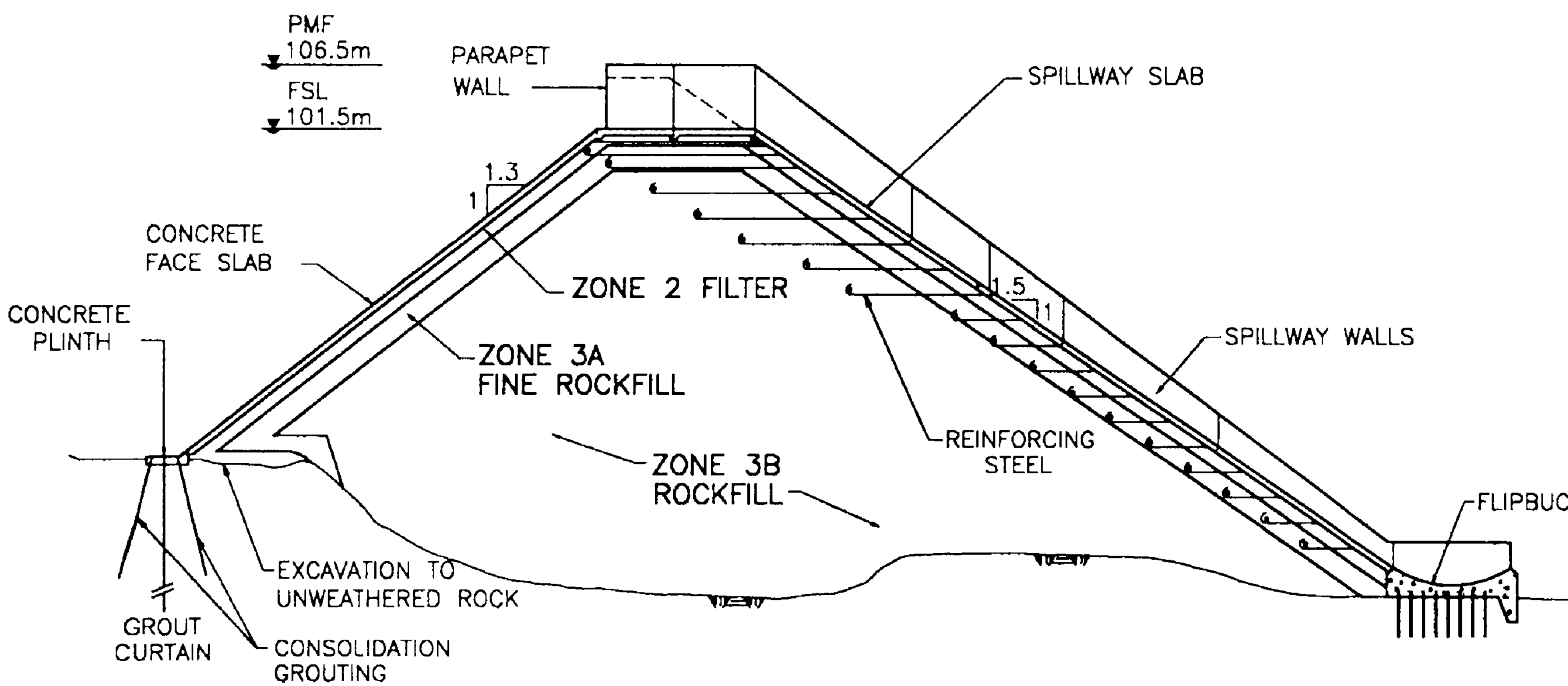


Figure 5: Roseau River Dam Spillway Section

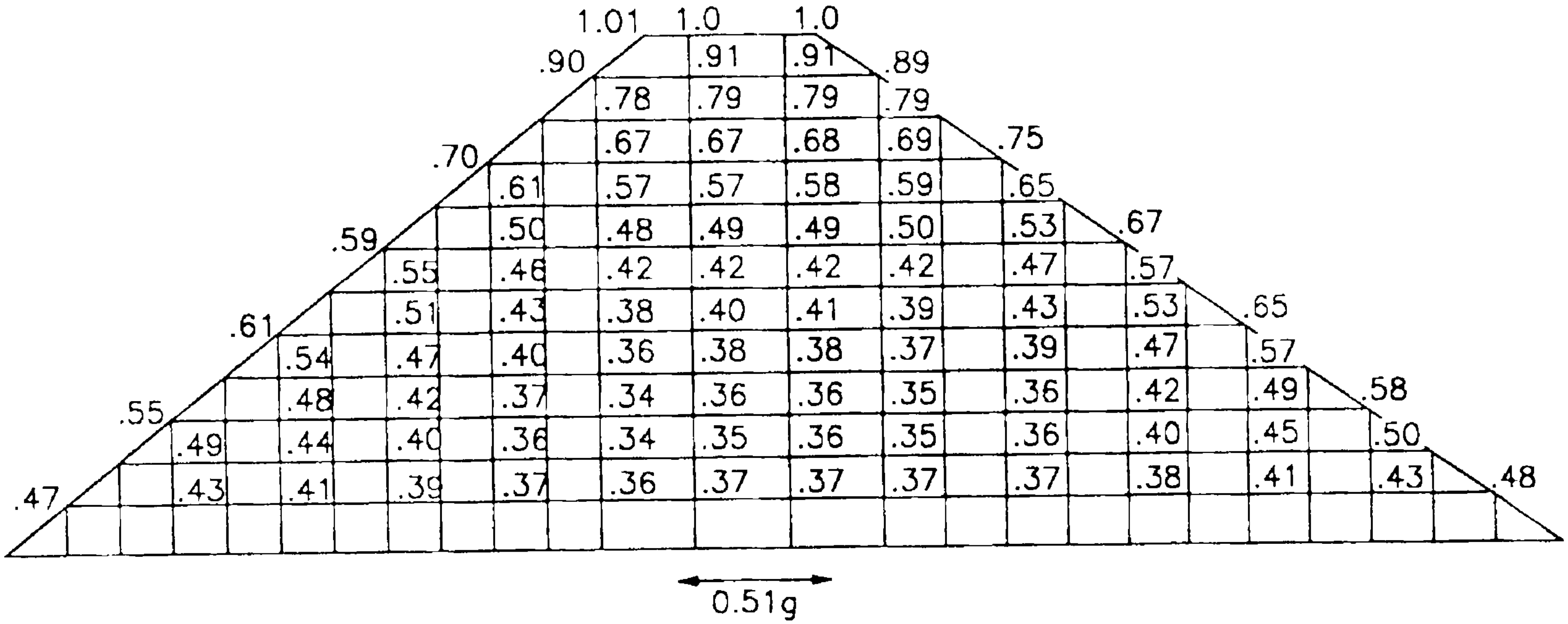


Figure 6: Finite Element Mesh and the Peak Accelerations

computed from the response analysis and appropriately scaled to account for three-dimensional effects. The deformation of each failure wedge is obtained by doubly integrating the portion of the spatially averaged acceleration time history that exceeds the yield acceleration (Newmark, 1965). Figure 7 shows the trial failure wedges selected.

The maximum estimated deformations ranged from 40 mm to 160 mm and are associated with Surface 1 (See Figure 7). The results also indicate that deformations are largest at the top of the section and decrease as the wedge extends to lower elevations, consistent with the observations of Bureau et al. (1985). Similarly, estimated deformations for the parapet wall are approximately 50 mm for the DBE and 200 mm for the MCE.

Mitigative Designs

The estimated deformations are well within acceptable limits for a conventional well built concrete face rockfill dam, and would normally not require any mitigative measures. However, there was concern that the spillway chute slab and joints would be vulnerable to this amount of movement. Added protection was sought against the risk of rupturing of the water stops and joints, and dislocation of spillway slabs and walls leading to erosion and possible breaching of the dam. Consequently, the rockfill beneath the spillway will be reinforced as shown on Figure 5, and connected to the spillway slab. The design of the rockfill reinforcement was based on pseudo-static, force equilibrium methods of analysis, to calculate the total anchor load for a minimum factor of safety of 1.1 within the critical sliding wedge. The lengths and layout of the reinforcement were established by assuming an equivalent friction angle of 45° between the rockfill and the steel reinforcement. The effective embedment lengths were taken as that segment of the reinforcement installed beyond (behind) the location of the critical sliding wedge.

The deformation of the reinforced section was estimated by analyzing the three downstream "sliding wedges" shown on Figure 8. These surfaces are parallel to the inboard end of the reinforcement and are selected on the assumption that deformations on shallower surfaces will be restricted by the reinforcement. The estimated deformations are reduced to 10 mm to 70 mm for the MCE.

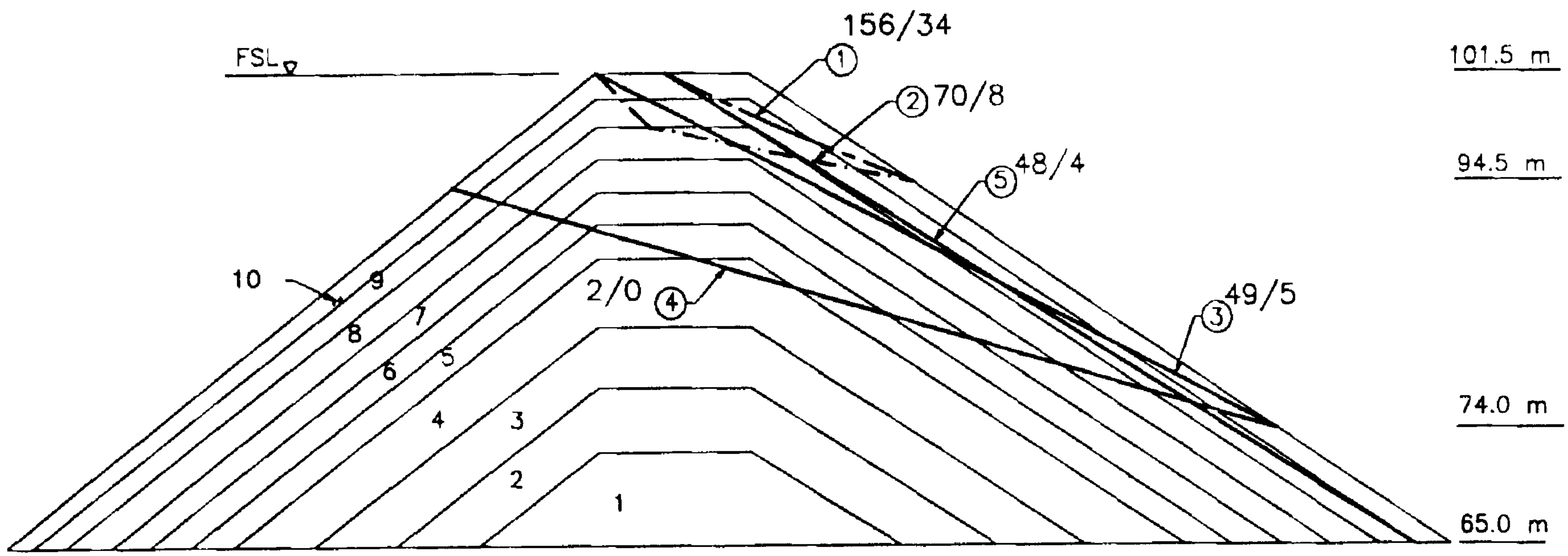
For the parapet wall, the analysis showed that three layers of reinforcement located below the wall foundation would reduce the deformations to a negligible amount for the DBE, and about 24 mm to 121 mm, depending on reservoir level for the MCE.

The dam configuration actually adopted for construction has lower crest fill level behind the parapet wall. This should further improve the seismic performance of the dam crest, and resulted in some minor changes to reinforcement layout from that shown here.

Frigate Bay Sewage Collection and Treatment

Project Description

The Frigate Bay Development Corporation is a St. Kitts government agency that is developing a residential and commercial/hotel complex surrounding an 18 hole golf course on a 100 acre coastal lowland area near the southern tip of St. Kitts. The area is partly developed with the golf course, Jack Tar Village hotel, several smaller beach motels and some private residences. Before the area can be developed to its full potential a modern sewage collection, treatment, and disposal system is needed to replace the existing system of septic tanks and Tile fields. The Canadian Government, through the Canadian International Development Agency (CIDA), recently funded a project to develop final design and tender documents for the preferred sewage option. The treatment plant will remove solids and floatables and provide for a minimum of 10 days retention in two 1000 m^3 forced air aeration lagoons. Following dosing with chlorine, the treated liquid effluent will be discharged to a holding pond in the golf course. This pond acts as a small reservoir for pumped irrigation of the golf course through an extensive system of buried sprinkler pipes.



LEGEND

1 ROCKFILL STRENGTH ZONES

2 SLIDING WEDGES ANALYZED

49/5 3-D / 2-D DEFORMATION, - mm.

Figure 7: Sliding Wedges Analyzed – Unreinforced Section

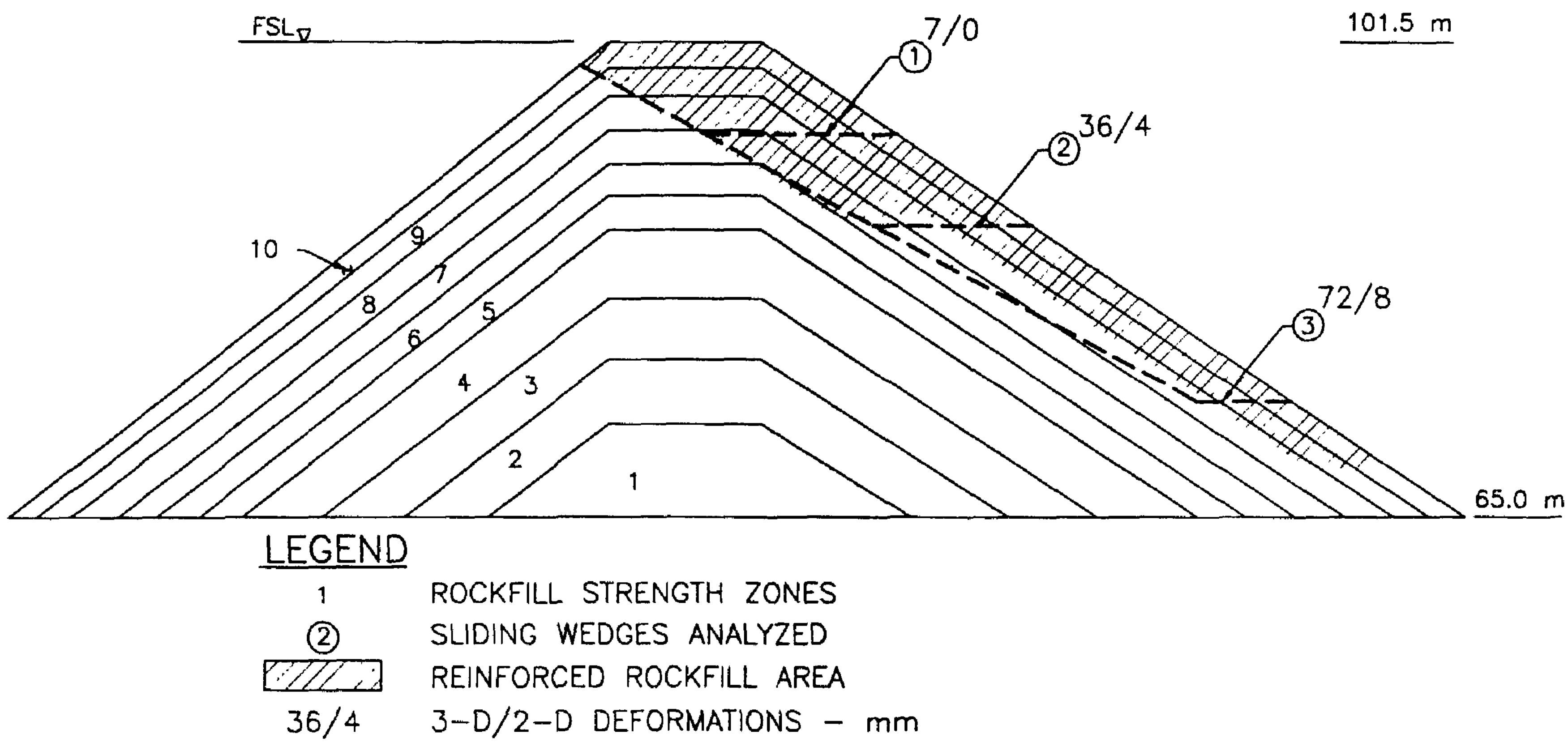


Figure 8: Sliding Wedges Analyzed – Reinforced Section

Foundation Conditions

The Frigate Bay development occupies a coastal strip of saline marsh that forms an isthmus connecting the older volcanic rocks of Timothy Hill, with the Conarce Hills. The coastal area is protected by a low perimeter dune but still floods during extreme rainfall or hurricane events. Much of the original brackish mangrove swamp behind the dunes was filled with dredged sand to create the golf course and hotel/casino complex. The treatment plant occupies an area 250 x 100 m at the south end of the remaining brackish pond and mangrove swamp known as Muddy Pond. The plant site is underlain by up to 2 m of peat and organic clay resting on a stratum of fine sand. These are geologically very young deposits and much of the sand is composed of a high percentage of pulverized seashells. Despite the high carbonate content there appears to be little cementation and there are layers of significant thickness in several drill holes where the fine sand has a low standard penetration test resistance (SPT), typically in the range of 5 to 13 blows/foot, indicating a loose cohesionless material.

Seismic Hazard and Design Ground Motions

The sewage collection system and treatment plant is a conventional facility posing no special environmental risk or risk to life in the event of sudden operational failure caused by ground shaking or ground rupture. Therefore, in contrast to the Roseau River dam, it was considered appropriate to use the Caribbean Uniform Building Code (CUBiC, 1985), to define the seismic hazard and design ground motions. The seismic provisions of CUBiC are not explicit with respect to the peak ground motion values or return periods associated with the seismic zone coefficients. The geotechnical engineer, therefore, must apply some judgement to arrive at an appropriate value or range or values for peak ground acceleration consistent with the code.

St. Kitts lies in seismic zone 3 of CUBiC. The seismic zones contained in CUBiC are modelled after the 1985 Uniform Building Code's four zones which have associated zonal coefficients of 1, 3/4, 3/8, 3/16, for use in the calculation of seismic base shear. In a later edition of the Uniform Building Code (UBC 1988), the numerical values of the zonal coefficients have been changed to correspond to the effective peak acceleration for each zone. The implied equivalent zone factors are shown in Table 1. Thus, one interpretation is that the effective peak acceleration, implied by Z = 0.75 for zone 3 in the CUBiC Code, is 0.3 g. This is consistent with early seismic hazard studies by Faccioli et al. (1983) and Shepherd (1989b). Shepherd (1987) prescribes a design acceleration of 0.2 to 0.3 g for St. Kitts-Nevis.

Considering these factors, the peak firm ground acceleration for evaluating subsoil resistance to liquefaction to a hazard level comparable with CUBiC, was taken as 0.3 g.

Table 1. Seismic Zone Coefficients

Zone No.	Z Factor UBC 1985 & CUBiC	Z Factor UBC 1988
1	3/16 - 1/4	0.1
2	3/8 - 1/2	0.15 - 0.2
3	3/4	0.3
4	1	0.4

Earthquake Induced Liquefaction

Using the adopted peak ground acceleration of 0.3 g as a surface acceleration, conventional liquefaction triggering analysis based on SPT (Seed, 1984) indicated that the loose foundation sand layers would liquefy in an earthquake of the intensity implied by the CUBiC zoning provisions. Figure 9 illustrates this by comparing the measured SPT blow-counts to the blow counts required to provide a safety factor of 1.3 against liquefaction triggering for a magnitude 7.0 earthquake. It indicates that the loose layers between 3.3 to 8 m depth (11 to 26 feet) are, in fact, vulnerable to liquefaction at ground shaking levels significantly lower than that implied by CUBiC.

However, not all of the treatment plant site is underlain by liquefiable sand. From the site investigations it appears that there is a narrow zone, running parallel to the shore line and roughly north-south through the centre of the plant site, where relatively dense sands overlie what is interpreted to be a coral reef buried at 3 m depth. Deeper alluvium, containing loose liquefiable sands, exists on both sides of this buried reef structure. Possibly the sands overlying the reef were compacted by wave action, while the loose materials on either side were deposited in a quiet water environment.

The consequences of widespread subsoil liquefaction at the site are that; shallow foundations would settle and tilt, the lagoon dykes would spread laterally and possibly result in tearing of the high density polyethylene liner, and any buried sumps, tanks, or empty piping could become buoyant and heave up to the ground surface.

Mitigative Designs

To eliminate the risk of liquefaction ground failure from the treatment plant site, the loose sands could be densified in-situ either by dynamic compaction, by vibro-replacement along columns, or possibly by blasting. Costs for stone column treatment were estimated to be several millions of dollars. To avoid this expense and still meet the life-safety objectives of CUBiC, it was decided to completely change the plant layout so that the control building, which is the only occupied structure, is located on the denser sand deposits underlain by the reef. The remaining facilities, including the primary tanks, aeration lagoons and pump station would remain on potentially liquefiable areas. With this arrangement, the earthquake hazard would present solely an economic risk that the owner has to accept. The risk includes the potential for structural damage to the control building, and ground failure leading to damage and possible loss of use of the treatment tanks and the lined earthfill lagoons.

A provision has been included in the construction contract for further site investigation to better define the extent of the buried reef and select the final position for the control building within pre-set limits. There is also an allowance for driving displacement compaction piles to densify the sand if foundations for part of the control building on the loose sands cannot be avoided.

Cost of Aseismic Design

For the Roseau River dam the costs of engineering studies, designs, and specifications, together with the cost of construction for the rockfill reinforcing steel and additional rockfill handling and placement costs, amounts to about 2% of the estimated capital cost of the dam, spillway, and outlet works.

For the Frigate Bay sewage scheme, early recognition of the earthquake risk prompted relocation of the control room to a potentially more stable part of the site. The only "extra" construction associated with the earthquake risk is the provision of compaction piling if needed and the associated exploratory testing still required. Representative cost figures are not available since the project has not yet gone to tender, but the combined cost of engineering and construction associated with the earthquake issue is expected to be less than 1% of the estimated capital cost of the works.

Conclusions

In this paper, we contrast the seismic design aspects of two different kinds of infrastructure projects: one a high hazard water storage dam and the other a low hazard sewage treatment plant. The different level of effort in establishing the seismic hazard and design ground motions as well as the respective mitigative measures for each project is

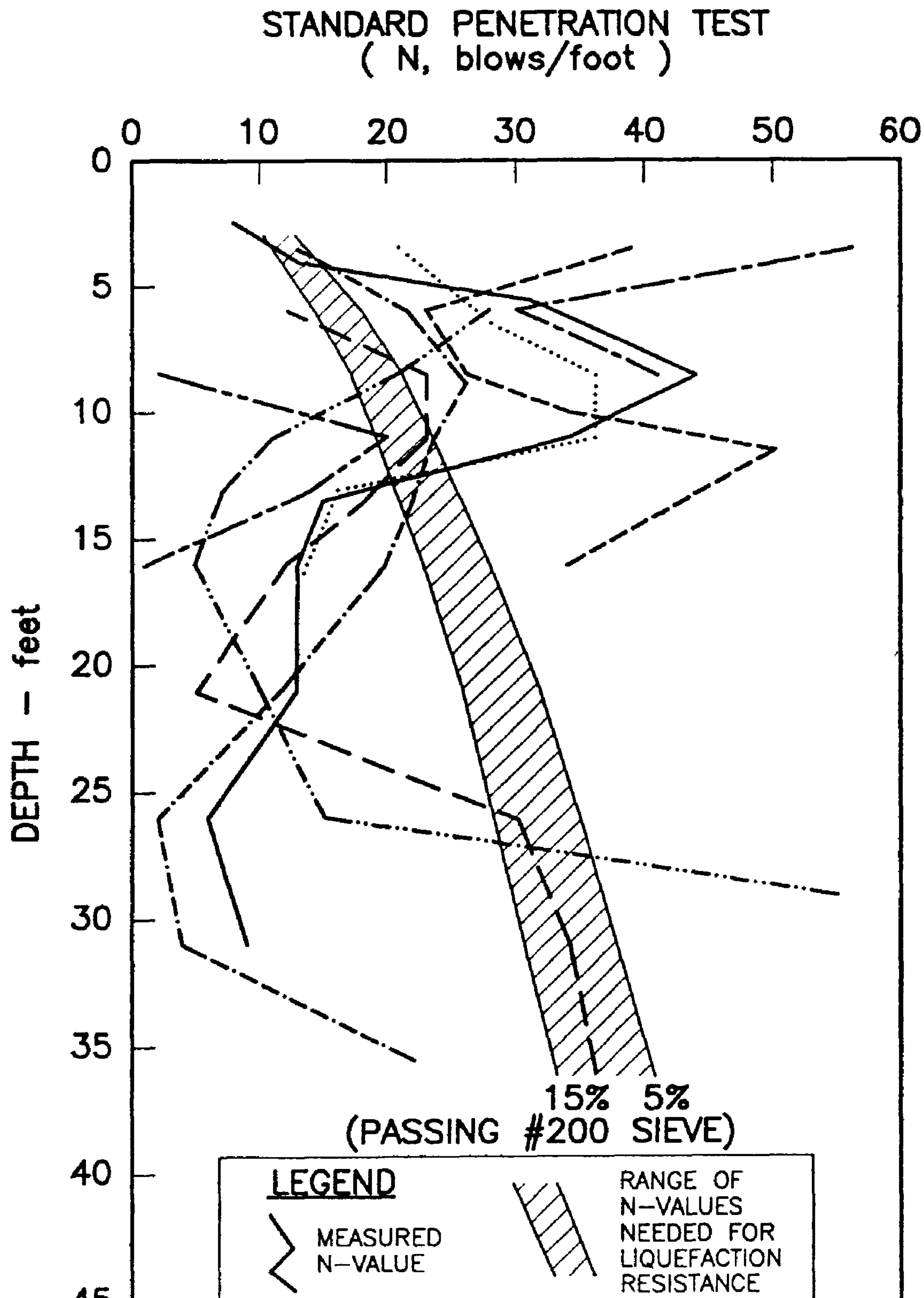


Figure 9: Standard Penetration Test Profiles

discussed. For the dam, the concern was over dynamic displacement of the spillway, and reasonably sophisticated analyses were employed. For the sewage plant, simple empirical charts for liquefaction resistance were employed. It is apparent that the subsoils are liquefiable and that precise definition of the design acceleration is unnecessary. The mitigative designs in both cases are straight forward and the engineering and construction costs are relatively low when considered in relation to the total capital cost.

Acknowledgements

The authors would like to thank CIDA for permission to publish information on these two projects, the Government of St. Lucia for permission to publish information on the Roseau River dam and Aspinall et al. for permission to use the information contained on Figures 2 and 3. The authors would also like to thank Dr. Alex Sy of Klohn-Crippen for his contributions to Figure 4.

References

Ake, J. (1990). Memorandum - U.S Bureau of Reclamation.

Aspinall, W.P., Shepherd, J.B., Woo, G., Wightman, A., Rowley, K.C., Lynch, L.L. and Ambeh, W.B. (1994). Seismic ground motion hazard assessment at a site near a segmented subduction zone: the Roseau Dam, Saint Lucia, West Indies. Earthquake Spectra, 10: 259-292.

Bureau, G., Volpe, R.L., Roth, W.H. and Udaka, T. (1985). Seismic analysis of concrete-face rockfill dams. Concrete-face rockfill dams - design, construction and performance, ASCE, pp. 479-508.

Elgamal, A.M., Scott, R.F., Succarieh, M.F. and Yan, L. (1990). La Villita dam response during five earthquakes including permanent deformation. Journal of Geotechnical Engineering (ASCE), 116: 1443-1462.

Faccioli, E., Taylor, L. and Shepherd, J.B. (1983). Recommendations on the level of lateral forces to be used for earthquake resistant design in the Caribbean region. Eastern Caribbean CCEO Seminar on Earthquake and Wind Engineering, Port of Spain, January.

Gazetas, G. and Dakoulas, P. (1991). Aspects of seismic analysis and design of rockfill dams. Proc. Second Int. Conf. on Recent Advances in Geotechnical Engineering and Soil Dynamics, St. Louis, Missouri, pp. 1851-1888.

Gillon, M.D. (1988). The observed seismic behaviour of the Matahina Dam. Proc. of the Second Int. Conf. on Case Histories in Geotechnical Engineering, Missouri, pp. 841-848.

Harder, L.F. (1991). Performance of earth dams during the Loma Prieta earthquake. Proc. Second Int. Conf. on Recent advances in Geotechnical Engineering and Soil Dynamics, St. Louis, Missouri, pp. 1613-1629.

Idriss, I.M., Lysmer, J., Hwang, R. and Seed, H.B. (1973). QUAD-4, a computer program for evaluating the seismic response of soil structures by variable damping finite element procedures. Report No. EERC73-16. University of California, Berkeley.

International Conference of Building Officials (1988). Uniform Building Code. Whittier, California.

Maley, R. et al. (1989). U.S. Geological Survey strong-motion records from the northern California (Loma Prieta) earthquake of October 17, 1989. U.S. Geological Survey Open-File Report 89-568.

Mejia, L.H. and Seed, H.B. (1983). Comparison of 2-D and 3-D dynamic analyses of earth dams. Journal of Geotechnical Engineering (ASCE), 109: 1383-1398.

Mejia, L.H., Sykora, D.W., Hynes, M.E., Fung, K. and Koester, J.P. (1991). Measured and calculated dynamic response of rockfill dam. Proc. Second Int. Conf. on Recent Advances in Geotechnical Engineering and Soil Dynamics, St. Louis, Missouri, pp. 1063-1070.

Newman, W. R. (1965). A report on general and economic geological studies, St. Lucia, West Indies. Prepared for the Government of St. Lucia.

Newmark, N.M. (1965). Effects of earthquakes on dams and embankments. Geotechnique, 5: 139-160.

Principia Mechanica Ltd. (1985). PRISK Manual.Report prepared for the Central Electricity Generating Board, London.

Robson, G.R. (1964). An Earthquake catalogue for the Eastern Caribbean 1530-1960. Bull. Seism. Soc. Am., 54: 785-832.

Seed, H.B., Seed, R.B., Lai, S.S. and Khamenehpour, B. (1985). Seismic design of concrete-faced rockfill dams. Concrete face rockfill dams - design, construction and performance. ASCE, pp. 459-478.

Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R. (1985). Influence of SPT procedures in soil liquefaction resistance evaluations. Journal of Geotechnical Engineering (ASCE), 111:

Shepherd, J.B. (1989a). Eruptions, eruption precursors and related phenomena in the Lesser Antilles. In: Latter , J.H. (ed.). IAVCEI Proceedings in Volcanology 1 - Volcanic Hazards. Springer -Verlag, Berlin, pp. 292-311.

Shepherd, J.B. (1989b). Earthquake and volcanic hazard assessment and monitoring in the Commonwealth Caribbean - current status and needs for the future. In: Barker, D. (ed.). Proceedings Meeting of Experts on Hazard mapping in the Caribbean 1987, Kingston, Jamaica, pp. 50-60 .

Tomblin, J.F. (1975). The Lesser Antilles and Aves Ridge. In: Naim, A.E.M. and Stehli, F.G. (eds.). The Ocean Basins and Margins, Vol.3. Plenum Pub. Co., New York, pp. 467-500.

Wadge, G. and Shepherd, J.B. (1984). Segmentation of the Lesser Antilles subduction zone. Earth Planet. Sci. Lett., 71: 297-304.