ANNACIS ISLAND CABLE-STAYED BRIDGE - DESIGN FOR EARTHQUAKE

by

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ABSTRACT

The 465 m span Annacis Island cable-stayed bridge, currently under competitive design in both steel and concrete, will probably be the longest cable-stayed span in the world when it is completed in 1986. Being founded partially on soft delta deposits and located within 80 km of a deep seismic source zone, design of the bridge has required careful consideration of earthquake effects. This paper reports the seismic analytical and design approaches adopted for the foundations and superstructure of the steel alternative. Details covered include dynamic modelling of the subsoil layers, correlation of measured surface accelerations with those predicted by computer modelling from nearby rock accelerations, earthquake risk analysis, assessment of the soil liquefaction potential, compliance of deep piles and pier translation during ground shaking, dynamic modelling of the superstructure, ductility and displacement demands in the superstructure and details for construction.

INTRODUCTION

Annacis Island Bridge will cross the South Arm of the Fraser River approximately 19 kilometres upstream from its mouth. It is a high level four lane bridge with capacity for the future addition of two highway or rapid transit lanes. The superstructure is supported by two planes of closely spaced cables radiating from concrete portal towers, as shown in Figure 1. The suspended structure comprises a pair of steel plate girders approximately 2 m deep and 28 m apart, which support floorbeams at 4.5 m on centre carrying a 215 mm composite precast concrete deck. The concrete deck is continuous over the whole length of the bridge and carries most of the superstructure lateral and longitudinal loads. Lateral loads are resisted at both the main towers and the side towers, while longitudinal loads are resisted at the main towers only.

In recent geological time the site has experienced flooding by the sea, three glacial cycles, and considerable erosion and deposition. The site geology is complex (see Figure 1), and there is no similarity between the deposits on the two sides of the present river channel. On the South bank competent granular deposits, which have been glacially densified, are present at the surface and provide bearing for a shallow foundation. The North bank stratigraphy has about 55 m of recent soils overlying the more competent pre-glacial deposits. The recent deposits comprise 25 m of Fraser River sand overlying about 30 m of lightly over-consolidated clayey silt. The upper sand is somewhat loose, and will be densified prior to construction. The clayey silt may be subject to long term consolidation. The choice of foundation designs for the North pier therefore lies between deep piles to the pre-glacial deposits at depth and a shallow foundation resting on compacted sands over the clayey silt stratum which would have been subjected to forced consolidation.

It is clear from the stratigraphy that the seismic response in the North and South bank strata to rock accelerations at depth may be very different. The seismicity of the site was examined by Hofmann (1), who identified major source zones West of the site and in the Puget Sound region to the South. The sources are related to underthrusting of the oceanic crust, and the site is to the East of the Pacific plate and above the eastern extent of the underthrusting Juan de Fuca plate. The Puget Sound zone of deep earthquakes have reached magnitude $M = 7.1$ in historic time, and estimates of maximum expected magnitude of 7.3 have been made. Other earthquakes up to magnitude 7.3 have occurred within a 200 km radius of Vancouver during the last 40 years.

The seismic design philosophy for this structure and foundation works was based on the assumption that earthquake accelerations would not govern strength design aspects of a structure having basic vibration periods in the range 2 to 6 seconds. Therefore, structural design proceeded in advance of earthquake considerations, while recognizing the structural form, mechanisms and details necessary to ensure good earthquake performance. Then the "strength designed" structure and foundations were subjected to rigourous earthquake analysis where strength, displacement, ductility and total stability demands were carefully assessed.

SEISMIC DESIGN CRITERIA

The behaviour of a structure during a seismic event should result in consequences commensurate with the likelihood of the occurrence. Thus the moderate earthquakes that may occur with high probability in the life of the structure should cause no structural damage. Some damage, but no life loss, can be tolerated in the rare strong motion earthquake and it is desirable to have further ductility and displacement reserves still available for the catastrophic earthquake.

Conversion of the above vague statements into specific design criteria is a rather arbitrary process because of the difficulty of predicting probabilities of various levels of earthquake motion and the resulting structural response. Recent North American practice is reflected in the risk level chosen by the Applied Technology Council, where peak acceleration and velocity corresponding to a 475 year return period, or 0.1 probability of exceedence in 50 years was chosen to correspond to behaviour in which repairable damage was admissible. This risk level has also been adopted for the Canadian National Building Code, and for the Applied Technology Council model bridge code ATC-6.

For Annacis Bridge it was decided that a similar risk level be adopted. Thus for a strong motion earthquake, (475 year return period), the bridge, as a lifeline structure, should remain usable for emergency passage, although it may suffer repairable damage. However, the foundations, which would not be easily repaired, should remain essentially undamaged. For moderate earthquakes (return period 100 years) the structure should perform elastically without damage. Values of peak acceleration and velocity at the site for the 475 year return period have been based on E.M.R. Canada site specific data and a study of source zones and attenuation factors carried out by Hofmann (1). Values of acceleration of about 0.2 g and velocity of about 20 cm/sec. are reasonable.

INPUT MOTIONS

A number of earthquake records (Lake Hughes #4 and #9, 1971; Chile, 1965; Imperial Valley, 1979; Olympia, 1949) were selected for analysis of the bridge response. These records are all either rock records or surface records from strata similar to Annacis Island and are of the appropriate magnitude and epicentral distance to avoid introducing significant scaling problems. A broad period range of spectrum energy was covered by these records. The maximum surface acceleration recorded at Olympia (1949) was 0.28 g at a site 220 km South of Annacis Island. Although this acceleration is somewhat higher than values derived from statistical analysis of nearer earthquakes, it was decided to use this record as an upper bound design strong motion surface record. The other records, of slightly lower peak acceleration, were used as rock input records. Response spectra for typical rock input and North and South pier surface accelerations are shown in Figure 2.

The long period components of surface waves have the potential for influencing the structure, especially its displacements. Hofmann (1) estimated surface wave maximum amplitudes of the order of 70 mm based on a magnitude 6 earthquake at 20-30 km from the site. Fujino and Ang (2) considered surface wave effects from a severe strike-slip fault near Long Beach, and calculated peak velocities. Based on a comparison of the situation at Annacis with their assumptions, a peak surface velocity of 20 cm/sec, with a displacement limit of 60 mm was adopted.

DYNAMIC MODELLING OF THE SOIL

The seismic analysis of the underlying soil was carried out using a modification of the SHAKE program. The stratigraphic information input to SHAKE was taken from borehole records of the drilling and sampling. The dynamic shear modulus profile (at low strain levels) for the North main pier was evaluated directly from in situ cross hole seismic testing which provided a record of the shear wave velocity at various depths. At the South pier the shear moduli were estimated from borehole data, penetration resistance measurements, general correlations with similar materials and comparison with the in situ results obtained at the North bank.

The variation of shear modulus with strain level was based on (3) for the Fraser River sand, (4) for the clayey silt and (5) for the glacial soils. Both the damping ratio maximum values and the variation of the damping ratio with strain level were based on the published work of Hardin &

Drnevich (6) for sands and the clayey silt. Reasonant column laboratory test results were used to modify these values for the clayey silt at low levels of strain. The damping parameters for the glacial deposits were based on (5).

By coincidence, a surface acceleration record of the 1976 Pender Island earthquake was obtained on Annacis Island, at an epicentral distance of approximately 50 km and a rock record of the same earthquake was obtained at Lake Cowichan at an epicentral distance of approximately 60 km. The existence of these two records permitted additional (7) comparisons of measured surface accelerations with those computed from rock accelerations fed through the overlying soil. Comparisons in the form of Acceleration Response Spectra are shown in Figure 3.

The correlation between calculated and measured acceleration maxima is reasonably good. However, it should be noted that the computed short period accelerations reduce sharply with the distance below grade. The response peak in the measured accelerations between 0.5 and 1.1 seconds, which is not reproduced in the calculated accelerations, is thought to be due to the presence of surface waves.

SOIL LIQUEFACTION

The Fraser River sands at the site of the North Main Pier are poorly graded and are submerged. Consequently, where they can be shown to be loose, they must be considered susceptible to a liquefaction type failure. In situ field tests carried out at the North Pier indicate that the upper 10 m of river bed sand exists at a density below that considered critical for liquefaction failure in the design earthquake. The principal method used to determine this critical level of density was the empirical relationship which relates cyclic stress ratio to the penetration resistance (8). The dynamic shear stresses for various levels within the alluvial sand deposit were obtained from the SHAKE analyses and the penetration resistance was calculated from Standard Penetration test records.

In order to prevent liquefaction failure of the upper sands this stratum will be artificially compacted with timber piles driven at 2 m centres over an area which extends laterally about 15 m beyond the pilecap on all sides.

PILE COMPLIANCE

One foundation alternative has long (approximately 70 m) 910 mm diameter, steel pipe piles supporting the North main pier. Earthquake generated horizontal relative movements in soil layers within 70 m of the surface will cause flexure of the piles. If it is assumed that the relative stiffnesses of the soil and the piles force full compliance of the piles with the soil, then an upper bound on pile flexure demand is obtained.

In order to ascertain the magnitude of this effect, time histories of lateral motion in each soil layer were computed by double integration of the soil layer accelerations obtained by the SHAKE program. Zero soil displacements at the beginning and end of the acceleration record were the assumed boundary conditions. A typical pile flexure time history is shown in Figure 4.

Although the deformation curve has double curvature, the amplitudes of displacement produced under these rock accelerations do not pose a strength or stability problem for the 910 mm diameter pipe piles. Maximum bending stresses due to curvature were of the order of 4 MPa and secondary bending stresses due to P-Δ effects amounted to about 12 MPa. At this level of rock acceleration, the impressed pile curvatures could become a problem at larger pile diameters or in non ductile piles.

SOIL DEFORMATION ANALYSIS BY NEWMARK'S METHOD

The main piers will be protected by sand fill extending out into the river channel and sloping down to river bed level on the river side, see Figure 1. This change in surface grade near the piers causes a shear stress in the foundation deposits. During an earthquake this static shear bias tends to promote pier movement in the downslope direction. On the North side, the clayey silt deposit is relatively weak under the conditions of rapid stressing and inadequate drainage which would prevail during an earthquake. For this reason a series of Newmark (9) analyses were carried out on the North main pier to estimate the permanent set which might be caused at the tower during an earthquake. The results of one series of Newmark analyses carried out for an earthquake of maximum acceleration equal to 0.2 g is shown in Figure 5. This diagram indicates the amount of tower movement computed for a range of undrained shear strengths of clayey silt and a range of river bed levels.

The curves show three levels of soil strength and represent the conditions where the clayey silt has a strength equal to the lower bound of all laboratory triaxial test results (A); a strength typical for a marine clay which has not had the benefit of over-consolidation (B); a strength which is the mean value of the most credible of the laboratory work caried out at a specialist laboratory on specimens specifically prepared for this type of analysis (C). The Newmark analysis indicates that a reasonable upper bound for permanent movement of the North tower would be about 0.1 m or less.

DYNAMIC MODELLING AND ANALYSIS OF SUPERSTRUCTURE

A three dimensional full bridge computer model of the superstructure was created for aeroelastic analysis and it was convenient to use this for seismic analysis also, because it permitted lateral and longitudinal seismic bridge responses to be analyzed on the same computer model. The model incorporated lateral torsional coupling, which is significant in an open girder section. Catenary cable equations were used to establish dead load geometry and forces, thereafter tangent cable stiffnesses were used for linear dynamic analysis.

It was originally intended to use a conventional base acceleration input analysis method with modal responses being computed by response spectra techniques. However, examination of the ground surface motions at each main pier, generated by common bedrock acceleration records, showed such large differences of response, see Figure 2, that it was considered unrealistic to assume common ground accelerations and displacements at the two locations. Also, for a structure with such widely separated main supports, it was considered important to examine the effects of relative pier motion due to surface waves. Therefore, a time history response approach was adopted, with separate imposed displacements at the base of each main pier. Selected groups of displacements and forces, which defined tower and superstructure responses, were audited every time step.

The energy content of the ground acceleration is predominantly in the period range 0.1 to 1.5 seconds, while the lowest nine natural frequencies of vibration of this structure have periods longer than 2.0 seconds. This marked period separation of lower mode structure response from the input energy results in relatively small effective maximum earthquake forces on the structure. Under strong motion earthquake ground acceleration of 0.28 g, the effective maximum lateral and longitudinal accelerations experienced by the mass of the superstructure are 9.1% and 3.2% g respectively. Factored lateral wind loads are equivalent to approximately 5% g and these govern the lateral load design of the deck and side towers. In the longitudinal direction, the dead load plus wind combination has a factored design load equivalent to approximately 2% g, but longitudinal tower bending is governed by traffic live loads.

A measure of the response to surface waves was obtained by calculating the superstructure response to an arbitrary three full sine waves of excitation at each of the predominant natural periods. Structural damping was assumed proportional to displacement. The deck/cable system is analogous to a flexible beam on an elastic foundation, and, in the absence of discrete rigid bearings, is extremely tolerant of vertical and lateral earthquake dynamic displacements and also relative motions of the supports. Table 1 lists some earthquake response maxima in the structure.

From the results shown, it is obvious that the bridge is able to resist a moderate earthquake elastically and the strong motion earthquake without serious damage. Inelastic lateral deformations in the towers make only minor ductility demands for the strong motion earthquake. This means that, subject to the provision of properly designed ductile details at the appropriate locations, the structure has significant additional earthquake resistance capacity for the catastrophic earthquake.

CONSTRUCTION DETAILS

The general principles followed in the seismic aspects of detail design were, firstly, to keep things simple and, secondly, to create details which could handle larger than anticipated displacements without loss of structural integrity. The ability of the steel pipe piles to handle bending due to relative soil movement over their length has already been mentioned, but equally important is the capacity of the pile to pilecap connection to handle flexure of the pile without spalling of the cap. The pile is filled with reinforced structural concrete at the top and ample vertical rebar is well lapped into the pilecap.

Generous rebar anchorage lengths and laps, together with amounts of confining steel in excess of code minima, are used in all areas of the towers, such as the base and in the portal to tower connection zones, where ductility is required.

The structural connection between the deck and the towers is detailed carefully to meet the service requirements of traffic, temperature, and wind, while still providing strength and displacement capacity for the strong motion earthquake and breakaway reserve capacity in the event of excessively large displacement demands. A large rubber bridge bearing mounted in the vertical plane provides a flexible spring (14 KN/mm) connection for vertical displacements and horizontal displacements parallel to the centreline of the bridge, and a stiff spring (280 KN/mm) for horizontal displacements normal to the centreline of the bridge. In the event of failure of this shear block due to excessive displacements, the deck will physically engage the tower. "Damage panels" in the deck will absorb energy while being crushed against a tower strong point.

Other points of connection of the deck occur at the tie down piers N2, S2 and at flanking piers N3, S3 (see Figure 1), where the main bridge abuts the approach spans and the expansion joints are located. Structural integrity is assured by the use of very long bearing engagement over piers N3, S3 and the installation of earthquake restrainers between the main bridge and approaches. Sacrificial damage slabs are provided on each side of the deck expansion joints. The tie down piers are flexible longitudinally and are ductile frames laterally. Ample extra vertical capacity is provided against tie down lift off due to vertical earthquake accelerations.

CONCLUSION

Based on a thorough analysis of acceleration and velocity response, it has been shown that, with appropriate choice of foundation and superstructure configuration and, more particularly, choice of structural constraints such as bearings, a long span cable stayed bridge can be effectively isolated from the acceleration excitation of a strong motion earthquake and also can handle the displacement demand. Major increases in ultimate seismic performance of the structure are attainable at small premium.

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Table 1

Earthquake Response Maxima in the Bridge Superstructure

Excitation Olympia 1949 @ 0.28 g	Lateral Response	Longitudinal Response
Maximum total base shear	9.1% g	3.2% g
Maximum deck displacement	280 mm (horiz)	144 mm (horiz)
	52 mm (vert)	90 mm (vert)
Maximum deck displacement (Surface Wave)	-	350 mm (horiz)
Tower top displacement	225 mm	130 mm (horiz)
B.M. @ tower base	.69 Mu	0.29 Mu
B.M. in lower crossbeam @ tower	1.55 Mu	-

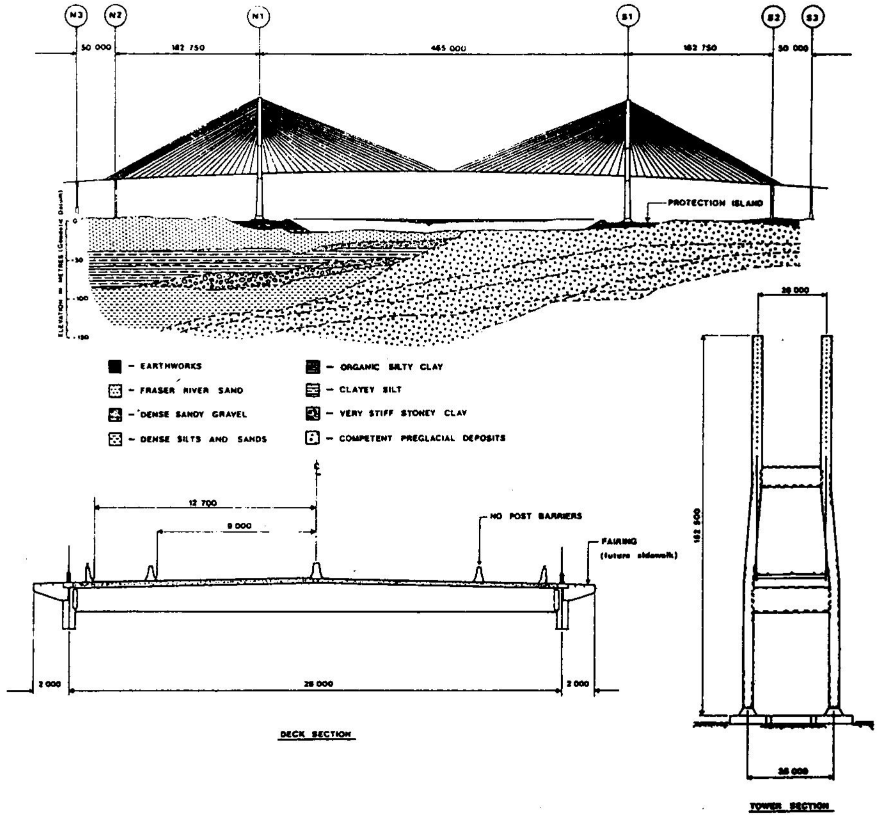

FIGURE — 1

ELEVATION AND SIMPLIFIED GEOLOGY

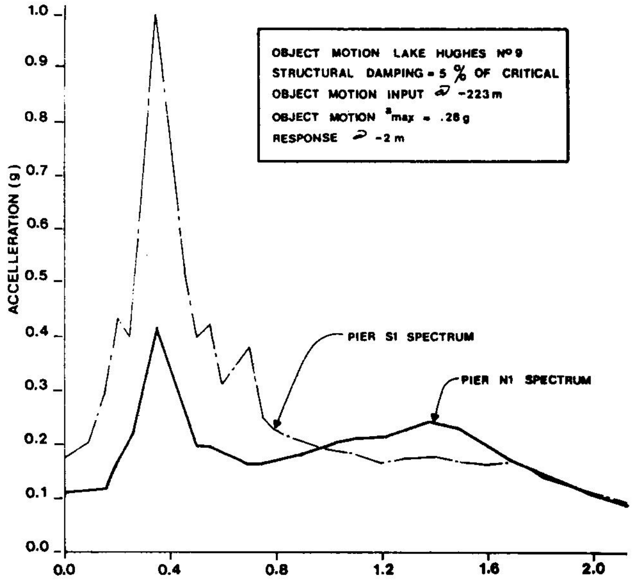

FIGURE – 2

COMPARISON OF ACCELERATION RESPONSE SPECTRA
AT
NORTH AND SOUTH PIERS

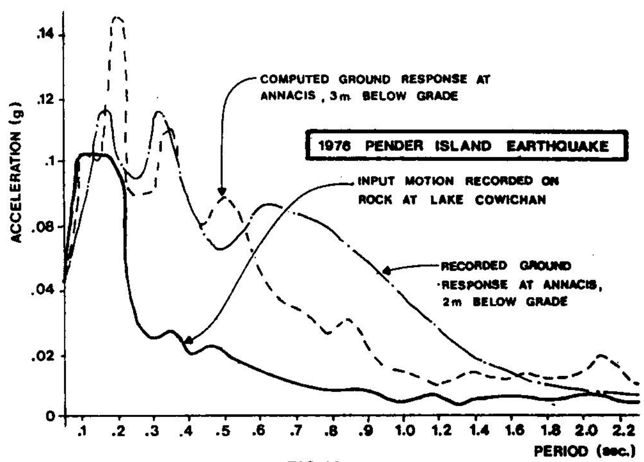

FIGURE – 3

COMPARISON OF RECORDED AND COMPUTED GROUND
ACCELERATION RESPONSE SPECTRA AT ANNACIS ISLAND

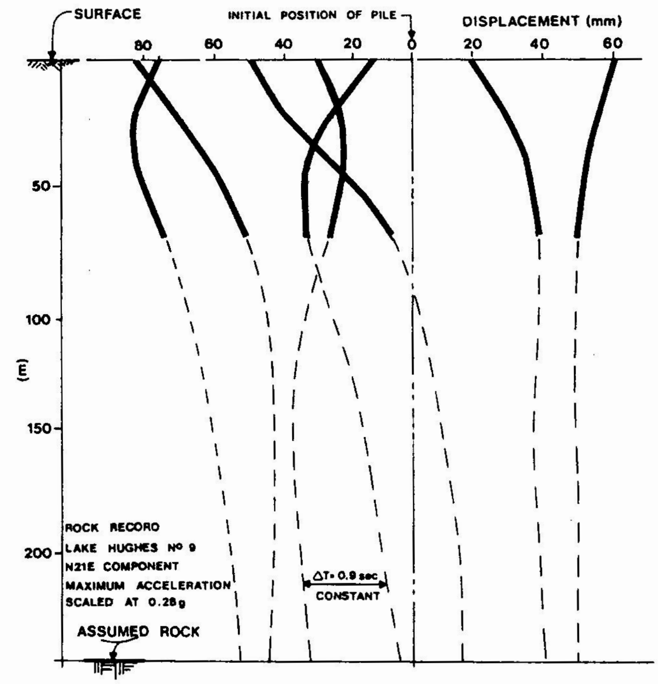

FIGURE - 4

TIME HISTORY OF PILE LATERAL DISPLACEMENT AT PIER N1

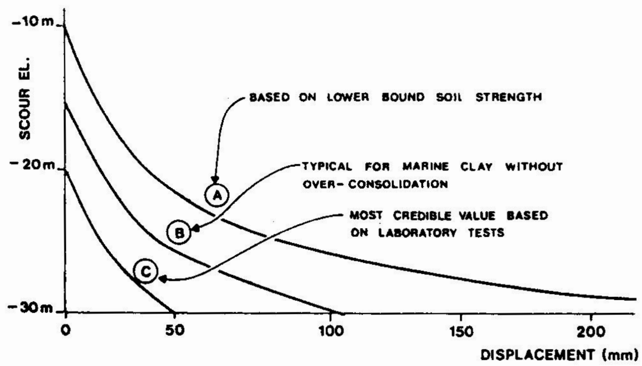

FIGURE - 5

CALCULATED LATERAL MOVEMENT

OF

NORTH PIER IN AN EARTHQUAKE