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Seismic evaluation of gravity dams – Practical aspects

B.H. Fan & J.J. Sled
B.C. Hydro, Vancouver, B.C., Canada

ABSTRACT: The apparent good performance of concrete gravity dams in past earthquakes must be viewed in the proper perspective: that no such dams on record, with maximum reservoir, have been subjected to the most severe ground motion assessed possible for their respective sites. Despite recent advances made in research, seismic evaluation of concrete gravity dams remains largely a subjective process. A comprehensive review of the state-of-the-art is provided in this paper, from the vantage point of a practitioner. Two case studies are presented to illustrate a practical approach for bounding the seismic evaluation of concrete gravity dams.

1 INTRODUCTION

Concrete gravity dams were traditionally designed for static loads and checked for seismic effects due to nominal earthquake forces, typically based on pseudo-static computations with seismic coefficients of 0.1 or less and assuming uniform distribution of acceleration along the height of the dam. Hydrodynamic effects were either neglected or based on incompressible water acting on rigid dams (Westergaard, 1933). The stringent requirements placed on factors of safety against sliding, overturning and overstressing were capable of being met.

2 REVIEW OF STATE-OF-THE-ART

2.1 Linear elastic analysis

Dynamic response of concrete gravity dams began to gain attention in the mid-1960s, accentuated by the damage to the Koyna Dam in the 1967 M6.5 earthquake. Seismic forces came to be more realistically evaluated.

A two-level design earthquake approach was introduced for the evaluation and design of dams: the Operating Basis Earthquake (OBE), sometimes also referred to as the Design Basis Earthquake (DBE), and the Maximum Design Earthquake (MDE). It is usually required that a dam should be capable of resisting, without significant damage, the DBE which will probably occur during the life of the dam. The dam must also be capable of surviving the MDE without the uncontrolled release of the reservoir. For a given dam, the MDE is usually dependent on the hazard potential of the dam (B.C. Hydro, 1988).

Important advances have been made in the past two decades in the dynamic analysis of seismic response of concrete dams. The effect of dam flexibility and water compressibility began to be considered, followed by the inclusion of dam-reservoir and dam-foundation interaction effects, as well as the effects due to foundation compliance and damping and wave absorption by reservoir bottom materials (Fenves and Chopra, 1984).

Simplified dynamic analysis procedures have also been proposed, based on data generated in extensive parametric studies using FEM analyses in the time domain (Fenves and Chopra, 1986). Recent research also indicates the importance of reservoir quasi-resonance effects, depending on the relative fundamental frequencies of independent vibration of the dam-foundation and the reservoir. These studies have been conducted largely on 2-D mathematical models.

The overall result of considering dynamic responses is that seismic forces thus derived are greatly increased from their traditional values. The application of these larger forces to concrete gravity dams often finds them unable to meet the old stability criteria. The emphasis on safety evaluation then shifted to stresses, damage and post-earthquake stability.

2.2 Nonlinearity

Nonlinearity in the dynamic response of gravity dams began to attract the interest of researchers in the late 1960s. Various post-yield constitutive relationships were put forward for dam concrete. Cracking was first investigated with smeared crack models using elastoplastic analysis procedures and later modelled with the application of fracture mechanics (Reich, Cervenka and Saouma, 1991). Opening of vertical contraction joints would also cause nonlinearity in the dam's response, and so would cavitation of the reservoir water. Attempts have recently been made to quantify the sliding response of concrete gravity dams during earthquakes (Chopra and Zhang, 1991).

2.3 Uplift

Opinions vary widely on the subject of uplift pressure during earthquakes in seismically induced cracks. They range from zero uplift (USBR, 1977) to uplift unchanged from its pre-earthquake value (FERC, 1991). Analytical and experimental research is underway in an attempt to provide more answers in these areas (Amadei, Illangasekare and Chinnaswamy, 1991; Bruhwiler and Saouma, 1991).

3 PRACTICAL ASPECTS

Following the designation of a hazard classification for the dam and the selection of design ground motion parameters, two tasks face the practitioner in the seismic evaluation of concrete gravity dams. Firstly the level of investigation warranted must be established. Secondly a strategy must be planned which utilizes the resources and technology available in order to obtain answers, fragmental as they may be, and combine them to form a best-estimate evaluation.

3.1 Pseudostatic and pseudodynamic analyses

Due to its inability to account for dynamic behaviour of structures, the pseudostatic analysis is generally discounted as a meaningful investigative tool. However, when used with realistic seismic coefficients, it provides helpful "indices" of seismic safety of gravity dams in a screening level assessment.

The advantage of the pseudodynamic method lies in its ability to approximate dynamic behaviour of the dam despite its simplicity. It is very useful in parametric studies to identify critical factors to be included in the more extensive and expensive dynamic analyses.

3.2 Reponse spectrum vs time history

A linear dynamic analysis may be carried out with either response spectra or time histories as input. Nonlinear dynamic analysis procedures, however, are time-domain oriented.

The advantage of time history analysis is that it produces real-time response and yields information on the duration and recurrence of the peak response which is useful in forming a judgmental assessment of the damage.

A response spectrum may be a smoothed design response spectrum (DRS) constructed following standard procedures such as those proposed by Newmark and Hall (Newmark and Hall, 1982). It has the advantage of extreme simplicity, in that it can be constructed from 1 to 3 ground motion parameters. The amplification factors used in its construction are generally based on an arbitrary suite of time histories.

An alternative with greater relevance is to construct spectra for short-listed time histories that are considered to best reflect site seismic conditions and to statistically obtain from these spectra the DRS. It should be cautioned that one should avoid compounding conservatism, for example, by selecting the mean plus one standard deviation value for the DRS from spectra and scaling it to the mean plus one standard deviation values of peak ground acceleration.

3.3 Two-dimensional vs three-dimensional analysis

Concrete gravity dams are usually straight and in relatively wide valleys. Consequently, the need for 3-D modelling is the exception rather than the rule. When 3-D modelling is used, the analyst must be satisfied that conditions at vertical contraction joints allow the necessary transmission of stresses to enable 3-D action.

3.4 Linear vs nonlinear analysis

A linear analysis is generally adequate for the evaluation of local safety in terms of computed elastic stresses compared to assessed material strengths. Positive assessment of local safety in critical areas of the dam usually leads to a positive assessment of global or overall safety.

On the other hand, a negative assessment of local safety does not necessarily mean global failure, unless a concurrent combination of local failures is identified that lead to the formation of a failure mechanism. It usually indicates the need to investigate the possibility of redistribution of stresses through nonlinear behaviour.

3.5 Discussion

The most rigorous state-of-the-art analytical tool is not always necessary in every case of seismic evaluation of concrete gravity dams. The selection depends on the tools available in the particular engineering office, as much as on the level of analysis warranted by the situation. The following two case studies demonstrate different levels of deterministic analyses applied to the seismic assessment of concrete gravity dams and the results obtained pieced together to form an evaluation.

4 JOHN HART DAM

B.C. Hydro's John Hart Project, completed in 1947, comprises both earthfill and concrete gravity structures. The Main Concrete Dam, the subject of this case study, is 30 m high and 200 m long and consists of two lengths of non-overflow sections with a central 3-bay gated spillway section 40 m wide. Deficiency investigations commenced in 1985. The dam is designated as high incremental hazard for earthquake and the MDE is therefore the MCE (Maximum Credible Earthquake). The MCE Peak Horizontal Ground Acceleration (PHGA) assessed for the site was 0.6g and the Peak Vertical Ground Acceleration (PVGA) was assumed to be 2/3 of the PHGA, or 0.4g.

4.1 Pseudostatic analysis

A screening level analysis was first carried out with the pseudostatic method applied to typical sections of the non-overflow and spillway dams. For the computation of stresses and the extent of cracking, the seismic coefficients were assumed to be equivalent to the MCE PHGA and PVGA divided by gravitational acceleration.

Examination of six acceleration time histories, short-listed from records of past earthquakes in the States of California and Washington on the U.S. west coast, showed that the ratio of the sustained peaks to the maximum peak in each case varies between 0.54 and 0.63. For stability calculations where a sustained effect is more relevant, therefore, the seismic coefficients were reduced to 2/3 of the values used in stress computations. Angle of internal friction of 50° and cohesive strength of 100 kPa were assumed for the concrete-rock interface. Results indicated marginal stability during the MCE event.

4.2 Dynamic analysis

Dynamic analysis followed, using the EAGD-84 computer code (Fenves and Chopra, 1984) on a plane stress finite element model of a typical section of the non-overflow dam as shown on Fig. 1. This model was selected based on the dam monoliths being in a straight alignment in a relatively open valley and with vertical contraction joints neither keyed nor grouted.

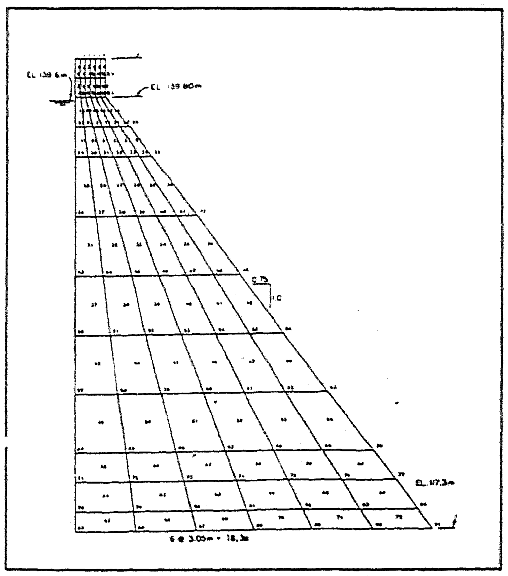

Fig. 1 John Hart non-overflow section 2-D FEM model

Sensitivity tests were made using the same six acceleration time histories from past earthquakes on the U.S. west coast. They were scaled to the PHGA for the John Hart MCE. Results were found to be sensitive to the direction of application of the horizontal ground motion. The time history producing the largest heel tension at the dam-foundation interface was designated as the design time history. It was used as input for all the dynamic analyses of representative dam sections.

The maximum vertical heel tensile stresses for the DBE and MCE cases were determined to be 570 kPa and 1,300 kPa respectively. These compare with the corresponding pseudostatic results of 210 kPa and 660 kPa. The amplifications were due to two factors: dynamic amplification effect and stress concentration at the heel. Fig. 2 shows the contours of maximum vertical stresses under MCE conditions.

4.3 Post-earthquake analysis

To assess the lower bound of post-earthquake stability of the non-overflow section, static analysis was carried out for normal reservoir level and with the dam section cracked through. Full uplift was assumed to develop over various percentages of the base width. It was thus computed that the section would not slide if full reservoir uplift did not develop under more than 60% of the base.

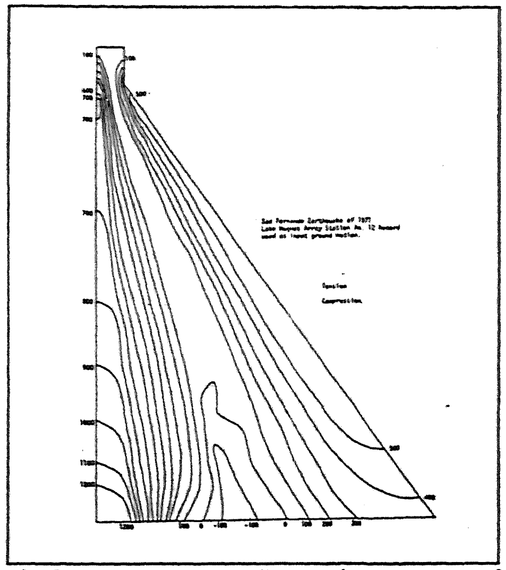

Fig. 2 John Hart non-overflow section - contours of maximum vertical stresses (kPa) for MCE + static loads

4.4 Assessment

The John Hart non-overflow section was assessed to be able to survive the MCE with some separation of the concrete from the foundation rock in the heel region, a performance considered acceptable for this very severe event. The remedial work carried out for the non-overflow section was the drilling of additional drain holes to relieve uplift and improve post-earthquake stability.

The spillway section was similarly assessed to be able to survive MCE ground motions in the direction of river flow. However, in order to enable the piers to survive MCE motions in the cross-valley direction, large capacity post-tensioned anchors were installed.

5 CLEVELAND DAM

Cleveland Dam is located on the Capilano River in a suburb of Vancouver, British Columbia. Retaining a water supply reservoir for the Greater Vancouver Regional District (GVRD), the 40 year old concrete gravity dam is made up of 13 monoliths for a crest length of 195 m, 30 m of which is in a deep canyon where maximum dam height reaches just over 100 m. The canyon section of the dam is straight in plan. The monoliths flanking the central canyon blocks, referred to as the abutment blocks, are aligned in a gentle arc of about 275 m radius and are much more modest in height (average 20 m). Vertical contraction joints were keyed and grouted. A 21.3 m wide spillway occupies the central part of the canyon section and is controlled by a 7 m high drum gate.

A 3-phase investigation was commenced in 1989 to assess the stability of the dam. Due to the highly populated and developed area downstream, the dam is classified as a high hazard structure for earthquake and the MDE is therefore the MCE. The investigation was conducted by the Vancouver engineering firm of Klohn Crippen Consultants Ltd. (KCCL). The authors, through B.C. Hydro International Ltd. (BCHIL) provided assistance to KCCL in the seismic evaluation of the dam.

5.1 Phase 1 - Screening level review and analyses

Phase 1 consisted of a review of design documentation and static, pseudostatic and pseudodynamic computations. The objective was to identify conditions that are unquestionably safe, unquestionably unsafe or requiring further investigation. The PHGAs established for the site were 0.2g for the DBE and 0.5g for the MCE. PVGAs were assumed to be 2/3 of the corresponding PHGAs.

Original design calculations made in 1946 included trial load analyses to approximate the transfer of loads to the canyon walls. Horizontal and vertical accelerations of 0.1g had been assumed. The original trial load results were scaled to the new seismic parameters for the investigation.

The investigation concluded that the abutment blocks would have adequate stability for all conditions except the MCE. It was also concluded that the canyon section would perform acceptably for all loading conditions provided the 3-D action is achievable and the canyon walls have adequate strength to support this action. It was further concluded that the 3-D action of the canyon blocks under lateral loads would subject the transition blocks, i.e. abutment blocks immediately adjacent to the canyon blocks, to additional loads. 3-D analysis was recommended to investigate these aspects.

5.2 Phase 2 - Dynamic analysis

Linear elastic analysis were carried out on a 3-D FEM model of the dam-foundation complex (Fig. 3). Linear static and dynamic analyses were made with the PC-version of the ANSYS computer code (SASI, 1989). The FEM model contained 1,744 elements for the dam and 4,335 elements for the foundation. The drum gate was modelled as a stiff steel plate. Other appurtenant structures were modelled as lumped masses. The response spectrum option was used for the earthquake load cases. The reservoir was simulated with lumped nodal masses on the water face of the dam.

Geological investigations carried out in Phase 2 concluded that the foundation rock is strong and relatively incompressible as a rock mass, with the exception of stress relieved zones (up to 10 m in thickness) on either side of the canyon. Three zones were assumed with different deformation modulii: Zone 1 adjacent to both canyon walls (Eh=1,380 MPa; Ev=6,900 MPa), Zone 2 immediately below the abutment blocks (Eh=Ev=6,900 MPa) and Zone 3 representing the rest of the foundation (Eh=Ev=27,600 MPa). It was estimated that consolidation grouting could increase the Zone 1 horizontal deformation modulus to 6,900 MPa.

Fig. 3 is an isometric view, looking downstream, of the first mode shape. It is apparent that the dam geometry, as well as the low deformation modulus in the Zone 1 rock, has a dominant effect on dam response. The top part of the canyon section is observed to undergo significant transverse deflection relative to the abutment blocks, indicative of horizontal flexural action. The first mode response was found to contribute only 50% to 60% to the overall response, much lower than that for typical gravity dams.

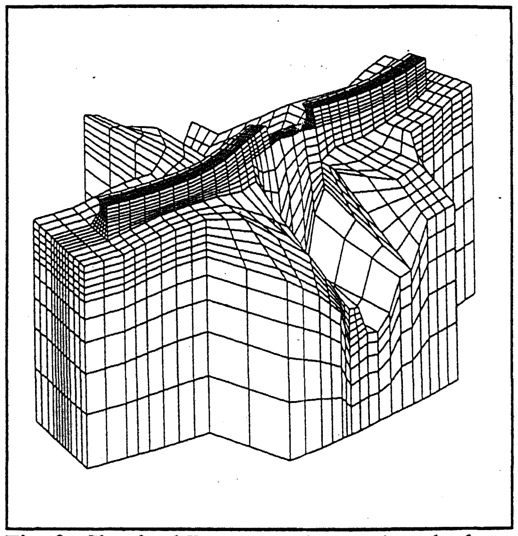

Fig. 3 Cleveland Dam - Fundamental mode shape

Analyses made for the MCE case first assumed 5% structural damping in the dam. High tensile stresses were indicated, fairly well distributed though mainly in the upper section of the canyon blocks. Damping was then increased to 7% to approximate the effect of cracking (Corps of Engineers, 1985). This resulted in a general stress reduction of about 12%. Damping values larger than 7% were judged to be unwarranted due to the localized nature of the high tensile stresses.

Analyses were performed for cases with and without foundation grouting. First mode frequency was found to increase from 6.2 Hz to 7.2 Hz due to the increase in the Zone 1 foundation deformation modulus. Reductions in stresses ranged from 40% at the base of the transitition blocks to no noticeable reduction in the upper section of the canyon blocks. A general reduction in the depths of tension zones was also observed.

Fig. 4 shows vertical stress contours (MCE load case) on a representative 2-D slice taken from the canyon section of the 3-D model, for the foundation improved case and 7% structural damping in the dam. The largest tensile stress contour (labelled "I") is 3,530 kPa, while the smallest (labelled "A") is 160 kPa.

Based on the results of a modest concrete strength evaluation program, a threshold tensile strength value of 1,720 kPa was selected for assessment of crack extents. On this basis it was assessed that, under MCE conditions, the upper part of the canyon blocks could experience severe cracking. While this assessment helped focus attention on potential problem areas, it was recognized to be indicial at best. A more rigorous concrete strength evaluation program, as well as an investigation into the various factors that could influence the dynamic response of the dam, such as contraction joint opening, foundation damping and reservoir bottom absorption, would be required for a better informed assessment.

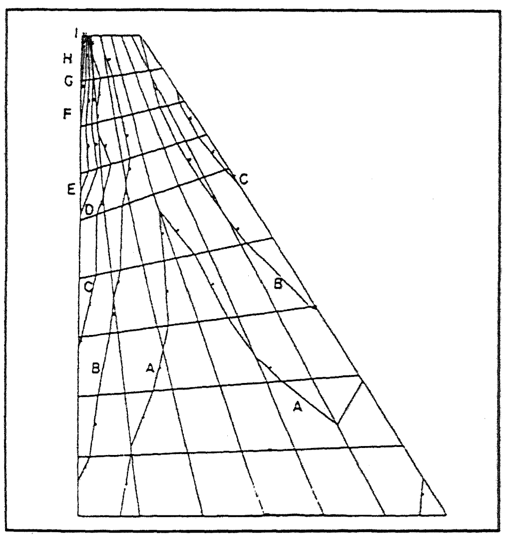

Fig. 4 Canyon section vertical stress contours - MCE load case

5.3 Phase 3 - Concrete evaluation and correlation of analysis results

A comprehensive concrete coring and testing program was carried out in Phase 3. Cores were obtained from 22 locations in mass concrete and at lift joints and contraction joints. Efforts in retrieving intact cores at contraction joints were unsuccessful and tensile strength across these joints was therefore assumed to be zero. Collation of test results led to the assumptions of static tensile strength in mass concrete of 4,140 kPa and dynamic tensile strength in mass concrete having a lower bound of 5,280 kPa and an upper bound of 7,590 kPa. Equivalent strengths at lift joints were assumed to be 1/2 those in mass concrete.

The lack of tensile strength across contraction joints means that the seismic response of the top part of the canyon blocks, as predicted by the 3-D elastic analysis carried out in Phase 2, would be substantially different. The behaviour of the upper portion will more closely resemble that of vertical cantilevers, restrained by the lower portion which is wedged in the narrow canyon. The keyed contraction joints will provide some lateral load sharing of neighbouring cantilevers. Therefore, 2-D analysis would provide upper bound cantilever response.

Pseudodynamic analyses were then carried out, following the procedures proposed by Fenves and Chopra (Fenves and Chopra, 1986), on representative 2-D sections of the abutment, the transition and the canyon blocks, to study the parametric effects of various values of structural and foundation damping and reservoir bottom absorptiveness.

Maximum tensile stresses of 690 kPa and 360 kPa were computed for the transition and the abutment blocks, respectively. These were considered acceptable for MCE conditions. For the canyon section, EAGD-84 anaysis was considered necessary to better define the high tensile stresses computed at the base.

A reservoir bottom wave reflection coefficient between 0.75 and 0.9 was considered to be appropriate. Due to the extreme sensitivity observed in the parametric study of the response to this particular parameter, it was decided to err on the safe side and to proceed with the EAGD-84 analysis with wave reflection coefficient of 0.9. Three time histories were selected from the suite from which the design response spectrum was derived. Fig. 5 shows the distribution of vertical stresses in the canyon section at the instant when maximum tensile stresses at the water face of the dam occur.

Based on the upper bound of vertical stresses predicted by the EAGD-84 analysis and on the lower bound of assessed tensile strength at lift joints, a tension crack width, under MCE conditions, equal to 18% of the width of dam was estimated for the critical horizontal section.

Post-earthquake stability was computed for the same horizontal section, assuming a friction angle of 45° and cohesion of 350 kPa, on the uncracked portion, for concrete sliding on concrete. The factor of safety would exceed 1.3 provided crack widths were not greater than 45% of section width. These calculations did not consider crack propagation or cracking on the downstream face of the dam.

5.4 Assessment

Cleveland Dam as a whole does not fall into any of the usual classifications of dam types, although it is generally referred to as a concrete gravity dam. No single procedure is available to adequately carry out the complete analysis. A number of different analyses were carried out to estimate the safety bounds to arrive at an evaluation of the dam's seismic stability.

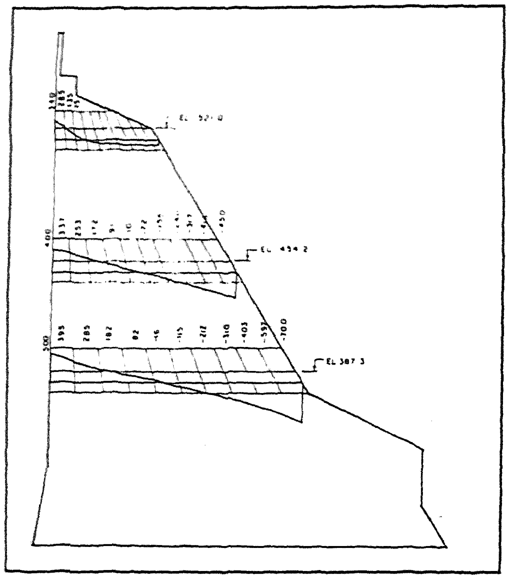

Fig. 5 Canyon section - snapshot of vertical stresses (Imperial units)

The most conservative interpretation of the stress distribution within the dam, and of the concrete strengths, indicate that some cracking could occur on the upstream face of the dam after a MCE event, but that this cracking would not result in an uncontrolled release of the reservoir. Post-earthquake stability of the cracked dam would be acceptable provided the reservoir level is reduced fairly rapidly after the MCE event and that the damage is assessed and repaired before the reservoir is restored to full operation.

Foundation grouting and restoration of foundation drains were proposed to improve dam performance.

6 CONCLUSIONS

The good performance of concrete gravity dams in past earthquakes could be attributed as much to good fortune as to good design, as no such dams are known to have been subjected to their MCE events with full reservoir.

Any one available analytical procedure by itself usually does not enable a rigorous evaluation of the seismic safety of a gravity dam. Each dam is different and challenges the ingenuity of the engineer to assemble a set of proper procedures to arrive at a practical evaluation.

Due to the grave consequences of an error, proposed new analytical procedures must be corroborated by experimental research and extensive field tests by practising engineers before their general application.

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